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Efficiency of Eurocode 8 design rules for steel and steel-concrete composite structures

A. Braconi [a], S. Caprili [b,*], H. Degee [c], M. Guendel [h], M. Hjiaj [e], B. Hoffmeister [d], S.A. Karamanos [f], V. Rinaldi [g], W. Salvatore [b], H. Somja [e]

[a] Thy Marcinelle Rivagroup, Rue de l'Acier, BP 1002, Charleroi, Belgium
[b] Department of Civil and Industrial Engineering University of Pisa, Italy
[c] Faculty of Engineering Technology, University of Hasselt, Belgium
[d] RWTH Aachen University, Institut für Stahlbau, Aachen, Germany
[e] Institut National des Sciences Appliquées de Rennes, France
[f] University of Thessaly, Volos, Greece
[g] Arcelor Mittal S.A. Luxembourg
[h] Wölfel Beratende Ingenieure GmbH + Co. KG, Max-Planck-Str. 15, 97204 Höchberg, Germany

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ABSTRACT

Seismic design codes allow the realization of structures able to dissipate energy through cyclic plastic deformations located in specific regions, selected to involve the largest number of structural elements. The capacity design approach requires an opportune selection of the design forces and an accurate definition of structural details in the plastic hinges. The structural elements in which plastic hinges are located are over-sized with respect to the seismic actions obtained by the use of the design spectrum, while the elements that shall remain elastic are over-sized with respect to dissipative elements. The capacity design methodology requires an accurate control of the localization of plastic hinges, strongly influenced by the actual mechanical properties of materials. In the present work, developed within the European research project OPUS, different case studies were designed according to Eurocodes and subjected to a deep structural analysis, aiming to evaluate the effective allowable ductility (behaviour factor) with respect to what imposed during the design phase and taking into account the effective mechanical behavior of materials.

1. Introduction

Seismic design codes [1–4] allow the exploitation of plastic resources realizing ductile structures able to dissipate the seismic energy stored during the earthquake through cyclic plastic deformations located in the suitably chosen "dissipative zones". Plastic deformations shall be located within the structures in order to guarantee the involvement of the largest number of structural elements. The larger is the number of the plastic hinges, the larger is the attainable global structural ductility and the smaller is the deformation demand at local level (Fig. 1).

The design of dissipative zones (i.e. plastic hinges) in correspondence of the selected elements and the development of an efficient energetic dissipation, without significant decreases of strength and stiffness, are obtained through a proper methodology called *capacity design* and an accurate definition of structural details of elements, joints and connections. Obviously, the choice of dissipative elements depends on the structural typology: different dissipative elements/mechanisms are foreseen for Moment Resisting Frames (MRF), Concentrically Braced Frames (CBF) and Eccentrically Braced Frames (EBF).

For example, in multi-storeyMRF buildings, the condition $\sum M_{Rc} \geq 1.3 \times \sum M_{Rb}$[1] shall be checked in correspondence of each beam-to-column joint of the structure, being ΣM_{Rc} and ΣM_{Rb} the sums of the design values of the bending resistance of respectively columns and beams framing at a joint (Fig. 2), in order to allow the development of the largest number of plastic hinges and to dissipate the highest quantity of seismic energy. The above presented condition aims at avoiding the development of poor dissipative mechanisms such as soft-storey, providing columns with sufficient overstrength with respect to the beams. The 1.3 factor takes into account possible overstrength phenomena of materials used in beams with respect to the ones adopted for columns. Moreover, other specific criteria are adopted for the design of CFB and EBF structures, in which the dissipative elements are respectively the bracing system (X, inverted V and others) and link elements, opportunely designed in order to guarantee a uniform distribution of energy dissipation.

* Corresponding author at: Dept. of Civil and Industrial Engineering, University of Pisa, Largo L. Lazzarino 1, 56122 Pisa (IT). Tel.: +93 0502218246; fax: +39 0502218201.
E-mail address: silvia.caprili@ing.unipi.it (S. Caprili).

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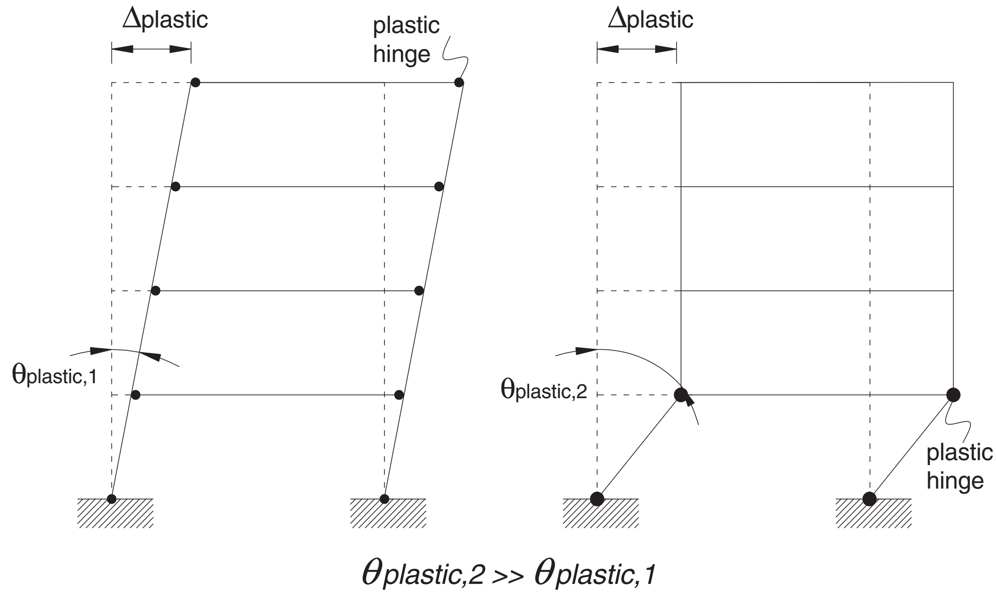

Fig. 1. Global ductility vs. local rotation demand for moment resisting frames.

In the modal analysis procedure, commonly used in the engineering practice, the possibility to exploit plastic resources is translated in lower values of design seismic action. To obtain the design spectrum, the elastic response one is divided by a reduction factor, summarizing the parameters governing the structural response, the available inelastic resources and the sensitivity to the second-order effects.

The aforementioned reduction factor is already introduced by several modern standards such as Eurocode 8 [1], in which it is identified as the *behavior factor* "*q*", or the US standards, Uniform Building Code UBC[3], NEHRP provisions [2], American Seismic Provisions for Structural Steel Buildings [4], in which the *reduction factor* "*R*" is defined. The larger the reduction factor is, the larger shall be the structural ductility and the lower can be the seismic design actions used for ultimate limit states (Life Safety – LS or Collapse Prevention – CP). In such a way it seems possible to obtain systematically structural solutions characterized by a reduced overall weight.

However, the exploitation of the plastic capacity can be limited by other criteria adopted in the design process: for example, in the assessment at serviceability limit state, the limitation of second order effects as well as the assessments of limit states associated to static load combinations shall be considered. The fulfilment of such conditions can limit the benefits of ductile design, leading to a structure whose seismic response can be quite far from the one supposed at design stage. Moreover, the elements in correspondence of dissipative zones of the structure often result over-sized with respect to the seismic actions obtained by the design spectrum, so that, in practice, only a small percentage of ductile resources can be effectively exploited. At the same time, as the capacity design rules are applied, the protected elastic elements are further over-dimensioned with respect to dissipative ones [5,6].

According to previous concepts, the seismic ductile design foresees an accurate control of plastic hinges' development that strictly depends on the distribution of plastic resistances of the structural elements: as a consequence, the *capacity design* methodology strongly depends on the actual mechanical properties of materials.

Despite what already presented, actual European production standards [7] do not provide adequate limitations for the mechanical properties of the steel products, evidencing the lack of agreement among provisions coming from different standards. As a consequence, the adoption of the aforementioned design approach is allowed by Eurocode 8 [1] for steel and composite steel-concrete structures only with the introduction of adequate safety factors and controlling that actual values of the mechanical properties do not modify the location of plastic hinges.

These conditions limit the adoption in the design practice of the steel and steel-concrete composite structures, potentially a very interesting option in seismic zone because of the intrinsic ductility and dissipative

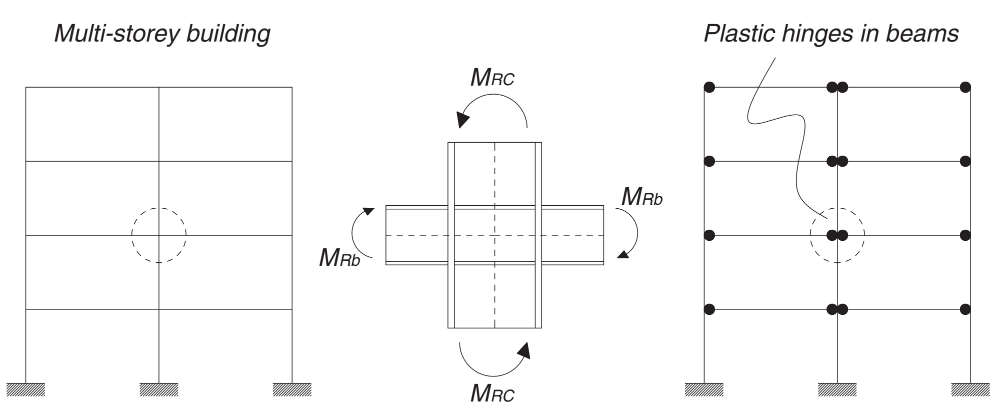

Fig. 2. Distribution of plastic hinges to allow the maximum dissipation of seismic energy in MRF structures.

capacity of the steel. Eurocode 8 [1], in particular, imposes additional checks on material properties in dissipative zones as, for example, in the steel members where the yielding stress shall be upper limited by the overstrength coefficient, γ_{OV} fixed equal to 1.25 (1.25 times the nominal yielding value f_y).

Some first evaluations on the seismic reliability of steel structures taking into account variability of the steel properties were executed on concentrically braced frames [8] and eccentrically braced frames [9]; anyway, a deep comprehensive study, able to clarify the aforementioned open problems, is still missing.

The OPUS research project, *Optimizing the seismic Performance of steel and steel-composite concrete strUctures by Standardizing material quality control*, funded by European Commission through Research Fund for Coal and Steel [10], aimed at assessing the influence of material properties' scattering on the final structural performance of steel and steel concrete composite structures designed in earthquake-prone areas. In particular, in the framework of this project, a representative set of case studies, including MRFs, EBFs and CBFs, was designed according to the design provisions of the Eurocodes and afterwards subjected to a deep analysis of the structural performance. A suitably developed probabilistic procedure then allowed estimating the failure probability associated to all relevant collapse modalities previously identified for each structural typology [11–13].

