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ORIGINAL ARTICLE

Extension of the improved upper-bound pushover analysis for seismic assessment of steel moment resisting frames with setbacks

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Abstract
Pushover analysis technique is a key tool for the performance-based seismic design that has been largely adopted in the new generation of seismic codes. Therefore, more precise and reliable performance predictions are highly demanded. Improved upper-bound (IUB) pushover analysis is one of the advanced nonlinear static procedures (NSPs) that has been recently developed. This procedure adequately estimates the response of regular and tall buildings. In this study, IUB is extended to assess the seismic response of irregular buildings with setbacks. To this end, an adjustment of the IUB lateral load distribution is implemented by integrating a third mode of vibration to control the response of these complex buildings. Fifteen multi-storey steel frames with different types of setbacks including a reference structure are used to test the accuracy of the proposed procedure by comparing its results to those from other NSPs and the nonlinear time history analysis (NTHA). The findings show the superior capacity of the extended IUB in predicting the seismic response of buildings with different levels and types of setbacks.

Keywords Setbacks buildings · Nonlinear static procedures · Improved upper-bound · Pushover analysis · Lateral load distribution

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1 Introduction

The current trend in adopting the concept of performance-based seismic design in seismic codes requires efficient analysis techniques capable of assessing the seismic performance of a large variety of building configurations. NSPs are actually viewed as the most favourite alternative to perform such tasks. Although, several developments have so far been achieved to enhance the capabilities of these procedures, other challenging issues still need to be addressed. Setbacks are common and frequent irregularities in buildings that result from abrupt discontinuities in the construction elevation. Such irregularities can significantly affect the seismic performance of these buildings. Over the past few decades, extensive research work has been undertaken in an attempt to shed light on the behaviour of setback buildings (Humar and Wright 1977; Aranda 1984; Shahrooz and Moehle 1990; Wood 1992; Wong and Tso 1994; Pinto and Costa 1995; Mazzolani and Piluso 1996; Bosco et al. 2002; Das and Nau 2003; Romão et al. 2004; Lignos and Gantes 2005; Athanassiadou 2008; Sarkar et al. 2010). Yet, the conflicting conclusions regarding the effects of setbacks on the global and local behaviour of structures suggest that further research work is still needed to resolve the inconsistent results about the seismic behaviour of this type of building structures. Currently, the nonlinear time history analysis (NTHA) may be viewed as the most appropriate and accurate method for estimating the seismic responses of structures. However, its use is limited in practice as it requires some qualifications that are beyond the competence level of engineers. The nonlinear static or pushover analysis procedures (NSPs) especially the recent improved versions have been brought to the forefront of seismic design and assessment of complex structural behaviour and constitute an efficient alternative to the NTHA.

The conventional version of the pushover method considers the inelastic behaviour assuming that the dynamic response of the building is dependent on the elastic fundamental (first) mode during the analysis (Freeman 1998; Fajfar 1999). This hypothesis gives good results when dealing with regular structures. However, when higher modes affect the structural response, like in high-rise or irregular buildings, this assumption can lead to inadequate results.

In this regard, several researchers have recently proposed enhancements to overcome the above-mentioned limitation. The modal pushover analysis (MPA), developed by Chopra and Goel (2002) is a multi-run procedure that considers primarily the few first vibration modes that have significant effects on the structural response, where a modal distribution corresponding to the ith vibration mode is applied to the structure during each run of the MPA procedure. Then, the final response is obtained by combining the results obtained from each modal run using the square-root-of-sum-of-squares (SRSS) or the complete quadratic combination (CQC). This method was later improved and extended to assess the seismic response of irregular buildings and bridges (Chopra and Goel 2004; Mao et al. 2008; Paraskeva and Kappos 2010; Reyes and Chopra 2011; Belejo and Bento 2016).

In the same context, Jan et al. (2004) designed an upper bound (UB) pushover procedure considering only the first two modes of vibration using the modal combination with the absolute sum (ABSSUM) method to determine the lateral loading pattern and evaluate the target displacement. The authors studied a set of two-dimensional tall building frames and concluded that the UB procedure is more efficient when dealing with flexible buildings. However, it was found that, in most cases, this procedure underestimates the responses of the lower storeys of the building (Jan et al. 2004).

To address this drawback, Poursha and Samarin (2015) modified and extended the UB method (MUB and EUB) by combining the results of the conventional and UB pushover methods. In this case, the target displacement is supposed to be equal to the average of roof displacements calculated by the NTHA; this makes the two multi-run procedures difficult to be applied in practice.

Later, Rahmani et al. (2018) proposed the so-called improved upper-bound (IUB) pushover analysis, which involves an adjustment of the contribution of the second mode of vibration applied to both the lateral load pattern and the target displacement formulas using a correction factor. Then, the corrected applied lateral load pattern is combined with the scaled uniform and first mode lateral patterns to get the final lateral load distribution (envelope). It was shown that the improved upper-bound pushover procedure gives better results for seismic assessment of tall steel building frames compared to those obtained from the MUB and other pushover procedures. These findings are in good agreement with those reported by El-Esnawy et al. (2020).

Earlier, the conventional N2 method (Fajfar 1999) has been enhanced and extended in order to take into consideration higher mode effects (Fajfar et al. 2005; Kreslin and Fajfar 2011a, b). The results of the N2 procedure were then adjusted using correction factors based on the linear dynamic response spectrum analysis. Recently, Zarrin et al. (2021a, b) have proposed a multi-mode N2 (MN2) method to assess the seismic performance of steel buildings and jacket-type offshore platforms. The MN2 method employs the algebraic sum combination rule to calculate the modal response of the linear dynamic response spectrum analysis instead of the quadratic modal combination rule that is used in the extended N2 method.

Concurrently, a consecutive modal pushover (CMP) procedure has been developed by Poursha et al. (2009) in order to assess the seismic response of high-rise buildings. The method is based on enveloping the results obtained from single-stage and multi-stage pushover analyses. The single-stage pushover analysis is used to control the responses of lower storeys of tall buildings, while the multi-stage procedure controls the seismic demands at mid and upper storeys. Khoshnoudian and Kiani (2012) modified the CMP method for the purpose of investigating the seismic response of one-way asymmetric-plan tall buildings. Additionally, Poursha et al. (2014), extended the CMP method to account for the torsional effects on the seismic behaviour of two-way asymmetric-plan high-rise buildings under bi-directional seismic ground motions. Recently, Zarrin et al. (2021c) have developed an updated consecutive modal pushover (UCMP) procedure that was applied to two case studies of jacket-type offshore platform models.

Furthermore, Poursha and Amini (2015) developed a single-run multi-mode pushover (SMP) procedure. In this approach, the lateral load pattern is calculated using the algebraic summation of the modal storey forces. Also, Behnamfar et al. (2016) conducted another study to enhance Sahraei and Behnamfar's work (2014), which was based on the storey drift to construct the lateral load vector. Afterwards, in the extended version, a new modal combination rule for storey drifts was employed to combine the storey drifts instead of using the square-root-of-sum-of-squares (SRSS) rule. Liu and Kuang (2017), developed the so-called spectrum-based pushover analysis (SPA) where both the lateral load vector and the target displacement were computed based on the modal response analysis. Later, this SPA approach was adopted to study several types of buildings (Liu et al. 2018, 2020).

A new generation of pushover analyses, called the generalised pushover analysis (GPA) was developed by Sucuoğlu and Günay in 2011 to take into account the contributions of the most important vibration modes. Series of pushover analyses using various generalised force vectors were performed. Each generalised force vector integrated the modal lateral forces in order to replicate the effective lateral force distribution when the inter-storey drift at a given level of the building reaches its maximum value during the seismic response. It is worth emphasising that in this approach, the envelope values produced by the set of GPA are used to derive the final response. In 2014, the GPA was modified and extended to investigate the seismic behaviour of bridges and irregular buildings (Cao and Yuan 2014; Kaatsiz et al. 2017). In this line, Ferraioli (2017) developed and evaluated a multi-mode pushover procedure to estimate the seismic response of steel moment-resisting frames. The new multi-run and multi-mode procedure predicted accurately the seismic demands of steel moment-resisting frames at different intensity levels of input ground motion.

In the year 2019, Guan et al. (2019) adopted a simplified approach for the purpose of calculating the structural seismic response by combining the responses obtained from three conventional pushover analyses with different lateral load distributions, including the uniform, first mode and concentrated distribution loadings. Likewise, Habibi et al. (2019) proposed a conventional method with optimal lateral load pattern that was obtained through an optimization procedure.

In a different way, some authors designed a number of adaptive pushover procedures in an attempt to consider the effects of damage and stiffness degradation, as well as the higher mode effects, by updating the applied load patterns in each phase during the nonlinear loading (Bracci et al. 1997; Gupta and Kunnath 2000; Mwafy and Elnashai 2001; Antoniou and Pinho 2004a, b; Kalkan and Kunnath 2006; Ferracuti et al. 2009; Shakeri et al. 2010, 2013; Abbasnia et al. 2014a, b; Tarbali and Shakeri 2014; Amini and Poursha 2017; Sürmeli and Yüksel 2018; Rahmani et al. 2019; Jalilkhani et al. 2020). Unfortunately, the adaptive procedures made the pushover analysis more complex and difficult to apply in practice (Kreslin and Fajfar 2011a).

Most of the abovementioned research work focused on the behaviour of tall, regular buildings. While, only a few of them investigated setback buildings. In 2016, Ferraioli et al. proposed an adaptive capacity spectrum method to assess the seismic behaviour of steel regular and irregular moment-resisting frames and concluded that the multi-modal pushover procedures (with invariant load patterns) give a more accurate assessment of seismic demands. However, these procedures become increasingly inaccurate when the peak ground acceleration and elevation irregularity go up. In the same year, Bohlouli and Poursha (2016) investigated a set of setback steel moment-resisting frames using four advanced pushover procedures. The obtained results made it evident that the accuracy of these procedures is particularly influenced by the geometrical configuration of the setback frames. In addition, Rooshenas (2020) investigated the seismic behaviour of tall concrete structures with partial infilled masonry panels leading to elevation irregularities in terms of mass and stiffness distributions. The results obtained from different pushover procedures were compared, and the author recommended that the effects of higher modes and those of infill panels should be considered when analysing this type of building.

The present work aims to extend the improved upper-bound (IUB) in order to take into account the effect of setback on the seismic response of mid-rise buildings. In addition to the first two modes of vibration used in the IUB method, a third mode, which potentially depicts the irregularity feature, is employed to generate the applied load vector. The proposed extended IUB version was applied to fifteen 10-storey buildings with different setback configurations, including a reference structure. The performance of this new version was evaluated in terms of target displacement, storey displacements, storey drift, and plastic hinge rotations, which were compared to those obtained from other pushover procedures, using the nonlinear time history analysis (NTHA) as a benchmark.

