



Prediction of seismic demands for ancillary elements in vertically irregular steel frames

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ABSTRACT

This paper investigates the elastic and inelastic seismic response of non-structural ancillary components within multi-storey steel frames exhibiting typical vertical irregularities. For this purpose, a large set of 188 steel moment framed structures, characterised by various combinations of setbacks within their elevations, is considered. The frames, which are designed according to European code provisions, are described in detail in terms of their configurations, member sizes, and dynamic characteristics. Within these structures, non-structural components, characterised by various periods and strength reduction factors are mounted at individual floors and assessed in terms of both their displacement and acceleration response under seismic loading. Detailed nonlinear dynamic time-history analyses are carried out using a suite of 100 ground motion records from both far-field and near-field sources, selected from the PEER NGA database in accordance with FEMA-695 requirements, scaled at intensity levels consistent with both elastic and inelastic conditions. The results are used to assess the suitability of existing models which are primarily developed to predict the seismic response of non-structural components in regular frames. The results reveal, with 95 % confidence, that frame irregularities have a direct and significant effect on the seismic response of non-structural components, and therefore, various modifications to current models are necessary to capture adequately the influence of vertical frame irregularities. Based on the findings, unified analytical models are proposed and shown to predict, well within an accuracy of 10 %, the seismic displacement and acceleration response of non-structural components mounted within the floors of both regular and irregular multi-storey steel frames.

1. Introduction

Various classifications for non-structural components (NSCs) have been used in the past yet, for building structures, they can be broadly categorised in terms of configuration as either typical infill elements or floor-mounted ancillary systems. For the latter, early research focused on ancillary components within nuclear facilities [1], and gradually expanded to more common applications, such as medical equipment, piping systems, or bookshelves [2–4]. Several studies have shown that NSCs are highly vulnerable to earthquake-induced damage, with the monetary losses typically representing the largest part of the total investment, often reaching up to 80 % [5–8]. Other studies examined the response of different types of NSCs using experimental and numerical approaches [9–13], based on which different categories were suggested depending on their response characteristics. For instance, based on the weight of NSCs, they were classified as either light or heavy components

in terms of the adequacy of coupled or uncoupled analysis methods between the NSCs and the primary structure [14–19]. Tests were also carried out to assess the performance of specific forms of NSCs rather than the interaction with the primary structure [20,21]. Overall, apart from the monetary losses from damage to NSCs, even in the absence of structural damage [2–4], the detrimental effects on the functionality and resilience of the structure can be significant and require careful consideration in design.

In order to enable appropriate assessment of the seismic response of NSCs, it is necessary to provide reliable practical approaches for predicting the imposed displacement and acceleration demands. Recent studies on assessing the response of ancillary components include the investigation by Obando & Lopez-Garcia [22], which assessed the displacement demands in NSCs with due account for their inelastic behaviour. Based on a series of time-history analyses on NSCs attached to 3-storey steel frames and 5-storey reinforced concrete frames, it was shown that the inelastic displacement response of NSCs was less

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Nomenclature			
AF	Overall amplification factor	h	Height of primary structure
AF_{Rnsc}	Amplification factor for spectral acceleration accounting for inelastic behaviour of ancillary component	L	Length of primary structure at the ground floor
AF_{Rstr}	Amplification factor for spectral acceleration accounting for the inelastic behaviour of primary structure	L_1	Length of primary structure at the roof level
λ_0	Modification factor for reverse function of non-structural components mounted on irregular frames	$M_{re(irre)}$	Mean value of non-structural components response within regular (irregular) frame
λ_1	Modification factor for fundamental period reduction function of ancillary components mounted on irregular frames	Q	Quantitative coefficient for structural irregularity
λ_2	Modification factor for second natural period reduction function of ancillary components mounted on irregular frames	R_{ps}	Force reduction factor for primary structure
β	Modification factor for spectral acceleration of ancillary components mounted on irregular frames	R_{nsc}	Force reduction factor for ancillary component
C_R	Inelastic displacement ratio	S_a	Spectral acceleration
$C_{R,0}$	Reverse function for inelastic displacement ratio prediction	S_{an}	Spectral acceleration of ancillary component
C_{R,T_1}	Fundamental period reduction function for inelastic displacement ratio prediction	$S_{an,code}$	Spectral acceleration of ancillary component from the 2nd generation draft of Eurocode 8
C_{R,T_2}	Second natural period reduction function for inelastic displacement ratio prediction	$S_{an,p}$	Spectral acceleration of ancillary component from the regular frames model
		S_{ean}	Spectral acceleration of ancillary component from elastic design spectrum
		$S_{ep,i}$	Spectral acceleration of the primary structure at the i^{th} vibration mode from elastic design spectrum
		T_{an}	Natural period of ancillary component
		$T_{1(2)}$	First (second) period of primary structure
		Φ_{ij}	i^{th} mode shape value at j^{th} floor for primary structure
		Γ_i	Modal participation factor of i^{th} mode for primary structure
		z	Attachment height of ancillary component

sensitive to the fundamental period of the primary structure, compared with their corresponding elastic displacement response. Their conclusions on the displacement response were also consistent with the findings from other studies [23–25]. Based on their assessments, Obando & Lopez-Garcia [22] proposed a predictive relationship for the inelastic seismic displacement ratio of NSCs. This work was subsequently extended by Bravo-Haro et al. [25] to account for higher levels of inelasticity in the NSCs as well as validation under an extensive set of realistic ground motion records of different types. Further improvements were also recently proposed by the authors [26–28], to address several limitations related to the ability of these models to deal with high levels of inelasticity in the NSCs and possible inelasticity in the primary structure, as well as inaccuracies at higher modes of vibration. It should be noted however that these studies were based on, and validated for, regular multi-storey frame configurations.

Apart from the studies undertaken on assessing the displacement demands, the acceleration response of NSCs has also been examined in several investigations as discussed below. Prediction models for the acceleration response are also used in codified provisions as a basis for determining the seismic design actions on ancillary elements. For example, the current Eurocode 8 [29] assumes a linear distribution of the component accelerations along the height of the structure and proposes an over-simplified prediction equation which was shown in a number of studies to exhibit several limitations [26–32]. Modifications were therefore proposed by revising the vertical distribution of component acceleration and incorporating the influence of the inelastic behaviour of primary structures [30–32]. Vukobratovic & Fajfar [33] also proposed a series of improved relationships for determining the spectral accelerations of NSCs, by further accounting for higher vibration modes and subsequently suggesting an upper bound around the fundamental period of the primary structure [34,35]. These relationships have been incorporated into the draft of the upcoming second-generation version of Eurocode 8 [36]. However, it was shown through a recent study by the authors [27] that further modifications are necessary to address a number of limitations related primarily to the inelastic response of both the NSCs and the primary structures. Importantly, as for the investigations on displacement demands, these studies focused primarily on ascertaining the response for regular frame

configurations.

As many multi-storey buildings exhibit various types of irregularities, particularly in terms of setbacks in elevation, it is important to assess the response of NSCs in such structures. Previous studies [37–42] have focused on the influence of plan eccentricities and shown that when the period of NSCs was tuned with that of the primary structure, their seismic acceleration responses were amplified near the torsional periods. It was also shown [40–42] that the inelastic behaviour of the primary structures to which the NSCs were attached could significantly reduce the torsional amplification. In order to account for the torsional effect on the seismic response of NSCs, Laguardia et al. [43] suggested a torsional parameter to account for the eccentricity between the centres of mass and stiffness of the primary structure. Similarly, Aldeka et al. [44] conducted a detailed numerical investigation and suggested torsional amplification factors involving the top floor rotation and angular acceleration. In contrast, further research is needed to evaluate the influence of vertical irregularities in multi-storey frames on the seismic response of NSCs.

This paper extends previous research by examining the seismic displacement and acceleration response of ancillary NSCs within multi-storey frames exhibiting vertical irregularities. A set of 188 code-compliant steel moment frames, characterised by various combinations of setbacks within their elevation, are considered in the study. Nonlinear dynamic analyses are carried out using a suite of 100 ground motion records, incorporating 44 far-field and 56 near-field records with and without forward directivity velocity pulses. The ancillary components are modelled as single-degree-of-freedom with varying periods and strength reduction factors, considering elastic and inelastic irregular primary structures with setbacks. This detailed approach ensures that the findings are representative of realistic seismic scenarios, addressing limitations of previous studies, discussed above, that focused primarily on either elastic response or specific limited conditions. The influence of vertical irregularity of the primary structures on the seismic response of the ancillary NSCs is assessed and quantified, and unified relationships, which are applicable to both regular and irregular frames, are proposed for predicting the displacement and acceleration response. These relationships bridge the gap between existing predictive models, which typically exclude irregularities, and the practical need for simplified

code-compatible solutions.

2. Structural configurations and loading conditions

2.1. Primary frames and ancillary components

Three sets of initial regular frames, consisting of 8 three-storey frames (Set A), 15 five-storey frames (Set B), and 16 seven-storey frames (Set C), were adopted as the basis for the primary frame modelling in this study. These reference frames [27] were designed to Eurocode 8 [29], by considering various combinations of soil conditions and serviceability drift limits [45]. Each frame had three spans of 6 m, a first-storey height of 4.5 m, and a typical storey height of 3.5 m, as shown in Fig. 1. The considered seismic level was defined by a peak ground acceleration of 0.35 g. In total, 188 vertically irregular steel moment resisting frames were considered, with various combinations of setbacks, as shown in Figs. 2 to 4. The frames conform to Eurocode 3 [46] and Eurocode 8 [29] provisions, and represent extensions of the regular frames considered in the studies of Bravo-Haro et al. [45] and Ding et al. [27]. The number of setback spans ranges from one to two spans across the three-storey (Set A), five-storey (Set B), and seven-storey (Set C) frames. With respect to the number of setback storeys, Set A frames include either one- or two-storey setbacks, Set B frames include either two- or four-storey setbacks, while Set C frames include two-, four-, and six-storey setback configurations. Each frame is named according to its corresponding frame index and setback characteristics as XY-MN; in this notation, 'X' denotes the frame set, 'Y' represents the frame number within its corresponding set, 'M' indicates the number of setback spans, and 'N' refers to the number of setback storeys. Detailed information on the cross-sections, as well as the first two natural periods (T_1 and T_2) of the frames, are provided in Tables D1-D in the Appendix.

Fig. 5 depicts the distribution of the first two natural periods of the 188 irregular frames. A notable concentration of fundamental periods around 0.7–1.1 s is observed. The first-mode period T_1 of the frames covers a wide range from 0.3 s to 1.5 s, whereas the distribution of T_2 is comparatively narrower (varying from 0.1 s to 0.55 s). These period distributions indicate that the primary structures considered have distinct fundamental and second natural periods which are generally not closely spaced. It is also useful to show that the frames can be classified as vertically irregular based on Eurocode 8 [29,36] provisions. In general, regularity implies relatively uniform distribution of stiffness and

mass. The 2nd generation Eurocode 8 draft provisions [36] prescribe an upper limit of 20 % for the percentage change in stiffness and mass between two consecutive floors for a building to be considered as regular. Figs. 6 and 7 depict the mass and stiffness distributions, respectively, for Set A frames with one-span and one-storey setbacks, Set B frames with one-span and two-storey setbacks, and Set C frames with one-span and two-storey setbacks. Evidently, the 20 % criterion is not met at the locations of abrupt changes at the interfaces of the regular parts and the setbacks. These abrupt changes are all above 20 %, and in some cases exceed 70 %. In addition, according to the current Eurocode 8 [29] provisions, in the presence of setbacks, a structure is characterised as regular in elevation if the percentage reduction of length between the top setback and the ground floor does not exceed 30 %, and the respective length reduction across two consecutive storeys is not greater than 10 %. These criteria are also not satisfied in the frames considered herein, which are therefore characterised as irregular.

At each floor of the 188 irregular primary structures, a series of ancillary NSCs is examined, with varying period (T_{an}) from 0.05s to 4.00s using a 0.05s step. Both elastic and inelastic NSCs are considered, with the level of inelasticity (R_{nsc}), varying from 1 to 6 (i.e., $R_{nsc} = 1, 2, 3, 4, 6$ and 6), noting that R_{nsc} also represents the demand-to-capacity ratio (i.e. force reduction factor). The primary structures and the NSCs were modelled in the nonlinear finite element program OpenSees [47]. As in previous studies by the authors [27], the lumped plasticity approach was adopted for the primary structures. Rotational springs were placed at column and beam ends to simulate the plastic hinges. These rotational springs adopted the Ibarra-Median-Krawinkler model [48,49]. For the NSCs, a lumped mass was attached to a vertical cantilever, fixed at the base at each floor level, as illustrated in Fig. 8. Since the weight of the NSCs is considerably smaller than that of the floors to which they are attached, decoupled analysis was performed by applying a pre-established floor response acceleration time-history as input at the base of the NSCs.

2.2. Ground motion input

Two distinct sub-sets of seismic acceleration records are used in this study, namely far-field and near-field records, to capture a broad range of ground motion characteristics. These sub-sets, sourced from the PEER NGA-West2 database [50,51], include a total of 100 ground motion records – 44 far-field and 56 near-field, derived from 50 seismic events.

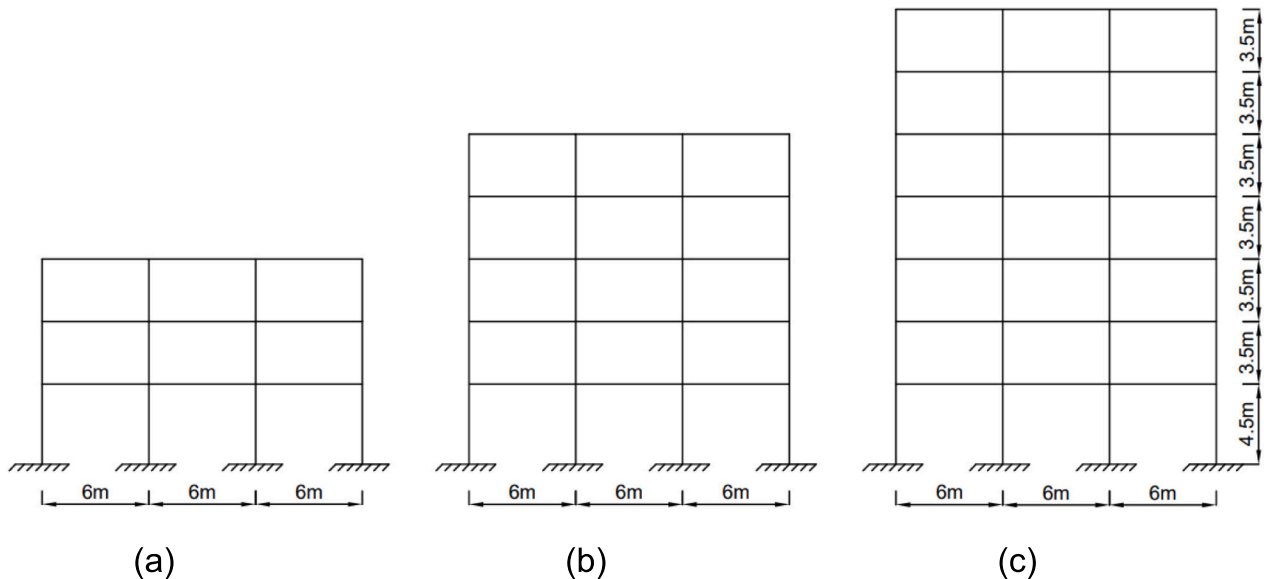


Fig. 1. Elevations of regular frames [27], from left to right: Set A, Set B, and Set C frames.