In the present paper the main results obtained by aforementioned performance analysis of the case studies are illustrated and discussed whereas the final project report [10] includes all the main results on influence of material properties' scattering on final structural performance.

The assessment of the seismic performance of all the structures was executed developing 2D non-linear models of the main resisting frames: the models were elaborated and calibrated through a complete benchmarking process aimed at the simulations of three structural typologies. The models of the buildings were analyzed employing Push-Over analysis (PO) and Incremental Dynamic Analysis (IDA) in order to characterize the structural behavior of case studies and to determine the relevant collapse modalities.

The analysis on structural case studies was deeply detailed in order to assess the real *behavior factor*(q) of all models and to determine the level of Peak Ground Acceleration (PGA) able to activate the collapse modalities previously identified. IDA technique was applied for identifying the PGA activating relevant collapse criteria and, in combination with the Ballio-Setti procedure [16,17], for assessing their actual q factor.

The Ballio-Setti procedure was furthermore modified for taking into account the discrepancy at first mode frequency between the target spectra and real spectra of earthquake time-histories. Such procedure allowed comparing, for each considered MRF,CFB or EBFcase-study, the real values of the q-factor with the ones adopted in the design process, highlighting eventual discrepancy with actual design standards [10,14,15].

2. Design of the case studies

A fully representative sample of case studies, including MRF,CBF and EBF structures was designed according to the prescriptions of the Eurocodes: the geometries and the morphologies were suitably chosen in order to be consistent with the foresees activities (offices, car parks, industrial storages, electrical power plants or ware house/light industrial activities); some examples of the designed buildings are presented in the Fig. 3. Two levels of seismic actions were selected for being representative of the low and high seismic hazard areas, respectively characterized by a peak ground acceleration (PGA) equal to 0.10-0.15g and 0.25g; the static loads were chosen on the basis of the housed activities according to Eurocode 1 [18] and the wind action was evaluated fixing equal parameters for all structures. In the Table 1 the information related to the use category, the live loads and the environmental loads (snow, wind and earthquake) are presented, whereas geometry, chosen resisting systems for vertical and horizontal loads and floor system schemes are listed in the Table 2. In particular, the column related to "secondary beams" refers to the presence of additional secondary beams for decoupling the gravity loads from the seismic loads acting in correspondence of the beams included in the eccentrical bracing systems.

The design procedure was carried out in compliance with the actual European and international standards [1,18–23].

The design process of the selected structures was executed adopting different strategies or techniques able to optimize the cross sections' size and to avoid the over-sizing of structural members, that is a relevant aspect especially for buildings designed in low seismicity areas, whose seismic forces can be also lower than wind ones. The optimal design was not reached in all of the cases because of design procedures, checks and limitations imposed by the Eurocodes.

In the case of MRFs, the assessment of design checks for both static- and seismic- load combinations leaded to over-sized beams respect to the simple seismic strength's requirements. In addition, the capacity design approach, beam-to-column resistance hierarchy, drift limitations and the sensitivity to second order effects strongly conditioned the final sizing of the elements, according to the following equations:

$$E_i^{c.d.} = E_i^{gravity} + 1.1 \cdot \gamma_{OV} \cdot \min(\Omega_j) \cdot E_i^{seismic} \tag{1}$$

$$\sum M_{Rd,PL}^{column} \geq 1.3 \cdot \sum M_{Rd,PL}^{beam} \tag{2}$$

$$\nu \cdot q_d \cdot d_e = \nu \cdot d_r \leq d_{\text{limit}} \tag{3}$$

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq \beta \tag{4}$$

being $E_i^{gravity}$ the solicitation acting on the i-th member coming from gravitational loads, $E_i^{seismic}$ the solicitation acting on the i-th member due to seismic action, $E_i^{c.d}$ the solicitation acting on the i-th member to be adopted following the capacity design approach, γ_{ov} the material over-strength (equal to 1.25), $M_{Rd,PL}^{column}$ and $M_{Rd,PL}^{beam}$ the design resistant bending moments of columns and beams respectively, d_r the drift coming from the analysis using the design response spectrum for seismic action, d_e the elastic drift coming from analyses and d_{limit} its maximum allowed value, ν the reduction factor taking into account the lower return period of the seismic action associated with the damage limitation requirement, q_d the displacement behaviour factor, P_{tot} and V_{tot} respectively the total vertical actions and the horizontal ones on the i-th floor, θ the coefficient indicating the sensitivity to second order effects and β the value of θ coefficient in relation to which linear or non linear analyses shall be adopted.

The Ω factor presented in Eq. (1), representing the structural over-strength, i.e. how higher is the strength of the more solicited dissipative member ($R_{d,i}$) respect to the maximum level of solicitations due to the seismic combination ($E_{i,dissipative}^{seismic}$), is expressed by:

$$\Omega_i = \alpha \cdot \frac{R_{d,i}}{E_{i,dissipative}^{seismic}} \tag{5}$$

being α a coefficient equal to 1 for MRFs and CBFs and 1.5 for EBFs. The over-sizing of dissipative members and the adoption of limitations for interstorey drift ratio notably increased the size of columns and of beams leading, at the end of the design process, to structures with a large amount of strength and ductility resources, higher than the ones effectively required by the seismic loads. In order to cope with these problems, in the case of MRFs, an appropriate design process, focused on the selection of an optimized behaviour factor harmonized with strength requirements coming from static load combinations was followed. This procedure leaded, in many cases, to the configurations characterized by the adoption of a medium ductility behaviour instead of an high ductility one, both in the case of high and low seismicity areas, and to the following use of lower behaviour factors.

In the case of EBF configurations (buildings 3, 4 and 16) a similar problem was revealed concerning the sizing of the seismic links, since the interaction between static and seismic combinations strongly influenced the structural design often obliging to over-size seismic links' sections; in addition, the control of links over-sizing inside EBF configuration was completed checking that difference between Ω_i of links was not more than 25%, according to what presented by the Eq. (6):

$$\frac{\Omega_{\max}}{\Omega_{\min}} \leq 1.25 \tag{6}$$

being Ω_{max} and Ω_{min} the maximum and the minimum values of the structural over-strength factors for dissipative members.

In the case of buildings 3 and 4, characterized by the same disposition of structural elements both in plan and elevation but designed for different levels of PGA and with a different behaviour of link elements (Table 2), a suitable technical solution of the floor system was used in order to decuple static effects from seismic effects on the seismic links and to consequently reduce the over-strength of the final design: beams containing seismic links were coupled with other beams (i.e. beam duplication) devoted to carry only vertical loads. This procedure allowed optimizing some structural solutions, leading to an utilization ratio of the links (i.e. internal forces/resistance, S/R) equal to 1.0 with

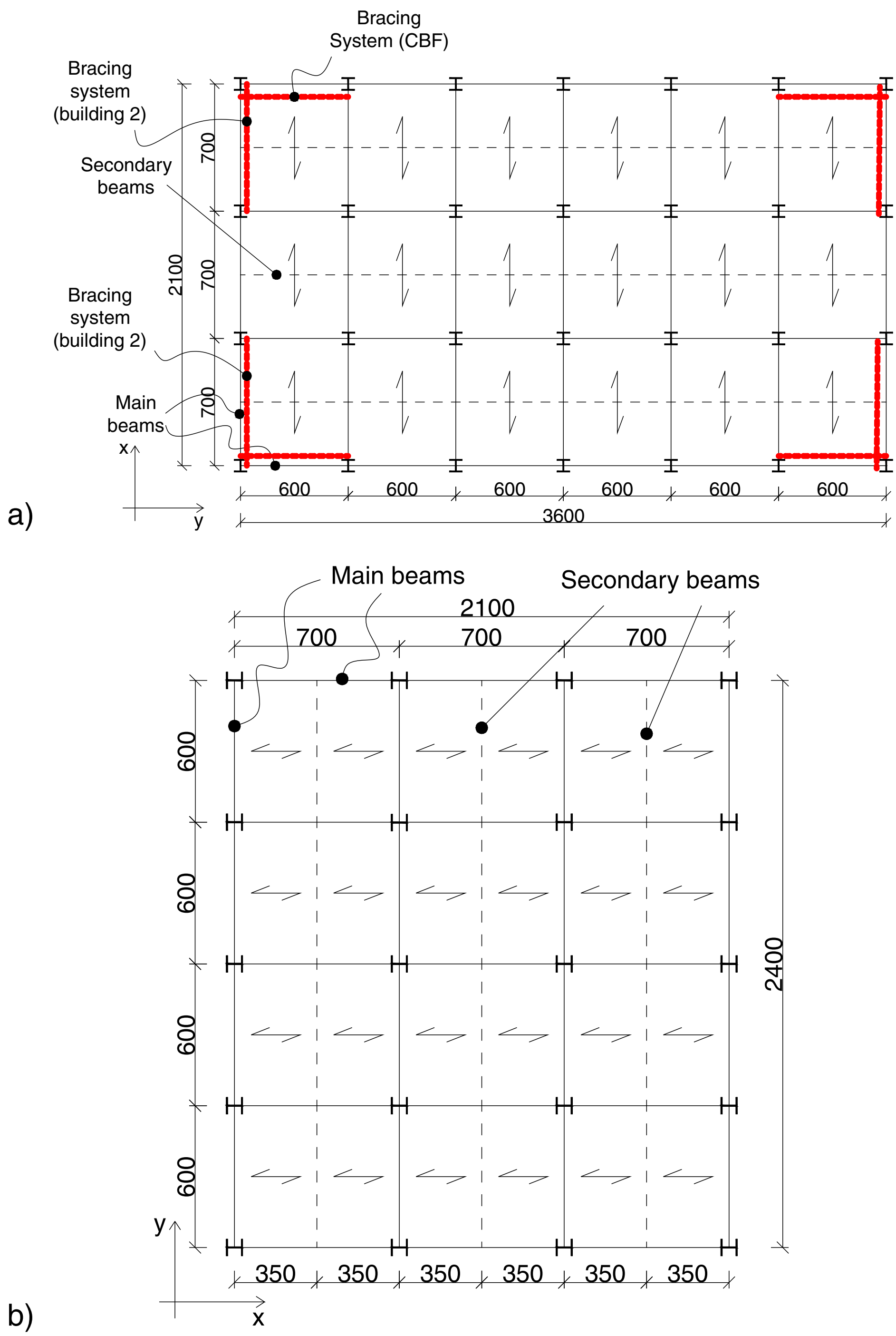

Fig. 3. Main plans and elevation schemes for: a) buildings 1-2, d) buildings 6, 7, 8, 10, 11 (MRF and MRF-CBF), c) buildings 3, 4 (EBF or CBF), d) building 5, e) building 13, f) building 14 (MRF and CBF), g) buildings 12 and 15 (MRF and CBF), h) building 16 (EBF).