2 Extension of the improved upper bound pushover analysis

2.1 Lateral load pattern

According to the study by Davoudi et al. (2016), the applied lateral load $\boldsymbol{f}$ can be written as:

$$\boldsymbol{f} = 0.48\boldsymbol{f}_{\boldsymbol{m}1} \pm 0.26\boldsymbol{f}_{\boldsymbol{m}2} \pm 0.26\boldsymbol{f}_{\boldsymbol{m}3} \tag{1}$$

$\boldsymbol{f}_{\boldsymbol{m}1}$,$\boldsymbol{f}_{\boldsymbol{m}2}$ and $\boldsymbol{f}_{\boldsymbol{m}3}$ are the load vectors of the first, the second, and the third modes of vibration, respectively. In which:

$$\boldsymbol{f}_{\boldsymbol{mi}} = \omega_i^2 \mathbf{M}\boldsymbol{\varphi}_{\mathbf{i}} q_i \tag{2}$$

where ω_i is the natural frequencies for the i-th mode, respectively; $\mathbf{M}$ is the mass matrix; $\boldsymbol{\varphi}_{\mathbf{i}}$ and q_i are the normalised mode shape and modal coordinates of the i-th mode. If the modal loads are substituted by their corresponding formulas in Eq. 1, it becomes

$$\boldsymbol{f} = 0.48\left(\omega_1^2 \mathbf{M}\boldsymbol{\varphi}_1 q_1\right) \pm 0.26\left(\omega_2^2 \mathbf{M}\boldsymbol{\varphi}_2 q_2\right) \pm 0.26\left(\omega_3^2 \mathbf{M}\boldsymbol{\varphi}_3 q_3\right) \tag{3}$$

Multiplying and dividing by the factor $0.48q_1$, then Eq. (3) can be expressed as:

$$\boldsymbol{f} = 0.48q_1\left[\left(\omega_1^2 \mathbf{M}\boldsymbol{\varphi}_1\right) \pm \frac{0.26}{0.48}\left(\omega_2^2 \mathbf{M}\boldsymbol{\varphi}_2 \frac{q_2}{q_1}\right) \pm \frac{0.26}{0.48}\left(\omega_3^2 \mathbf{M}\boldsymbol{\varphi}_3 \frac{q_3}{q_1}\right)\right] \tag{4}$$

Because the applied load in pushover analysis starts from zero, the $0.48q_1$ factor has no influence on the load distribution. Therefore, Eq. (4) may be stated (using simply the plus sign) as follows:

$$\boldsymbol{f}' = \left[\left(\omega_1^2 \mathbf{M}\boldsymbol{\varphi}_1\right) + \frac{0.26}{0.48}\left(\omega_2^2 \mathbf{M}\boldsymbol{\varphi}_2 \frac{q_2}{q_1}\right) + \frac{0.26}{0.48}\left(\omega_3^2 \mathbf{M}\boldsymbol{\varphi}_3 \frac{q_3}{q_1}\right)\right] \tag{5}$$

The $\left(q_i/q_1\right)$ ratio can be calculated using the following equation:

$$\frac{q_i}{q_j} = \left|\frac{\Gamma_i S_{di}}{\Gamma_j S_{dj}}\right| \tag{6}$$

where Γ_{i} and Γ_{j} are the modal participation factors of the i-th ($i = 2$ or 3 and $j = 1$) mode of vibration, S_{di} and S_{dj} are the corresponding displacements obtained from the elastic displacement response spectrum.

The ratio $0.26/0.48$ in Eq. (5) is considered as a correction factor C_r to adjust the contribution of the second and third modes of vibration. Its value is given by

$$C_r = \frac{0.26}{0.48} \approx 0.5 \tag{7}$$

Instead of taking only the two first modes of vibration like in the IUB procedure (Rahmani et al. 2018). In this study, the upper-bound load vector $\mathbf{f}''_{\boldsymbol{UB}}$ is given by

$$\mathbf{f}''_{\boldsymbol{UB}} = \boldsymbol{f}' = \omega_1^2 \mathbf{M}\boldsymbol{\varphi}_1 + 0.5 \cdot \left(\omega_2^2 \mathbf{M}\boldsymbol{\varphi}_2 \frac{q_2}{q_1}\right) + 0.5 \cdot \left(\omega_3^2 \mathbf{M}\boldsymbol{\varphi}_3 \frac{q_3}{q_1}\right) \tag{8}$$

The final lateral load pattern $\mathbf{f}_{EIUB}$ is defined as the envelope load pattern of the two lateral load patterns (uniform $\mathbf{f}_{Unif}$, and the corrected upper bound load $\mathbf{f}''_{UB}$ distributions). In which the $\mathbf{f}_{Unif}$ is given by:

$$\mathbf{f}_{Unif} = r_u \omega_1^2 \mathbf{M} 1 \tag{9}$$

r_u is set equal to 0.8 to ensure that the uniform load distribution is dominant at lower and mid storeys (Rahmani et al. 2018). 1 is a unity vector.

The i-th value of the applied load pattern at the i-th floor $\mathbf{f}_{i,EIUB}$ becomes:

$$\mathbf{f}_{i,EIUB} = max\left(\mathbf{f}''_{i,UB}, \mathbf{f}_{i,Unif}\right) \tag{10}$$

2.2 The target displacement

Rahmani et al. (2018), and based on the upper-bound pushover analysis developed by Jan et al. (2004), determined the target displacement at the roof of the structure U_r as follows:

$$U_r = U_{rM1}\left(1 + \frac{q_2}{q_1} C_r\right) \tag{11}$$

where U_{rM1}, the target displacement at the building's roof, is calculated using the capacity spectrum method (CSM) described in ATC-40 (ATC-40 1996), or other methods like the N2 method (Fajfar and Gašperšič 1996), by applying the first mode load pattern. The same correction factor C_r used in the lateral load pattern is adopted to adjust the target displacement (C_r is equal to 0.5).

By adding the contribution of the third mode, the formula of the target displacement for the EIUB procedure is given by:

$$U_r = U_{rM1}\left(1 + \frac{q_2}{q_1} C_r + \frac{q_3}{q_1} C_r\right) \tag{12}$$

2.3 Summary of the EIUB procedure

EIUB follows the same steps as the IUB procedure (Rahmani et al. 2018). The EIUB procedure can be summarised in the following steps:

1. Calculate the natural frequencies of the structure, ω_n, and the mode-shapes $\boldsymbol{\varphi}_n$, such that the lateral component of $\boldsymbol{\varphi}_n$ at the roof equals unity.
2. Determine the upper-bound of the contribution of 2nd and 3rd modes q_i/q_1 as provided by Eq. (6), respectively, using the elastic response spectrum of the specified earthquake records.
3. Calculate the lateral load distribution vector across the building's height using Eq. (10).
4. Determine the target roof displacement U_r using Eq. (11) or Eq. (12).
5. Use the lateral load determined in step 3 to perform a pushover analysis until the target displacement estimated in step 4 is reached.
6. Determine the maximum values of the seismic responses from the single-run analysis (step 5).

The contribution of the different load patterns in the final applied load pattern of each approach (UB, IUB, and EIUB) is summarised in Table 1, together with the target displacement formulas of each procedure.

3 Numerical analyses

3.1 Description of the studied structures

Fifteen two-dimensional (2D) 10-storey moment-resisting frames were used to validate the proposed procedure. This includes one reference regular frame selected from the literature (Behnamfar et al. 2016), and fourteen frames with vertical irregularity generated by introducing setbacks in the reference regular frame. Setbacks occur at various elevations of the buildings. All the structures are three-bay frames. The bays are 5 m long, with a consistent floor height of 3.2 m. The plan view of the buildings and the configuration of the frames are shown in Figs. 1 and 2, respectively. Three two-digit numbers following the F letter (the letter F stands for frame) in the frame's name represent the number of storeys in each bay of the frame from left to right, respectively. The lateral load-resisting system of the structures is the special steel moment resisting frame (SMRF). The buildings are designed according to the Iranian seismic code (Standard No. 2800 2005) as well as the American Institute of Steel Construction (AISC-ASD 2010). All of the structures are intended to meet the drift criterion as well as the strong column/weak beam philosophy (Standard No. 2800 2005). The gravity loads are assumed to be uniformly distributed considering the tributary load distribution of the slab and columns on beams, with 32.5 kN/m as the floor dead load and 10 kN/m as the live load. The seismic masses at the floor level include the dead load plus 20% of the live load. The panel zone effect is ignored. The buildings are residential buildings (Group 3) and located on medium soil (type C according to the National Earthquake Hazards Reduction Program (NEHRP 2009)) in a region of high seismicity. The factor V_d/W (design base shear versus weight of the building) is varied from 0.053 to 0.062 (the behaviour factor is set to 10 (R = 10)) for all the buildings according to the Iranian standard (2005). The design base shear of each frame is depicted in Fig. 2.

Tables 2 and 3 exhibit material and section details of the structures under study (Fig. 3). The reference (Behnamfar et al. 2016) provides further information on the reference structure.

3.2 Ground motion records selection

The nonlinear time history analysis and the static non-linear procedures were performed using a set of twenty-one ground motion records. The records were chosen from records on soil Type C that were compatible with the design assumptions of the structures under study, with magnitudes ranging from 6 to 7.5. All data comes from the strong-motion database of the Pacific Earthquake Engineering Research Center (PEER 2021). Figure 4 depicts the elastic spectra of the selected ground motion records (with a damping ratio of 5%), as well as the target spectrum (design spectrum) for the F-10-05-05 frame. In the time range of $0.2T_1$ to $2T_1$ (where T_1 is the period of the first mode of vibration of the studied building), there is a satisfactory match between the geometric mean spectrum of the chosen records and the target spectrum. It is worth noting that at any time within the specified

Table 1 Summary of UB, IUB and EIUB applied load patterns

Contribution	UB	IUB	EIUB
1st mode load pattern	$\omega_1^2 m\varphi_1$	$\omega_1^2 m\varphi_1$	$\omega_1^2 m\varphi_1$
Uniform load pattern	-	$r_u\omega_1^2 m_i$	$r_u\omega_1^2 m_i$
Higher modes load pattern	$\omega_2^2 m\varphi_2 \frac{q_2}{q_1}$	$\omega_2^2 m\varphi_2 \frac{q_2}{q_1}.C_r$	$\omega_2^2 m\varphi_2 \frac{q_2}{q_1}.C_r + \omega_3^2 m\varphi_3 \frac{q_3}{q_1}.C_r$
Combination	$\omega_1^2 m\varphi_1 + \omega_2^2 m\varphi_2 \frac{q_2}{q_1}$	$max\left(\omega_1^2 m\varphi_1, r_u\omega_1^2 m_i, \omega_1^2 m\varphi_1 + \omega_2^2 m\varphi_2 \frac{q_2}{q_1}.C_r\right)$	$max\left(r_u\omega_1^2 m_i, \omega_1^2 m\varphi_1 + \omega_2^2 m\varphi_2 \frac{q_2}{q_1}.C_r + \omega_3^2 m\varphi_3 \frac{q_3}{q_1}.C_r\right)$
Target displacement	$U_{rTLP}\left(1 + \frac{q_2}{q_1}\right)$	$U_{rM1}\left(1 + \frac{q_2}{q_1}C_r\right)$	$U_r = U_{rM1}\left(1 + \frac{q_2}{q_1}C_r + \frac{q_3}{q_1}C_r\right)$