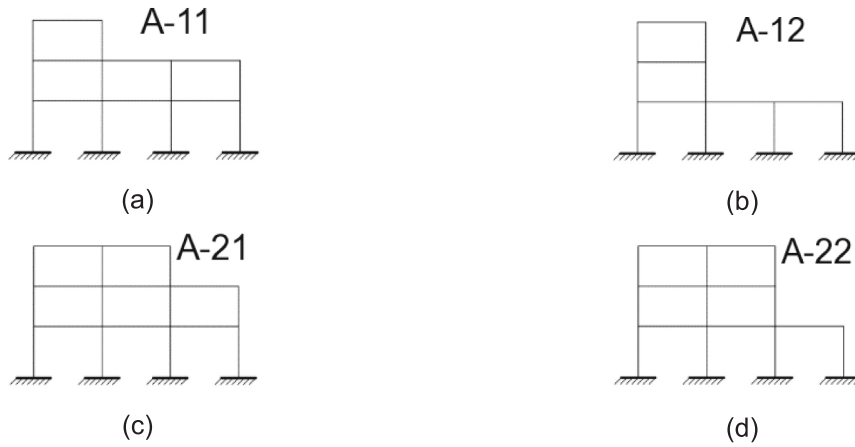


Fig. 2. Elevations of irregular Set A frames, considering various combinations of setbacks.

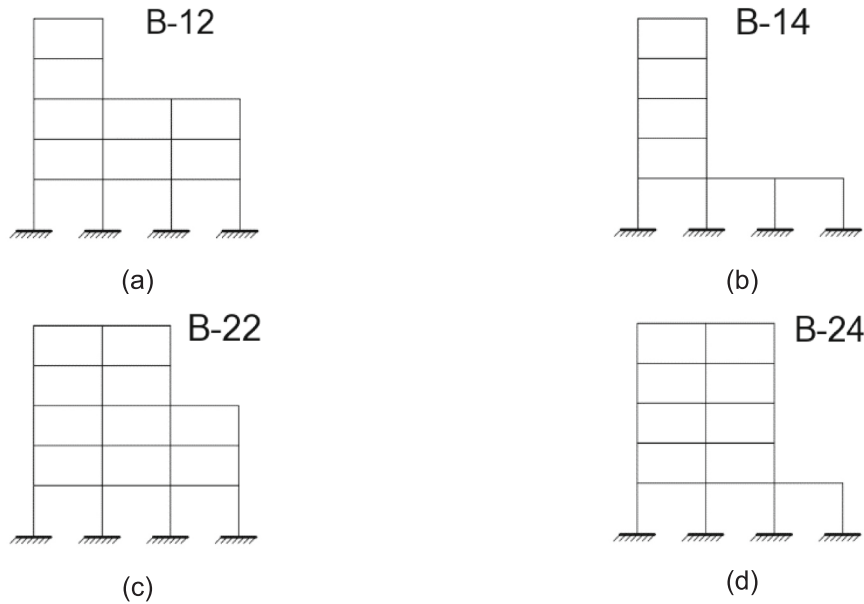


Fig. 3. Elevations of irregular Set B frames, considering various combinations of setbacks.

Notably, half of the near-field records exhibit velocity pulses. The records are classified in accordance with the definitions considered in FEMA-695 [52] which stipulates that the 12 km epicentre-to-site distance boundary between near-field and far-field records is generally consistent with the near-fault region of the maximum credible earthquake (MCE) design values maps in ASCE/SEI 7-05 [53]. Detailed information about the seismic events corresponding to the far-field and near-field records is provided in Tables 1 and 2, respectively. For the far-field records, the distance from the epicentre to the station ranges from 12 to 35 km, while the near-field records are obtained at stations closer to the epicentre, with a distance up to 12 km. Most of the selected earthquake events exhibit large magnitudes, ranging from about 6.5 to 8.0. The acceleration response spectra for all ground motions within each group of records, alongside their respective arithmetic mean spectrum, are plotted in Fig. 9. As Fig. 9 shows, the selected ground motion records exhibit rich spectral content in the low period range, and all records are within the same order of magnitude.

3. Seismic response assessment

Detailed nonlinear time-history analyses were firstly carried out for

the 188 irregular frames, using the set of 100 records, scaled at two intensity levels to represent elastic and inelastic primary structures. For the latter, a low strength reduction factor $R_{ps} = 2$ was considered, as the NSC response is typically examined for elastic or limited damage performance levels in the primary structure. The required scaling factor (SF) was computed from Eq. (1), as follows:

$$SF = R_{ps} \frac{V_1}{S_a(T_1) m \gamma_1} \quad (1)$$

Where R_{ps} is the inelasticity level of the primary structure ($R_{ps} = 1.2$), V_1 is the base shear corresponding to the first plastic hinge in the primary structure as obtained from pushover analyses, m is the total mass of the primary structure, and γ_1 is the effective mass participation for the first vibration mode of the primary structure.

In this section, the numerical results are used to assess the extent of applicability of the predictive models for inelastic displacement ratios and spectral accelerations of NSCs proposed previously by the authors [27] for regular frames.

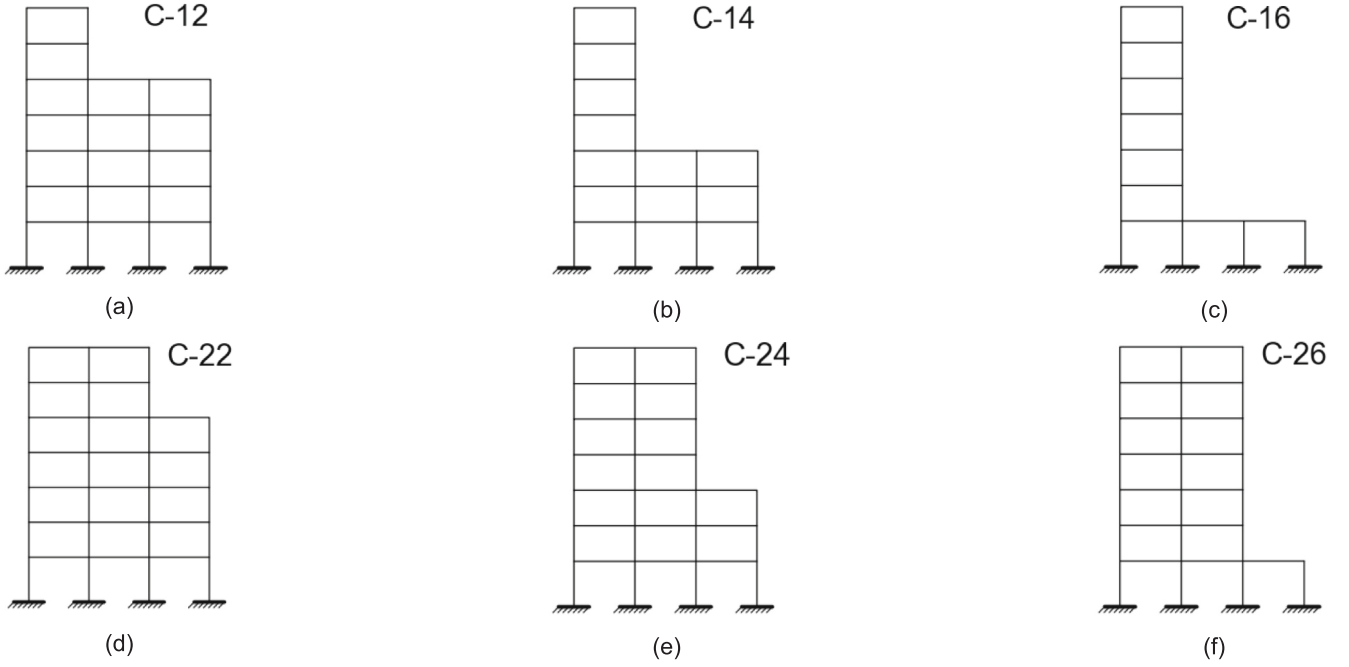


Fig. 4. Elevations of irregular Set C frames, considering various combinations of setbacks.

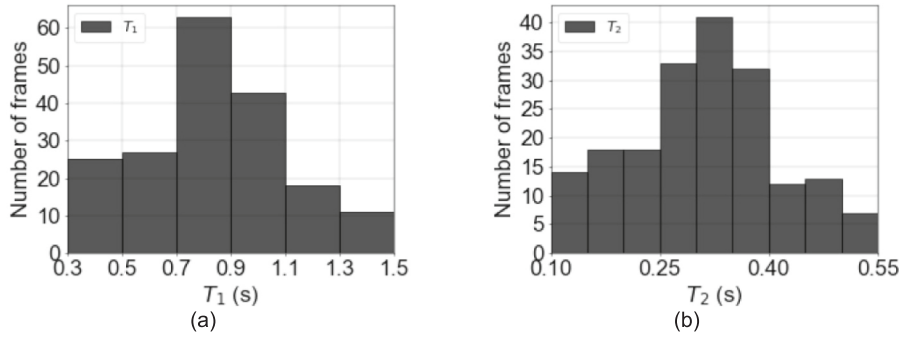


Fig. 5. Distribution of natural periods of irregular frames: (a) T_1 , and (b) T_2 .

3.1. Inelastic displacement ratios

The displacement response was evaluated through the inelastic displacement ratio (C_R) which is the ratio of the inelastic displacement of the NSC to its corresponding elastic displacement. Predictive relationships for the C_R of NSCs were previously proposed by Ding et al. [27], based on detailed numerical analyses of 38 regular steel moment resisting frames subjected to 100 ground motion records, as given in Eq. (2).

$$C_R = C_{R,0} + C_{R,T_1} + C_{R,T_2} \quad (2)$$

where $C_{R,0}$ is the function describing the overall trend of C_R , while C_{R,T_1} and C_{R,T_2} determine the local minima around the first two natural periods of the primary structures. These are defined in Eqs. (3)–(7) below.

$$C_{R,0} = 1 + \frac{2R_{nsc} - 1}{\left| 8 \left(\frac{T_{an}}{T_1} \right)^{1.1} - 1.0 \right|} \quad (3)$$

$$C_{R,T_1} = \frac{0.36R_{nsc} - 0.46}{\sqrt{2\pi} \cdot \sigma_{T_1}} e^{-\frac{\left(\frac{T_{an}}{T_1} - 1.0 \right)^2}{2\sigma_{T_1}^2}} \quad (4)$$

$$\sigma_{T_1} = 0.059R_{nsc} + 0.05 \quad (5)$$

$$C_{R,T_2} = \frac{-0.05R_{nsc} + 0.382}{\sqrt{2\pi} \cdot \sigma_{T_2}} e^{-\frac{\left(\frac{T_{an}}{T_1} - \frac{T_2}{T_1} \right)^2}{2\sigma_{T_2}^2}} \quad (6)$$

$$\sigma_{T_2} = -0.059 \ln(R_{nsc}) + 0.12 \quad (7)$$

where T_{an} is the NSC period; T_1 and T_2 are the fundamental and second-mode period, respectively, of the primary structure; and R_{nsc} is the inelastic behaviour factor (force reduction factor) of NSCs, varying from 1 to 6.

The above predictive relationship for C_R is applicable to NSCs attached to elastic regular primary structures. If the NSCs are attached to inelastic regular primary structures, Eq. (8) should be considered, as follows:

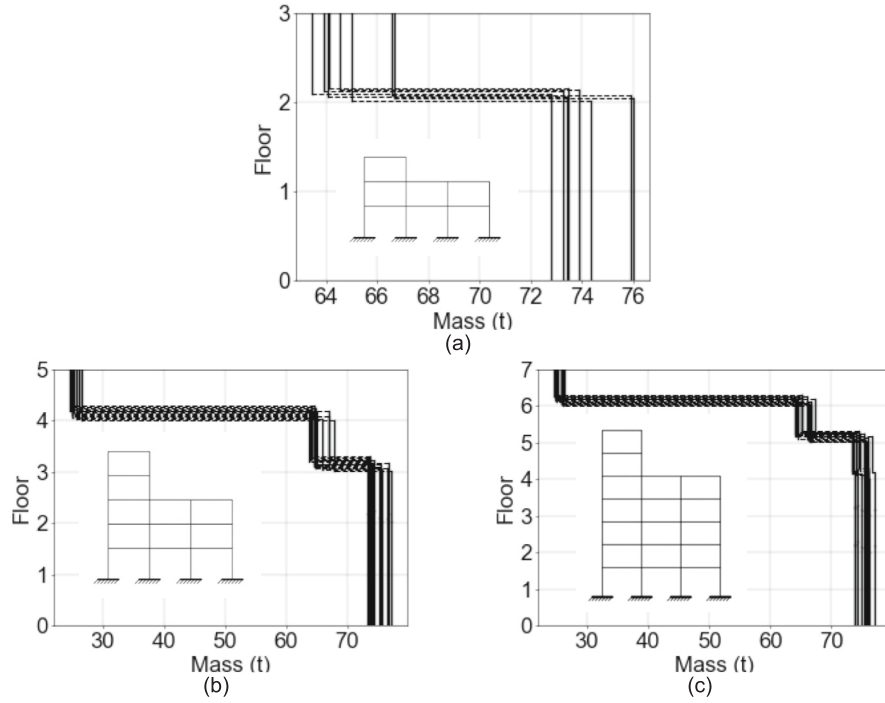


Fig. 6. Mass distribution over height for: (a) 8 Set A frames with one-span one-storey setback, (b) 15 Set B frames with one-span two-storey setback, (c) 16 Set C frames with one-span two-storey setback.

