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Seismic response of plane steel MRF with setbacks: Estimation of inelastic deformation demands

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Abstract

An extensive parametric study on the inelastic seismic response of plane steel moment resisting frames (MRF) with setbacks is presented. A family of 120 such frames, designed according to the European seismic and structural codes, are subjected to an ensemble of 30 ordinary (i.e. without near-fault effects) earthquake ground motions scaled to different intensities in order to drive the structures to different limit states. The statistical analysis of the created response databank indicates that the number of stories, beam-to-column strength ratio, geometrical irregularity and limit state under consideration strongly influence the heightwise distribution and amplitude of inelastic deformation demands. Nonlinear regression analysis is employed in order to derive simple formulae which reflect the aforementioned influences and offer, for a given strength reduction (or behaviour) factor, three important response quantities, i.e. the maximum roof displacement, the maximum interstorey drift ratio and the maximum rotation ductility along the height of the structure. A comparison of the proposed method with the procedures adopted in current seismic design codes reveals the accuracy and efficiency of the former.

Keywords: Setbacks; Vertical irregularity; Deformation demands; Steel MRF; Seismic design; Seismic assessment

1. Introduction

A common type of geometrical irregularity in building structures, inherited from various architectural requirements, is the presence of setbacks, i.e. the presence of abrupt reductions of the floor area at specific levels of the elevation. The increasing number of damage statistics after seismic events has provided strong evidence that setback buildings exhibit inadequate behaviour though they were designed according to the current state of knowledge existing in seismic codes. This inferior seismic performance has been attributed to the combined action of structural irregularities, i.e., to the combined non-uniform distributions of mass, stiffness and strength along the height of setback frames. Therefore, simple yet effective procedures to estimate seismic inelastic deformation demands (i.e. damage) in setback frames are certainly needed. Moreover, in order for the structural irregularity to be taken into account in the early stages of the building design, these procedures should be adjusted to the framework of current seismic design codes that mainly employ the elastic static or the elastic dynamic (response spectrum) analysis.

The lower level of a setback frame (with the largest number of bays) is usually termed as the "base", while the upper level (with the smallest number of bays) as the "tower". It has been found [1,2] that interstorey drifts of the tower are larger than those of the corresponding regular structure at the same height level, whereas the opposite is true for the base. Based on both analytical and experimental studies, Shahrooz and Moehle [3] observed concentration of damage in members near setbacks or in the tower. Wong and Tso [4] studied the elastic response of setback structures and found that the first mode is capable of representing the displacement response. According to the study of Pinto and Costa [5], the response of regular and setback structures is similar. Duan and Chandler [6] concluded that both static and dynamic analyses for design are inadequate from preventing concentration of damage in members near the setback level and they identified the need for imposing increased strength on the tower. Mazzolani and

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Piluso [7] presented extensive numerical investigations aimed at evaluating the differences in the behavior factor of setback and corresponding regular frames. The main conclusion of their work is that the presence of setbacks does not lead to significant worsening of the seismic response, i.e. to significant decrease of the behaviour factor, especially for structures designed to develop a collapse mechanism of global type. Based on field evidence, Kappos and Scott [8] did not arrived at definite conclusions regarding the possible locations of concentration of damage in setback frames. Chen et al. [9] pointed out that the tower experiences a local vibration mode resulting in higher mode effects. Romao et al. [10] found that setback buildings exhibit an adequate seismic performance when compared to the regular ones, while Tena-Colunga [11] did not identify undesirable concentration of plastic deformation in the neighborhood of the setback. Khoury et al. [12] identified excessive response in the upper tower stories, especially when the structure is irregular both in plan and elevation. The above literature review reveals that the seismic behaviour of setback frames is a rather controversial issue since some works [5,7,10,11] indicate adequate seismic performance, while some others [3,6,12] show the opposite for those frames. Therefore, more research work is needed in order to better understand the elastic and inelastic seismic response of setback frames.

The current practice [13,14] for estimating maximum deformations of building structures adopts procedures which use equivalent single-degree-of-system (SDOF) systems. The characteristics of the SDOF system are established by using the results of a pushover analysis and hence, the aforementioned procedures are more suitable for seismic evaluation of pre-designed structures and not for the design of new ones.

Current seismic codes, such as the EC8 [13], provide criteria for the definition of vertical irregularity due to the presence of setbacks and this distinction between regular and irregular structures has implications on the method of analysis of irregular structures (a response spectrum analysis is mandatory) and on the value of the behaviour factor (a 20% decrease of the q factor is dictated). Nevertheless, the rules adopted by seismic codes for estimating deformation demands apply both to regular and irregular in elevation buildings. Specifically, maximum displacements and interstorey drifts are calculated by multiplying their yield values under the reduced design lateral forces, with the strength reduction (or behaviour) factor, q, meaning that the equal-displacement rule is assumed to be valid. Furthermore, the above multiplication assumes that the shape of the maximum displacement and interstorey drift profiles remain constant during the seismic excitation. The validity of both assumptions has not yet been checked for steel MRF with setbacks, while an alternative to the equal-displacement rule has not yet been presented.

In the displacement-based design (DBD) procedure [15] there exists correlation between the maximum roof displacement, $u_{r,\max}$, and the maximum interstorey drift ratio, $IDR_{\max}$, along the height of the structure. To the best of authors' knowledge, a correlation study between $IDR_{\max}$ and $u_{r,\max}$ for steel MRF with setbacks has not as yet been presented.

The scope of the paper is to examine and quantify the influence of changes in structural (setback configuration, number of stories, beam-to-column strength ratio) and ground motion characteristics on the heightwise distribution and amplitude of inelastic deformation demands in plane steel MRF with setbacks. For that purpose, a family of 120 plane setback steel MRF are subjected to an ensemble of 30 ordinary (i.e. without near-fault effects) ground motions scaled to different intensities in order to drive the structures to different damage levels. The central tendency of the heightwise distribution of drift demands is examined in detail. Then, nonlinear regression analysis is employed in order to derive simple formulae which offer a direct estimation of inelastic deformation demands. Emphasis was given to the ability of the proposed formulae to be adjusted to the framework of design methods (FBD [13] and DBD [15]) which adopt elastic analysis. In detail, the paper offers, for a given strength reduction (or behaviour) factor, three valuable response quantities, i.e. the maximum roof displacement, the maximum interstorey drift ratio and the maximum rotation ductility, μ_θ, along the height of the structure. The strength reduction factor refers to the point of the development of the first plastic hinge in the building and thus, pushover analysis and estimation of the overstrength factor are not required. A comparison of the proposed method with the procedures adopted in current seismic design codes reveals the efficiency of the former.