Table 2 Section details of the beams and columns for the studied frames

Dimensions of beams					Dimensions of columns		
Section	h_t (cm)	t_w (cm)	b_f (cm)	t_f (cm)	Section	d (cm)	t (cm)
B1	40	1	22.5	2	C1	35	2.5
B2	35	0.88	22.5	2	C2	30	2
B3	30	0.8	20	1.5			

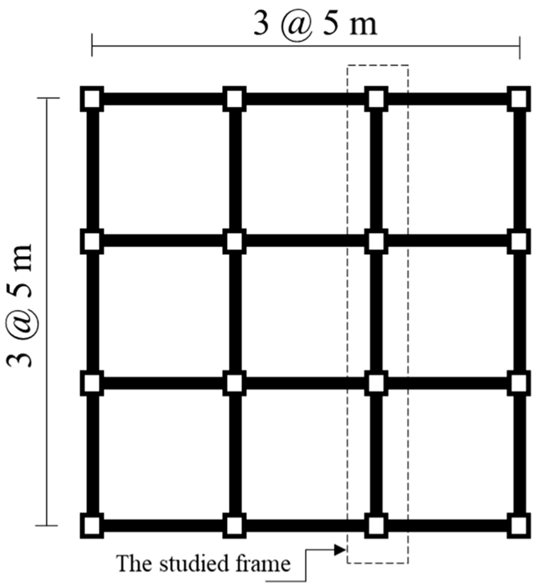


Fig. 1 Plan view of the studied buildings

period range, the average spectrum of the individual spectra does not fall below 90% of the target response spectrum (ASCE 2016). Table 4 lists other characteristics of the selected ground motion data.

3.3 Structural modelling

The computer program SAP2000 (Computers and Structures, Inc. 2013) was used to perform the NSPs and the nonlinear time history analysis (NTHA). The nonlinearity of the structural elements is modelled by employing elastic elements coupled with concentrated plastic hinges. FEMA-356 (FEMA 2000) specifies the properties of the plastic hinges at the ends of beams and columns. For columns, the interaction of axial forces and bending moments (P-M33 in SAP2000) is considered. For beams, the bending moment is considered to control the formation of the hinges. The generalised force–deformation relationship model used for modelling the hinges is shown in Fig. 5. More details about the determination of a, b and c parameters of the model in Fig. 5 can be found in FEMA-356 (2000). The connections are considered in this study as fully rigid and the panel zone effect is neglected. For both nonlinear static and dynamic analysis, the $P-\Delta$ effect is included. The

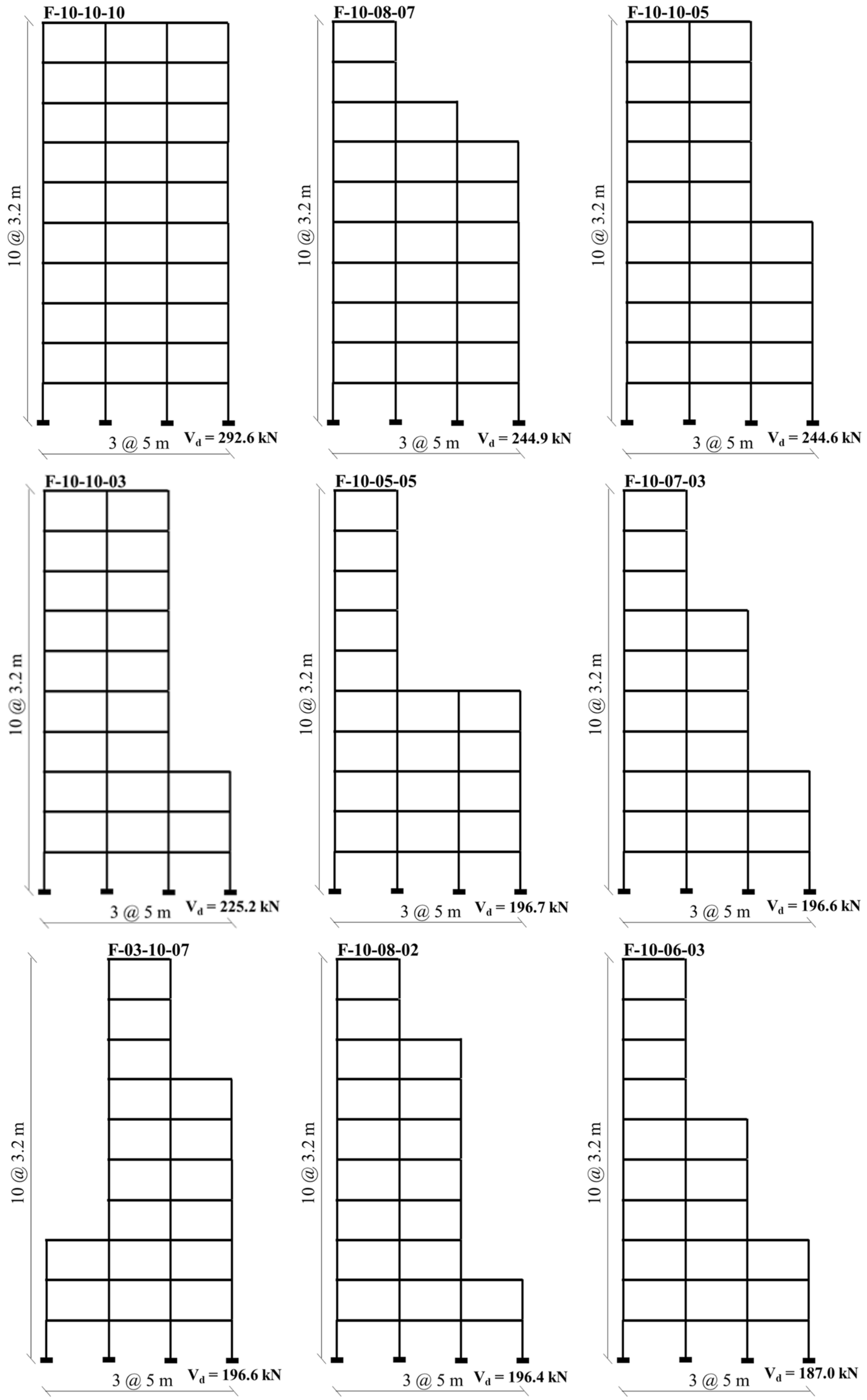


Fig. 2 Geometric configurations of the steel frames

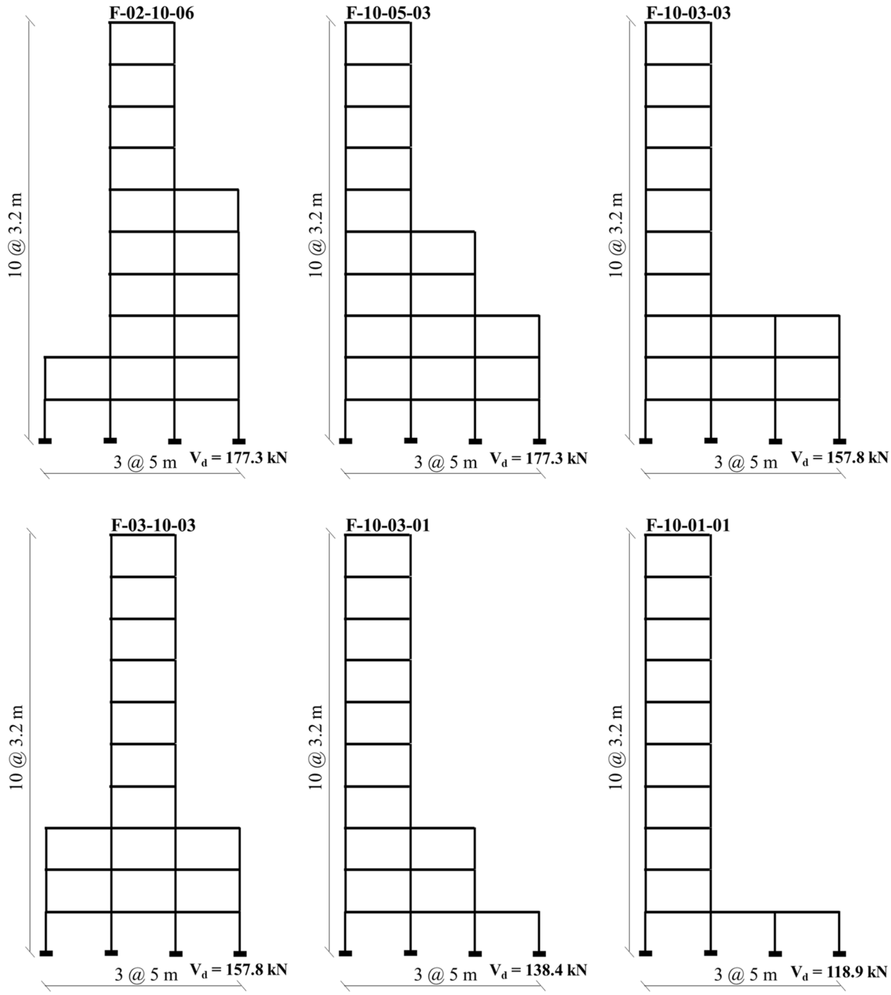


Fig. 2 (continued)

analytical solution for the NTHA was performed using the Newmark step-by-step numerical integration scheme, and the Rayleigh damping was used, with a damping ratio of 5% for the first and third modes of vibration (Chopra 2012).