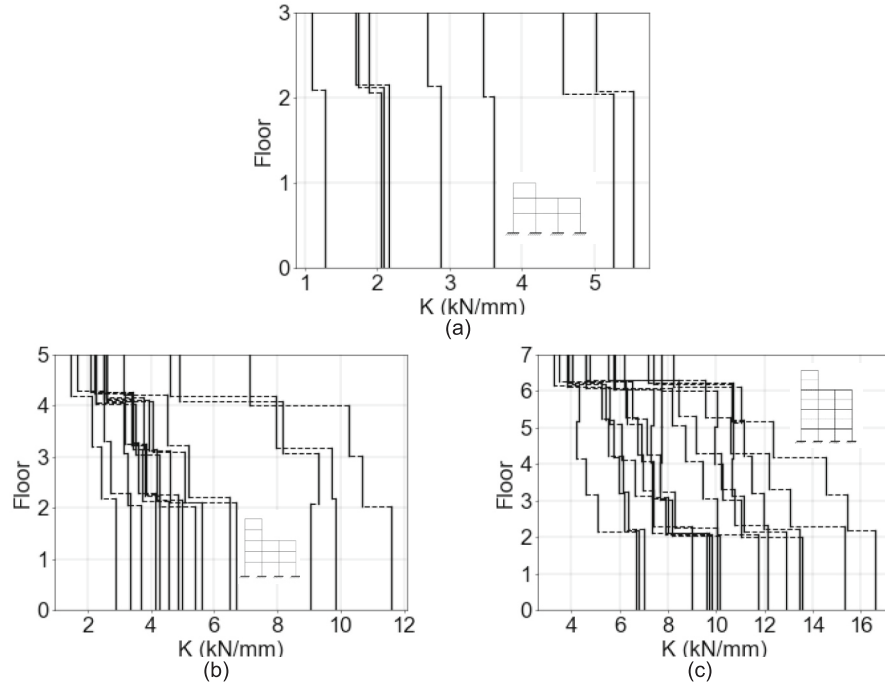


Fig. 7. Stiffness distribution over height for: (a) 8 Set A frames with one-span one-storey setback, (b) 15 Set B frames with one-span two-storey setback, (c) 16 Set C frames with one-span two-storey setback.

$$C_{Rps} = C_R \left/ \left[\left(\frac{-30 \left(\frac{T_{am}}{T_1} \right)^2 + 20 \frac{T_{am}}{T_1} - 2.3}{45 \left(\frac{T_{am}}{T_1} \right)^4 + 6} + 1.0 \right) \bullet (-0.05R_{nsc} + 1.32) \right]^{R_{ps}-1} \right. \quad (8)$$

In Fig. 10, the average C_R values for NSCs attached to all the irregular

frames and for all ground motion records are plotted against the NSC period, normalised by the first-mode period of the primary structures, along with the corresponding predictions based on the regular frame prediction model [27]. As shown in the figure, the regular frames model [27], which has been demonstrated to represent faithfully the response of NSCs in regular frames, also offers close predictions for NSCs mounted on irregular structures within the lower period range and accurately

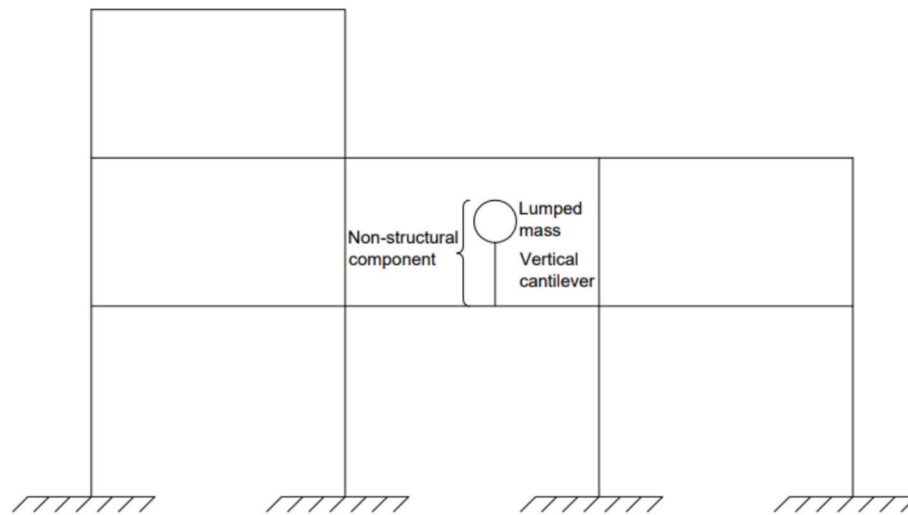


Fig. 8. Representation of non-structural component, modelled as a vertical cantilever on the floor of the primary structure.

Table 1

Description of far-field seismic events.

Name of event	Year	Magnitude (M)	Distance (km)	Soil type
San Fernando, LA	1971	6.6	25.9	D
Friuli, Italy	1976	6.5	15.8	C
Imperial Valley, Delta	1979	6.5	22.5	D
Imperial Valley, El Centro	1979	6.5	13.5	D
Superstition Hills, El Centro	1987	6.5	18.5	D
Superstition Hills, Poe Road	1987	6.5	11.7	D
Loma, Prieta	1989	6.9	35.5	D
Loma, Prieta	1989	6.9	12.8	D
Manjil, Iran	1990	7.4	13.0	C
Landers, Coolwater	1992	7.3	20.0	D
Cape Mendocino, Rio Dell	1992	7	14.3	D
Landers, Yermo	1992	7.3	23.8	D
Northridge, Canyon Country	1994	6.7	12.0	D
Northridge, Beverly Hills	1994	6.7	17.2	D
Kobe, Japan	1995	6.9	25.2	C
Kobe, Japan	1995	6.9	28.5	D
Kocaeli, Turkey	1999	7.5	13.5	C
Duzce, Turkey	1999	7.1	12.4	D
Chi-Chi, Taiwan	1999	7.6	15.5	D
Kocaeli, Turkey	1999	7.5	15.4	D
Hector Mine, Hector	1999	7.1	12.0	C
Chi-Chi, Taiwan	1999	7.6	26.8	C

captures the critical local minimum. However, as the inelasticity in NSCs (i.e. R_{nsc}) increases, the accuracy significantly declines. In particular, for NSCs with lower levels of inelasticity (i.e. $R_{nsc} = 2$ to 4), the predictions compare reasonably well with the numerical results. As the inelasticity increases to $R_{nsc} = 5$ or above, the fidelity of predictions diminishes markedly.

Overall, it is evident that the models for regular frames [27] tend to provide a larger estimation for the C_R response when applied to irregular frames. It is worth noting that although the actual total displacement for NSCs in irregular frames is larger than that in regular frames, the increase in the inelastic displacement is significantly smaller than that of the elastic part [40–42]. Therefore, the C_R values in irregular frames tend to be smaller than those in regular frames. Whilst these models perform well for the regular structures that they were intended for, they clearly become less reliable for irregular frames necessitating further improvements to account for their specific characteristics. These limitations are addressed through the development of a unified predictive

Table 2

Description of near-field seismic events.

Name of event	Year	Magnitude (M)	Distance (km)	Soil type	Pulses
Gazli, U.S.S.R.	1976	6.8	5.5	C	Without
Imperial Valley-06	1979	6.5	4.0	D	Without
Imperial Valley-06	1979	6.5	8.4	D	Without
Imperial Valley, El Centro	1979	6.5	3.5	D	With
Imperial Valley, El Centro	1979	6.5	3.6	D	With
Irpinia, Italy	1980	6.9	10.8	B	With
Nahanni, Canada	1985	6.8	9.6	C	Without
Nahanni, Canada	1985	6.8	4.9	C	Without
Superstition Hills, Parachute	1987	6.5	3.5	D	With
Loma, Prieta	1989	6.9	10.7	C	Without
Loma, Prieta	1989	6.9	3.9	C	Without
Loma, Prieta	1989	6.9	8.5	C	With
Cape Mendocino	1992	7.0	7.0	C	Without
Erzican, Turkey	1992	6.7	4.4	D	With
Cape Mendocino	1992	7.0	8.2	C	With
Landers, Lucerne	1992	7.3	3.7	C	With
Northridge, LA	1994	6.7	8.4	C	Without
Northridge	1994	6.7	12.1	D	Without
Northridge, Sylmar	1994	6.7	5.3	C	With
Northridge, Rinaldi	1994	6.7	6.5	D	With
Kocaeli, Turkey	1999	7.5	5.3	D	Without
Chi-Chi, Taiwan	1999	7.6	6.5	C	Without
Chi-Chi, Taiwan	1999	7.6	11.2	C	Without
Kocaeli, Turkey	1999	7.5	7.4	B	With
Chi-Chi, Taiwan	1999	7.6	6.7	D	With
Chi-Chi, Taiwan	1999	7.6	7.7	C	With
Duzce, Turkey	1999	7.1	6.6	D	With
Denali, Alaska	2002	7.9	8.9	C	Without

model in Section 4.1.

3.2. Acceleration response

As discussed before, the limitations of the current Eurocode 8 provisions [29] in estimating the acceleration response of NSCs have been highlighted in previous studies, including that by the authors [27]. It has also been shown that although several of these shortcomings have been addressed in the second-generation draft of Eurocode 8 [36], further improvements were still necessary, including the following: (i) significant underestimation of spectral acceleration in the range between the first two natural periods of the primary structure, (ii) inaccurate

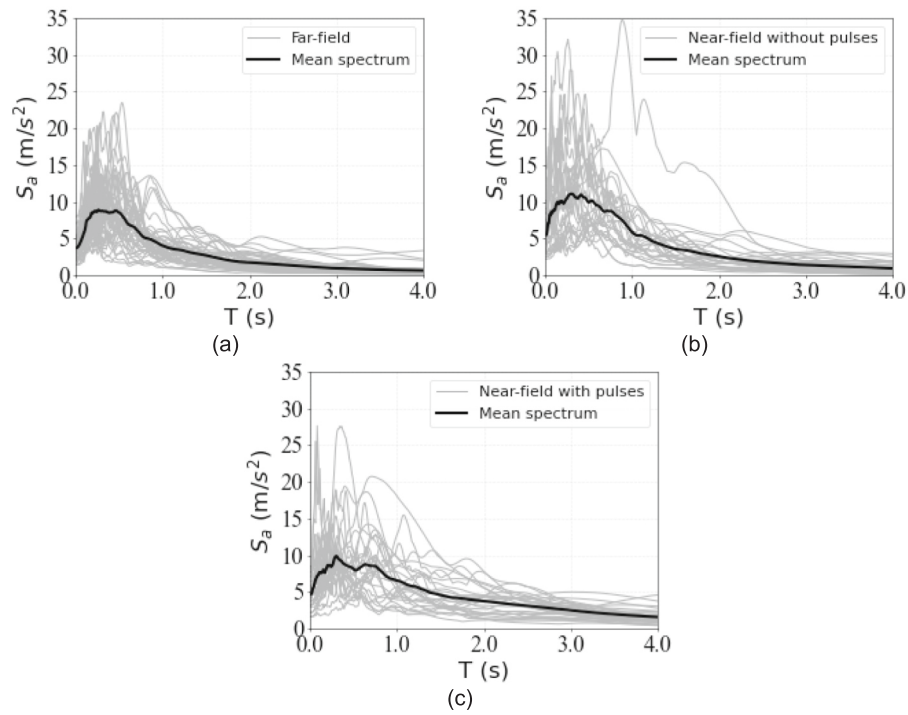


Fig. 9. Unscaled spectral acceleration and arithmetic mean spectrum for each type of records: (a) far-field records, (b) near-field records without velocity pulses, (c) near-field records with velocity pulses.

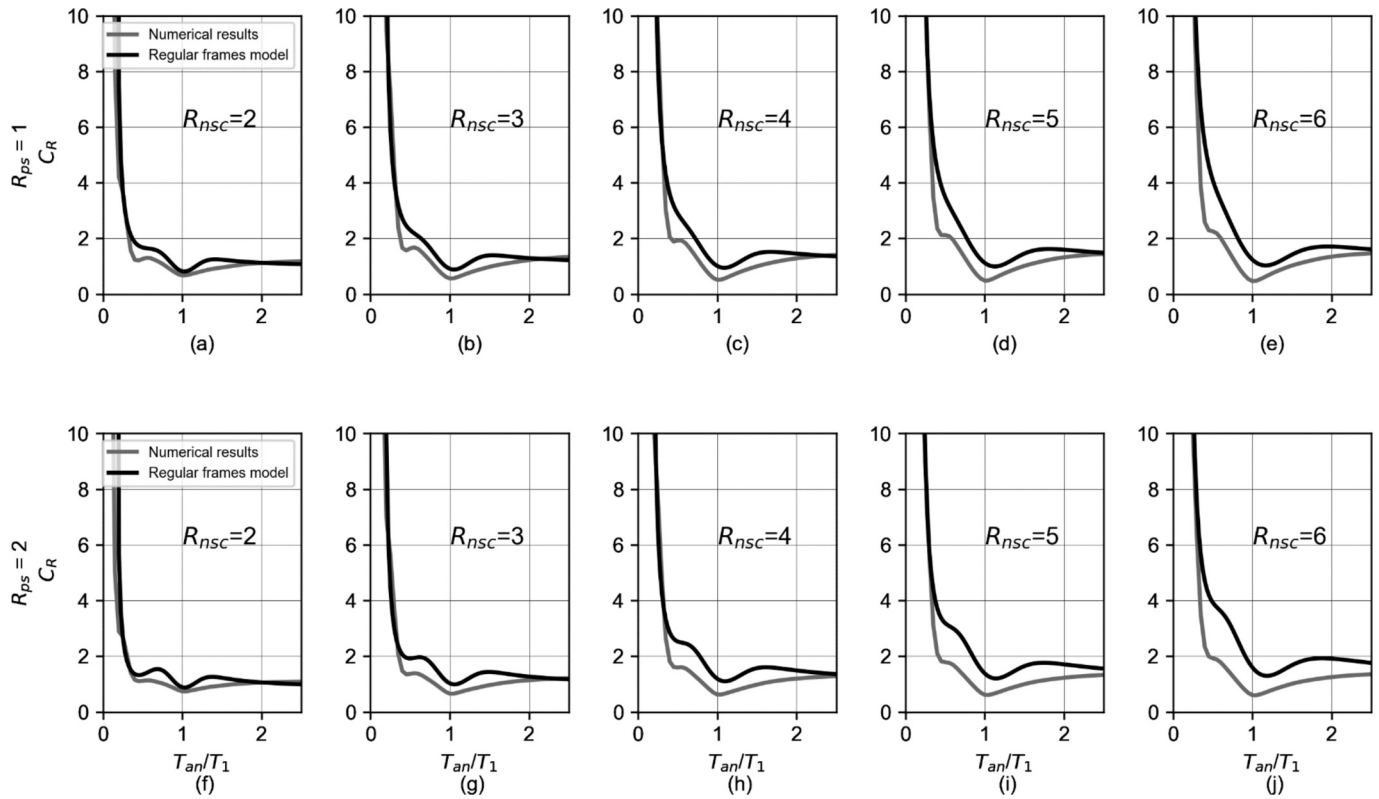


Fig. 10. C_R values from numerical results and regular frame models [27] against T_{an}/T_1 , for all irregular frames based on average response for all types of ground motion records. Each row corresponds to a level of inelasticity of the primary structure, and each column corresponds to an inelasticity level of the NSC.

predictions for NSCs in inelastic primary structures, and (iii) oversimplified linear approximation of the effects of inelasticity in NSCs. To address these deficiencies, in a previous study by the authors on NSCs

mounted in regular frames [27], modifications were introduced based on the results of detailed nonlinear time-history analyses. These modifications are given in Eq. (9) below, alongside the predictive equations

for NSCs in the draft second-generation Eurocode 8 as defined in Eq. (10).