2. Plane setback steel moment resisting frames used in this study

The study is based on frames which are plane and orthogonal with storey heights and bay widths equal to 3 m and 5 m, respectively. It should be pointed out that a bay width from 4 to 6 m is the usual case in European practice but quite low compared to that of the American practice. The frames (Fig. 1) are low-to-mid-rise (number of storeys, n_s, with values 3, 6 and 9) and represent 40 different geometrical irregularities due to the presence of setbacks with 33.3% (one missing bay) and 66.6% (two missing bays) reductions of the floor area. It is observed (Fig. 1) that the study covers buildings: (a) with large setbacks in the upper floors (e.g. frame No. 27); (b) tower-like structures with one large setback in their lowest part (e.g. frame No. 20) and (c) buildings with setbacks occurring in various height levels (e.g. frame No. 13). Following the footsteps of Mazzolani and Piluso [7], this work aims at describing and quantifying the irregularity due to the presence of setbacks through simple geometrical indices Φ_s and Φ_b which, with reference to Fig. 2, are given by the following formulae

$$\Phi_s = \frac{1}{n_s - 1} \cdot \sum_{i=1}^{i=n_s-1} \frac{L_i}{L_{i+1}} \tag{1}$$

$$\Phi_b = \frac{1}{n_b - 1} \cdot \sum_{i=1}^{i=n_b-1} \frac{H_i}{H_{i+1}} \tag{2}$$

where n_s is the number of stories of the frame and n_b is the number of bays of the first storey of the frame. A large value of

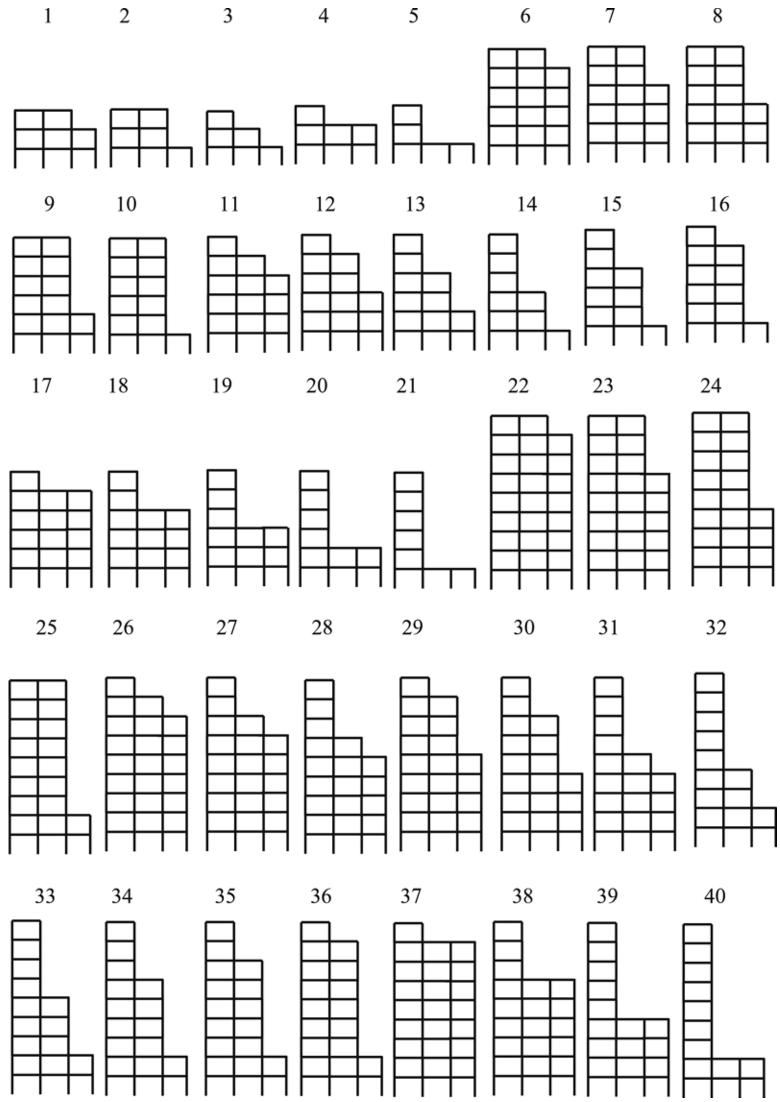

Fig. 1. Geometries of the setback steel MRF considered in this investigation.

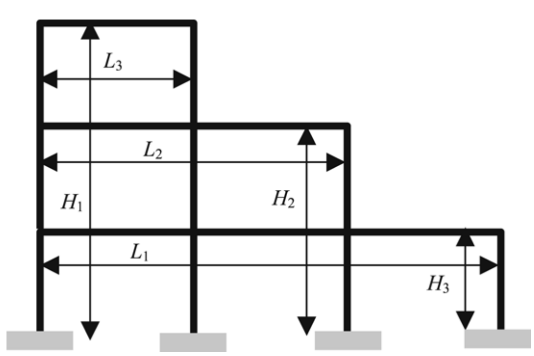

Fig. 2. Geometry of frame with setbacks for definition of geometrical irregularity indices.

the Φ_s index reveals large reductions of the floor area, a large value of the Φ_b index reveals a tower-like structure, while for the extreme case of a regular frame without setbacks both of the above indices take their minimum value, i.e. unity.

The frames are designed in accordance with the structural Eurocodes EC3 [16] and EC8 [13] on the basis of an automated computer-aided [17] response spectrum analysis and design procedure. Commercially available cross-sections [18] (HEB profiles for the columns and IPE profiles for the beams) are used in order to avoid discrepancies between strength and stiffness, i.e. the use of generic frames is avoided. The yield stress of the material is set equal to 235 MPa, while gravity load on the beams is assumed equal to 27.5 kN/m (dead and live loads of the floors). The expected earthquake ground motion is defined by the design spectrum of the EC8 [15] with peak ground acceleration, PGA, equal to 0.4g and soil class B. The design process of the frames resulted in optimum cross-sections of the columns which satisfy both the requirements for strength/stiffness [16] and the capacity design rule [13]. For each of the frames, the column cross-sections were subsequently increased two times in order to obtain three different values of the column-to-beam strength ratio, a, defined as [19]