4 Results and discussions

The new extension of the IUB is evaluated by comparing its outcomes to those of NTHA, which are calculated as the mean values of the maximum seismic responses to the set of predefined ground motions (Table 4), in terms of target displacements, total drifts, interstorey drifts, and plastic hinge rotations. For comparison purposes, results from the 1st

Table 3 Beams and columns of the studied frames

Building	Storeys	Columns	Beams	Mechanical properties
10-storey	1–4	C1	B1	$F_y = 235MPa$ $E = 200GPa$
	5–6	C1	B2	
	7–8	C2	B2	
	9–10	C2	B3	

E -Young Modulus; F_y – steel yield stress

Table 4 Details of the ground motion records

ID	Earthquake Name	Year	Station Name	Magnitude	Distance (km)
1	Parkfield	1966	Cholame—Shandon Array #8	6.19	12.9
2	Imperial Valley-06	1979	Delta	6.53	22.03
3	Imperial Valley-06	1979	El Centro Array #12	6.53	17.94
4	Victoria_ Mexico	1980	SAHOP Casa Flores	6.33	39.1
5	Morgan Hill	1984	Gilroy Array #3	6.19	13.01
6	Chalfant Valley-02	1986	Bishop—LADWP South St	6.19	14.38
7	Superstition Hills-02	1987	Brawley Airport	6.54	17.03
8	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	18.2
9	Landers	1992	Desert Hot Springs	7.28	21.78
10	Landers	1992	Mission Creek Fault	7.28	26.96
11	Big Bear-01	1992	San Bernardino	6.46	33.56
12	Kobe_ Japan	1995	Abeno	6.9	24.85
13	Kobe_ Japan	1995	Sakai	6.9	28.08
14	Gulf of Aqaba	1995	Eilat	7.2	43.29
15	Duzce_ Turkey	1999	Bolu	7.14	12.02
16	Tottori_ Japan	2000	TTR006	6.61	35.15
17	El Mayor-Cucapah_ Mexico	2010	Chihuahua	7.2	18.21
18	El Mayor-Cucapah_ Mexico	2010	MICHOACAN DE OCAMPO	7.2	13.21
19	Darfield_ New Zealand	2010	DFHS	7	11.86
20	Darfield_ New Zealand	2010	Kaiapoi North School	7	30.53
21	El Mayor-Cucapah_ Mexico	2010	Westside Elementary School	7.2	10.31

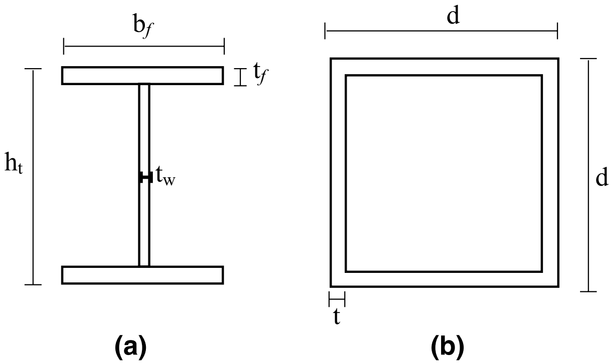


Fig. 3 Types of used sections, **a** beams, **b** columns (Behnamfar et al. 2016)

mode pushover, IUB (Rahmani et al. 2018), and OLLP (Habibi et al. 2019) procedures are also included.

4.1 Preliminary results

Table 5 presents the modal characteristics of the studied frames in terms of the period and the modal mass participation ratio for the first three modes of vibration, as well as the (q_i/

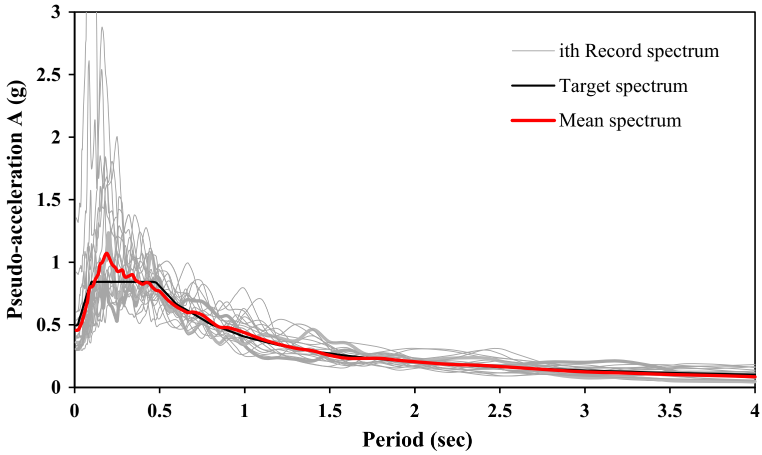
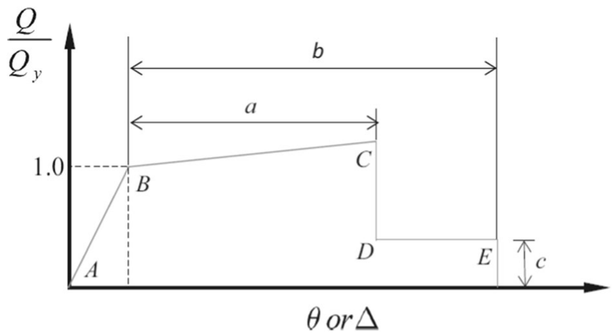


Fig. 4 Mean and individual scaled pseudo-acceleration responses spectra for the F-10-05-05 frame

Fig. 5 Generalised force deformation relationship for plastic hinges (FEMA 2000)



q_1) ratio. As seen in Table 5, the fundamental (first) period of the regular (reference) frame F-10-10-10 is the largest, while the lowest period corresponds to the F-10-05-05 frame (-23.5%). For the modal mass participation ratio, Table 5 illustrates the decrease of the first mode ratio for the setback frames compared to the regular one. The two frames F-10-03-03 and F-03-10-03 have the smallest percentage of the ratio (less than 50%). On the contrary, the increase in the modal mass participation ratio of the second mode is noticeable for the setback buildings. For instance, the mass ratio exceeds 25% for the four frames F-10-05-05, F-10-05-03, F-10-03-03 and F-03-10-03. The data in Table 5 are arranged in descending order according to the value of the third mode modal mass participating ratio. Seven frames have a ratio greater than 8.5%, while eight frames have a ratio smaller than 7.5%. The modal mass participation ratio of the third mode can reach 10% for the F-10-03-03 and F-10-03-01 frames.

The (q_i/q_1) ratio (Table 5) gives also information about the contribution of the second and third modes of vibration to the seismic response of the structures relative to the first mode. This ratio (Eq. (6)), which considers both the modal participation and displacement spectral amplitudes (S_{di}), shows that the contribution of the second mode is higher than the third mode for all the frames, with a ratio ranging from 15 to 30% (the maximum value

Table 5 Modal characteristics of the studied frames

Frame	Periods (Sec)			Modal Mass Participation Ratios (%)			q_i/q_1 (%)	
	Mode 1	Mode 2	Mode 3	Mode 1	Mode 2	Mode 3	q_2/q_1	q_3/q_1
F-10-03-03	1.43	0.55	0.36	49.80	29.30	10.30	24.01	7.36
F-10-03-01	1.47	0.54	0.34	54.70	21.80	10.00	20.33	6.04
F-03-10-03	1.41	0.55	0.36	49.60	30.00	9.90	24.33	7.33
F-02-10-06	1.34	0.61	0.33	62.10	17.50	9.70	24.68	5.31
F-10-07-03	1.33	0.59	0.36	65.90	16.00	8.80	26.84	6.45
F-03-10-07	1.33	0.59	0.36	66.00	15.90	8.80	26.32	6.52
F-10-06-03	1.31	0.61	0.34	61.20	20.80	8.50	26.73	5.45
F-10-08-02	1.43	0.55	0.35	68.60	13.20	7.20	22.52	7.41
F-10-10-03	1.57	0.59	0.36	67.80	17.50	6.40	17.53	4.64
F-10-01-01	1.61	0.55	0.31	61.50	12.20	6.20	15.22	3.71
F-10-05-03	1.32	0.61	0.34	55.40	27.50	6.20	26.43	5.50
F-10-08-07	1.40	0.56	0.36	76.90	9.40	5.30	24.86	7.15
F-10-10-05	1.52	0.63	0.34	70.90	16.40	4.20	18.89	3.88
F-10-10-10	1.70	0.61	0.35	77.00	11.80	4.10	14.40	3.38
F-10-05-05	1.30	0.65	0.33	59.90	25.30	3.90	29.06	4.35

belongs to the F-10-05-05 frame, and the minimum to the regular frame). However, the third mode to the first mode ratio $(q_3/q_1)_{UB}$ exceeds 5% for ten frames (Table 5), which the proposed method will take into account.

Table 6 Target displacement of the studied buildings

Frame	Target Displacement (cm)					Error (%)			
	NTHA	Mode 1	OLLP	IUB	EIUB	Mode 1	OLLP	IUB	EIUB
F-10-01-01	21.28	22.40	24.70	24.10	24.52	5.26	16.07	13.25	15.23
F-10-03-03	23.74	21.90	24.20	24.52	25.33	− 7.74	1.95	3.30	6.71
F-10-05-03	24.38	22.40	25.40	26.60	27.25	− 8.12	4.18	9.10	11.77
F-10-05-05	25.57	23.50	27.20	26.91	27.42	− 8.10	6.37	5.24	7.23
F-10-07-03	24.26	22.20	25.40	25.18	25.90	− 8.51	4.68	3.77	6.74
F-10-08-07	23.63	22.00	29.90	24.29	25.06	− 6.89	26.54	2.80	6.06
F-03-10-03	23.61	21.70	23.90	24.34	25.13	− 8.08	1.24	3.10	6.45
F-10-10-03	21.73	22.50	25.00	24.47	24.99	3.53	15.03	12.59	14.99
F-10-10-05	22.88	22.80	25.80	24.95	25.40	− 0.37	12.74	9.03	11.00
F-10-10-10	22.59	23.36	25.80	25.04	25.43	3.41	14.21	10.84	12.57
F-10-08-02	23.20	21.60	26.60	24.03	24.83	− 6.90	14.66	3.58	7.03
F-02-10-06	24.18	22.30	26.20	25.05	25.64	− 7.78	8.35	3.60	6.04
F-03-10-07	24.22	22.10	27.10	25.01	25.73	− 8.75	11.89	3.26	6.23
F-10-06-03	24.97	22.60	26.50	25.62	26.24	− 9.49	6.13	2.60	5.09
F-10-03-01	22.51	21.80	23.90	24.02	24.67	− 3.15	6.18	6.71	9.60

4.2 Target displacement prediction

Table 6 shows clearly that the conventional pushover analysis with first mode lateral load distribution underestimates the target displacement and gives, in most cases, negative error values with a maximum of 9.49%. The underestimation of the target displacement will affect the storey drift results (§4.4). The errors in estimating the target displacements range between 5% and 15.23% using the EIUB's target displacement formula (Eq. (12)), whereas the errors resulting from the OLLP procedure vary between 1.24% and 26.54%. Using Eq. (11), the IUB procedure gives a reasonable error for the target displacement less than that obtained by Eq. (12) or the OLLP procedure. For that, the use of Eq. (11) is preferable to calculate the target displacement even for the EIUB procedure. It should be noted that when using Eq. (11), the IUB and EIUB procedures give the same results in terms of target displacement.