$$S_{an,p} = S_{an,code} \bullet AF \bullet AF_{Rps} \bullet AF_{Rnsc} \quad (9)$$

$$S_{an,code,ij} = \frac{\Gamma_i \phi_{ij}}{\left| \left(\frac{T_{an}}{T_{p,i}} \right)^2 - 1 \right|} \sqrt{\left(\frac{S_{ep,i}}{R_{ps}} \right)^2 + \left[\left(\frac{T_{an}}{T_{p,i}} \right)^2 S_{ean} \right]^2} \quad (10)$$

where $S_{an,p}$ is the spectral acceleration response of NSCs from the regular frames model [27]; $S_{an,code}$ is the spectral acceleration response of NSCs derived from the draft second generation Eurocode 8 [36]; $S_{ep,i}$ is the spectral acceleration for the i^{th} vibration mode of the primary structure according to the input elastic design spectrum, while S_{ean} is that for the fundamental mode of the NSC according to the same input elastic design spectrum; $S_{an,code,ij}$ is the spectral acceleration of the NSC for the i^{th} vibration mode at the j^{th} floor, noting that the $S_{an,code}$ is derived by combining $S_{an,code,ij}$ through the square root of the sum of squares method; T_{an} is the fundamental period of the NSC, while $T_{p,i}$ is the i^{th} period of the primary structure; R_{ps} is the structural behaviour factor of the primary structure; Γ_i is the modal participation factor of the i^{th} vibration mode of the primary structure; Φ_{ij} is the i^{th} mode shape value at the j^{th} floor of the primary structure; AF is the overall amplification factor, as defined from Eq. (11); AF_{Rstr} is the amplification factor accounting for the influence of inelasticity of the primary structure, as defined from Eq. (12); AF_{Rnsc} is the amplification factor accounting for the influence of inelasticity of the NSC, as defined from Eq. (13).

$$AF = \frac{-1560 \left(\frac{T_{an}}{T_1} \right) + 1072}{6834 \left(\frac{T_{an}}{T_1} \right)^4 - 9952 \left(\frac{T_{an}}{T_1} \right)^2 + 4434} + 1.0 \quad (11)$$

$$AF_{Rstr} = \left[\frac{791 \left(\frac{T_{an}}{T_2} \right)^2 - 328 \left(\frac{T_{an}}{T_2} \right) + 46}{1664 \left(\frac{T_{an}}{T_2} \right)^4 + 129} + 1.0 \right]^{R_{str} - 1} \quad (12)$$

$$AF_{Rnsc} = \begin{cases} 1 & R_{nsc} = 1 \\ \left(0.1 \left(\frac{T_{an}}{T_1} \right) - 0.02 \right) R_{nsc} + \left(-0.1 \left(\frac{T_{an}}{T_1} \right) + 1.0 \right) & R_{nsc} \neq 1 \end{cases} \quad (13)$$

To assess the applicability of the current [29] and second generation [36] Eurocode 8 provisions, as well as the predictive relationships for regular frames [27], in estimating the acceleration response of NSCs in irregular structures, comparative evaluations are presented in Figs. 11 and 12. In Fig. 11, the mean acceleration response of the examined

NSCs, based on all 188 irregular frames and 100 ground motion records, is presented against the corresponding predictions based on the Eurocode 8 procedures, for elastic NSCs and both elastic and inelastic primary frames. Although the predictions of the second generation Eurocode 8 [36] are superior to those of the current Eurocode 8 [29], there are still notable differences compared to the numerical results, particularly in the presence of inelasticity in the primary frames. In Fig. 12, the mean numerical results from the NSCs mounted on the 188 irregular frames examined herein and subjected to the set of 100 ground motion records, are compared against the predictions based on Eq. (9), considering various levels of inelasticity in NSCs, and both elastic and inelastic primary frames. It is evident that significant differences still exist between the numerical results for irregular frames and the prediction relationships, which were derived for regular frames, with a notable underestimation of the response at resonance peaks.

3.3. Statistical tests

To assess the influence of the presence of frame irregularities on the seismic response of NSCs, a series of hypothesis tests is conducted herein. A null hypothesis is considered to stipulate that the presence of irregularities does not influence the seismic response of NSCs. In this case, it is implied that the mean value of the seismic response in the case of irregular structures, M_{irre} , across the set of 100 ground motions, is equivalent to that of the regular structures, M_{re} , under identical conditions, as indicated in Eq. (14).

$$H_{null} : M_{irre} = M_{re} \quad (14)$$

To test this hypothesis, the 'Z-test' is employed to determine the presence of a statistically significant difference between the two independent samples, each assumed to follow a log-normal distribution [54]. To decide on the acceptance or rejection of the null hypothesis, a conventional significance level of $\alpha = 5\%$ is applied, representing the risk of incorrectly rejecting the hypothesis. Within the context of a normal distribution, a significance level of $\alpha = 5\%$, or 0.025 in each tail, translates to a Z-value of 1.96. Thus, if the calculated Z-score exceeds an absolute value of 1.96, the null hypothesis is rejected, indicating a significant influence of the presence of irregularity on the seismic response of NSCs at the 5% significance level.

Prior to conducting the Z-test, it is imperative to confirm that the data samples from both the regular and irregular frames conform to a log-normal distribution. Given the identified critical issue with the modified equations, namely their inaccurate predictions for NSCs at the fundamental periods, this verification focuses on the seismic response at the fundamental periods across different structural frames, both regular and irregular. Both the inelastic displacement ratio (C_R) and the seismic acceleration response (S_{an}) of NSCs at the fundamental period are selected as the basis for the distribution verification across 80 NSC oscillators whose natural period varies from 0.05s to 4.0s. The corre-

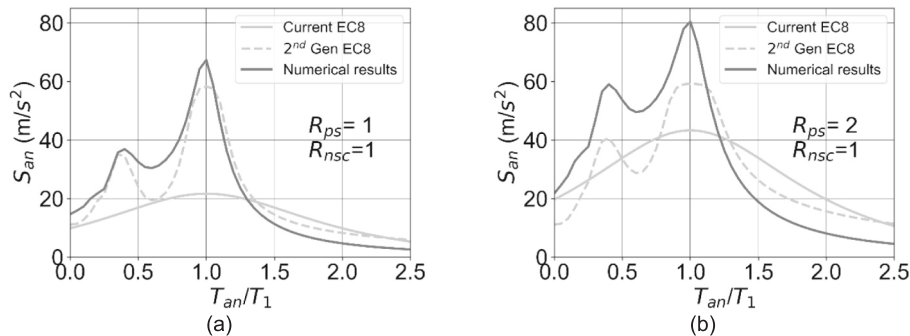


Fig. 11. Spectral acceleration of elastic NSCs ($R_{nsc} = 1$) from numerical results, current Eurocode 8 [29], and 2nd generation Eurocode 8 [36], based on average response of all irregular frames, subjected to all ground motions: (a) $R_{ps} = 1$, and (b) $R_{ps} = 2$.

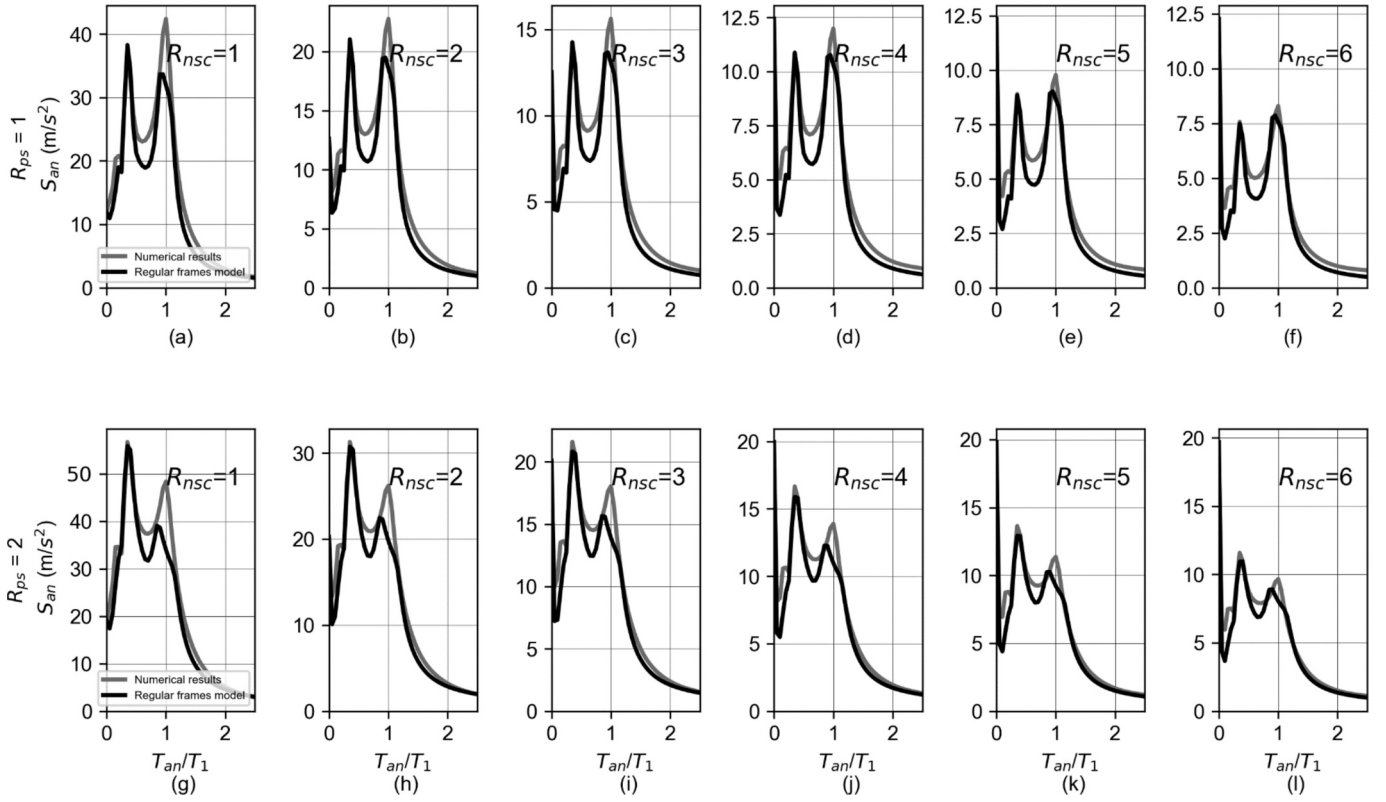


Fig. 12. Spectral acceleration of numerical results and predictions from regular frame models [27] against $\frac{T_{an}}{T_1}$, considering all types of irregular frames (including Sets A, B and C frames) based on average response for all types of ground motion. Each row corresponds to a level of inelasticity of the primary structure, and each column corresponds to an inelasticity level of the NSC.

sponding Quantile-Quantile (Q-Q) plots for C_R and S_{an} are presented in Figs. 13 and 14, respectively, where scatter data samples are plotted together with a solid theoretical line and bounded by two 95% confidence interval lines. If the scatter data samples align precisely with the theoretical line, it indicates conformance to a standard normal distribution. Nonetheless, samples are also considered to conform if the majority fall within the 95% confidence interval lines. Given that only a few points lie outside these interval lines in the illustrated figures (respectively 4.0% and 3.7% data points in Figs. 13 (a) and (b), and 6.8% and 7.4% data points in Figs. 14 (a) and (b)), it is reasonable to conclude that the data samples from both the regular and irregular structures largely follow a normal distribution.

Upon confirming the distributional characteristics of the data samples, Z-scores are computed, and the mean, along with the mean plus or minus one standard deviation curves, are shown in Figs. 15 (a) and (b) for regular and irregular frames, respectively. As shown in the figures,

nearly all Z-scores exceed the critical value of ± 1.96 , notably with the Z-score for the response at the fundamental period $\frac{T_{an}}{T_1} = 1.0$ reaching up to -15 and 15 , respectively. This indicates, with 95% confidence, that the frame irregularities significantly influence the seismic response of the NSCs.

4. Unified prediction models for regular and irregular frames

4.1. Prediction of inelastic displacement ratios

As discussed in Section 3.1 above, the prediction equations proposed for regular structures [27] cannot capture accurately the C_R response for irregular frames, particularly in terms of the local minimum around the fundamental period of the primary structure. These issues are addressed in this section in order to improve the capability of the model and enable its application for NSCs in both regular and irregular structures.

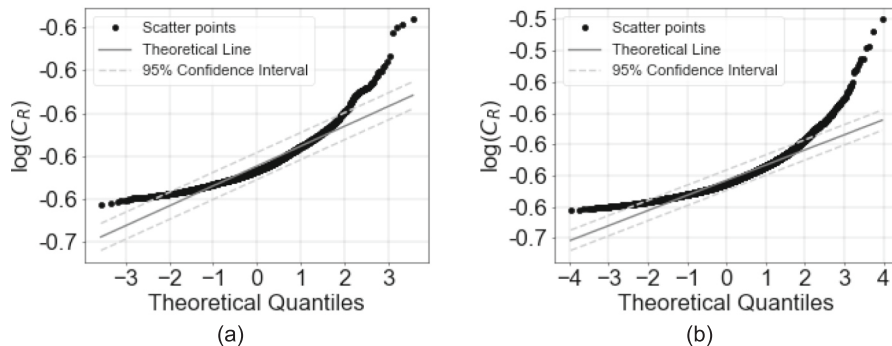


Fig. 13. Quantile-Quantile plots of C_R at T_1 for (a) all 39 regular frames, and (b) all 188 irregular frames.

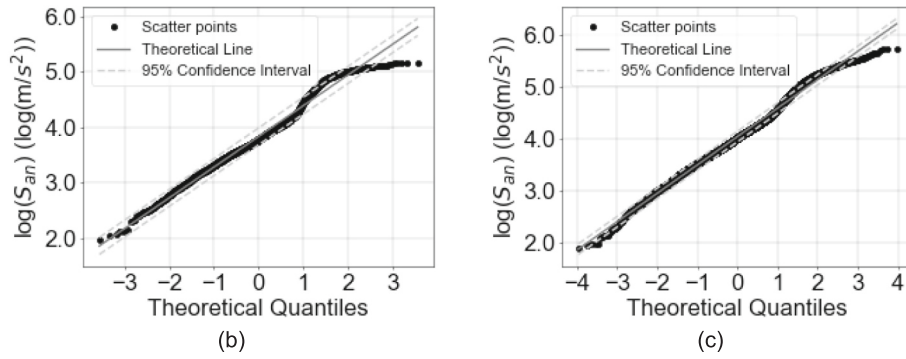


Fig. 14. Quantile-Quantile plots of S_{an} at T_1 for (a) all 39 regular frames, and (b) all 188 irregular frames.

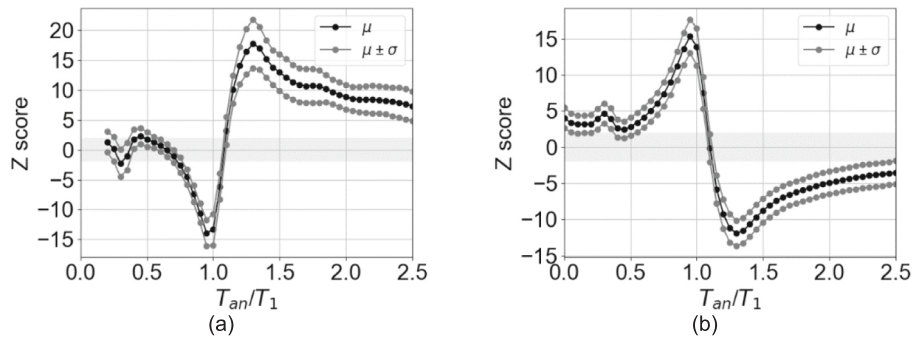


Fig. 15. Z scores for: (a) C_R , and (b) S_{an} , considering the differences between all regular and irregular frames based on the average response using all types of ground motions.