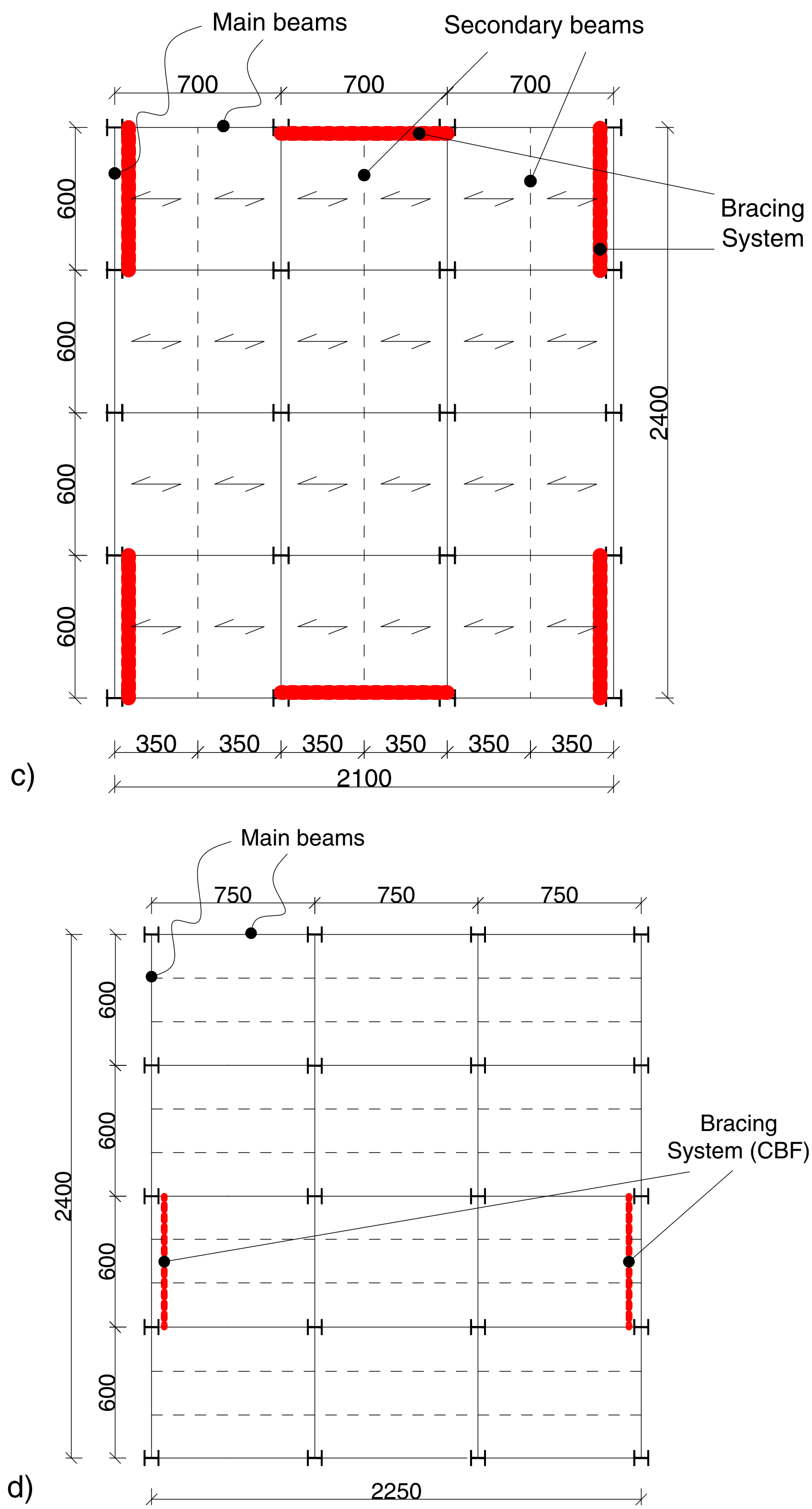

Fig. 3 (*continued*).

a following over-strength coefficient Ω up to 1.5, producing, on the other hand, an increment of bracing sections. The main characteristics of the buildings with a MRF resisting scheme are summarized in Tables 3, 4 and 5.

Similarly to the case of MRFs, the design of EBFs was strongly influenced by second order effects and by the respect of the drift limitation: as an example, the sizing of the braces in building 3 in X-direction was determined in relation to the satisfaction of the limitation of the maximum drift up to 0.05%, leading to an oversizing of elements, even if not directly related to the capacity design approach. Moreover, in the case of building 4 the most critical design condition was the fulfilment of buckling of members in compression, with a following ratio S/R

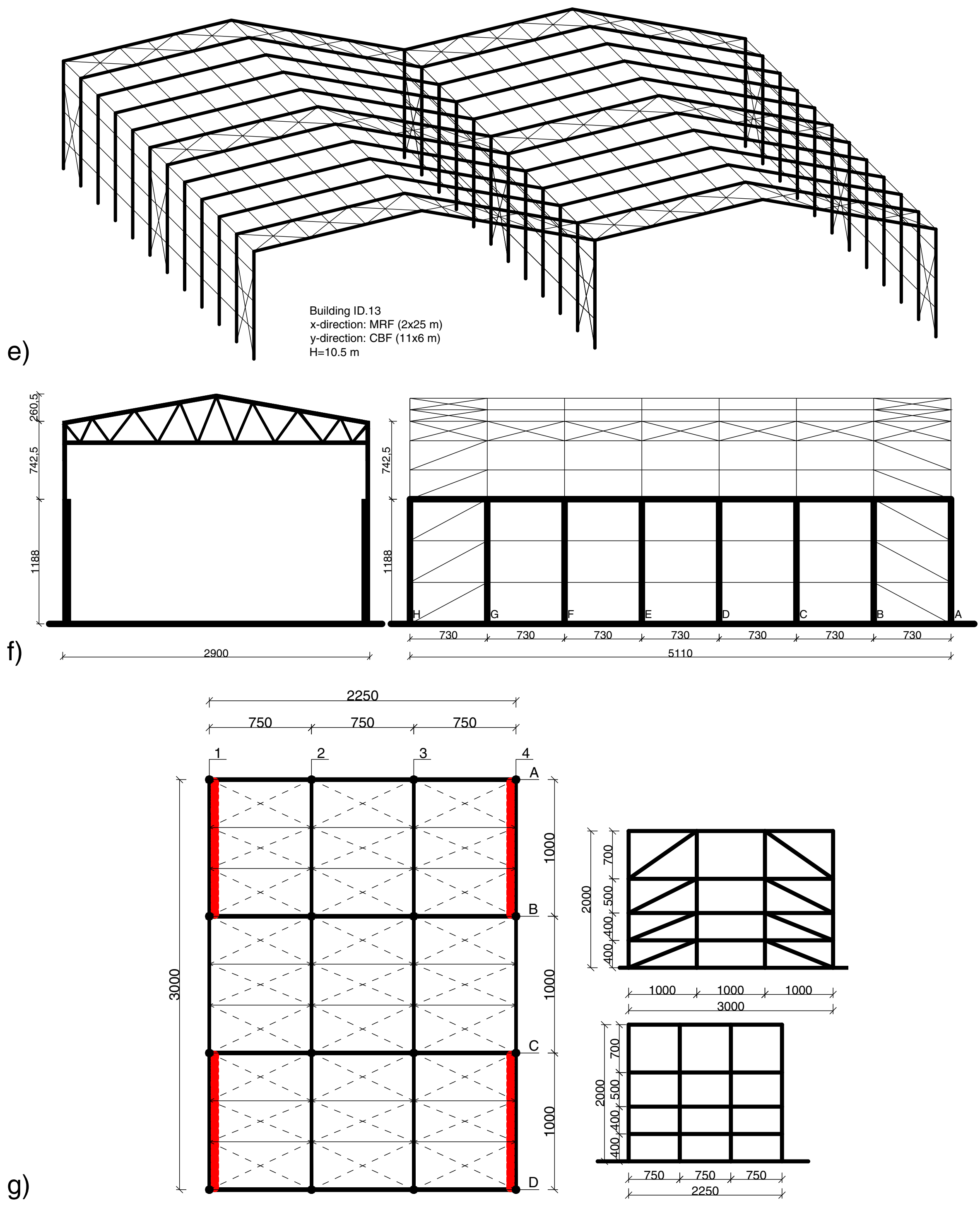

Fig. 3 (*continued*).

nearly about 1, once again not directly related to the capacity design criteria. The main characteristics of EBF steel buildings are summarized in Tables 4 and 6.

Concerning CBF solutions, the design process proposed by [1] obliged to perform several repetitive designing in order to fulfil problems related to slenderness ratio (λ):

$$1.3 \le \lambda \le 2.0 \qquad (7)$$

and the assessment of Eqs. (1), (3) and (4).

Only in some cases the optimization was reached arriving to a solicitation/strength ratio near to 1; for other solutions, different steel qualities were adopted for steel bracings at different floor levels in order to optimize design checks and to satisfy all the prescriptions for seismic design. The main characteristics of CBFs are briefly summarized in Tables 3, 4 and 6.