4.3 Total drift prediction

The mean total drift profiles obtained by the NTHA plus and minus the standard deviation, denoted NTHA + STD and NTHA—STD, respectively, as well as those derived from 1st mode pushover, OLLP, IUB, and EIUB procedures, are presented in Fig. 6. Compared to the NTHA results, OLLP underestimates the total drift at lower storeys and overestimates it at upper-storeys in most cases. For six cases, the first mode pushover analysis underestimates total drift, particularly at upper-storeys. At lower storeys, the procedure can lead to a more accurate estimation than the OLLP procedure. The best results of the 1st mode pushover analysis are observed for the regular frame in which the 1st mode dominates the response according to Table 5, with a modal participating mass of 77%.

It can be noticed from Fig. 6 that the two procedures IUB and EIUB lead to a good and safer estimation of the response when including the two or three first modes of vibration, which can be excited by the geometric irregularity along the height of the buildings.

4.4 Storey drift prediction

Figure 7 presents the storey drifts predicted by the NTHA and the NSPs procedures, while Fig. 8 depicts the differences between the NSPs predictions and the reference values of the NTHA. Both figures confirm that the OLLP underestimates the response of all the studied frames except the regular frame F-10-10-10. The largest error in predicting the storey drift by the OLLP reached 126% for the F-10-08-07 frame. The first mode pushover analysis results are close to the NTHA results for the regular frame, but it fails to predict the response in most of the studied cases. It underestimates the lower storey drifts significantly, with a maximum error of 50.30% recorded in the F-10-05-03 frame. Moreover, this procedure underestimates also the response at the upper storey of all the setback frames. The IUB and EIUB procedures provide a good estimation of the response along with the height of the studied frames. The results obtained from the IUB are closer to those of the NTHA compared to the corresponding results of the OLLP and 1st mode pushover procedures. The storey drifts predicted by the IUB at the lower levels are satisfactory. However, IUB underestimates the response at the upper storeys of the setback frames, where the error exceeds—15% in nine cases. Figures 7 and 8 indicate that the prediction of the storey drifts has been improved using the EIUB compared to IUB results. For the upper storeys,

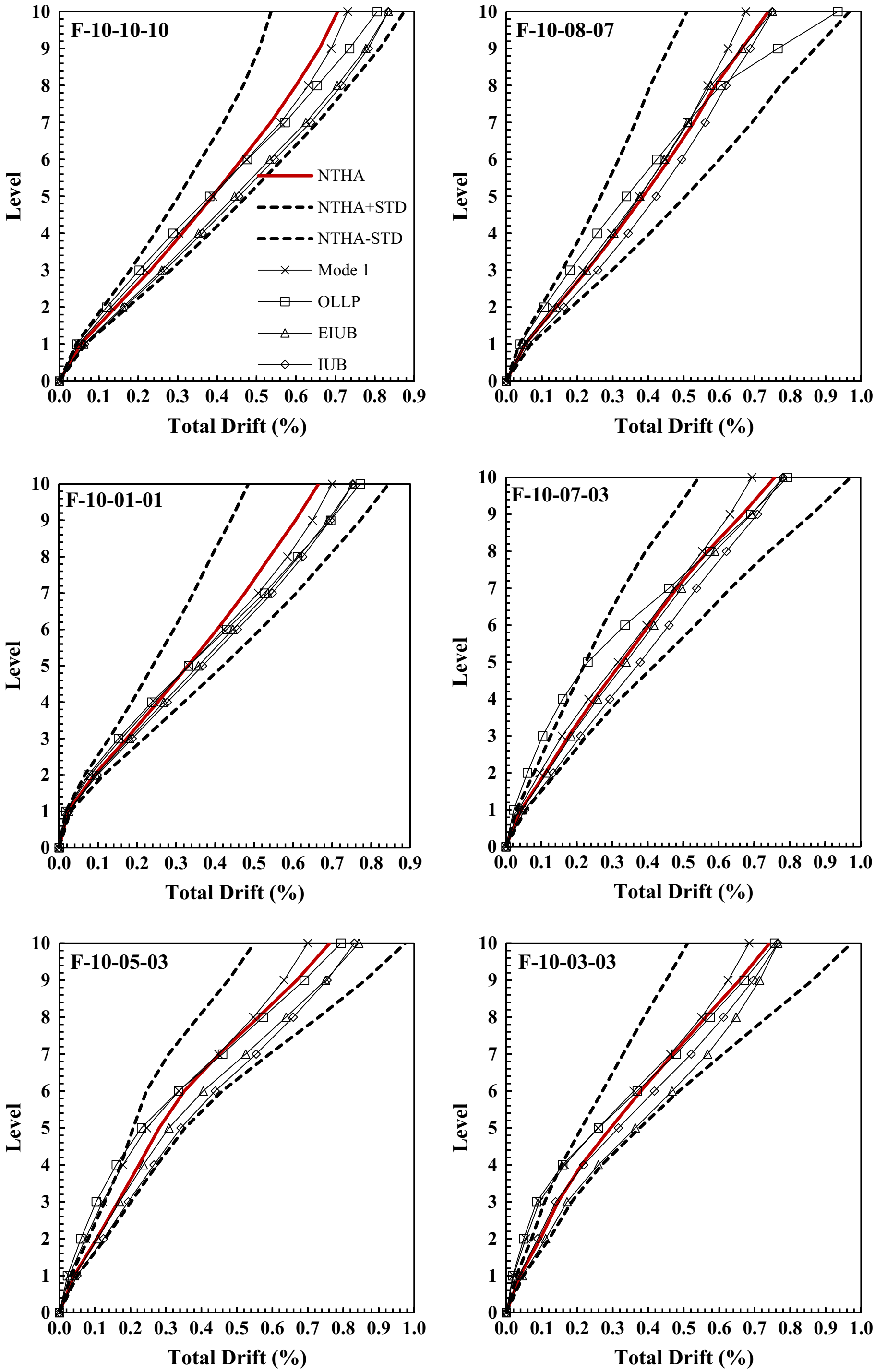


Fig. 6 Total drift of studied buildings

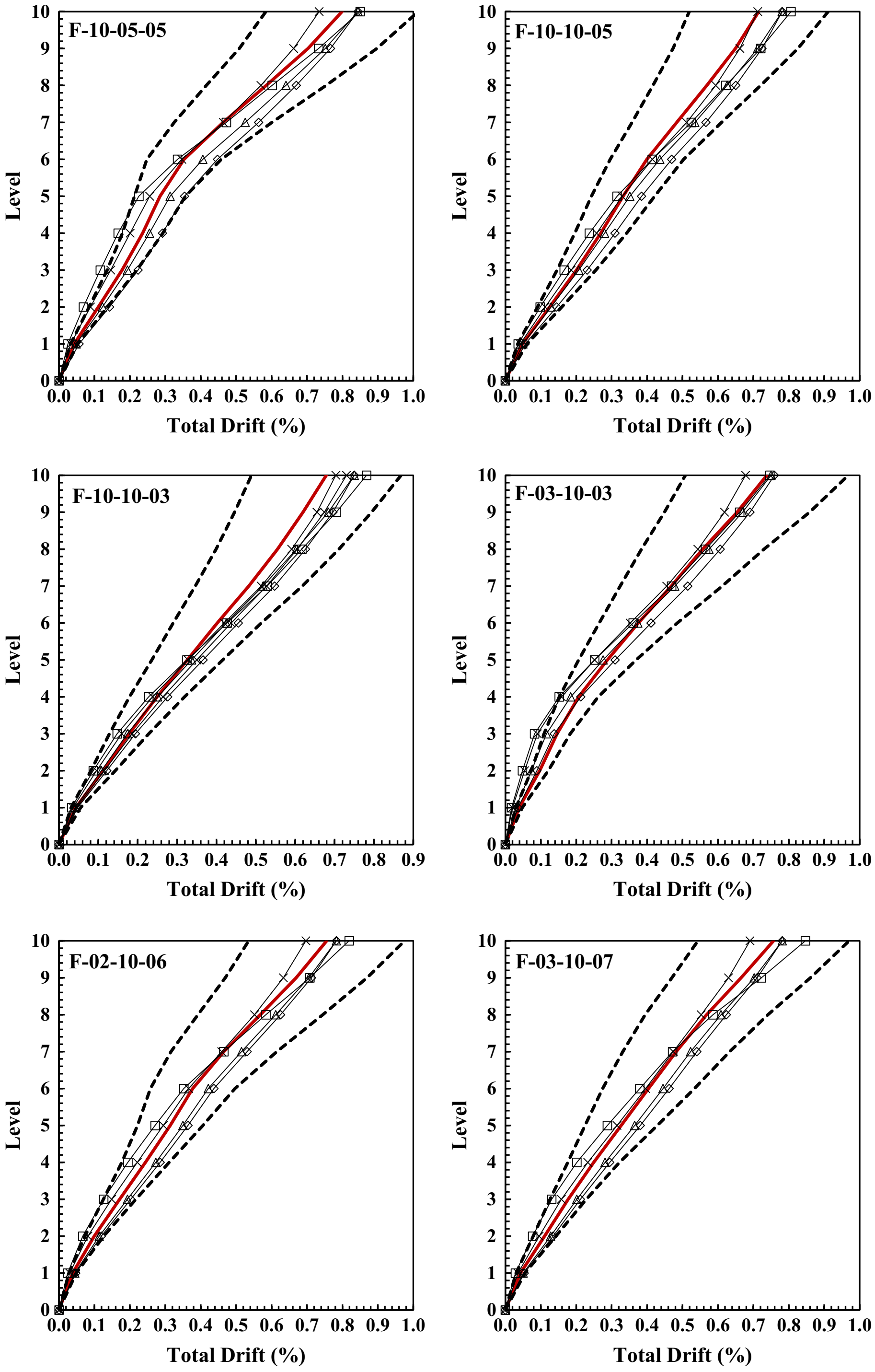


Fig. 6 (continued)