$$a = \frac{M_{RC,1,av}}{M_{RB,av}} \tag{3}$$

Table 1
Data pertinent to the setback steel MRF considered in this investigation

Geom.	Φ_s	Φ_b	α	T (s)	Geom.	Φ_s	Φ_b	α	T (s)	Geom.	Φ_s	Φ_b	α	T (s)
1	1.25	1.25	1.31	0.67	14	1.30	2.50	2.11	0.79	27	1.19	1.23	3.05	1.21
1	1.25	1.25	1.90	0.59	14	1.30	2.50	2.83	0.74	28	1.19	1.35	2.32	1.25
1	1.25	1.25	2.67	0.54	15	1.30	2.75	1.64	0.85	28	1.19	1.35	2.57	1.22
2	1.25	2.00	1.31	0.66	15	1.30	2.75	2.11	0.80	28	1.19	1.35	3.10	1.18
2	1.25	2.00	1.90	0.59	15	1.30	2.75	2.83	0.75	29	1.19	1.36	2.28	1.34
2	1.25	2.00	2.67	0.54	16	1.30	3.10	1.79	0.98	29	1.19	1.36	2.53	1.30
3	1.75	1.75	1.31	0.56	16	1.30	3.10	2.31	0.91	29	1.19	1.36	3.05	1.26
3	1.75	1.75	1.90	0.50	16	1.30	3.10	3.09	0.85	30	1.19	1.52	2.28	1.26
3	1.75	1.75	2.67	0.45	17	1.40	1.10	1.90	1.01	30	1.19	1.52	2.53	1.23
4	2.00	1.25	1.31	0.59	17	1.40	1.10	2.44	0.94	30	1.19	1.52	3.05	1.19
4	2.00	1.25	1.90	0.52	17	1.40	1.10	3.28	0.88	31	1.19	1.53	2.32	1.24
4	2.00	1.25	2.67	0.47	18	1.40	1.25	1.67	0.88	31	1.19	1.53	2.57	1.20
5	2.00	2.00	1.31	0.57	18	1.40	1.25	2.16	0.82	31	1.19	1.53	3.10	1.17
5	2.00	2.00	1.90	0.51	18	1.40	1.25	2.89	0.76	32	1.19	2.13	2.43	1.32
5	2.00	2.00	2.67	0.46	19	1.40	1.50	1.67	0.84	32	1.19	2.13	2.70	1.29
6	1.10	1.10	1.79	1.07	19	1.40	1.50	2.16	0.78	32	1.19	2.13	3.25	1.25
6	1.10	1.10	2.31	0.99	19	1.40	1.50	2.89	0.73	33	1.19	2.15	2.32	1.28
6	1.10	1.10	3.09	0.93	20	1.40	2.00	1.93	0.77	33	1.19	2.15	2.57	1.24
7	1.10	1.25	1.79	1.02	20	1.40	2.00	2.15	0.75	33	1.19	2.15	3.10	1.21
7	1.10	1.25	2.31	0.95	20	1.40	2.00	2.59	0.72	34	1.19	2.25	2.32	1.28
7	1.10	1.25	3.09	0.89	21	1.40	3.50	1.90	0.83	34	1.19	2.25	2.57	1.25
8	1.10	1.50	1.64	0.95	21	1.40	3.50	2.11	0.81	34	1.19	2.25	3.10	1.21
8	1.10	1.50	2.11	0.90	21	1.40	3.50	2.54	0.77	35	1.19	2.39	2.28	1.31
8	1.10	1.50	2.83	0.84	22	1.06	1.06	2.15	1.50	35	1.19	2.39	2.53	1.28
9	1.10	2.00	1.64	0.97	22	1.06	1.06	2.39	1.45	35	1.19	2.39	3.05	1.24
9	1.10	2.00	2.11	0.92	22	1.06	1.06	2.88	1.41	36	1.19	2.56	2.28	1.40
9	1.10	2.00	2.83	0.86	23	1.06	1.25	2.18	1.43	36	1.19	2.56	2.53	1.36
10	1.10	3.50	1.64	1.01	23	1.06	1.25	2.42	1.38	36	1.19	2.56	3.05	1.33
10	1.10	3.50	2.11	0.96	23	1.06	1.25	2.92	1.34	37	1.25	1.06	2.15	1.43
10	1.10	3.50	2.83	0.90	24	1.06	1.63	2.06	1.38	37	1.25	1.06	2.39	1.39
11	1.30	1.23	1.79	0.94	24	1.06	1.63	2.29	1.34	37	1.25	1.06	2.88	1.34
11	1.30	1.23	2.31	0.88	24	1.06	1.63	2.76	1.30	38	1.25	1.25	2.28	1.26
11	1.30	1.23	3.09	0.82	25	1.06	2.75	2.06	1.45	38	1.25	1.25	2.53	1.23
12	1.30	1.43	1.64	0.87	25	1.06	2.75	2.29	1.41	38	1.25	1.25	3.05	1.19
12	1.30	1.43	2.11	0.82	25	1.06	2.75	2.76	1.36	39	1.25	1.63	2.32	1.25
12	1.30	1.43	2.83	0.77	26	1.19	1.13	2.18	1.39	39	1.25	1.63	2.57	1.22
13	1.30	1.75	1.64	0.82	26	1.19	1.13	2.42	1.35	39	1.25	1.63	3.10	1.18
13	1.30	1.75	2.11	0.77	26	1.19	1.13	2.92	1.31	40	1.25	2.75	2.28	1.39
13	1.30	1.75	2.83	0.72	27	1.19	1.23	2.28	1.29	40	1.25	2.75	2.53	1.35
14	1.30	2.50	1.64	0.83	27	1.19	1.23	2.53	1.25	40	1.25	2.75	3.05	1.31

where $M_{RC,1,av}$ is the average of the plastic moments of resistance of the columns of the first storey and $M_{RB,av}$ is the average of the plastic moments of resistance of the beams of all the stories of the frame. For a given level of ductility and for given cross-sections of beams, an increase of a will delay the mechanism action (simultaneous appearance of plastic hinges at the end of beams and at the base of the columns of the first storey).

The aforementioned process led to a family of 40 (geometrical configurations) * 3 (values of the parameter a) = 120 plane steel MRF with setbacks. Data of the frames, including the geometrical configurations and values for Φ_s, Φ_b, α and fundamental periods of vibration, T, are presented in Table 1.

3. Seismic analyses and response databank

The family of the buildings described in Section 2 of the paper was subjected to an ensemble of 30 ordinary ground motions selected from the PEER [20] ground motion database. The term ordinary excludes ground motions which are characterized by distinct pulses in their velocity and displacement time histories. The moment magnitude, M_w, the closest distance to the causative fault, D, the characteristic period, T_c, and the peak ground acceleration, PGA, of the 30 ground motions are presented in Table 2. The characteristic period T_c was calculated by employing the iterative algorithm of Riddell and Newmark [21] that divides the response spectrum into three period ranges: constant spectral displacement (long periods), constant spectral acceleration (short periods) and constant spectral velocity (intermediate periods).