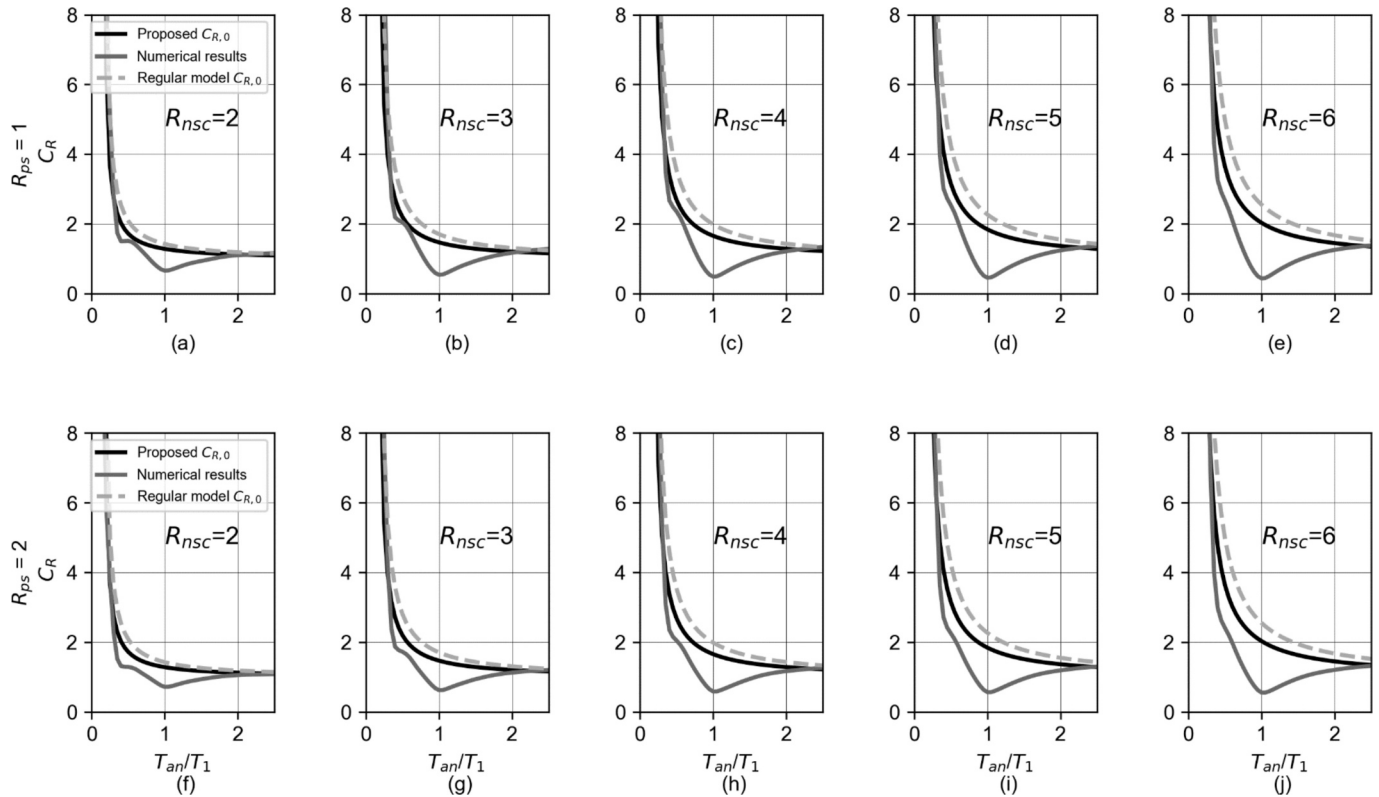


Fig. 16. C_R values against $\frac{T_{an}}{T_1}$ from numerical results and predictions from the regular frames model [27] (for only $C_{R,0}$) considering average results from all irregular frames and all ground motion records. Each row corresponds to a level of inelasticity of the primary structure, and each column corresponds to an inelasticity level of the NSC.

The first issue is related to the overestimation of C_R through the first term in Eq. (2), $C_{R,0}$, in the regular frames model [27], which is defined from Eq. (3). Fig. 16 shows that the $C_{R,0}$ curves based on the regular frames model [27] significantly diverge from those obtained from the numerical simulations for irregular structures. For regular frames, the $C_{R,0}$ curves were shown [27] to closely represent the results, being much closer to the local maximum between the first two natural periods of the primary structures. To rectify this inaccuracy in the case of irregular structures, a modification is proposed which involves adjusting the formulation of $C_{R,0}$ by introducing an additional coefficient λ_0 , as presented in Eqs. (15) and (16).

$$C_{R,0} = 1 + \frac{2R_{nsc} - 1}{\lambda_0 \left| 8 \left(\frac{T_{gm}}{T_1} \right)^{1.1} - 1.0 \right|} \quad (15)$$

$$\lambda_0 = e^{(0.004R_{nsc} - 0.007) \cdot Q} \quad (16)$$

where Q is a quantitative coefficient reflecting the structural irregularity and is defined according to Eq. (17), considering the combination of the dimensionless vertical setback ratio and the square of the fundamental period of the primary structure. It characterises the irregularity of the primary frame, and reduces with decreasing irregularity level, ultimately becoming null for regular frames. It is therefore expected that the coefficient λ_0 should converge to unity for regular frames (i.e. for $Q = 0$).

$$Q = \left(\frac{L - L_1}{L} \right) \cdot T_1^2 \quad (17)$$

where T_1 is the fundamental natural period of the irregular primary structure; L is the total length (i.e. width) of the frame at ground level, and L_1 is the length (width) of the setback at the top of the frame.

As shown in Fig. 17, the modified $C_{R,0}$ curve based on Eqs. (15)–(16) is in close agreement with that obtained from the numerical simulations. In addition, Figs. 17 (a) and (b) show the fitting for determining λ_0 based on the numerical results of the 188 irregular structures including different levels of inelasticity in the primary structures ($R_{ps} = 1$ and $R_{ps} = 2$), while Figs. 18 (a) to (e) collectively show the fitting for NSCs with varying inelasticity ($R_{nsc} = 2$ to $R_{nsc} = 6$).

As shown in Fig. 17, the modification factor λ_0 is not notably affected by the inelasticity in the primary structure (R_{ps}). Accordingly, in the following parts, the regression assessments are performed based on the seismic response of NSCs from both elastic and inelastic primary structures. By examining the distribution of the scatter points for λ_0 , it can be observed that the coefficient λ_0 exhibits a nonlinear relationship with the fundamental period T_1 of the primary structure. Hence, the regression equation for λ_0 is expressed through an exponential function, as indicated before in Eq. (16). As shown in Figs. 17 and 18, the average scatter points are displayed alongside the 95% confidence bands and

95% prediction bands, with confidence bands tightly surrounding the regression curves. The coefficients of determination are $R^2 = 0.7$ in these figures, but capture well the distribution trend of scatter points.

The second issue to address is related to C_{R,T_1} and C_{R,T_2} in Eq. (2) for the regular frames model [27]. Observations from the Z-test in Section 3.3 indicate that the inelastic displacement ratios of NSCs are highly influenced by the irregularity of the primary structure. It was shown before that the regular frames model [27] is unable to capture the peak values of the local minima occurring around the fundamental and second-mode periods of the primary irregular structures. A viable approach to addressing this is to account for the specific characteristics of the irregular frames, and subsequently make appropriate adjustments to C_{R,T_1} and C_{R,T_2} . Hence, additional modification factors, λ_1 and λ_2 , are introduced, as shown in Eqs. (18) and (19), respectively.

$$C_{R,T_1} = \lambda_1 \frac{(0.36R_{nsc} - 0.46)}{\sqrt{2\pi} \cdot \sigma_{T_1}} e^{-\frac{\left(\frac{T_{gm}}{T_1} - 1.0 \right)^2}{2\sigma_{T_1}^2}} \quad (18)$$

$$C_{R,T_2} = \lambda_2 \frac{(-0.05R_{nsc} + 0.382)}{\sqrt{2\pi} \cdot \sigma_{T_2}} e^{-\frac{\left(\frac{T_{gm}}{T_1} - \frac{T_2}{T_1} \right)^2}{2\sigma_{T_2}^2}} \quad (19)$$

Using a similar procedure as before, and with reference to Fig. 19, λ_1 is found to exhibit a strong correlation with the quantitative coefficient Q . In contrast to λ_0 , λ_1 shows a linear relationship, as stipulated in Eq. (20) below.

$$\lambda_1 = -0.0036 \cdot Q + 1.00 \quad (20)$$

In Fig. 19, the mean of the 188 scatter points for λ_1 from the numerical results of the irregular structures is depicted alongside the 95% confidence interval and prediction bands. The linear regression models, characterised by a gradual decrease, are in close agreement with the scatter points from the numerical results, underscoring their reliability with high coefficients of determination of $R^2 = 0.8$.

The coefficient λ_2 exhibits the same distribution trends against the quantitative coefficient Q for NSCs attached to either elastic or inelastic irregular primary structures and for different levels of NSC inelasticity ($R_{nsc} = 2$ to 6). This means that the adjustments for the second reduction function are not notably influenced by the inelasticity in the NSCs or the primary structures. Fig. 20 depicts λ_2 against Q for all the results, with the relationship for λ_2 given in Eq. (21).

$$\lambda_2 = \frac{1}{Q + 2.5} + 0.6 \quad (21)$$

Following the above modifications for the inverse and reduction functions, the adequacy of the unified proposed model, which employs the revised $C_{R,0}$, C_{R,T_1} , and C_{R,T_2} terms in Eqs. (3)–(7), is examined herein. Fig. 21 assesses the model accuracy across all 188 irregular frames. As

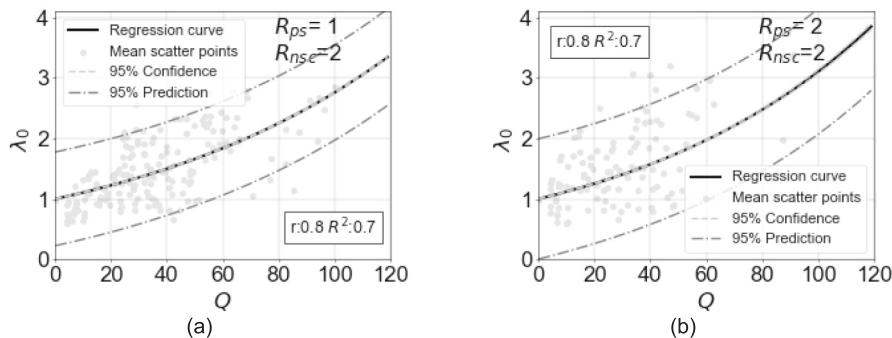


Fig. 17. Modification factor λ_0 against the coefficient Q for inelastic NSCs ($R_{nsc} = 2$) attached to primary structures with different levels of inelasticity: (a) $R_{ps} = 1$, and (b) $R_{ps} = 2$.

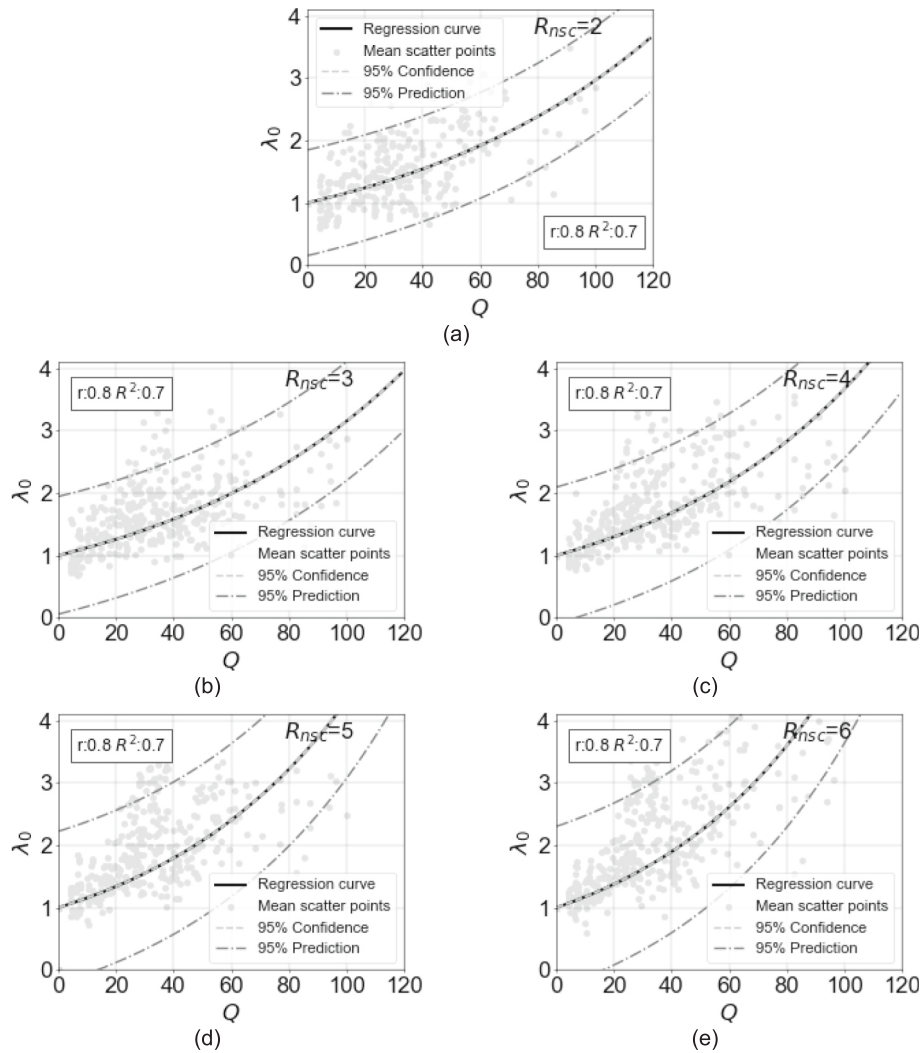


Fig. 18. Modification factor λ_0 against the coefficient Q for NSCs with different levels of inelasticity, (a) to (e): $R_{nsc} = 2$ to $R_{nsc} = 6$.

shown in the figure, the final model mitigates the two issues related to the overestimation around the two local minima associated with the first two natural periods of the primary structures. Furthermore, the C_R curves are now in close agreement with the numerical results across the full period range of the primary structures. While very minor discrepancies of up to 10 % between the predictions and numerical results exist, the overall precision of the model is markedly enhanced.