Design of composite steel/concrete structures was executed in agreement with Eurocodes' prescriptions [1,19,21] and with what presented in the scientific literature [24,25]. Lateral torsional buckling was supposed to be prevented for beams as well as for columns, in order to ensure a stable behavior of the members during the

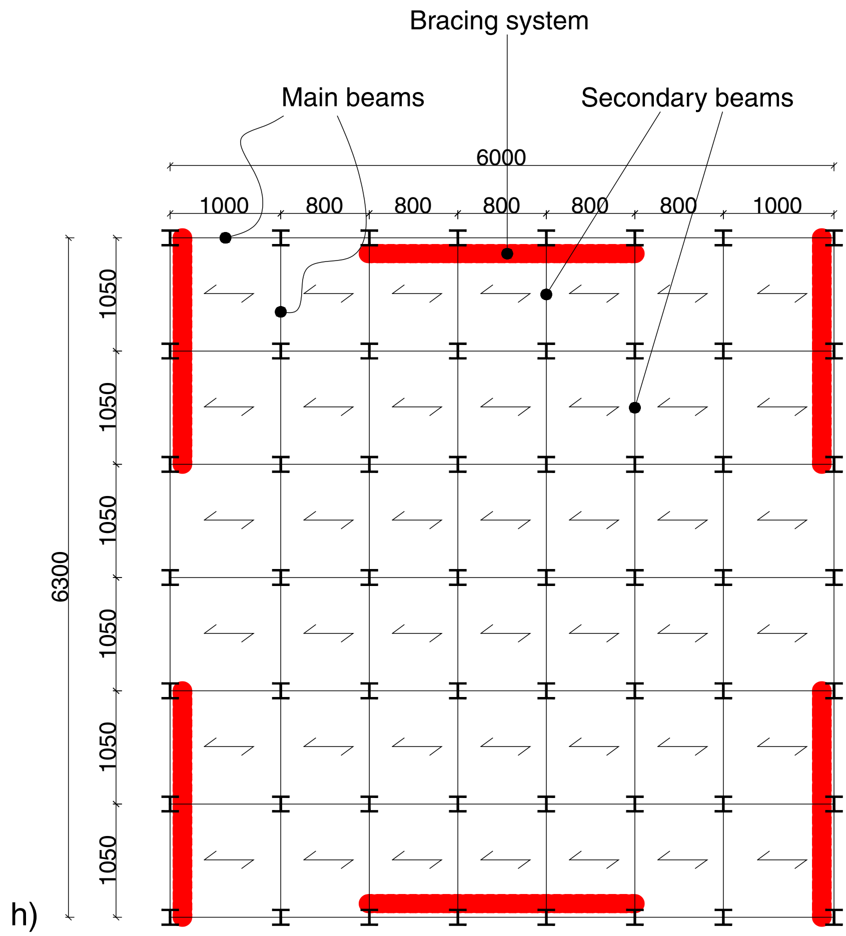

Fig. 3 (*continued*).

development of the plastic hinges. Columns were designed with the increased actions in order to respect the strong column-weak beam principle. More details of the design of composite structures can be found in [12].

Obviously, the adoption of several different braces' or beams' sections, different steel grades for braced configurations and the selection of behaviour factors for MRF optimized solutions are not commonly adopted in the current engineering design practice; as a consequence, it can be argued that the design procedure proposed by Eurocode 8 [1], without a complete and long conceptual preliminary phase, can lead to structural solutions characterized by performance rather higher than the ones strictly required by seismic load combination. What already presented confirms that EN1998-1[1] design procedure generally allows the realization of safe structures, evidencing, at the same time, that the impossibility of reaching a full optimized structural solution represents a limit that can endanger the competitiveness of these structural typologies. Table 7 summarizes the values of q factors adopted in the design (q_{design}) of bi-dimensional frames of the case study buildings.

Table 1
Structural typologies and design loads used for case studies.

Building ID n°	Building type	Material	Live Load	Snow	Wind	PGA [g]
			kN/m^2	kN/m^2	kN/m^2 (m/s)	
1	Office	Steel	3,00	0,85	0,39	0,10
2	Office	Steel	3,00	0,85	0,39	0,10
3	Office	Steel	3,00	1,00	1,10	0,25
4	Office	Steel	3,00	1,00	1,10	0,15
5	Office	Steel	3,00	1,40	(30 m/s)	0,25
6	Office	Composite beams/ Steel columns	3,00	1,11	1,40	0,10
7	Office	Composite beams and columns	3,00	1,11	1,40	0,10
8	Office	Composite beams/ Steel columns	3,00	1,11	1,40	0,25
10	Office	Composite beams/ Steel columns	3,00	1,11	1,40	0,10
11	Office	Composite beams and columns	3,00	1,11	1,40	0,25
12	Industrial	Steel	5,00	1,40	(30 m/s)	0,25
13	Industrial	Steel	Crane load (10 tons)	1,40	(30 m/s)	0,25
14	Industrial	Steel	Crane load (370+140 tons)	0,85	0,39	0,25
15	Industrial	Steel	5,00 kN/m^2 + add. dead loads (6,8 kN/m^2)	0,85	0,39	0,10
16	Car Park	Steel	2,50	1,00	1,10	0,25

Table 2
Structural and geometrical characteristics of designed case studies.

ID	Storeys	X – direction				Y – direction			
		Resisting System	Span	Second. beam	H_{storey} [m]	Resisting system	Span	Second. beam	H_{storey} [m]
1	5	MRF	3x7m	Yes	3,5	CBF	4x6m	No	3,5
2	5	CBF	3x7m	Yes	3,5	CBF	6x6m	No	3,5
3	5	EBF shear	3x7m	No	3,5	EBF shear	4x6m	Yes	3,5
4	5	EBF bending	3x7m	No	3,5	EBF bending	4x6m	Yes	3,5
5	5	MRF	3x7,5m	Yes	3,5	CBF	4x6m	Yes	3,5
6	5	MRF	3x7m	Yes	3,5	Not designed	4x6m	No	3,5
7	5	MRF	3x7m	Yes	3,5	Not designed	4x6m	No	3,5
8	5	MRF	3x7m	Yes	3,5	Not designed	4x6m	No	3,5
10	5	EBF shear	3x7m	No	3,5	CBF	4x6m	No	3,5
11	5	EBF shear	3x7m	No	3,5	CBF	4x6m	No	3,5
12	4	MRF	3x7,5m	Yes	4+4+5+7	CBF	3x10m	No	4+4+5+7
13	1	MRF	2x25m	Yes (purlins)	10,5	CBF	11x6m	Yes (purlins)	10,5
14	1	MRF truss girder	1x29m	No	21,9	CBF	7,30m	No	17,6
15	4	MRF	3x7,5m	No	4+4+5+7	CBF	3x10m	Yes	4 +4 + 5+7
16	2	EBF shear	5x8m 2x10m	No	4+4	EBF shear	6x10.5m	Yes	4+4

3. Development of structural models

3.1. Modelling and analysis of simple benchmarks

The case studies were analysed adopting sophisticated models able to capture all relevant non-linear phenomena at structural and material level, suitable for pursuing the main aims of the present work, i.e. the determination, for each structure, of the relevant collapse criteria and of their corresponding PGA activation levels and the evaluation of the seismic structural performance in terms of behaviour factor q.

Given that the numerical simulations were carried out by different partners of the OPUS project [10] using different software like Abaqus

Table 3
Steel buildings with MRF-CBF structures. In the case of building n°5: (*) for the A and E MRFs, (**) for the B, C, D frames, (') for the 1 and 4 CBFs, ('') for the 2 and 3 frames, ext: refers to external column, int: refers to interior columns.

ID	ST		Mat.	Lev.	Beams		Columns		Braces
n°	x-dir	y-dir			x-dir	y- dir	x-dir	y- dir	y-dir
1	MRF	CBF	S235	1	IPE400	IPE500	HEB400	-	CHS139.7x12.5
				2	IPE400	IPE500		-	CHS139.7x10.0
				3	IPE400	IPE500		-	CHS139.7x8.0
				4	IPE400	IPE500		-	CHS114.3x8.0
				5	IPE400	IPE500		-	CHS114.3x4.0
5	MRF	CBF	S355	1	IPE330* IPE450**	IPE270' IPE330''	HEA360 (ext*), HEA400 (int*), HEB300 (ext**), HEB360 (int**)	HEA360 (ext'), HEA400 (int''), HEB300 (ext'), HEB360 (int'')	S100x8
				2	IPE330 PE450**	IPE270' IPE330''	HEA360 (ext*), HEA360 (int*), HEB300 (ext**),	HEA360 (ext'), HEA360 (int''),	S90x8
				3	IPE330* IPE450**	IPE270' IPE330''	HEB360 (int**)	HEB300 (ext'), HEB360 (int'')	S80x8
				4	IPE330* IPE450**	IPE270' IPE330''			S75x6
				5	IPE300* IPE450**	IPE330' IPE400''			S60x4
12	MRF	CBF	S355	1	HEB360	HEB400	HEB450	-	C273,0×8.0
				2	HEB360	HEB400		-	C273,0×8.0
				3	HEB360	HEB400		-	C273,0×7.1
				4	HEB360	HEB400		-	C273,0×7.1
	MRF	CBF	S460	1	HEB320	HEB340	HEB400	-	C273,0×7.1
				2	HEB320	HEB340		-	C273,0×7.1
				3	HEB320	HEB340		-	C244,5×7.1
				4	HEB320	HEB340		-	C244,5×7.1
13	MRF	CBF	S235	1	IPE500	IPE120	HEA500	-	2L120x120x20
	MRF	CBF	S275	1	IPE450	IPE120	HEA500	-	2L120x120x20
14	MRF + truss girders	CBF	S355 S235 for some braces	1	Truss girder as part of MRFs chords: HEA180	Simply supported purlins: HEA200.	main col.: HEB100	-	CHS193.7x8 (diss)/ 2HEA240 (nd)
				2	diag.: CHS139.7x4	Fully rigid connectors HEA140	roof col.: HEA450	-	CHS193.7x8 (diss)/ 2HEA240 (nd)
				3				-	CHS193.7x8 (diss)/ 2HEA240 (nd)
				4				-	*CHS168.3x6.3 (diss)/2HEA240 (nd)
				5				-	*CHS168.3x6.3 (diss)/2HEA240 (nd)
15	MRF	CBF	S355 S235 for braces 4-5th floor	1	IPE550	HEA700	HEB700	-	CHS 244.5x8.0
				2	IPE500	HEA700		-	CHS 244.5x6.0
				3	IPE500	HEA700		-	CHS 193.7x10
				4	IPE500	HEA700		-	CHS 193.7x4

Table 4
Steel buildings with CBF and EBF structure.