The well-known program DRAIN-2DX [22] was used for performing nonlinear dynamic analyses. The analytical models of the frames were centreline representations in which inelastic behaviour was modelled by means of bilinear (hysteretic) point plastic hinges with 3% hardening [23]. Therefore, the

Table 2
Data pertinent to the ordinary earthquake ground motions considered in this investigation

Event	Station	M_w	D (km)	PGA (m/s^2)	T_c (s)
Kern country 1952/07/21	Taft	7.7	43	1.74	0.33
San Fernardo 1971/02/09	Castaic	6.6	29	2.63	0.50
Imperial Valley 1979/10/15	Calexico	6.6	15	2.70	0.25
Imperial Valley 1979/10/15	Delta	6.5	44	3.44	0.60
Coalinga 1983/05/02	Cantua Creek School	6.4	26	2.75	0.60
Loma Prieta 1989/10/18	Gilroy Array #4	6.9	16	4.09	0.40
Loma Prieta 1989/10/21	Coyote Lake Dam (SW Abut)	6.9	22	4.75	0.50
Loma Prieta 1989/10/18	SF Intern. Airport	6.9	64	3.23	0.70
Landers 1992/06/28	Desert Hot Springs	7.3	23	1.68	0.35
Northridge 1994/01/17	LA - Centinela St	6.7	31	3.15	0.55
Northridge 1994/01/22	Castaic - Old Ridge Route	6.7	23	5.58	0.40
Northridge 1994/01/17	Hollywood - Willoughby Ave	6.7	26	2.41	0.90
Northridge 1994/01/17	LA - N Faring Rd	6.7	24	2.68	0.60
Northridge 1994/01/17	LA – Hollywood Stor FF	6.7	26	3.52	0.35
Northridge 1994/01/17	Glendale – Las Palmas	6.7	25	2.02	0.20
Northridge 1994/01/24	LA – Chalon Rd	6.7	24	2.21	0.60
Northridge 1994/01/18	Moorpark - Fire Sta	6.7	28	2.86	0.35
Northridge 1994/01/22	LA - S Grand Ave	6.7	37	2.84	0.40
Northridge 1994/01/17	Mt Wilson - CIT Seis Sta	6.7	36	2.29	0.20
Northridge 1994/01/17	San Gabriel - E. Grand Ave.	6.7	42	2.51	0.25
Northridge 1994/01/17	Canoga Park - Topanga Can	6.7	16	4.12	0.60
Northridge 1994/01/17	LA - Century City CC North	6.7	26	2.51	0.45
Northridge 1994/01/17	LA - City Terrace	6.7	37	3.10	0.30
Northridge 1994/01/17	LA – Obregon Park	6.7	38	3.48	0.25
Northridge 1994/01/17	LA – Baldwin Hills	6.7	31	1.65	0.45
Northridge 1994/01/17	LA - Wonderland Ave	6.7	23	0.17	0.50
Northridge 1994/01/23	Pasadena - N Sierra Madre	6.7	39	2.40	0.40
Northridge 1994/01/17	Leona Valley #3	6.7	38	0.11	0.50
Northridge 1994/01/23	Big Tujunga, Angeles Nat F	6.7	24	2.40	0.30
Kobe 1995/01/16	Kakogawa	6.9	26	3.38	0.35

modelling is more representative of steel frames with an overall response that is not significantly influenced by the deformations of panel zones and connections. In addition, diaphragm action was assumed at every floor due to the presence of the slab, P-delta effects were also taken into account, while Rayleigh damping corresponding to 3% of critical damping at the first two modes was adopted.

It is of significant interest to study the heightwise distribution and amplitude of deformation demands at various performance levels (i.e. damage levels) of the setback frames. Thus, for every pair of structure and accelerogram, the scale factors of the accelerogram were identified for the following (SEAOC [15] seismic design manual) performance levels: (1) occurrence of the first plastic hinge; (2) $IDR_{\max}$ equal to 1.8%; (3) $IDR_{\max}$ equal to 3.2% and (4) $IDR_{\max}$ equal to 4.0%. Please note that for the frames considered in this study, the interstorey drift ratio which corresponds to first yielding has values lower than 0.8% and thus, the above code-dictated [15] values of the $IDR_{\max}$ cover a wide range of structural damage.

The results of the 4 (performance levels) * 30 (ground motions) * 120 (frames) = 14 400 nonlinear dynamic analyses were post-processed in order to create a databank with the response quantities of interest. These response quantities are the maximum interstorey drift ratio of each storey, the maximum roof displacement and the maximum rotation ductility (evaluated at the end of beams and columns over the entire structure) in the building.

Moreover, the strength reduction factor, q, was calculated as the ratio of the scale factor of the accelerogram which drives the structure to a specific performance level over the scale factor of the accelerogram which corresponds to the occurrence of the first plastic hinge, while the roof displacement ductility, μ, is defined as the maximum (inelastic) roof displacement over its value that corresponds to the occurrence of the first plastic hinge. The aforementioned definitions of the strength reduction factor and roof displacement ductility take into account the small (but existing) variations of the response quantities that correspond to first yielding with respect to the signature of the ground motion [24].

4. Heightwise distribution of elastic and inelastic deformation demands

In this section, the central (counted sample median) value of the distribution of the maximum interstorey drift ratio along the height of setback steel MRF is examined. It is of interest to study the shape of the maximum interstorey drift profile for various geometrical irregularities and for different levels of inelastic deformation. For instance, the variation of the median value of the maximum interstorey drift ratio along the height of four 3-storey setback steel frames associated with the four damage-based performance levels of Section 3 is shown in Fig. 3(a)–(d), while the same information pertinent to 9-storey setback steel frames is shown in Fig. 4(a)–(d). It is observed