Based on the above modifications, the inelastic displacement ratio of NSCs attached to irregular or regular frames, can be estimated using the unified model proposed in this Section. The application of the model relies on the basic structural characteristics of the frames and the NSCs, and can be summarised as follows: (i) Firstly, the periods and inelastic behaviour factor of the NSC are determined; (ii) Subsequently, based on the first and second mode periods and inelasticity level of the primary structure, and also considering its irregularity (defined through the quantitative coefficient Q in Eq. (17)), the modification factors λ_0 , λ_1 , and λ_2 can be evaluated from Eqs. (16), (20), and (21), respectively; (iii) These are then applied to Eqs. (15), (18), and (19), respectively, to determine the inverse function $C_{R,0}$, and reduction functions C_{R,T_1} and C_{R,T_2} ; (iv) Finally, if the NSCs are attached to an elastic primary structure, their C_R values can be determined from Eq. (2), while if they are attached to an inelastic primary structure, Eq. (8) should be applied. Furthermore, for practical design purposes, the predictive equations of

C_R can be used to derive the inelastic displacement response of NSCs from the elastic response. Once the desired peak component acceleration of NSCs is determined, their elastic displacement response can be obtained by considering their force-deformation relationships. Subsequently, these elastic displacement responses can be used in conjunction with the predictive equations for C_R to determine the inelastic displacement response.

4.2. Prediction of spectral acceleration response

In the Z-tests presented in Section 3.3, it was shown that the frame irregularity influences the seismic acceleration response of NSCs. This, together with the comparative assessments carried out in Section 3.2, indicated that the regular frame model [27] for predicting the acceleration response in regular frames requires further improvement to ensure its accuracy when applied to irregular structures. Therefore, modifications are suggested below in order to develop a final unified model for predicting the spectral acceleration response of NSCs mounted within either regular or irregular multi-storey frames.

A modification factor β that accounts for irregularity is introduced herein, and the proposed unified model for the spectral acceleration of the NSCs is presented in Eq. (22) below.

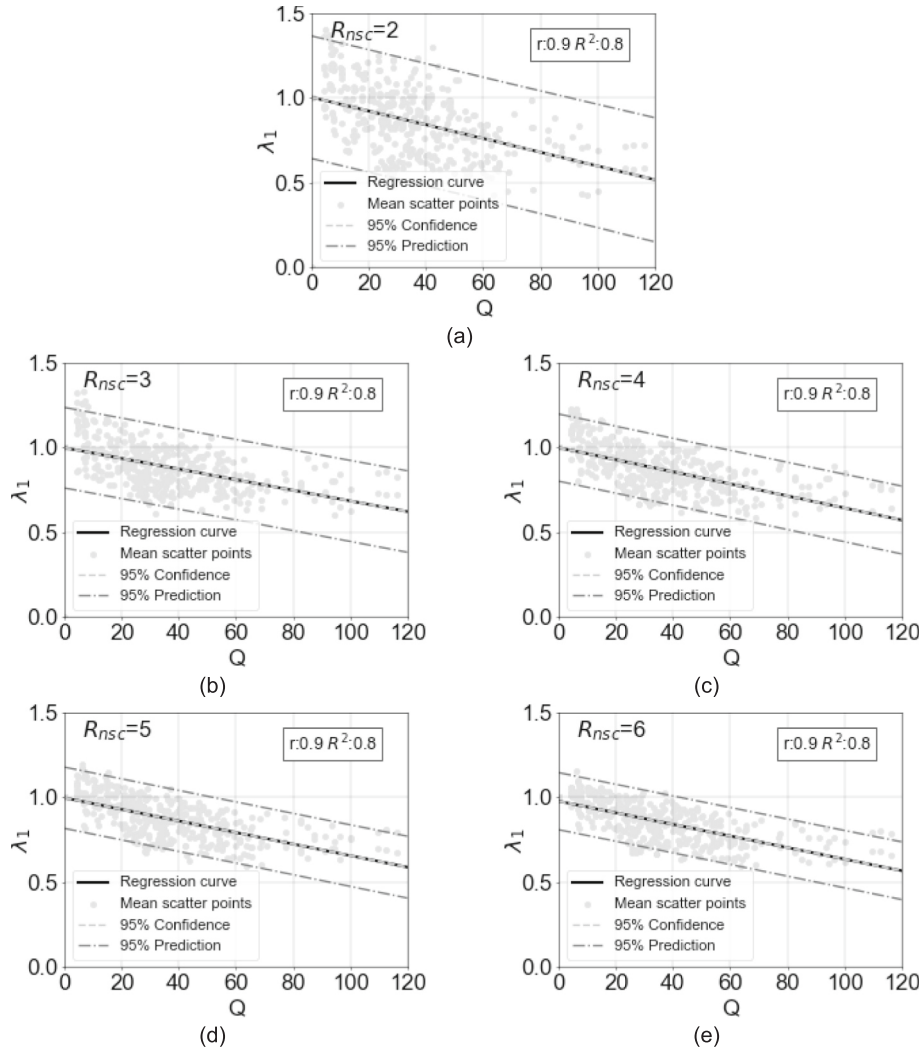


Fig. 19. Modification factor λ_1 against the coefficient Q for NSCs with different levels of inelasticity, (a) to (e): $R_{nsc} = 2$ to $R_{nsc} = 6$.

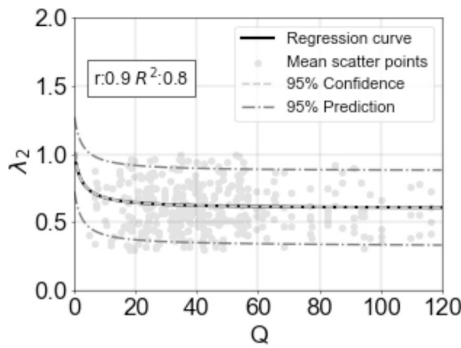


Fig. 20. Modification factor λ_2 against the coefficient Q for NSCs attached to elastic and inelastic primary structures, considering all NSC inelasticity levels ($R_{nsc} = 2$ to $R_{nsc} = 6$).

$$S_{an} = \beta \cdot \frac{\Gamma_i \phi_{ij}}{\left| \left(\frac{T_{an}}{T_{p,i}} \right)^2 - 1 \right|} \sqrt{\left(\frac{S_{ep,i}}{R_{ps}} \right)^2 + \left[\left(\frac{T_{an}}{T_{p,i}} \right)^2 S_{ean} \right]^2} \cdot AF \cdot AF_{R_{ps}} \cdot AF_{R_{nsc}} \quad (22)$$

In Figs. 22 (a) and (b), the modification factors for structural irregularity, β , are depicted for elastic NSCs ($R_{nsc} = 1$) mounted on elastic

($R_{ps} = 1$) and inelastic ($R_{ps} = 2$) primary structures, respectively. The regression analyses for these structures demonstrate robust predictions, with coefficients of determination R^2 of 0.7 for both cases, indicating a strong predictive accuracy. Furthermore, the regression curves fall within the 95% confidence intervals, and most of the scatter points are within the 95% prediction bands. Also, as the difference between the two cases is not significant, the spectral acceleration response of NSCs from both elastic and inelastic primary structures are considered together. Based on the regression, Eq. (23) is proposed to evaluate the modification factor β , accounting for the irregularity of the primary structures and the inelasticity of the NSCs.

$$\beta = \left(-\frac{1}{Q + 2.5} + 1.4 \right)^{[-0.07(R_{nsc} - 1) + 1.00]} \quad (23)$$

In Figs. 23 (a) to (e), the regression curves are depicted alongside two bounds: the 95% confidence interval lines and the 95% prediction lines. It can be observed that the regression analyses for all inelastic NSCs are robust.

The suggested modification factor, β , as defined in Eq. (23), can be incorporated into the predictive model for regular frames [27]. The accuracy of this adjustment for the seismic response of NSCs at the fundamental period is assessed through a comparative evaluation including the numerical results, the predictions derived from the second-generation Eurocode 8 draft [36], and the unified predictive model proposed in this paper. Fig. 24 shows the overall comparisons, based on

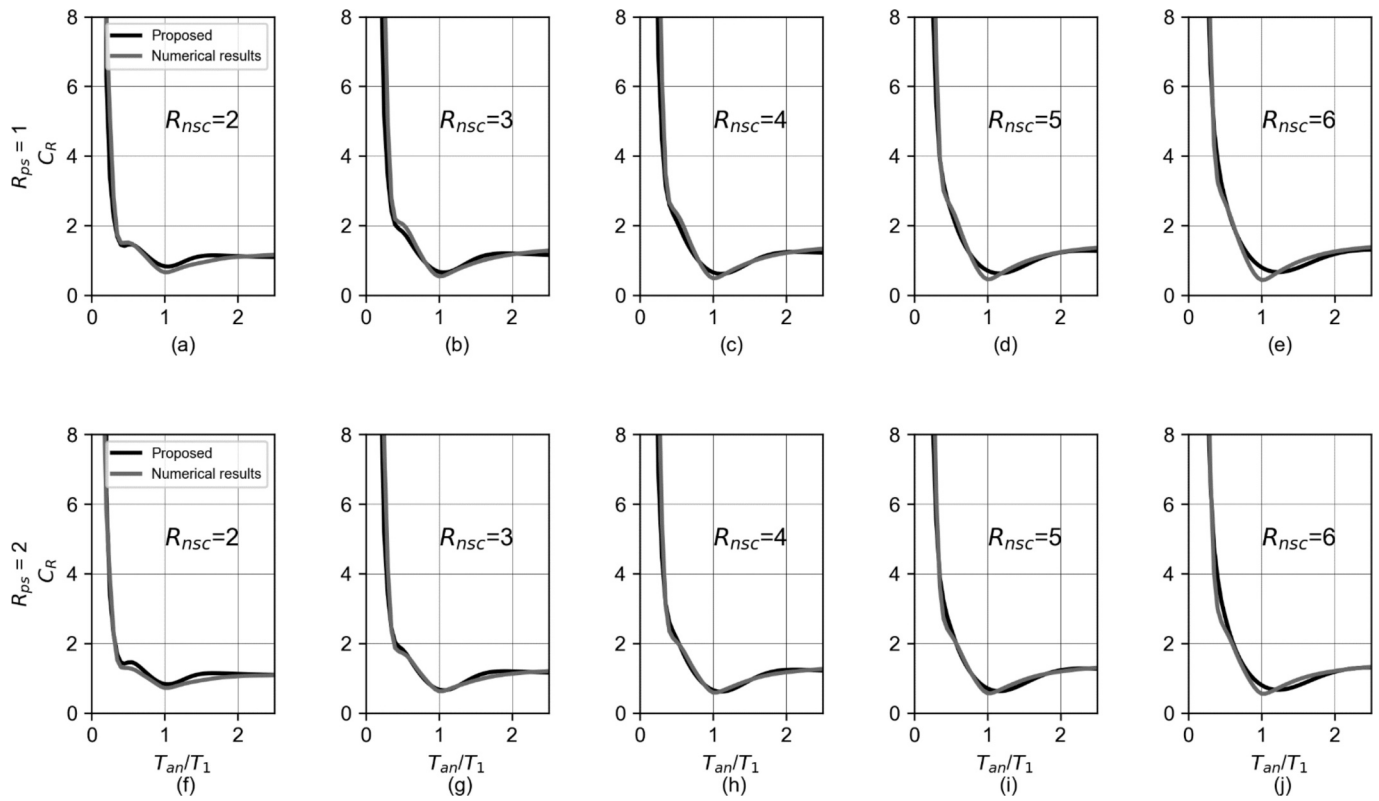


Fig. 21. C_R vs $\frac{T_{an}}{T_1}$ from numerical results and proposed unified models for all 188 irregular frames and all ground motion records. Each row corresponds to a level of inelasticity of the primary structure, and each column corresponds to an inelasticity level of the NSC.

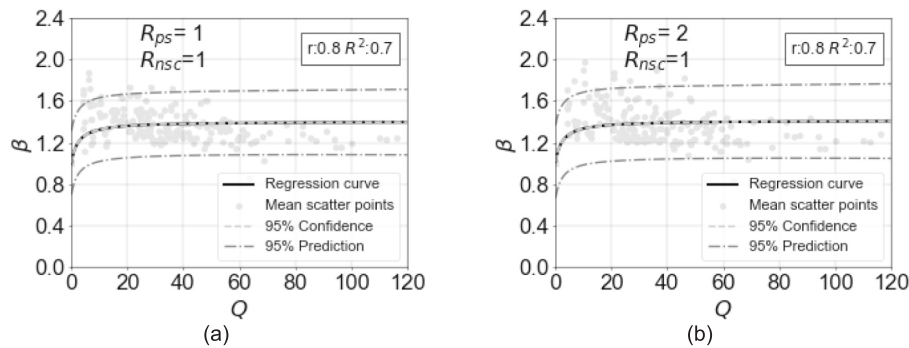


Fig. 22. Modification factor β against the coefficient Q for elastic NSCs ($R_{nsc} = 1$) mounted on all 188 irregular (a) elastic ($R_{ps} = 1$), and (b) inelastic ($R_{ps} = 2$) frames, based on average response using all ground motion records.

average results from the 188 irregular frames under 100 ground motion records. Evidently, the proposed model significantly improves the accuracy of the predictions from the second-generation Eurocode 8, following closely the numerical results from this study. The maximum difference between the proposed model and the numerical spectral accelerations is 8 %, and is observed at a period close to half the fundamental period of the primary structure.

Since the proposed unified acceleration model is based on the seismic response of NSCs at roof levels, it is interesting to evaluate its applicability to NSCs at other floor levels. To provide a detailed evaluation, the height-wise distribution of spectral acceleration of the NSCs, averaged over all 188 irregular frames and 100 ground motion records, and normalised with respect to the peak ground acceleration, is presented in Fig. 25, along with the distribution relationships derived from the current [29] and draft second generation Eurocode 8 [36]. As shown in the figure, while, for practical purposes, the relationship between

component maximum acceleration and structural height can be broadly considered as ‘approximately’ linear, it does exhibit nonlinearities, particularly from mid-height upwards. The proposed unified acceleration model effectively captures this distribution, closely following the numerical results at all floor levels throughout the height of the primary structure. It provides much closer estimates than the design provisions in the current Eurocode 8 [29], in terms of both values and distribution. By comparing the proposed unified acceleration model with the second generation Eurocode 8 [36], it becomes clear that the accuracy of the latter is limited to elastic ($R_{nsc} = 1$) NSCs attached to elastic ($R_{ps} = 1$) irregular primary structures. When either the NSCs or the primary structures undergo inelastic behaviour, its predictive accuracy reduces significantly. In contrast, the proposed unified acceleration model addresses these shortcomings, offering consistently close estimates of the numerical results. It is also worth noting that the NSCs at the roof level, which were the basis for the development of the proposed unified model,