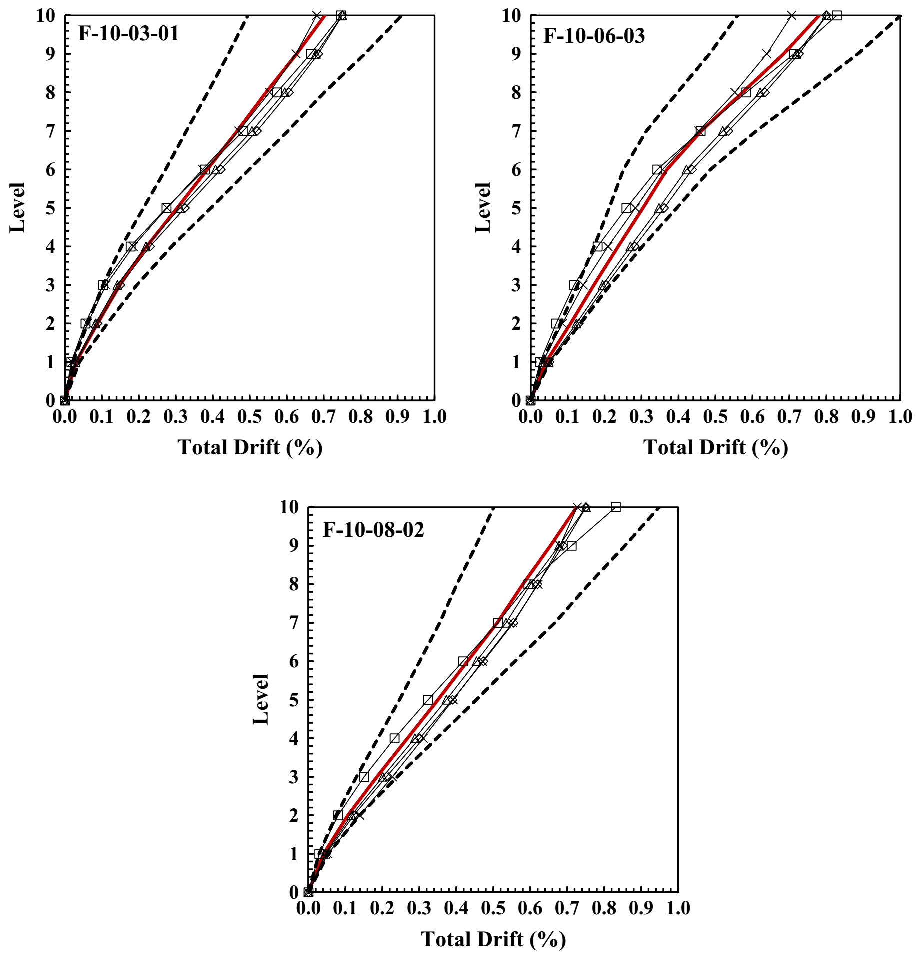


Fig. 6 (continued)

EIUB gives more conservative estimations of the storey drifts than the IUB procedure in all cases, particularly in frames with a high contribution of the third mode of vibration.

Figure 9 shows the mean absolute error of the storey drifts obtained by the NSPs regarding the NTHA results. The EIUB procedure provides more accurate estimates of the storey drift for all the setback frames compared to other NSPs. The maximum mean absolute error for the proposed procedure reached 18.71% in the case of the regular frame. This error remains less than 15% for the setback frames. The mean absolute errors of the OLLP predictions exceed 20% for all models, with a maximum error of 41.68% recorded for the F-10-08-07 model. In addition, the first-mode pushover analysis obtained less accurate estimations of storey drifts. The procedure cannot predict the response given large errors in three cases with an error higher than 20%. The IUB procedure gives errors less than the first-mode pushover analysis errors in five cases, with a maximum mean absolute error of 21.63% in the F-10-05-05 frame.

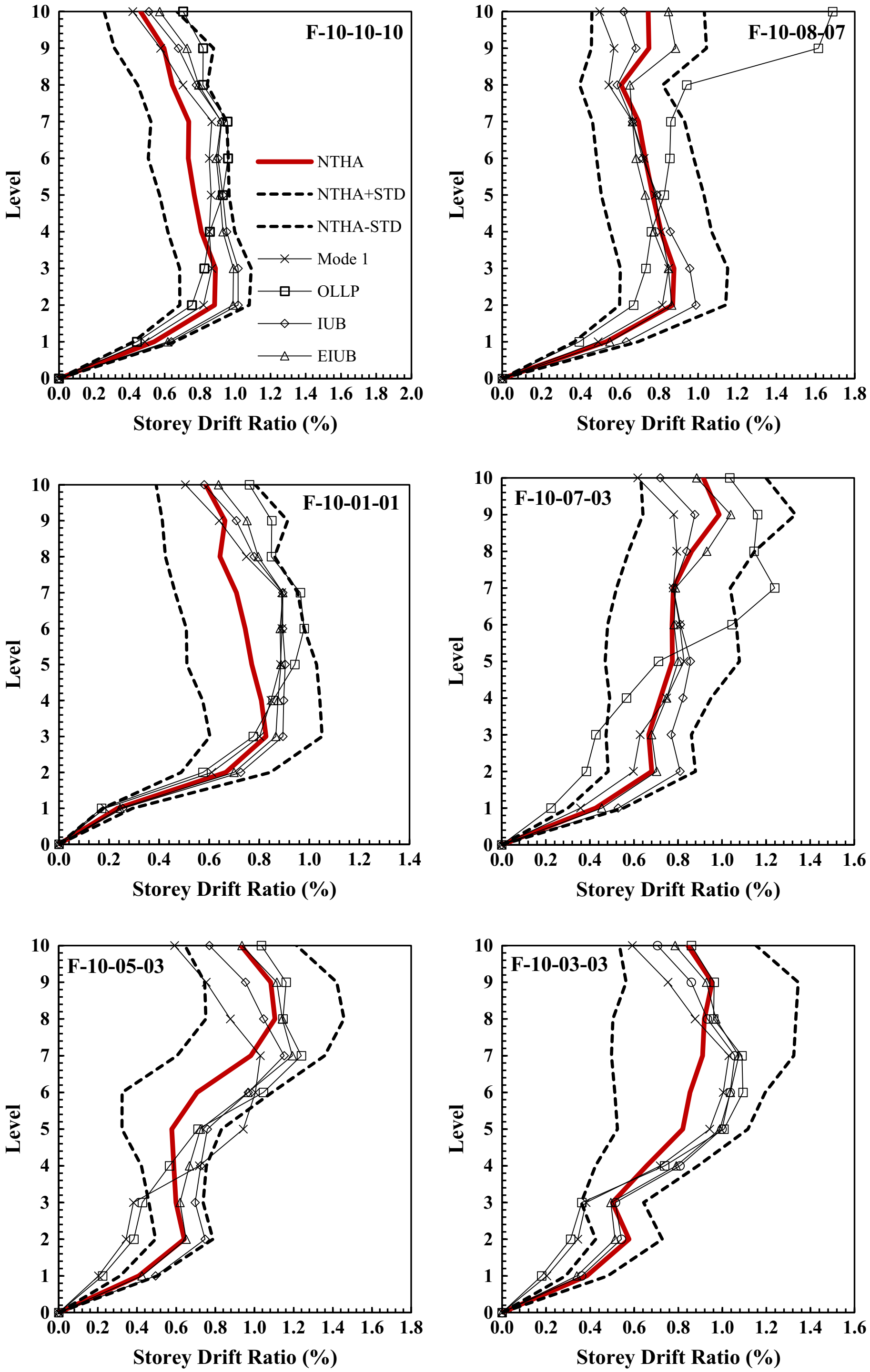


Fig. 7 Storey drift of studied buildings

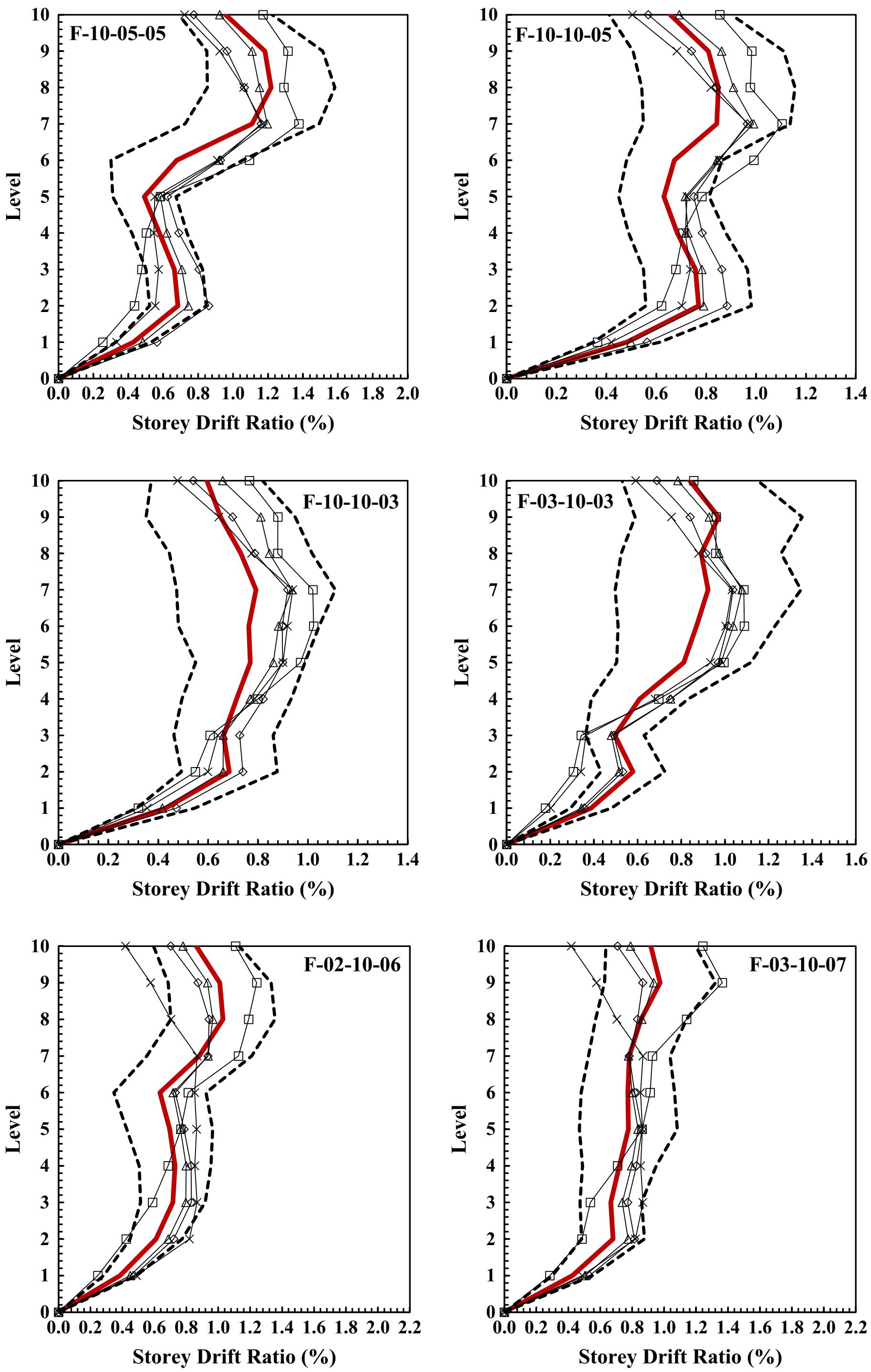


Fig. 7 (continued)