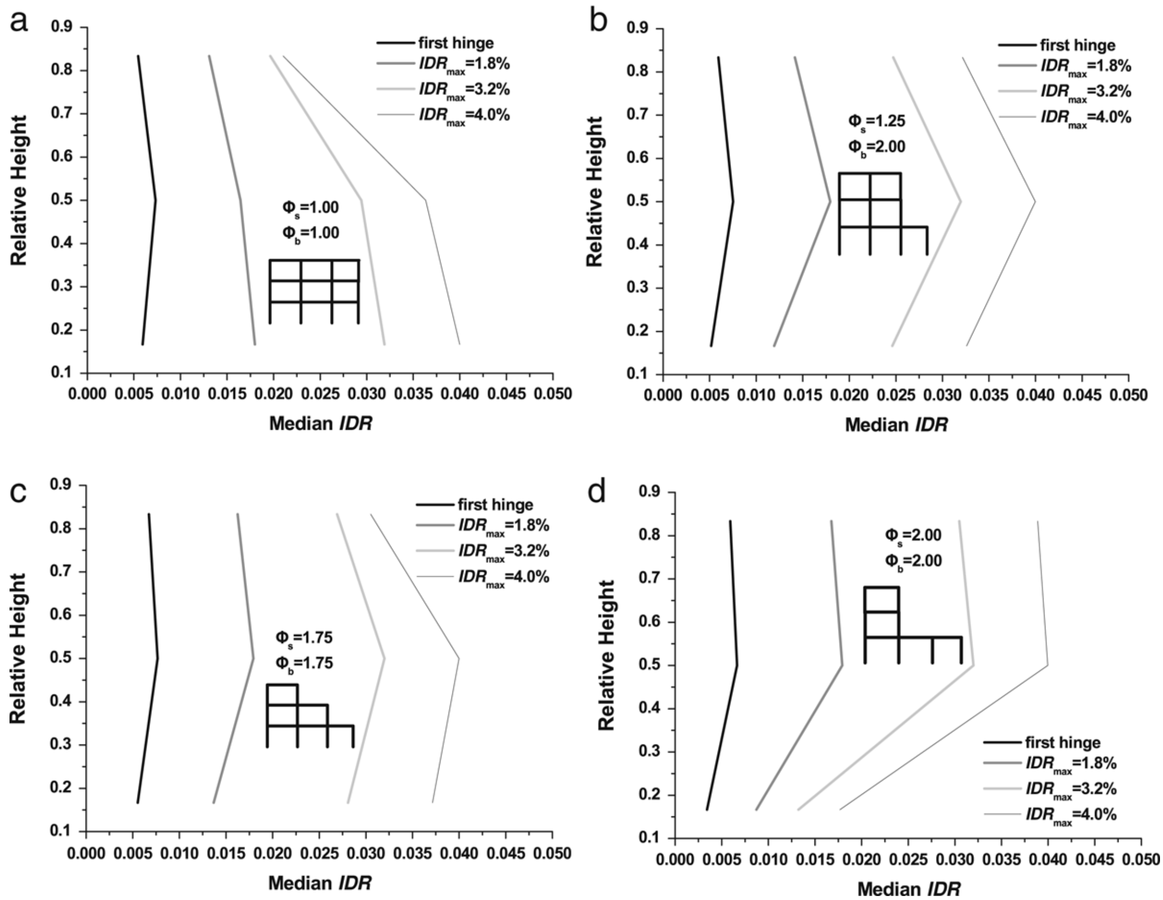

Fig. 3. Central value (counted sample median) of distribution of maximum interstorey drift ratio along the height of 3-storey setback steel MRF associated with four performance levels: first plastic hinge; $IDR_{\max} = 1.8\%$; $IDR_{\max} = 3.2\%$ and $IDR_{\max} = 4.0\%$.

that:

(1) All the frames experience uniform distributions of elastic deformation demands (limit state associated with the development of the first plastic hinge).
(2) Regular frames (Figs. 3(a) and 4(a)) distribute inelastic deformations uniformly along their height for limit states associated with an $IDR_{\max}$ equal or lower than 1.8%. For larger values of inelastic deformation, they show a clear tendency to concentrate damage in the lower stories. The latter tendency increases with an increasing level of inelastic deformation.
(3) Tower-like buildings (Figs. 3(b), (d), 4(b) and (d)) concentrate damage in the tower, near to the mid-height of the building. This tendency increases with an increasing level of inelastic deformation. The base is the part experiencing the least damage for buildings with large values of the Φ_s index (Figs. 3(d) and (d)), while for buildings with smaller values of the Φ_s index (Fig. 3(b) and (d)), the least damage is found both in the base and in the upper storeys.
(4) Buildings with setbacks occurring at various height levels (Fig. 4(c)), concentrate damage in the neighborhood of the setbacks and this trend becomes more evident with an increasing level of inelastic deformation.
(5) For limit states associated with an $IDR_{\max}$ larger than 1.8%, interstorey drifts in the tower of setback structures are larger than those of the corresponding regular structures at the same height level, whereas for limit states associated with an $IDR_{\max}$ smaller or equal to 1.8%, are of the same order.
(6) For all limit states, interstorey drifts at the base of setback structures are smaller than those of the corresponding regular structures at the same height level.

5. Estimation of inelastic deformation demands

In this section, simple formulae to estimate inelastic deformation demands in plane steel MRF with setbacks are proposed. With R being any response quantity of interest, the counted sample median and the standard deviation of the natural logarithm of the ratio of the predicted (from the proposed formula) over the exact (from dynamic analysis) value of R, i.e. $R_{\mathrm{app}}/R_{\mathrm{exact}}$, are used in order to express the central value and the dispersion of the error introduced by the proposed relations. It should be noted that the results associated with the elastic range of the response (limit state associated with the occurrence of the first plastic hinge) are excluded from the response databank, since they distort the effectiveness of the formulae to predict deformation demands associated with the inelastic range of the response.