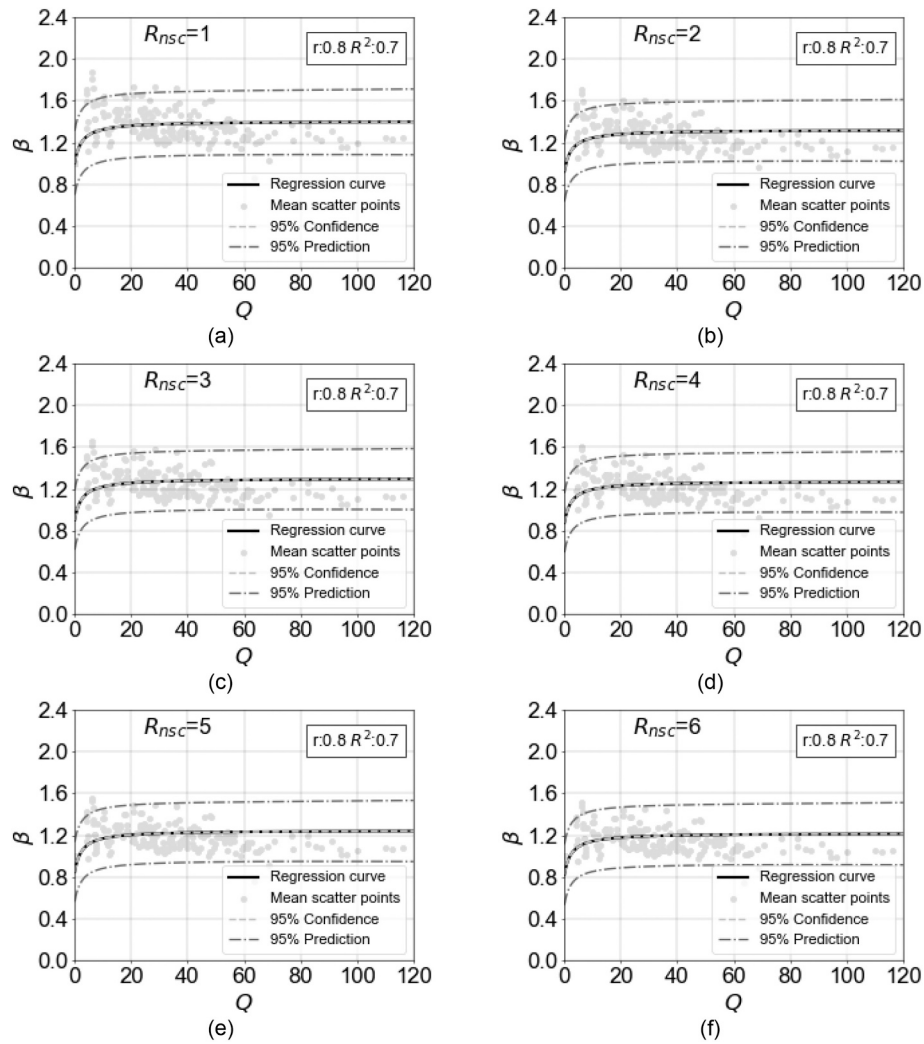


Fig. 23. Modification factor β against the coefficient Q for NSCs with different levels of inelasticity, (a) to (f): $R_{nsc} = 1$ to $R_{nsc} = 6$.

exhibit the most critical seismic acceleration responses. Based on these observations, the unified acceleration model can be applied to NSCs mounted on different floors of the primary structure, besides those at the roof level.

The procedure for using the proposed model for predicting the spectral acceleration of NSCs mounted in irregular frames can be summarised as follows: (i) The first step is to determine the key structural characteristics of both the NSCs and the primary structure, including their periods and inelastic behaviour factor; (ii) In addition, the dynamic properties of the primary structure (e.g. mode shapes and participation factors) should be obtained; (iii) Based on these, the spectral acceleration can be derived from Eq. (10); (iv) Subsequently, the modification factors AF , AF_{Rps} , AF_{Rnsc} and β can be computed from Eqs. (11)–(13) and Eq. (23), accounting for the influence of inelasticity in both the primary structures and NSCs, and any irregularity in the primary structure; (v) Finally, Eq. (22) can be used to determine the spectral acceleration.

5. Conclusions

This paper investigated the seismic response of non-structural ancillary components attached to vertically irregular frames, focusing on their inelastic displacement ratios and spectral accelerations. A series of ancillary components with varying periods and inelastic behaviour factors, mounted on 188 vertically irregular steel moment frames, were analysed. These secondary components were modelled as single-degree-

of-freedom systems fixed within the floors of the primary structures. The steel frames, with different elevation setbacks, were designed to cover a wide range of structural characteristics and were subjected to a set of 100 far-field and near-field ground motion records. The configurations, column and beam member sizes, and dynamic characteristics of the frames were presented in detail, and their vertical irregularities were affirmed based on European code provisions. The study was limited to planar frames with well-spaced dominant modes, hence did not account for spatial torsional effects nor for structures with closely-spaced periods of vibration.

The adequacy of existing prediction models for ancillary components mounted on regular frames was firstly evaluated. Detailed comparative assessments were carried out between the predictions from these models and the numerical results from the ancillary components mounted on the irregular frames examined in this study. The influence of irregularity on the seismic response of the ancillary components was assessed through statistical analyses. It was shown that the seismic displacement and acceleration response followed log-normal distributions, for both regular and irregular frames. Subsequently, a null hypothesis test indicated that the seismic displacement and acceleration responses of ancillary components were statistically significantly different between the regular and irregular frames. It was concluded therefore that existing models based on regular frames require various modifications to capture the effects of vertical irregularities on the ancillary component response.

Based on the results of the detailed numerical studies, modified

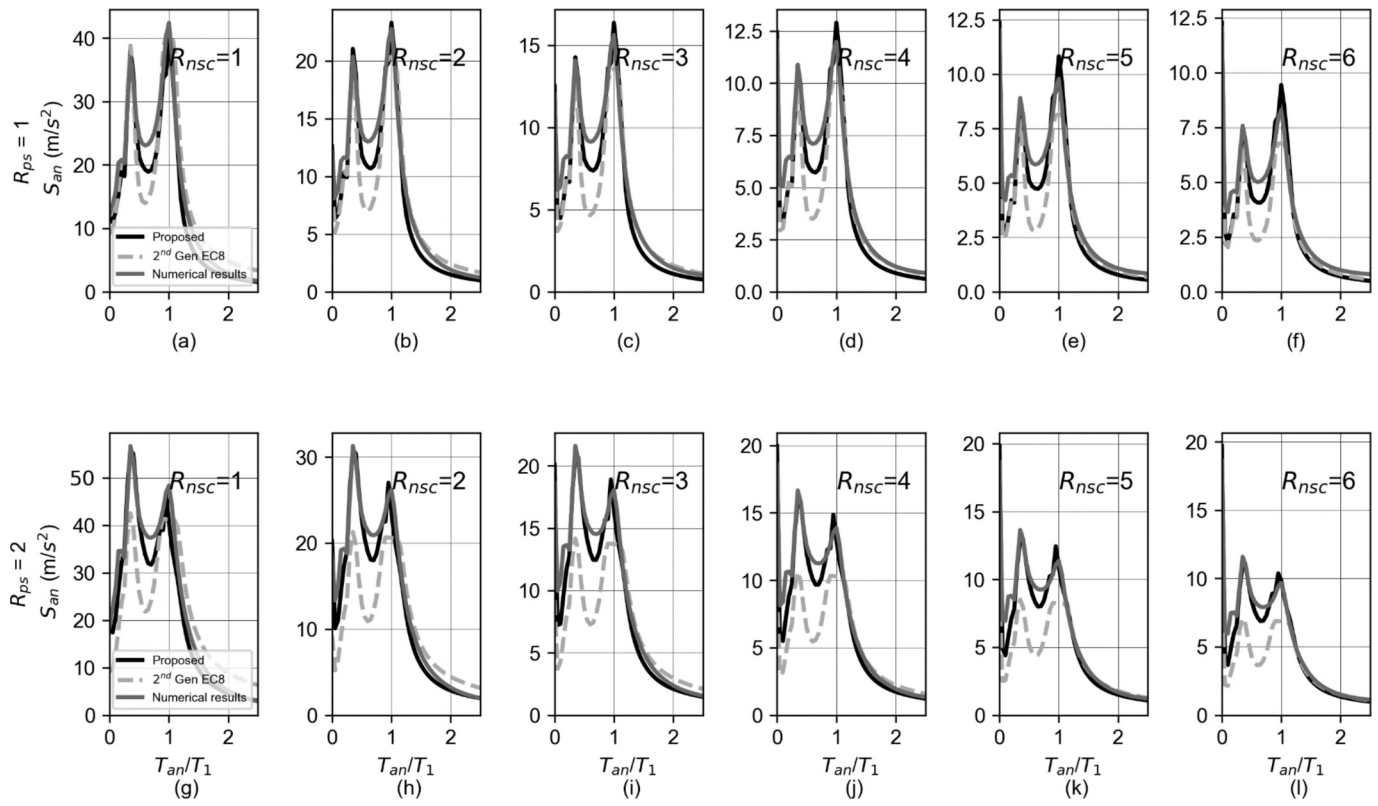


Fig. 24. Spectral acceleration of NSCs against $\frac{T_{an}}{T_1}$ from numerical results, 2nd generation draft of Eurocode 8 [36], and proposed unified models for all 188 irregular frames and all ground motion records. Each row corresponds to a level of inelasticity of the primary structure, and each column corresponds to an inelasticity level of the NSC.

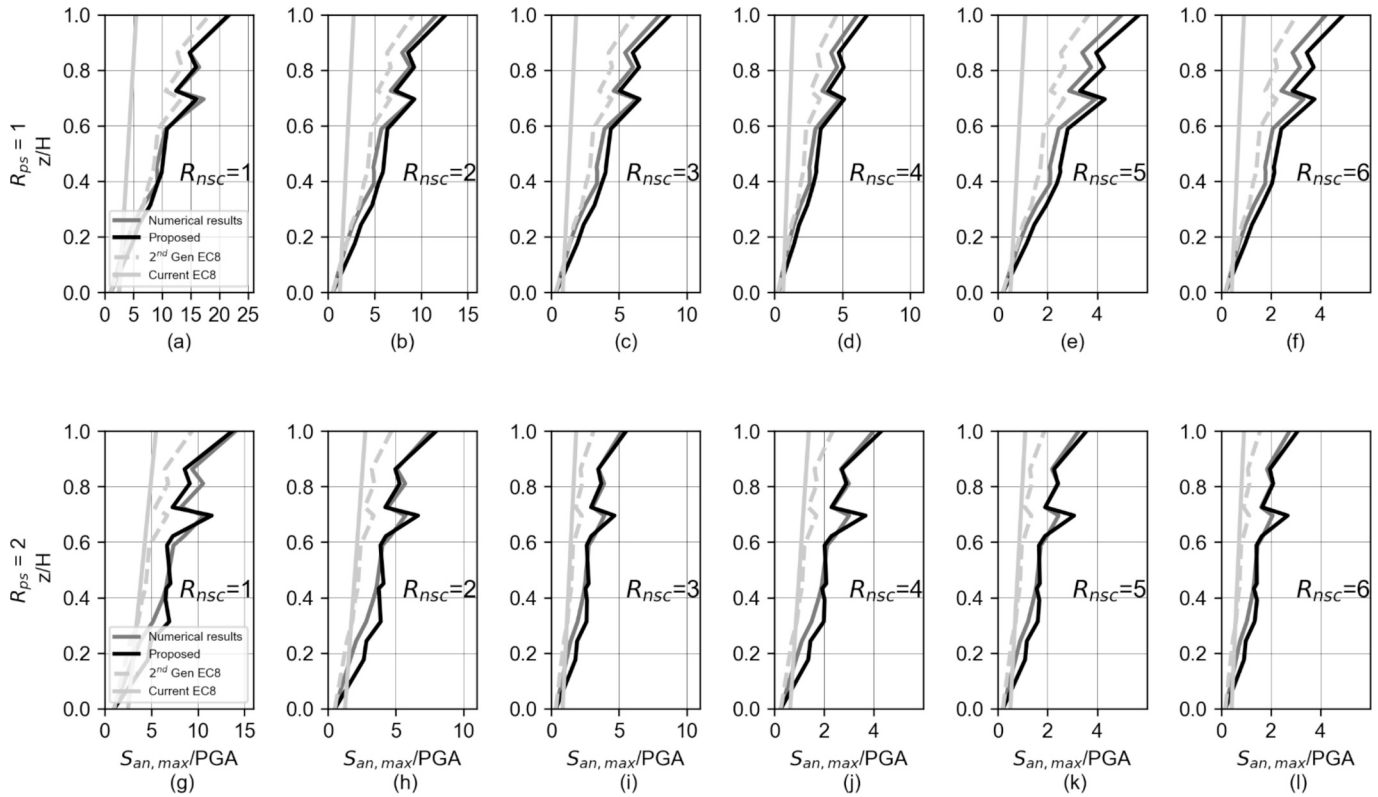


Fig. 25. Maximum spectral acceleration of NSCs along structural height from numerical results, proposed unified models, 2nd generation draft of Eurocode 8 [36], and current Eurocode 8 [29] for all 188 irregular frames and all ground motion records. Each row corresponds to a level of inelasticity of the primary structure, and each column corresponds to an inelasticity level of the NSC.

models for the prediction of both the displacement and acceleration response of non-structural ancillary components in steel moment frames were proposed, in order to provide a unified approach for both regular and irregular frames. After a series of detailed regression analyses and comparative evaluations, the proposed modified relationships were shown to address the shortcomings of existing models, which are intended for regular structures, particularly in terms of their inability to capture the response around the fundamental period of the frames. Additionally, the accuracy of the proposed unified acceleration model was further evaluated in terms of predicting the distribution, over the frame height, of the peak acceleration response of the ancillary components, through comparisons with the numerical results and predictions from the current and the proposed second-generation Eurocode 8 provisions. The proposed model was shown to provide faithful predictions, which are significantly improved in comparison with existing procedures, in terms of both the values and height-wise distribution at all floor levels of the primary structures.

Overall, this investigation demonstrated the accuracy of the proposed models for predicting the inelastic displacement ratio and acceleration response of non-structural ancillary components. The unified

models provide simple and reliable analytical approaches, without the need for undertaking nonlinear dynamic analysis, for determining the seismic response of ancillary components in both regular and irregular steel moment framed structures.

CRediT authorship contribution statement

X. Ding: Writing – original draft, Validation, Software, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **M. Liapopoulou:** Writing – review & editing, Supervision, Methodology, Conceptualization. **A.Y. Elghazouli:** Writing – review & editing, Supervision, Resources, Project administration, Methodology, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Appendix

Table D1

First two periods and member sizes for Set A structures^{1,2} (Frame A01 to A06). The column and beam sections are uniform in each floor and are presented here starting from the first floor to the top floor of each frame.