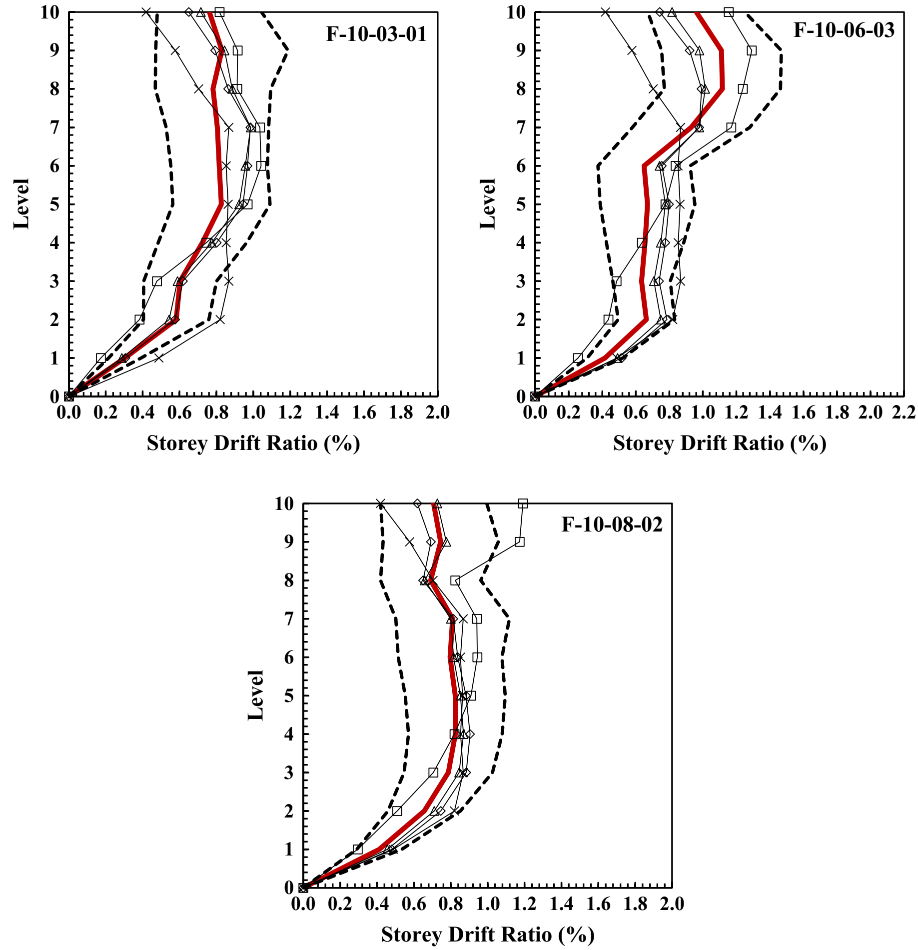


Fig. 7 (continued)

Inter-storey residual drifts given in Table 7 are evaluated using the NTHA for the six frames that endure some residual drifts. Low storey-drift residuals are observed for all frames. The maximum residual inter-storey drifts are recorded near the abrupt elevation discontinuities. The frame F-10-03-03 is the most affected with a maximum storey residual drift ratio of 1.25% and an average of the responses to all ground motions of 0.11%.

4.5 Plastic hinge rotations

The total plastic rotations of all hinges at each storey are computed for the EIUB and IUB procedures and compared to the mean values of the NTHA. This parameter is measured for only seven frames, which have a large contribution of the second and third modes of vibration in their responses (Table 5). As shown in Fig. 10, the storey plastic

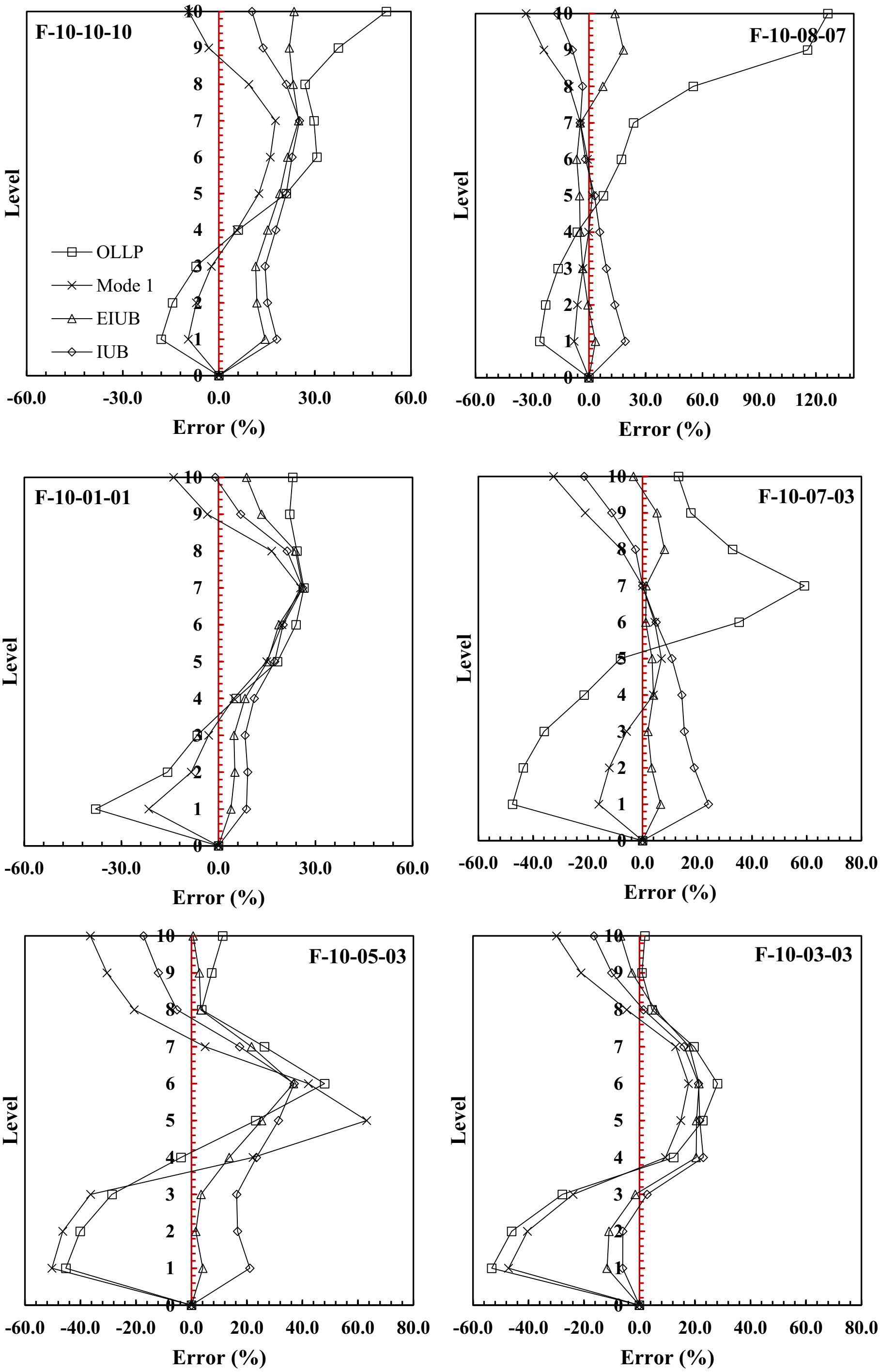


Fig. 8 Error in storey drift of studied buildings

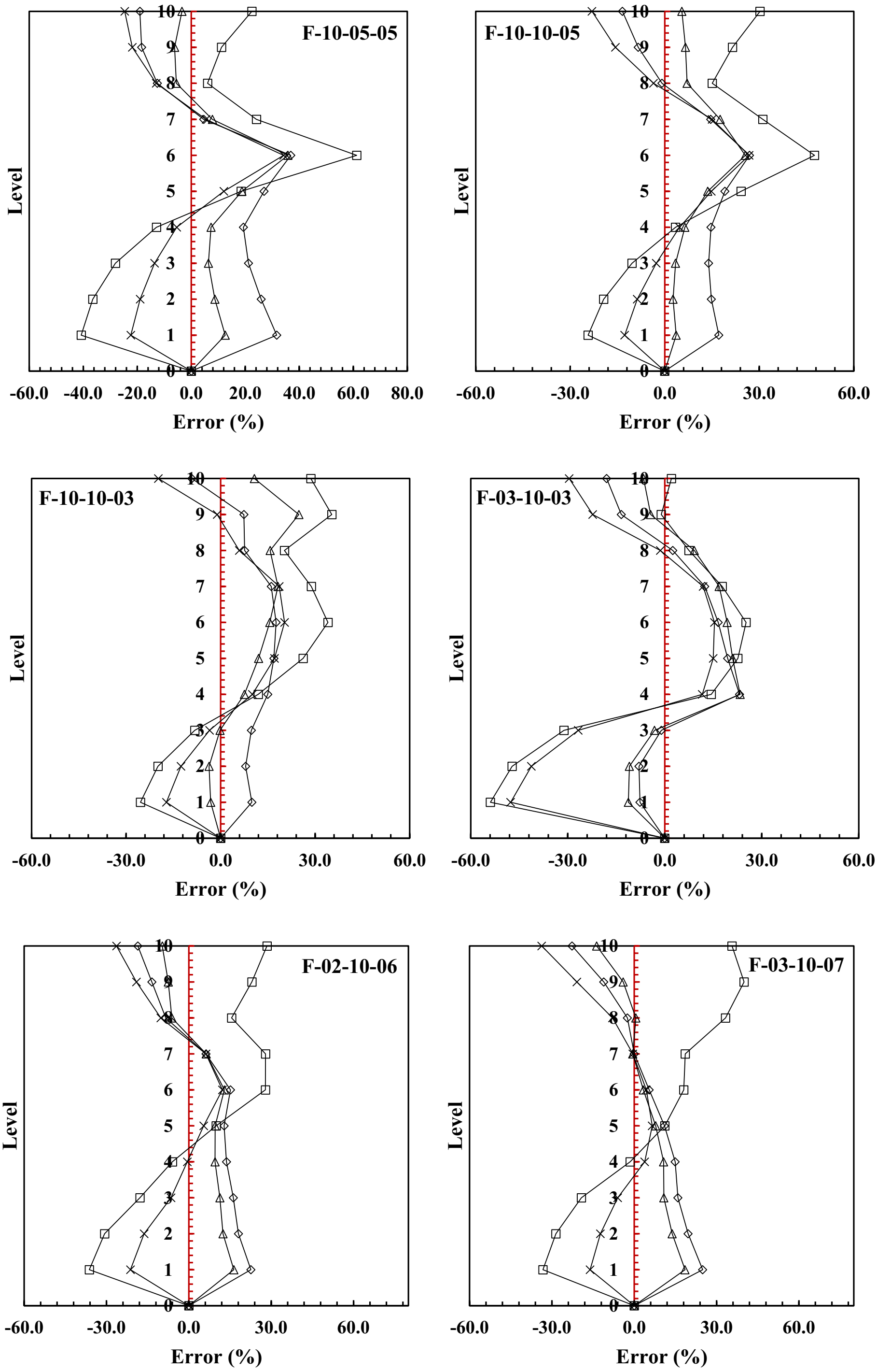
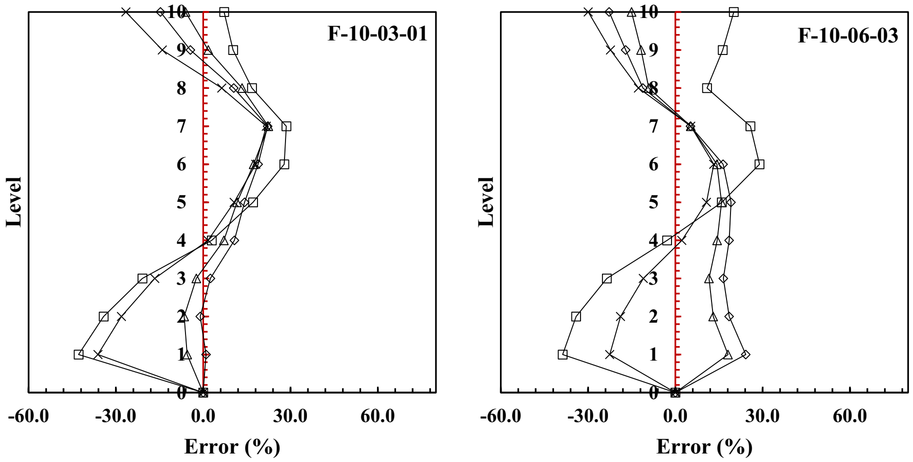