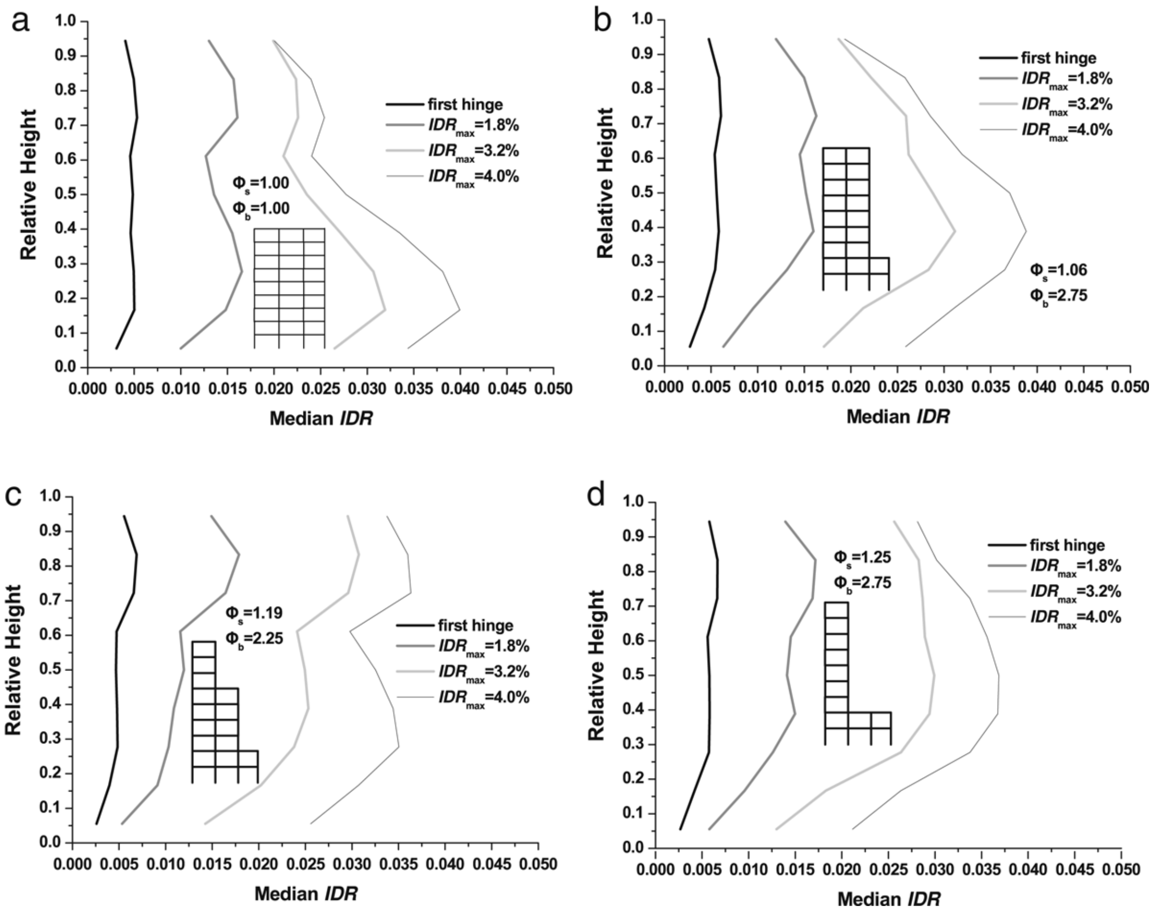

Fig. 4. Central value (counted sample median) of distribution of maximum interstorey drift ratio along the height of 3-storey setback steel MRF associated with four performance levels: first plastic hinge; $IDR_{\max} = 1.8\%$; $IDR_{\max} = 3.2\%$ and $IDR_{\max} = 4.0\%$.

5.1. Estimation of the maximum roof displacement

Current seismic codes [13] estimate approximately the maximum inelastic roof displacement by adopting the equal-displacement rule, i.e. by assuming that $\mu = q$. With respect to the whole databank, this rule leads to a ratio $u_{r,\max,\text{app}}/u_{r,\max,\text{exact}}$ with central value equal to 1.32 and dispersion value equal to 0.3 (Fig. 5(a)). It is therefore evident that the equal-displacement rule clearly overestimates the exact maximum inelastic roof displacement.

By analysing the response databank, no effect of the parameters n_s, Φ_b and a, or the period of vibration and the frequency content of the ground motion (ratio T/T_c), on the relationship between q and μ was identified. A small effect of the irregularity index Φ_s was observed and thus, use of the Levenberg-Marquardt algorithm [25] for nonlinear regression analysis, led to a formula for q of the form

$$q = 1.0 + 1.92 \cdot (\mu - 1)^{0.85} \cdot \Phi_s^{-0.17}. \quad (4)$$

The aforementioned relation is relatively simple, satisfies the physical constraint $q = 1$ for $\mu = 1$ and offers directly either the desired behaviour factor in order to limit ductility under a predefined value, or indirectly the maximum inelastic roof displacement given the behaviour factor and yield roof displacement. The degree of accuracy of Eq. (4) was evaluated by examining the statistical distribution of the ratio $u_{r,\max,\text{app}}/u_{r,\max,\text{exact}}$, as explained at the beginning of this section. This ratio was found to have a central value equal to 0.95 and dispersion value equal to 0.3 (Fig. 5(b)).

Median and dispersion values usually mask the existence of extreme error values. For instance, Fig. 5(b) indicates that for a small number of pairs of structure and accelerogram, the use of Eq. (4) may underpredict response by a factor of two.

5.2. Estimation of the maximum interstorey drift along the height of the frame

An interesting way for estimating the maximum interstorey drift ratio, $IDR_{\max}$, along the height of the frame is via correlation studies with the maximum roof drift $u_{r,\max}/H$. By analysing the response databank described in this paper, the ratio $\beta = u_{r,\max}/(H * IDR_{\max})$ was found to be strongly dependent on the number of stories and on the irregularity indices and thus, nonlinear regression analysis produced the following approximation for this ratio:

$$\beta = 1.0 - 0.13 \cdot (n_s - 1.0)^{0.52} \cdot \Phi_s^{0.38} \cdot \Phi_b^{0.14}. \quad (5)$$

The above-mentioned relation is simple, satisfies the physical constraint $\beta = 1$ for $n_s = 1$ and can be used either for

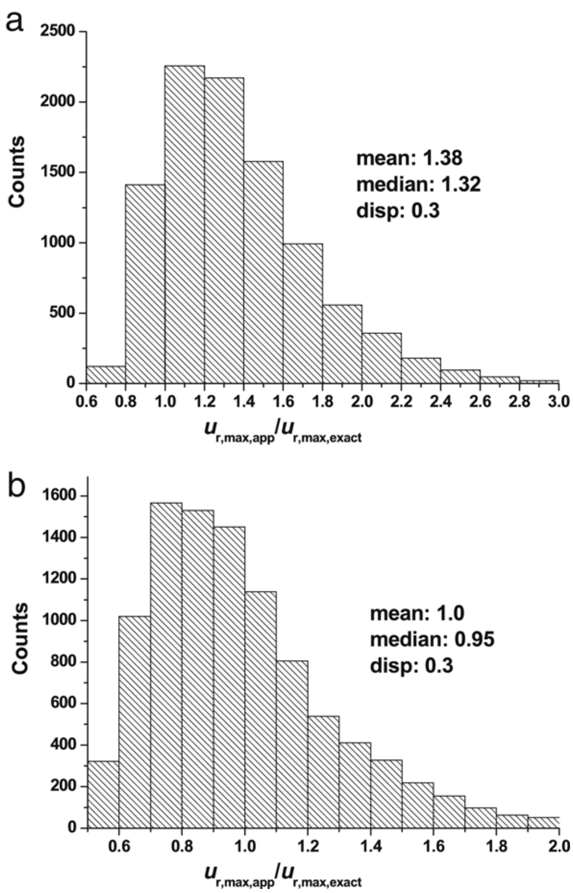

Fig. 5. Distribution of the ratio $u_{r,\max,\text{app}}/u_{r,\max,\text{exact}}$ on the basis of (a) the equal-displacement rule and (b) the proposed relation.

predicting the $IDR_{\max}$ given the $u_{r,\max}$, or vice-versa, i.e. predicting the $u_{r,\max}$ given the $IDR_{\max}$. The latter calculation is needed for the initiation of deformation-controlled seismic design methods such as the DBD [15].

With the maximum roof displacement known ($u_{r,\max,\text{exact}}$), Eq. (5) leads to a ratio $IDR_{\max,\text{app}}/IDR_{\max,\text{exact}}$ with a central value equal to 1.0 and dispersion value equal to 0.17 (Fig. 6(a)).