ID/ST/Lev.			Beams		Col.	Braces		Links		Ω_i	
			x-dir	y-dir		x- dir	y- dir	x	y	x	Y
2	CBF S235	1	IPE 400	IPE 500	HEB 340	CHS139.7x12.5	CHS139.7x12.5	-	-	1.14	1.28
		2				CHS139.7x10	CHS139.7x10	-	-	1.11	1.19
		3				CHS139.7x8	CHS139.7x8	-	-	1.11	1.11
		4				CHS139.7x8	CHS114.3x8	-	-	1.32	1.20
		5				CHS139.7x4	CHS114.3x4	-	-	1.37	1.18
3	EBF S355	1	IPE 500	IPE 360	HEB 300-320	HEB220	HEB 280	HEB200	HEB200	1.66	2.12
		2				HEB220	HEB280	HEB180	HEB200	1.54	2.47
		3				HEB220	HEB260	HEB160	HEB160	1.53	2.00
		4				HEB200	HEB260	HEB140	HEB140	1.62	2.03
		5				HEB200	HEB 260	HEB120	HEB100	1.86	2.24
4	EBF S355	1	IPE 500	IPE 360	HEB 240-260	HEB 200	HEB 200	IPE270	IPE270	1.68	1.99
		2						IPE270	IPE270	1.87	1.74
		3						IPE240	IPE240	1.63	1.78
		4						IPE220	IPE220	1.66	1.76
		5						IPE160	IPE160	1.51	1.61
16	EBF S275	1	IPE 600	IPE 600	HEB 240	HEB 280	HEB 260	HEB320	HEB300	1.53	1.57
		2						HEB360	HEB280	1.88	1.91

[26], FineLG [27], Dynacs [28] and OpenSees [29], a calibration process was executed comparing their simulation capabilities with simple benchmarks: different structures (a bracing element, a portal frame and a concentrically braced frame, Fig. 3) were modelled and used as calibration case studies: modelling parameters employed for the definition of numerical models of the structures previously design were consequently fixed [10]. In particular:

- Benchmark n°1: simple bracing elements with initial imperfection modelled according to Eurocode 3 to directly introduce buckling phenomena. Four different cross sections (H, U, Tubular and L) were considered, two steel grades qualities, (S235 and S355) and two different slenderness (low bound $\lambda=1.3$, upper bound $\lambda=2$). A cyclic displacement history was adopted for the analysis: the elastic buckling displacement is $\delta_0 = \frac{N_{b,Rd} \cdot L}{EA}$[19], being $N_{b,Rd}$ the buckling resistance of the considered element with length L, area section A and elastic modulus E. For the cyclic displacement history is used $\delta = \{0, -\delta_0, +\delta_0, -2\delta_0, +2\delta_0, -3\delta_0, +3\delta_0, \ldots\}$. The scheme of benchmark 1 is presented in the Fig. 4a.
- Benchmark n°2: simulation of a simple portal frame subjected to static vertical loads (P) and to push-over load (V). Columns' profile HEA 400 and beam's profile IPE 400 were adopted. The vertical load applied P was equal to $P = 0.3 \cdot N_{pl,Rd} = 0.3 \cdot A \cdot f_{yk}/\gamma_{M0}$. The scheme of benchmark 1 is presented in the Fig. 4b.
- Benchmark n°3: portal frame with concentric braces (UPN 160), according to the selected value of $\lambda=1.5$ (Eurocode 8 boundaries 1.3-2.0). Columns' profile HEA 300 and beam's profile IPE 200 were adopted. The lateral force V is a load was applied according to the ECCS45 loading protocol. The scheme of benchmark 3 is presented in the Fig. 4c.

Fig. 5 presents the comparison of the results obtained from different software adopted for the validation of the procedure. For sake of simplicity, the results of benchmark n°1 are presented only for the case of section HEA 200, steel grade S235 and slenderness ratio 1.3.

3.2. Elaboration of buildings' models

According to what decided in the framework of the European research project Opus [10], in order to have the possibility to directly compare the results of numerical analyses, the same bilinear schematization for the stress-strain model of steel was adopted in all the simulations.

The structural behaviour of buildings 1, 2, 14 and 15 was investigated by using the finite element program Dynacs [28]. The structures were modelled using bi-dimensional frames with fibre beam elements, with increasing element density in dissipative regions of the MRFs were

Table 5
Composite structures with MRF system (no braces), frames in Y direction are not designed.

ID	ST	Lev.	Beams (x-direction)					Columns
6	Composite beams/Steel columns	1	Fully rigid IPE 360	Concrete effective width [mm]	EC4	Mid.	1225	HEA 450
	X dir: MRF	2				End.	875	
	Y dir: not des.	3			Elastic EC8	M^-	700	
		4				M^+	525	
		5			plastic EC8	M^-	1400	
		Materials: S235 – C25/35				M^+	1050	
7	Composite beams/ Composite columns	1	Fully rigid IPE 360	Concrete effective width [mm]	EC4	Mid.	1225	HEA 400 Reinfor. steel 4Φ24 mm
	X dir: MRF	2				End	875	
	Y dir: not des.	3			Elastic EC8	M^-	700	
		4				M^+	525	
		5			plastic EC8	M^-	1400	
		Materials: S235 – C25/35				M^+	1050	
8	Composite beams/	1	Fully rigid IPE 360	Concrete effective width [mm]	EC4	Mid.	1225	HEA 400
	Steel columns	2				End	875	
	X dir: MRF	3			Elastic EC8	M^-	700	
	Y dir: not des.	4				M^+	525	
		5			plastic EC8	M^-	1400	
		Materials: S355 – C30/37				M^+	1050	

Table 6
Composite structures with bracing systems (EBF X-direction, CBF Y-direction).

ID	ST	Lev.	Beams		Bracings (X)		Bracings (Y)	Column
n°			x- dir	y- dir	Link	Braces	Braces	
10	Composite beams/ Steel columns. S275	1	IPE 270 + slab 0,18 m	IPE 270 + slab 0,18m	HEB 260	HEB 180	UPE 160	HEB 300
	X dir: EBF (vert. shear).	2			HEB 260		UPE 200	strong axis X
	Y dir: CBF	3			HEB 220		UPE 160	
		4			HEB 200		UPE 120	
		5			HEB 160		UPE 80	
11	Composite beams/ Steel columns. S355	1	IPE 270 + slab 0,18 m	IPE 270 + slab 0,18m	HEB 450	HEB 240	UPE 180	HEB 260 strong axis X, except
	X dir: EBF (vert. shear).	2			HEB 450		UPE 200	for ground floor (HEB280)
	Y dir: CBF	3			HEB 400		UPE 180	
		4			HEB 340		UPE 140	
		5			HEB 280		UPE 100	

plastic hinges are expected (i.e. columns' base, beam-column connections). The non-linear material behaviour was modelled adopting a bilinear law with kinematic hardening described by yielding stress, tensile stress and ultimate elongation. The braces were modelled using special developed non-linear springs elements, able to represent the elastic-plastic cyclic behaviour under tension, the global buckling under compression and also the cyclic degradation. The analyses took into account large deformations to evaluate the influence of the P-Δ effect.

Buildings 6, 7, 8, 10 and 11, characterized by a composite steel/concrete structure, were assessed using the non linear finite element software FineLG [27]. The composite elements were modelled using fibre beam elements including a steel part and a concrete one; the concrete part was assumed to be a rectangular beam element with width equal to the effective one evaluated according to Eurocode 4 [21]. For what concerns the steel part, a simplified bilinear constitutive relationship, using the yielding and tensile strength and the ultimate elongation, was used, as widely specified in [12]. Diagonal members of EBF and CBF structures were modelled using steel beam element directly taking into account the possible lateral buckling under compression. The model of seismic links in EBF needed to include the yielding in shear, generally not possible with the adoption of classical fibre models: as a consequence, the dissipative links were modelled through a classical non-linear beam element, whose mechanical parameters were calibrated using as reference a refined FEM model able to describe the shear deformation and yielding and the flexural behaviour of the link, as briefly schematized in the Fig. 6.

Buildings 5, 12 and 13 were modeled by means of the Abaqus software [26]: beams and columns were modelled through 3-node quadratic beams in plane for MRF and 3-node quadratic beams in space for concentrically braced frames. An elastic-plastic model with linear kinematic hardening was adopted for the modelling of steel structural elements.

Buildings 3, 4 and 16 were modeled by means of the numerical software OpenSees [29]. The structural elements, beams, columns, braces and links, were modeled using fiber elements in which the combined effect of axial forces and bending moments were directly taken into account together with the an appropriate constitutive law for modelling, when necessary, the behavior of the structural element exposed to shear solicitations. Columns, beams and shear links were modelled using one element per structural member, two elements were used for modeling each long bending link and four elements were employed for each brace. Buckling phenomena of braces were directly taken into account giving an initial imperfection equal to 1/500 of the brace length to the middle point of the brace itself, as represented in Fig. 7a; a similar imperfection was also assigned to the top of the columns in order to include in the analysis P-Δ effect (Fig. 7b). The value adopted for the

Table 7
Summary of main design characteristics, typology of storey slabs, total mass of the building, first vibration periods in the two main directions and values of q factor assumed for design.

ID	X – dir	Y – dir	PGA	Slab typology	T_{1x}	T_{1y}	Mass	q_{Design}	
	System	System	[g]		[s]	[s]		x	y
1	MRF	Not des.	0.10	composite 18 cm (not des.)	1,28	-	2975 t	2.35	3.99
2	CBF	Not des.	0.10	composite 18 cm (not des.)	1,45	-	2975 t	3.68	4
3	EBF shear	EBF shear	0.25	concrete slab cast on prefabricated trussed slab, 23 cm, wire	1,15	0,82	1747 t	6	6
4	EBF Bend.	EBF Bend.	0.15	mesh 1 ϕ8/150 mm, bars 2ϕ10 / slab rib.	1,06	1,24	1747 t	4	4
5	MRF S355	CBF S355	0.25	concrete, 12 cm (not des.)	1,85	0,83	1000 t	4	4
	MRF S460	CBF S460		concrete, 12 cm (not des.)	2,10	0,87	920 t		
6	MRF	Not des.	0.10	concrete, 12 cm - reinf. 3/m - 5ϕ10 (X) and 5ϕ8 (Y), upper and	0,74	-	1916 t	4	4
7	MRF	Not des.	0.10	lower layer.	0,71	-	1995 t	4	4
8	MRF	Not des.	0.25		0,61	-	1909 t	4	4
10	EBF vertical shear	CBF	0.10	concrete 18 cm - upper reinf. (X) 10ϕ10+4ϕ16, (Y) 10ϕ10; lower reinf. 10ϕ10 +2ϕ16 (X), 10ϕ10 (Y).	0,89	1,45	1750 t	4.00	4.00
11	EBF vertical shear	CBF	0,25	concrete 18 cm - upper reinf. (X) 10ϕ10+4ϕ16, (Y) 10ϕ10; lower reinf. 10ϕ10 +4ϕ16 (X), 10ϕ10 (Y).	0,83	1,45	1745 t	4.00	4.00
12	MRF S355	CBF S355	0.25	concrete 24 cm (not des.)	1,38	0,64	0,3Q+150 t	2.33	4
	MRF S460	CBF S460		concrete 24 cm (not des.)	1,64	0,68	0,3Q+150 t	1.63	
13	MRF S235	CBF S235	0.25	-	0,53	0,13	370 t	4.00	4.00
	MRF S275	CBF S275		-	0,56	0,32	350 t		
14	MRF truss girder	CBF	0.25	-	1,06	0,89	792 t	1.55	3.8
15	MRF	CBF	0.10	composite 18 cm (not des.)	1,76	1,33	4107 t	1.96	2.64
16	EBF shear	EBF shear	0.25	concrete slab cast on prefabricated trussed slab, 23 cm, wire mesh 1 ϕ8/150 mm, bars 2ϕ10 / slab rib.	0,45	0,40	5761 t	6.00	6.00