Frame	T_1	T_2	Element sections from first storey upwards	
			Column	Beam
A01-11	0.42	0.15	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A01-21	0.47	0.15	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A01-12	0.39	0.15	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A01-22	0.46	0.15	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A02-11	0.31	0.10	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A02-21	0.35	0.11	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A02-12	0.30	0.11	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A02-22	0.35	0.11	HEB500, HEB500, HEB450	IPE500, IPE500, IPE450
A03-11	0.59	0.20	HEB400, HEB400, HEB400	IPE400, IPE400, IPE360
A03-21	0.66	0.20	HEB400, HEB400, HEB400	IPE400, IPE400, IPE360
A03-12	0.56	0.21	HEB400, HEB400, HEB400	IPE400, IPE400, IPE360
A03-22	0.65	0.21	HEB400, HEB400, HEB400	IPE400, IPE400, IPE360
A04-11	0.33	0.11	HEM550, HEM550, HEM550	IPE550, IPE550, IPE500
A04-21	0.37	0.11	HEM550, HEM550, HEM550	IPE550, IPE550, IPE500
A04-12	0.31	0.12	HEM550, HEM550, HEM550	IPE550, IPE550, IPE500
A04-22	0.36	0.11	HEM550, HEM550, HEM550	IPE550, IPE550, IPE500
A05-11	0.78	0.26	HEB340, HEB340, HEB340	IPE330, IPE330, IPE300
A05-21	0.89	0.27	HEB340, HEB340, HEB340	IPE330, IPE330, IPE300
A05-12	0.74	0.28	HEB340, HEB340, HEB340	IPE330, IPE330, IPE300
A05-22	0.87	0.27	HEB340, HEB340, HEB340	IPE330, IPE330, IPE300
A06-11	0.65	0.25	HEB400, HEB400, HEB300	IPE360, IPE360, IPE300
A06-21	0.74	0.25	HEB400, HEB400, HEB300	IPE360, IPE360, IPE300
A06-12	0.63	0.24	HEB400, HEB400, HEB300	IPE360, IPE360, IPE300
A06-22	0.74	0.25	HEB400, HEB400, HEB300	IPE360, IPE360, IPE300

First two periods and member sizes for Set A structures (Frame A07 to A08)

Frame	T_1	T_2	Element sections from first storey upwards	
			Column	Beam
A07-11	0.49	0.17	HEB450, HEB450, HEB400	IPE450, IPE450, IPE400
A07-21	0.56	0.18	HEB450, HEB450, HEB400	IPE450, IPE450, IPE400
A07-12	0.47	0.18	HEB450, HEB450, HEB400	IPE450, IPE450, IPE400
A07-22	0.55	0.18	HEB450, HEB450, HEB400	IPE450, IPE450, IPE400
A08-11	0.59	0.19	HEB450, HEB450, HEB400	IPE360, IPE360, IPE360
A08-21	0.67	0.20	HEB450, HEB450, HEB400	IPE360, IPE360, IPE360
A08-12	0.56	0.20	HEB450, HEB450, HEB400	IPE360, IPE360, IPE360
A08-22	0.66	0.20	HEB450, HEB450, HEB400	IPE360, IPE360, IPE360

¹XY-MN: X – frame set, Y – frame number, M – span number of the setback and N – storey number of the setback.

²The period values in columns ' T_1 ' and ' T_2 ' are in seconds.

Table D2

First two periods and member sizes for Set B structures (Frame B01 to B03). The column and beam sections are uniform in each floor and are presented here starting from the first floor to the top floor of each frame.

Frame	T_1	T_2	Element sections from first storey upwards	
			Column	Beam
B01–12	0.31	0.14	HEM900, HEM900, HEM900, HEM900, HEM600	IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE450
B01–22	0.36	0.14	HEM900, HEM900, HEM900, HEM900, HEM600	IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE450
B01–14	0.35	0.12	HEM900, HEM900, HEM900, HEM900, HEM600	IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE450
B01–24	0.38	0.13	HEM900, HEM900, HEM900, HEM900, HEM600	IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE750 × 161, IPE450
B02–12	0.59	0.27	HEM600, HEM600, HE5500, HEM550, HEM400	IPE500, IPE500, IPE450, IPE450, IPE300
B02–22	0.70	0.26	HEM600, HEM600, HE5500, HEM550, HEM400	IPE500, IPE500, IPE450, IPE450, IPE300
B02–14	0.66	0.22	HEM600, HEM600, HE5500, HEM550, HEM400	IPE500, IPE500, IPE450, IPE450, IPE300
B02–24	0.73	0.25	HEM600, HEM600, HE5500, HEM550, HEM400	IPE500, IPE500, IPE450, IPE450, IPE300
B03–12	0.74	0.32	HEM600, HEM600, HE5500, HEM550, HEM400	IPE450, IPE450, IPE450, IPE450, IPE300
B03–22	0.86	0.31	HEM600, HEM600, HE5500, HEM550, HEM400	IPE450, IPE450, IPE450, IPE450, IPE300
B03–14	0.79	0.27	HEM600, HEM600, HE5500, HEM550, HEM400	IPE450, IPE450, IPE450, IPE450, IPE300
B03–24	0.88	0.29	HEM600, HEM600, HE5500, HEM550, HEM400	IPE450, IPE450, IPE450, IPE450, IPE300

First two periods and member sizes for Set B structures (Frame B04 to B09)

Frame	T_1	T_2	Element sections from first storey upwards	
			Column	Beam
B04–12	0.39	0.20	HEM650, HEM650, HEM6500, HEM650, HEM400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B04–22	0.46	0.19	HEM650, HEM650, HEM6500, HEM650, HEM400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B04–14	0.43	0.16	HEM650, HEM650, HEM6500, HEM650, HEM400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B04–24	0.47	0.18	HEM650, HEM650, HEM6500, HEM650, HEM400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B05–12	0.69	0.30	HEM550, HEM550, HEB5500, HEB550, HEB550	IPE450, IPE450, IPE400, IPE400, IPE300
B05–22	0.81	0.29	HEM550, HEM550, HEB5500, HEB550, HEB550	IPE450, IPE450, IPE400, IPE400, IPE300
B05–14	0.77	0.24	HEM550, HEM550, HEB5500, HEB550, HEB550	IPE450, IPE450, IPE400, IPE400, IPE300
B05–24	0.85	0.27	HEM550, HEM550, HEB5500, HEB550, HEB550	IPE450, IPE450, IPE400, IPE400, IPE300
B06–12	0.73	0.31	HEB500, HEB550, HEB550, HEB550, HEB400	IPE450, IPE450, IPE400, IPE400, IPE300
B06–22	0.86	0.31	HEB500, HEB550, HEB550, HEB550, HEB400	IPE450, IPE450, IPE400, IPE400, IPE300
B06–14	0.81	0.26	HEB500, HEB550, HEB550, HEB550, HEB400	IPE450, IPE450, IPE400, IPE400, IPE300
B06–24	0.89	0.29	HEB500, HEB550, HEB550, HEB550, HEB400	IPE450, IPE450, IPE400, IPE400, IPE300
B07–12	0.83	0.34	HEB500, HEB500, HEB500, HEB500, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B07–22	0.97	0.34	HEB500, HEB500, HEB500, HEB500, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B07–14	0.89	0.29	HEB500, HEB500, HEB500, HEB500, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B07–24	1.00	0.32	HEB500, HEB500, HEB500, HEB500, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B08–12	0.59	0.27	HEM550, HEM550, HEB550, HEB550, HEB400	IPE500, IPE500, IPE500, IPE500, IPE300
B08–22	0.69	0.27	HEM550, HEM550, HEB550, HEB550, HEB400	IPE500, IPE500, IPE500, IPE500, IPE300
B08–14	0.64	0.23	HEM550, HEM550, HEB550, HEB550, HEB400	IPE500, IPE500, IPE500, IPE500, IPE300
B08–24	0.71	0.25	HEM550, HEM550, HEB550, HEB550, HEB400	IPE500, IPE500, IPE500, IPE500, IPE300
B09–12	0.39	0.20	HEM700, HEM700, HEM600, HEM500, HEB400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B09–22	0.45	0.19	HEM700, HEM700, HEM600, HEM500, HEB400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B09–14	0.43	0.16	HEM700, HEM700, HEM600, HEM500, HEB400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400
B09–24	0.47	0.18	HEM700, HEM700, HEM600, HEM500, HEB400	IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE400

First two periods and member sizes for Set B structures (Frame B10 to B15)

Frame	T_1	T_2	Element sections from first storey upwards	
			Column	Beam
B10–12	1.07	0.45	HEB400, HEB400, HEB400, HEB400, HEB300	IPE360, IPE360, IPE330, IPE330, IPE330
B10–22	1.26	0.44	HEB400, HEB400, HEB400, HEB400, HEB300	IPE360, IPE360, IPE330, IPE330, IPE330
B10–14	1.17	0.38	HEB400, HEB400, HEB400, HEB400, HEB300	IPE360, IPE360, IPE330, IPE330, IPE330
B10–24	1.30	0.42	HEB400, HEB400, HEB400, HEB400, HEB300	IPE360, IPE360, IPE330, IPE330, IPE330
B11–12	0.84	0.38	HEB500, HEB500, HEB400, HEB400, HEB300	IPE500, IPE500, IPE400, IPE400, IPE300
B11–22	0.99	0.37	HEB500, HEB500, HEB400, HEB400, HEB300	IPE500, IPE500, IPE400, IPE400, IPE300
B11–14	0.92	0.31	HEB500, HEB500, HEB400, HEB400, HEB300	IPE500, IPE500, IPE400, IPE400, IPE300
B11–24	1.03	0.35	HEB500, HEB500, HEB400, HEB400, HEB300	IPE500, IPE500, IPE400, IPE400, IPE300
B12–12	0.65	0.31	HEB550, HEB550, HEB550, HEB550, HEB300	IPE500, IPE500, IPE450, IPE450, IPE300
B12–22	0.77	0.31	HEB550, HEB550, HEB550, HEB550, HEB300	IPE500, IPE500, IPE450, IPE450, IPE300
B12–14	0.72	0.26	HEB550, HEB550, HEB550, HEB550, HEB300	IPE500, IPE500, IPE450, IPE450, IPE300
B12–24	0.79	0.29	HEB550, HEB550, HEB550, HEB550, HEB300	IPE500, IPE500, IPE450, IPE450, IPE300
B13–12	0.87	0.38	HEB500, HEB500, HEB450, HEB450, HEB300	IPE400, IPE400, IPE360, IPE360, IPE300
B13–22	1.02	0.38	HEB500, HEB500, HEB450, HEB450, HEB300	IPE400, IPE400, IPE360, IPE360, IPE300
B13–14	0.96	0.31	HEB500, HEB500, HEB450, HEB450, HEB300	IPE400, IPE400, IPE360, IPE360, IPE300
B13–24	1.06	0.35	HEB500, HEB500, HEB450, HEB450, HEB300	IPE400, IPE400, IPE360, IPE360, IPE300
B14–12	0.88	0.37	HEB450, HEB450, HEB450, HEB450, HEB300	IPE400, IPE400, IPE400, IPE400, IPE300
B14–22	1.02	0.37	HEB450, HEB450, HEB450, HEB450, HEB300	IPE400, IPE400, IPE400, IPE400, IPE300
B14–14	0.94	0.32	HEB450, HEB450, HEB450, HEB450, HEB300	IPE400, IPE400, IPE400, IPE400, IPE300
B14–24	1.05	0.35	HEB450, HEB450, HEB450, HEB450, HEB300	IPE400, IPE400, IPE400, IPE400, IPE300
B15–12	0.83	0.36	HEB500, HEB500, HEB500, HEB400, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B15–22	0.97	0.35	HEB500, HEB500, HEB500, HEB400, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B15–14	0.90	0.30	HEB500, HEB500, HEB500, HEB400, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300
B15–24	1.01	0.33	HEB500, HEB500, HEB500, HEB400, HEB400	IPE400, IPE400, IPE400, IPE400, IPE300

Table D3

First two periods and member sizes for Set C structures (Frame C01 to C04). The column and beam sections are uniform in each floor and are presented here starting from the first floor to the top floor of each frame.

[illegible]

First two periods and member sizes for Set C structures (Frame C05 to C08)

Frame	T_1	T_2	Element sections from first storey upwards					
			Column			Beam		
C05-12	0.9	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE550, IPE550, IPE550, IPE450, IPE450, IPE300				
C05-22	1.0	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE550, IPE550, IPE550, IPE450, IPE450, IPE300				
C05-14	0.9	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE550, IPE550, IPE550, IPE450, IPE450, IPE300				
C05-24	1.0	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE550, IPE550, IPE550, IPE450, IPE450, IPE300				
C05-16	1.0	0.3	HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE550, IPE550, IPE550, IPE450, IPE450, IPE300				
C05-26	1.0	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE550, IPE550, IPE550, IPE450, IPE450, IPE300				
C06-12	1.0	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE300				
C06-22	1.1	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE300				
C06-14	1.0	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE300				
C06-24	1.1	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE300				
C06-16	1.1	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE300				
C06-26	1.2	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE300				
C07-12	0.8	0.3	HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C07-22	0.9	0.3	HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C07-14	0.7	0.3	HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C07-24	0.8	0.3	HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C07-16	0.8	0.3	HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C07-26	0.9	0.3	HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C08-12	1.2	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C08-22	1.4	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C08-14	1.2	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C08-24	1.4	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C08-16	1.3	0.4	HEB450, HEB450, HEB450, HEB400, HEB400, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				
C08-26	1.4	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB300	IPE400, IPE400, IPE400, IPE400, IPE400, IPE300				

First two periods and member sizes for Set C structures (Frame C09 to C12)

Frame	T ₁	T ₂	Element sections from first storey upwards											
			Column						Beam					
C09-12	0.5	0.2	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450						IPE750 × 137, IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE600, IPE400					
C09-22	0.6	0.2	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450						IPE750 × 137, IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE600, IPE400					
C09-14	0.5	0.2	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450						IPE750 × 137, IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE600, IPE400					
C09-24	0.6	0.2	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450						IPE750 × 137, IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE600, IPE400					
C09-16	0.6	0.2	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450						IPE750 × 137, IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE600, IPE400					
C09-26	0.6	0.2	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450						IPE750 × 137, IPE750 × 137, IPE750 × 137, IPE600, IPE600, IPE600, IPE400					
C10-12	1.2	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300						IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300					
C10-22	1.3	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300						IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300					
C10-14	1.1	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300						IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300					
C10-24	1.3	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300						IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300					
C10-16	1.3	0.4	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300						IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300					
C10-26	1.4	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300						IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300					
C11-12	0.8	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300						IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300					
C11-22	0.9	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300						IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300					

(continued on next page)

Table D3 (continued)

First two periods and member sizes for Set C structures (Frame C09 to C12)						
Frame	T_1	T_2	Element sections from first storey upwards			
			Column		Beam	
C11–14	0.8	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300		
C11–24	0.9	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300		
C11–16	0.9	0.3	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300		
C11–26	0.9	0.3	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300		
C12–12	1.2	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300		
C12–22	1.4	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300		
C12–14	1.1	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300		
C12–24	1.3	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300		
C12–16	1.3	0.4	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300		
C12–26	1.4	0.5	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300		
First two periods and member sizes for Set C structures (Frame C13 to C16)						
Frame	T_1	T_2	Element sections from first storey upwards			
			Column		Beam	
C13–12	0.9	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300		
C13–22	1.0	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300		
C13–14	0.9	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300		
C13–24	1.0	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300		
C13–16	1.0	0.3	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300		
C13–26	1.1	0.4	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300		
C14–12	1.1	0.5	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300		
C14–22	1.2	0.5	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300		
C14–14	1.0	0.5	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300		
C14–24	1.2	0.5	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300		
C14–16	1.2	0.4	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300		
C14–26	1.3	0.5	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300		
C15–12	0.8	0.4	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360		
C15–22	0.9	0.4	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360		
C15–14	0.8	0.4	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360		
C15–24	0.9	0.4	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360		
C15–16	0.9	0.3	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360		
C15–26	1.0	0.4	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360		
C16–12	0.8	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300		
C16–22	0.9	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300		
C16–14	0.8	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300		
C16–24	0.9	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300		
C16–16	0.9	0.3	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300		
C16–26	1.0	0.4	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300		

Data availability

Data will be made available on request.

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