While the descriptive statistics for predicting the $IDR_{\max}$ for a known maximum roof displacement ($u_{r,\max,\text{exact}}$) are encouraging, it is of significant interest to calculate the error introduced in the prediction of the $IDR_{\max}$ by combining the uncertainties of both Eqs. (4) and (5). For a given strength reduction factor q, i.e. given the approximate maximum roof displacement ($u_{r,\max,\text{app}}$), Eq. (5) leads to a ratio $IDR_{\max,\text{app}}/IDR_{\max,\text{exact}}$ with a central value equal to 0.93 and dispersion value equal to 0.32 (Fig. 6(b)). The prediction of the $IDR_{\max}$ should be compared with the prediction offered by current seismic codes [13]. According to EC8 [13], the maximum interstorey drift ratio is calculated on the basis of the maximum lateral displacements at the top and bottom of the storey under consideration, which in turn, are calculated on the basis of the equal-displacement rule. With respect to the response databank of this study, the procedure of seismic codes leads to a ratio $IDR_{\max,\text{app}}/IDR_{\max,\text{exact}}$ with a central value equal to 1.33 and dispersion value equal to 0.32 (Fig. 6(c)), indicating that the equal-displacement rule clearly

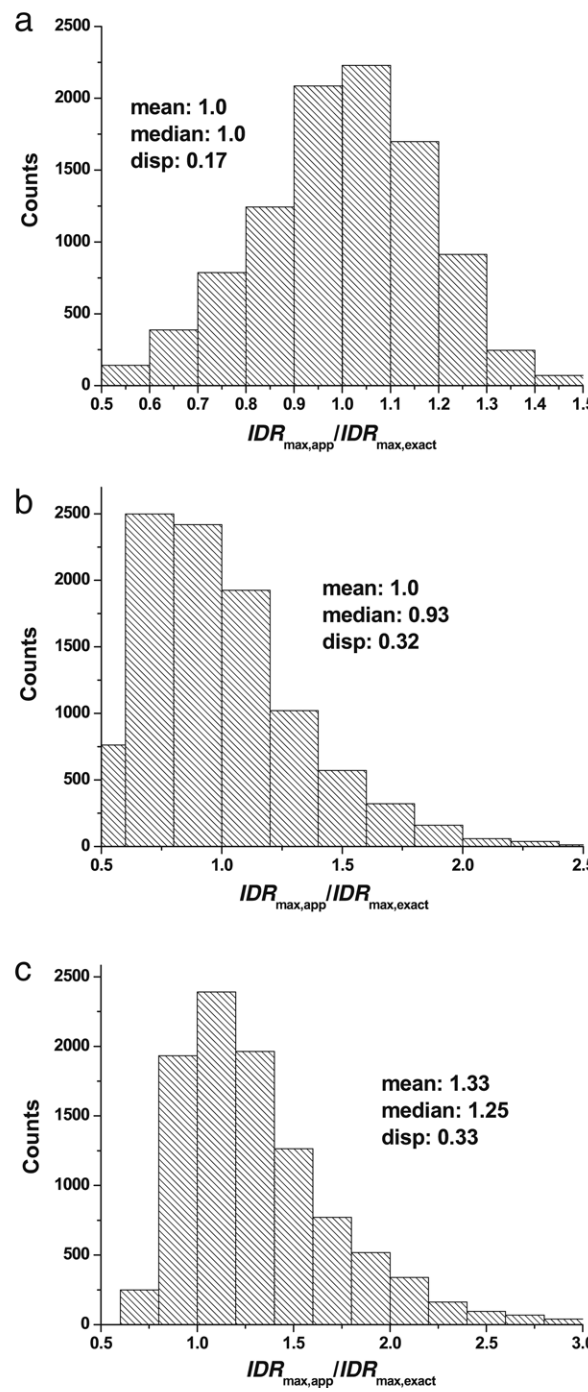

Fig. 6. Distribution of the ratio $IDR_{\max,\text{app}}/IDR_{\max,\text{exact}}$ on the basis of (a) the proposed relation with the $u_{r,\max}$ known; (b) the proposed relation with the q factor known and (c) the procedure adopted in EC8.

overestimates the maximum interstorey drift ratio along the height of plane steel MRF with setbacks.

5.3. Estimation of the maximum rotation ductility along the height of the frame

Here, the maximum rotation ductility μ_θ (evaluated at the end of beams and columns over the entire structure) is correlated with the strength reduction factor q. The statistical

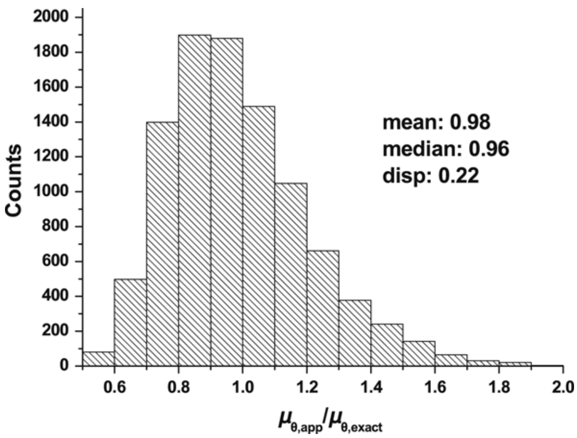

Fig. 7. Distribution of the ratio $\mu_{\theta,\text{app}}/\mu_{\theta,\text{exact}}$ on the basis of the proposed relation.

analysis of the present response databank has shown that the relation between q and μ_θ depends on the parameter α and on the irregularity index Φ_s. Nonlinear regression analysis produced the following approximation:

$$q = 1.0 + 0.7 \cdot (\mu_\theta - 1)^{1.29} \cdot \Phi_s^{-0.11} \cdot \alpha^{0.14}. \tag{6}$$

The aforementioned expression is simple and furthermore, satisfies the physical constraint $q = 1.0$ for $\mu_\theta = 1.0$. Eq. (6) offers a ratio $\mu_{\theta,\text{app}}/\mu_{\theta,\text{exact}}$ with a central value equal to 0.96 and dispersion value equal to 0.22 (Fig. 7).

5.4. Range of applicability of the proposed relations

Here, a brief discussion on the range of applicability of the proposed formulae is provided. The proposed relations are valid for ground motions recorded at locations far from the seismic source, i.e. for ground motions which do not exhibit near-fault characteristics. Moreover, the proposed relations cannot be regarded as valid for soft-soil sites, since these sites may produce nearly harmonic motions. In addition, the proposed techniques are valid for setback steel frames (1) for which a strain hardening equal to 3% may be regarded as a good choice for modelling the behaviour of plastic hinges, (2) for which a viscous damping equal to 3% may be regarded as a good choice, (3) which are designed according to rules that exclude panel zones from the dissipation of seismic energy, (4) with connections which can be assumed to be rigid and (5) with structural characteristics (n_s, a, Φ_s and Φ_b) in the range of values examined in this study (Table 1).