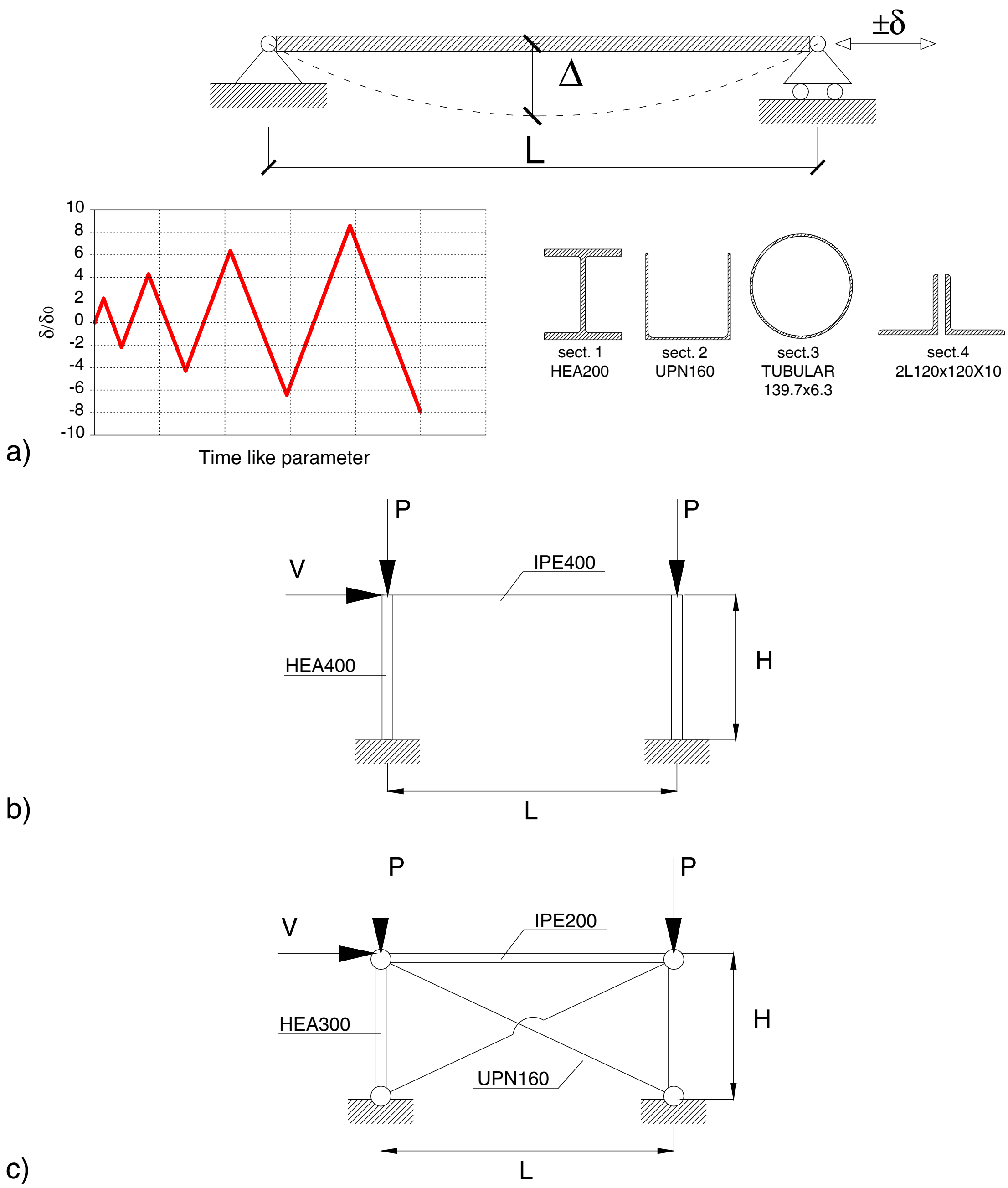

Fig. 4. a) Simple brace under tension/compression cycles, b) one storey-one bay portal frame, c) simple braced frame.

imperfection was evaluated using results from the literature and the provision of the Eurocodes.

The modeling of the flexural behavior of steel members (beams, braces and columns) was made using the Menegotto-Pinto law [30], characterized by bilinear elastic-plasticstress-strain curve with kinematic hardening and accurately calibrated in order to agree with results from the literature. Moreover, the force-angular distortion law, used for modelling the shear behavior of links, was a bilinear elastic-plastic law with kinematic hardening. The seismic mass of the building, evaluated according to Eurocodes' prescriptions, was divided between to the two (rigid) EBF frames in the two main directions of the building, with the application of concentrated masses in correspondence of the top of the columns.

Figs. 8 present some examples of three-dimensional models elaborated for the different case study buildings.

4. Analysis of structural performance

4.1. Definition of collapse criteria

Seismic demand levels are usually defined in relation to performance levels such as Damage Limitation (DL), Severe Damage (SD) and Near Collapse (NC). The investigations herein presented were carried out for the SD performance level, according to EN1998-3:2005 and corresponding to an earthquake hazard level with an average return period (T_R) of 475 years.

A crucial point in the assessment of the structures using non-linear static and dynamic analysis is the definition of the limit states and of the associated structural performance under seismic actions. As general rule, deformation criteria (i.e. the over-passing of the interstorey drift limit, the reaching of the ultimate rotation of beams…) or local ductility

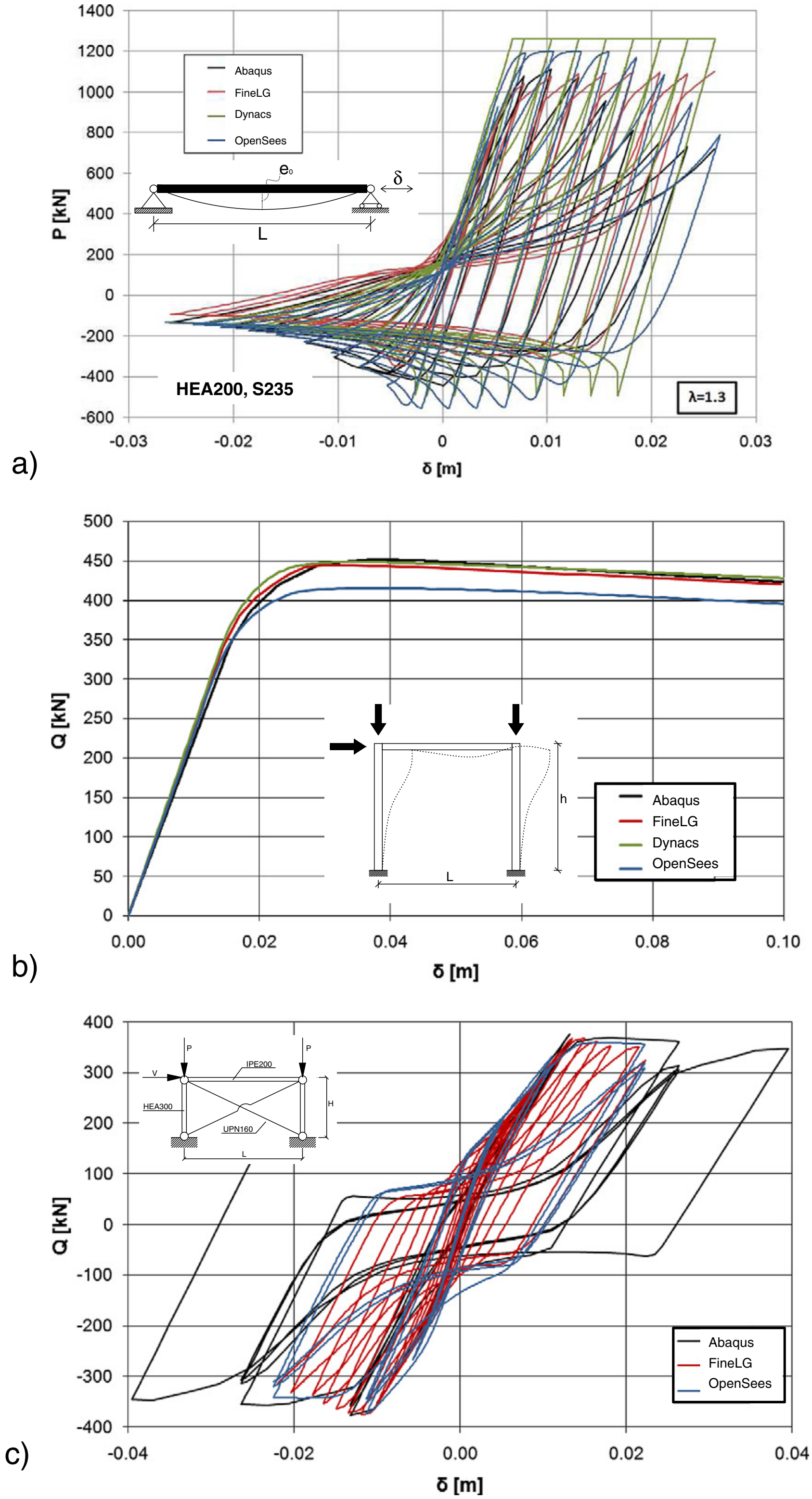

Fig. 5. Benchmarks adopted for the calibration of the software adopted and the validation of results: a) simple brace under cyclic action, section HEA200 steel grade S235 and slenderness ratio 1.3, b) one storey-one bay frame, c) simple braced frame one storey one bay.

criteria were selected as main indicators. Furthermore, non seismic-specific verifications as shear capacity, global buckling, and others shall be also carried out. The global deformation criteria as roof- and storey- drift were defined according to FEMA356 [23] and used only as indicative values. Additionally, the maximum forces acting in the connections and at foundation level were recorded for further investigations.

All of the verifications were carried out using a time-integratedapproach, i.e. only the maximum value of the monitored structural parameters, during a time history analysis, was recorded.