Fig. 8 (continued)



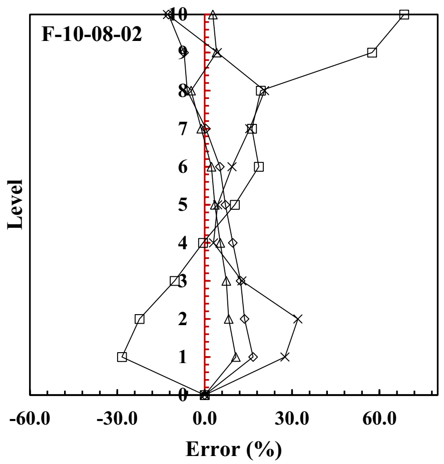


Fig. 8 (continued)

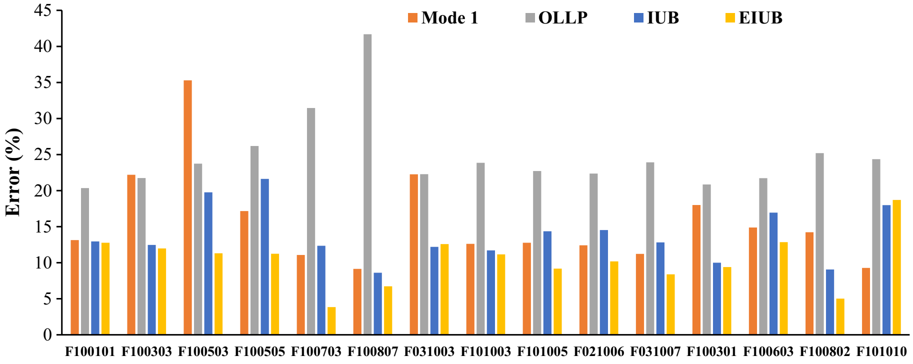


Fig. 9 Mean absolute error of the storey drifts for the studied buildings

Table 7 Mean and maximum residual inter-story drift

Frame	The inter-storey residual drift ratio (%)		Level
	Mean	Maximum	
F-10-03-01	0.08	0.23	4rd storey
F-10-03-03	0.11	1.25	4rd storey
F-03-10-03	0.06	0.65	4rd storey
F-10-07-03	0.04	0.32	8th storey
F-03-10-07	0.05	0.38	8th storey
F-02-10-06	0.04	0.31	9th storey

hinge rotations computed using the IUB and EIUB procedures are very close at lower storeys with no clear trend, as they tend to overestimate in some frames and underestimate in others. At upper storeys, IUB and EIUB procedures underestimate the plastic hinge rotations in all cases. The IUB procedure records the maximum error (40%) in the frame F-10-03-03. A steady improvement of the EIUB can be noticed at the upper storeys that tend to reduce the error for each case by about 12%. Here, it can be concluded that the advantage of the EIUB appears on the upper floors, where the higher modes affect most.

5 Conclusion

In this investigation, an extension of the so-called improved upper bound pushover analysis is developed to assess the seismic behaviour of setback buildings. The new single-run procedure uses the first three modes of vibration to create the upper-bound invariant lateral load vector. This vector load is moderated by a uniform load distribution to control the response at upper and lower storeys. The target displacement in this procedure is calculated by adjusting the target displacement corresponding to the first-mode pushover analysis to account for the second and third modes of vibration. Fourteen 2D setback mid-rise steel frames were studied to validate the developed procedure. NTHA with other single-run NSPs procedures (first mode, OLLP, and IUB) were used for comparison purposes. Within the limit of the small sample size of setback configurations, the evidence from this study suggests the following outcomes:

- The setback models considered in the present study show a tendency for a higher modal participating mass ratio of the second mode compared to the regular (reference) frame, and a consistent range of 5% to 10% for the third mode. This tendency is maintained for the (q_i/q_1) ratio, which integrates the spectral displacement amplitudes.
- Because the first mode of vibration dominates the response in regular mid-rise buildings, the first mode pushover analysis gives better results than the other advanced single-run procedures. However, the proposed procedure can give more conservative results in this case.

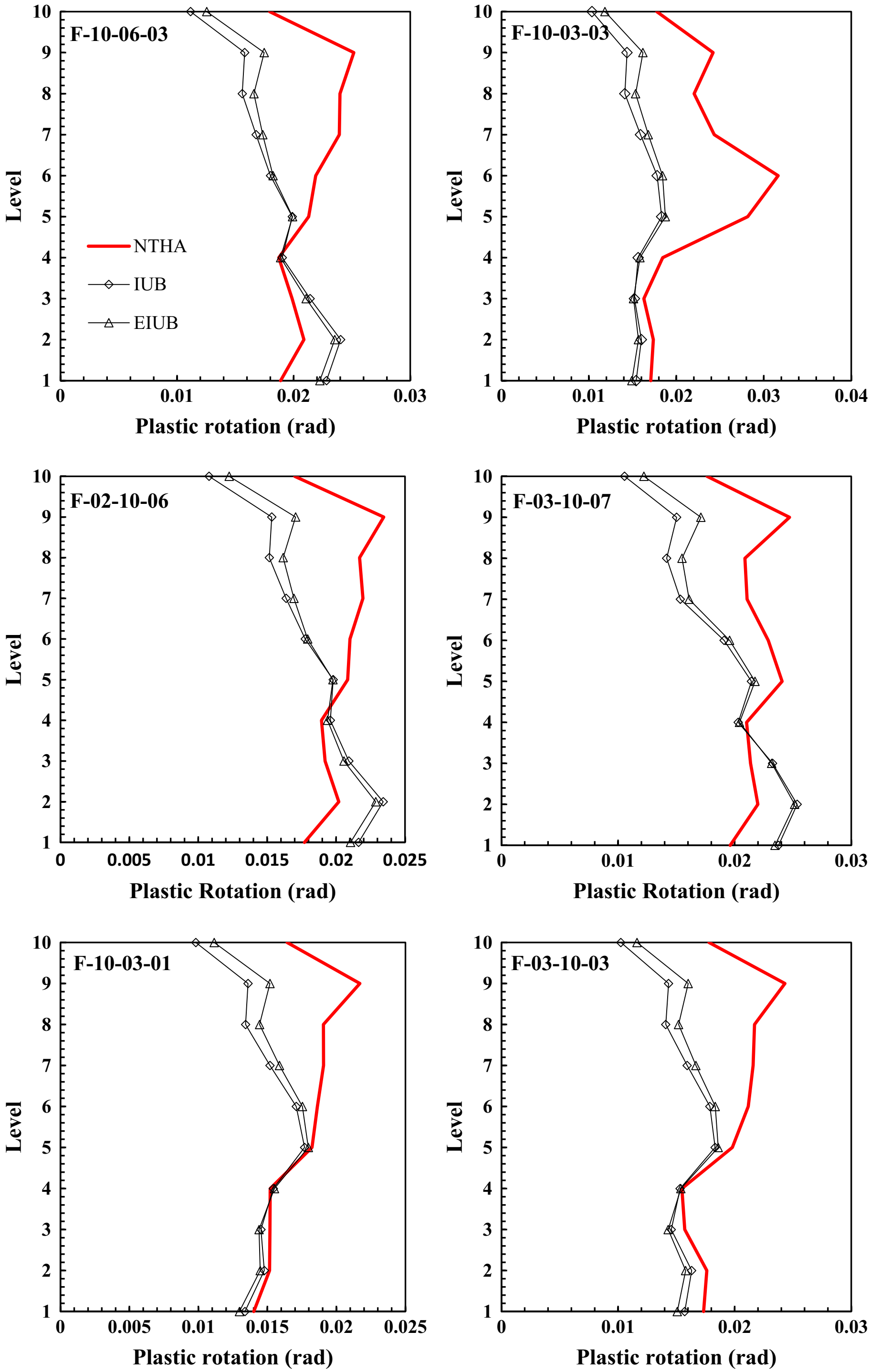


Fig. 10 Total plastic hinge rotations

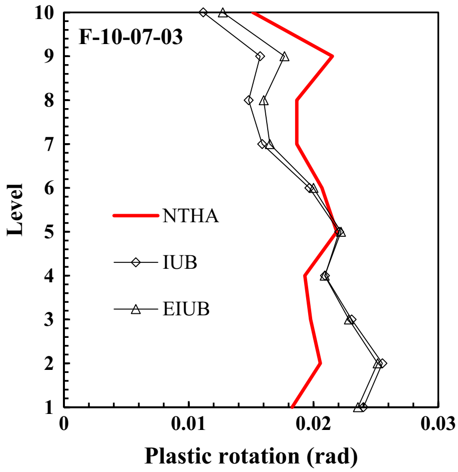


Fig. 10 (continued)

- The storey drifts predicted by the IUB are closer to those from NTHA compared to the corresponding results of the OLLP and 1st mode pushover procedures. However, IUB underestimates the response at the upper storeys of the setback frames.
- The proposed EIUB procedure improved the results of the IUB procedure, as it included the third mode of vibration of the setback frames. The effect is mostly felt at the upper storeys, and it gave better results than the other pushover procedures used in this study.
- The storey plastic hinge rotations estimated by the EIUB shows evident improvement compared to the IUB, even though both procedures underestimate the response at upper-storeys.

It is worth noting that the findings of the present study were obtained for a limited number of frames with different setback configurations. However, in order to generalise the conclusions of this study, other analyses should be done for various types of irregularities.

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Data availability The data that support the findings of this study are available from the corresponding author, Abdallah Yacine Rahmani, upon reasoned request.

Declarations

Conflict of interest The authors declare that there is no conflict of interests regarding the publication of this paper.

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