Since this study was based on a fixed bay width and storey height, one may claim that the inelastic seismic response of frames with similar "n_s and α" characteristics but with larger bay widths and/or storey heights can not be described by the proposed relations. It is apparent that more work is needed in order to reach a final conclusion.

Moreover, it should be pointed out that the proposed equations are not valid for frames with central towers since this study was based on setback frames with the steps being on one side.

It worthwhile noticing that the proposed formulae for estimation of deformations can be used not only at the end of the FBD [13] but also at the early stages of the DBD [15] as well as of hybrid force/displacement-based design methods [26].

6. Use of the proposed relations for performance-based seismic design

This section aims at demonstrating the major intention of the work presented in this paper, i.e. the development of a method which should be regarded as an alternative to seismic design procedures that use elastic analysis and dominate current seismic codes.

Consider the S235 setback steel frame shown in Fig. 8(a). The bay width is assumed equal to 5 m and the storey height equal to 3 m. The gravity load on beams is equal to 27.5 kN/m. The design ground motion is defined by the elastic acceleration design spectrum of the EC8 [13] seismic code with a PGA equal to $0.4g$ and a soil class B. The design of the frame is done according to the EC3 [16] and EC8 [13] structural codes with the aid of the commercial software package SAP2000 [17] on the basis of a response spectrum analysis and design process.

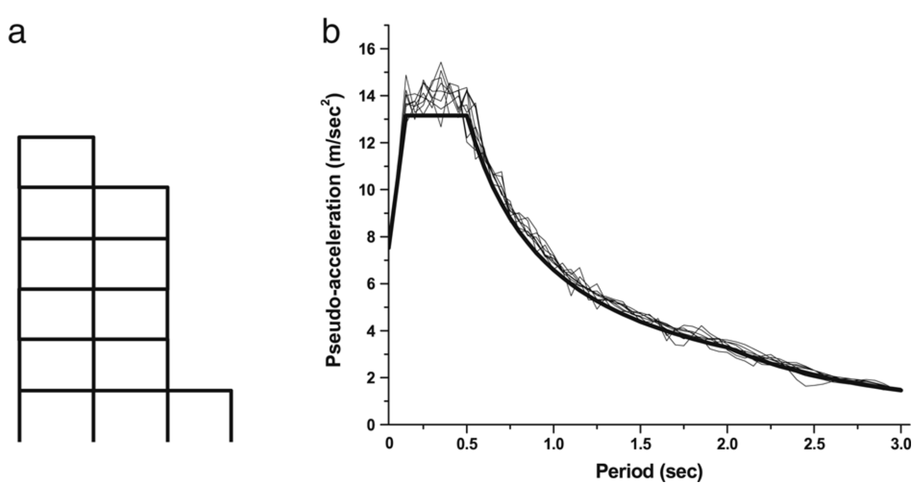

Fig. 8. Geometry of the frame (a) and response spectra of the ground motions (b) used in the design example.

HEB profiles [18] are used for the columns and IPE profiles [18] are used for the beams. A q factor equal to 5.2 ($0.8 * 6.5$) is selected and the design procedure yields HEB240-IPE300 cross-sections for the first and second stories, HEB220-IPE300 cross-sections for the third and fourth stories and HEB200-IPE270 cross-sections for the fifth and sixth stories. The maximum roof displacement and interstorey drift ratio under the reduced (divided by q) spectrum are equal to 0.06 m and 0.0044, respectively. Thus, the maximum inelastic roof displacement equals $5.2 * 0.06 = 0.31$ m and the maximum inelastic interstorey drift $5.2 * 0.0044 = 0.023$.

The irregularity indices Φ_s and Φ_b of the frame are computed on the basis of Eqs. (1) and (2) and found to be equal to 1.30 and 3.10, respectively. With the q factor and the Φ_s index known, Eq. (4) estimates the maximum roof displacement ductility to be equal to 3.65 and therefore, the maximum inelastic roof displacement equal to $3.65 * 0.06 = 0.22$ m. Given the maximum inelastic roof displacement and with the characteristics of the frame n_s, Φ_s, Φ_b known, Eq. (5) predicts a value of the $IDR_{\max}$ equal to 0.02.

Eight semi-artificial accelerograms compatible with the EC8 [13] spectrum were generated via a deterministic approach [27] on the basis of eight real seismic records of Table 2. The response spectra of these motions, in comparison with the EC8 spectrum, are depicted in Fig. 8(b). Nonlinear time history analyses of the designed frame under these motions yielded: $u_{r,\max} = 0.23$ (mean value with standard deviation $= 0.035$) and $IDR_{\max} = 0.022$ (mean value with standard deviation $= 0.003$). The results of nonlinear time history analyses reveal that the proposed procedure seems to be more accurate and efficient than the procedure of EC8 [13] for performance-based seismic design of plane steel moment resisting frames with setbacks.

7. Conclusions

In an effort to examine and evaluate the seismic inelastic deformation demands in setback steel MRF designed according to the guidelines of current seismic codes, an extensive analytical parametric study was undertaken. A family of 120 code-compliant setback steel MRF were subjected to an ensemble of 30 ordinary (i.e. without near-fault effects) earthquake ground motions scaled to different intensities in order to drive the structures to different performance levels.

It has been found that the level of inelastic deformation and geometrical configuration play an important role on the heightwise distribution of deformation demands. In general, the maximum deformation demands are concentrated in the "tower" for tower-like structures and in the neighbourhood of the setbacks for other geometrical configurations. The latter conclusions are more evident for high levels of inelastic deformation but they do not hold true for setback frames in the elastic range of the seismic response.

Based on regression analysis, a procedure in terms of simple formulae for estimating the maximum roof displacement, the maximum interstorey drift ratio and the maximum rotation ductility along the height of a setback steel frame, was developed. The procedure does not indicate where in the structure the maximum drifts will occur. Moreover, it does not depend on pushover analysis, since it demands only an elastic analysis up to the point of the development of the first plastic hinge in the building and therefore, is suitable for both seismic assessment of existing structures and direct deformation-controlled seismic design of new ones. It takes into account the influence of various structural characteristics of a setback steel frame, such as the number of stories, the geometrical irregularity and the beam-to-column strength ratio. Compared with the procedure adopted by current seismic design codes, it was found to be more accurate and efficient for performance-based seismic design of plane steel moment resisting frames with setbacks.

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