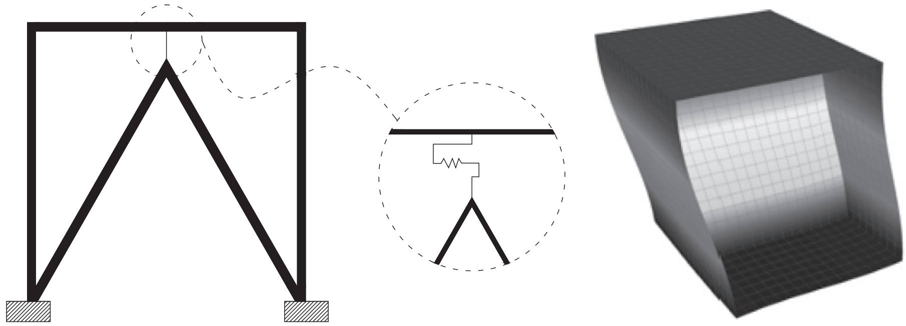

Fig. 6. Modelling of the seismic shear link.

Moreover, the global buckling and lateral torsional buckling were manually checked in the relevant time step of each simulation.

The limit axial load for the buckling of steel members in compression $N_{b,Rd}$ (columns and braces) was evaluated according to Eurocode 3 with expression (8), and was consequently strongly influenced by the mechanical properties of materials (yielding strength f_y):

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \tag{8}$$

in which f_y is the yielding strength of the material, A is the transversal area of the element's section, γ_{M1} is the partial safety coefficient and χ is the reduction factor for the relevant buckling mode.

All limit states considered in the case studies with moment-resisting (MRF) and concentrically braced steel frames (CBF) are presented respectively in Tables 8 and 9.

For eccentrically braced frames (EBF) one of the most conditioning collapse criteria was obviously the failure of link elements, in which plastic deformation are concentrated according to the capacity design approach. The plastic rotation (or angular distortion) of the shear links was evaluated using the ratio between the relative vertical displacement (δ) and the link length (e):

$$\frac{v_1 - v_2}{e} = \frac{\delta}{e} = \gamma_{LINK} \tag{9}$$

where for shear short link δ was evaluated as the relative vertical displacement between the two ends of the link, respectively v_1 and v_2(Fig. 9a) and for long bending links δ referred to the mid length of the element (Fig. 9b). The limits assumed for EBF are presented in the Table 10.

4.2. Pushover analysis and q-factor evaluation

Non linear static analyses (pushover – PO) were executed on the representative buildings previously described in order to evaluate the relevant collapse criteria and the corresponding available behaviour factor (q_{PO}). Pushover analyses were performed on each structure using a monotonically increasing triangular pattern of lateral loads, applying, at the same time, the vertical loads ($G+0.3Q$), due to structural weights (G) and live load (Q). The lateral loads were applied monotonically in a step-by-step nonlinear static analysis, as simply represented in the Fig. 10 in the case of building 5.

The behaviour factor (q_{PO}) was determined on the basis of the base shear-displacement curve (i.e. capacity curve) using the Eq. (10), taking into account two different contributions, the first one (μ) directly related to the structural ductility of the building (in terms of displacement) and the second one depending on the overstrength (Ω):

$$q_{PO} = \mu \cdot \Omega = \frac{d_u}{d_y} \cdot \frac{V_u}{V_y} \tag{10}$$

being d_y the displacement at the first plastic hinge, V_y the corresponding base shear, d_u the displacement at the first failure criteria and V_u its corresponding base shear [31].

4.3. Incremental dynamic analysis and q-factor evaluation

Incremental Dynamic Analyses (IDA) were executed on the designed buildings adopting artificial accelerograms, generated through the elastic response spectrum, gradually up-scaled in their peak ground acceleration until the collapse of the structure was reached. According to Eurocode 8 [1] prescriptions, seven earthquake time-histories of natural or artificially generated earthquakes were adopted. Artificial accelerograms meeting the elastic response spectra presented in Eurocode 8 and consistent with chosen hazard model were adopted

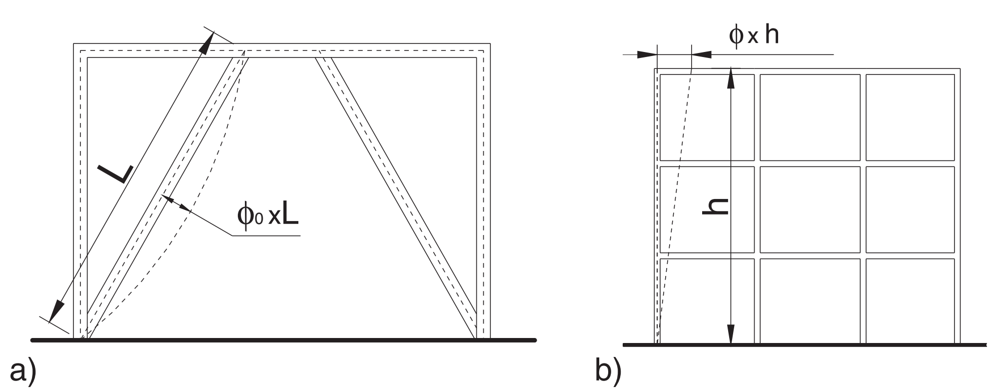

Fig. 7. a) General scheme of fibre elements and b) model of imperfections of braces and columns.

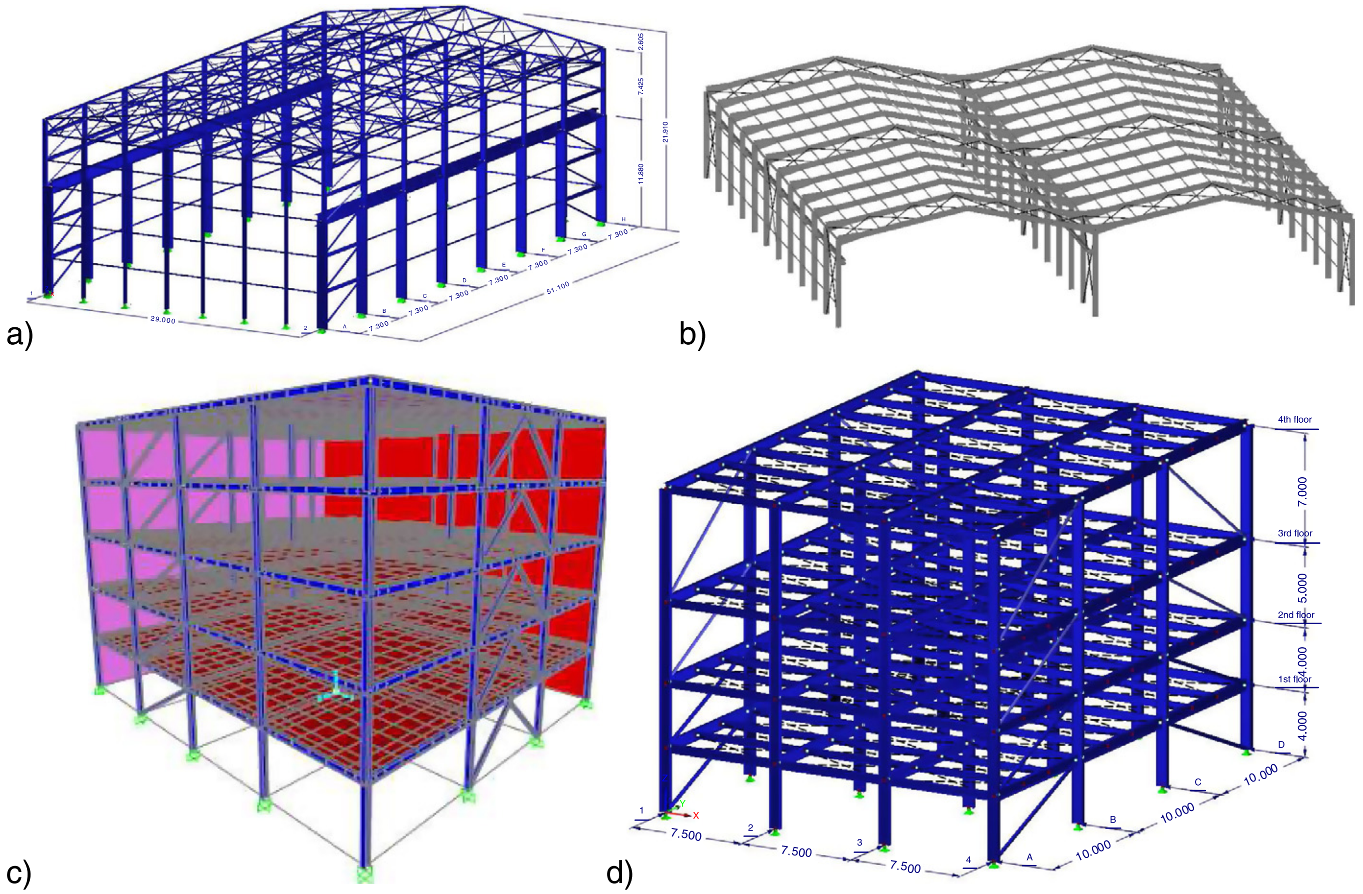

Fig. 8. a) Industrial building for electrical power plant activities; b) industrial building for warehouse/light activities; c) EBF and CBF configurations for offices; d) MRF and CBF configurations for industrial storage.

for numerical simulations. The program SIMQKE, developed by Gasparini and Vanmarcke [32] was used for generating the time histories.

Two types of seismic actions were considered: for the high seismicity areas if was assumed a PGA equal to 0.25 g and a type 1 spectrum for soil category B[1]; for the low seismicity areas a PGA equal to 0.10 g and a type 2 spectrum for soil type C[1](Fig. 11a, Table 11). A filter function, trapezoidal shaped, was defined in order to have a strong motion duration equal to 10 s and 5 s, respectively, for high and low seismicity (Fig. 11b). The chosen sampling interval of $\Delta t = 0.01$ s allowed a sufficient accurate calculation for the Eigen-frequencies up to 20 Hz (5 points for each period). Moreover, the verification of the accelerograms by determining the velocity and the displacement time histories showed that the displacements were running out (Fig. 12), evidencing the necessity of the application of a baseline correction in order to obtain a sufficient small displacement at the end of the record. The adequacy of the accelerograms was checked by determination of their elastic response spectra (Fig. 13): for periods lower than T_B the spectral acceleration S_a was slightly too high (Fig. 14). Anyway, the target spectrum was sufficiently met and so the requirements defined in EN1998-1:2005 [1].

The COV of the spectral values for the 7 accelerograms varied between 0.04 and 0.12 (Fig. 15) and the energy density of artificial accelerograms was generally higher than the one of natural accelerograms, since all interesting frequencies are included.

The PGA able to activate relevant collapse modes for each designed structure were used for the evaluation of the behaviour factor using a "traditional" Ballio-Setti[16,17] procedure slightly modified for taking

Table 8
Failure criteria for buildings with MRF. (*) for axial load ration $0.3 < n \leq 0.5$ linear reduction of rotation capacity in acc. to FEMA356; (**) Lateral torsional buckling of beams is prevented by RC-floor (no composite action).

	Type	Reference	Criteria
A	Dynamic instability (Global)	-	Limit
B	Maximum roof drift ratio (Global)	FEMA 356	Indicative
C	Inter-storey drift ratio (Global)	FEMA 356	Indicative
D	Ultimate rotation of plastic hinges (Local) *	EN1998-3	Limit
E	Shear capacity (Local)	EN1993-1	Limit
F	Lateral torsional buckling (Local) **	EN1993-1	Limit
G	Global buckling (Local)	EN1993-1	Limit
H	Joint forces	-	Evaluation
I	Foundation forces	-	Evaluation

Table 9
Failure criteria for buildings with CBF. (**) Lateral torsional buckling of beams is prevented by RC-floor (no composite action).

	Type	Reference Code	Criteria
A	Dynamic instability (Global)	-	Limit
B	Maximum roof drift ratio (Global)	FEMA 356	Indicative
C	Inter-storey drift ratio (Global)	FEMA 356	Indicative
L	Ultimate deformation, tension (Local)	EN1998-3	Limit
M	Ultimate deformation, compression (Local)	EN1998-3	Limit
E	Shear capacity (Local)	EN1993-1	Limit
F	Lateral torsional buckling (Local) **	EN1993-1	Limit
G	Global buckling (Local)	EN1993-1	Limit
H	Joint forces	-	Evaluation
I	Foundation forces	-	Evaluation

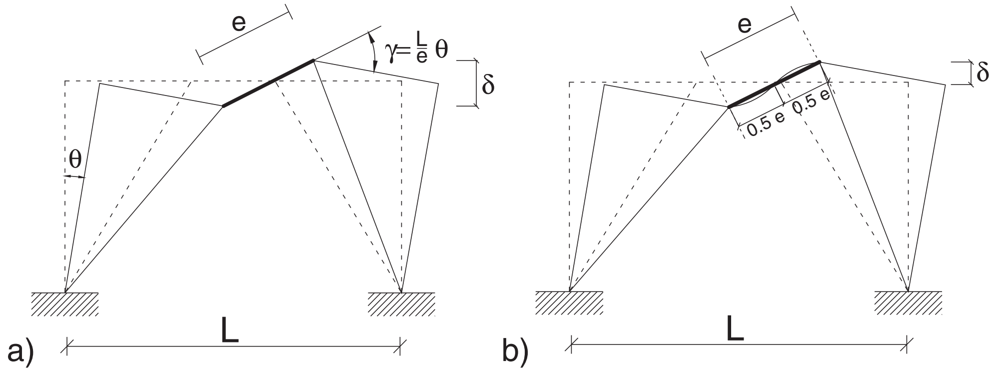

Fig. 9. Evaluation of link plastic rotation a) for short shear links, b) for long bending links.

into account the discrepancy at first mode frequency between the target spectra and real spectra of earthquake time-histories(Fig. 16). The available q-factors evaluated on the basis of IDAs'(q_{IDA}) results were determined according to Eq. (11):

$$q_{IDA} = \frac{\lambda_u}{\lambda_{e,static}} \cdot \frac{a_{s,art}}{a_{sd}} \tag{11}$$

where λ_u is the accelerogram multiplier at the first limit state, $\lambda_{e,static}$ is the multiplier of the equivalent static seismic forces corresponding to the first attainment of the plastic hinge in a non linear pushover analysis, $a_{s,art}$ is the acceleration of the spectrum of the accelerogram used in the current simulation and a_{sd} the acceleration of the design spectrum, both calculated at the fundamental period of the structure. In the classic Ballio-Setti approach, the second term, defined by the ratio between design spectrum PGA and artificial earthquake spectrum PGA, is not considered and the discrepancy between two PGA levels can consequently strongly influence seismic behaviour assessment and the assessment of the behaviour factor.

4.4. Material's mechanical properties investigation

The assessment of the effective behaviour factors, both coming from PO and IDA according to the methodology previously described, was executed considering the mean values of the real mechanical properties of materials: in the OPUS project [10] in fact, a wide statistical investigation of the real mechanical properties of the steel grades actually used in Europe for steel and composite steel-concrete structures (i.e. S235, S275, S355 and S460 with different levels of toughness) was executed, collecting experimental data from the two producers involved in the research project and related to steel profiles characterized by two different levels of thickness (0-16 mm and 16-40 mm). The adoption of the real values of the mechanical properties (in terms of yielding strength - $R_{e,H}$, tensile strength – R_m and ultimate elongation - A) allowed the identification of the effective behaviour factors in relation to the one adopted in the design process, evidencing the agreement or, otherwise, the discrepancy with the design procedure provided by the Eurocode 8 [1].

The complete data analysis is deeply presented in the Final Report of the OPUS project [10]; in the present paper, in order to determine the values effectively adopted for the numerical simulations, only some summarizing information are provided.

The scattering of the mechanical properties was analyzed in comparison with the requirements imposed by the production standards [7] and the seismic requirements imposed by EN1998-1-1[1] during the design. In the Fig. 16 the statistical distribution, represented as a domain contained in the 5%-95% percentile curves, is compared with the minimum yielding stress curve imposed by EN10025 [7] and with the maximum yielding stress allowable to a steel profile for being effectively

Table 10
Failure criteria for buildings with EBF.

	Type	Reference code	Criteria
A	Dynamic instability (Global)	-	Limit
B	Maximum roof drift ratio (Global)	FEMA 356	Indicative
C	Inter-story drift ratio (Global)	FEMA 356	Indicative
N	Ultimate rotation of link (Local)	FEMA 356	Limit
E	Shear capacity (Local)	EN1993-1	Limit
G	Global buckling (Local)	EN1993-1	Limit
H	Joints forces	-	Evaluation
I	Foundation forces	-	Evaluation

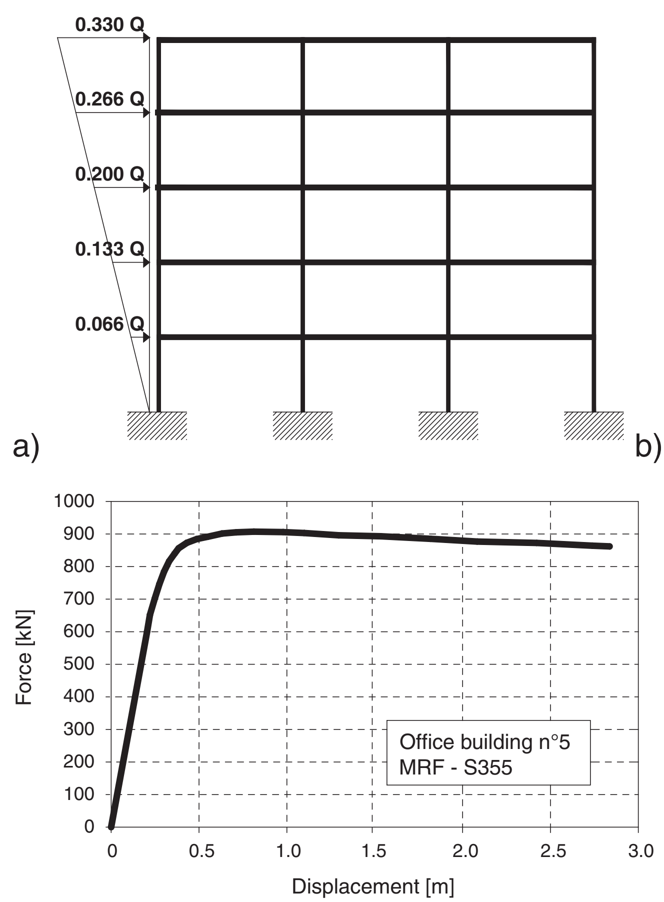

Fig. 10. a) Pushover analysis in Abaqus for 2D MRF (X-direction) of Office Building 5, b) Diagram Q versus δ, for pushover analysis in Abaqus, for MRF and S355.

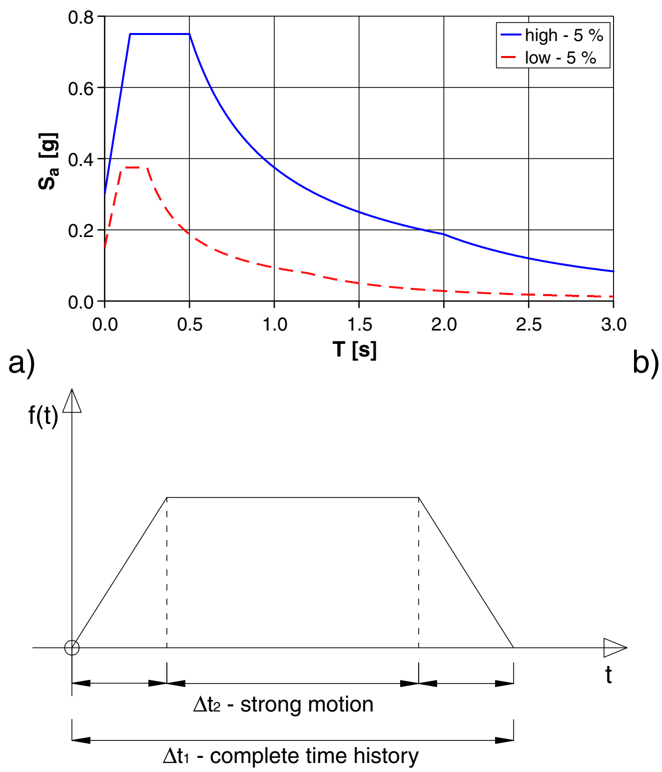

Fig. 11. Target spectra (a) and filter function (b) for the generation of artificial time histories.

employed in compliance with the Eurocode 8 [1]. It stands to reason that samples characterized by thickness lower than 16 mm were mostly contained in these limits (Fig. 17), whereas higher thickness samples have part of their statistical area outside the domain (i.e., exceeding the upper limitations imposed by Eurocode 8 [1]). This circumstance is due to the absence of an upper limitation for yielding stress in the steel production standard.

Similar considerations were carried out on data of the tensile strength (R_m) and of the ultimate elongation (A) and can be found in the research final report [10]. According to the results of the statistical investigation, main parameters (mean, standard deviation, Co.V and others) were adopted for the elaboration of a numerical model necessary to generate samples to be used in the non linear analyses when the seismic reliability should be assessed. On the other hand, the evaluation of behaviour factor – deterministically made – was carried out adopting the mean values of mechanical characteristics presented in Table 12.

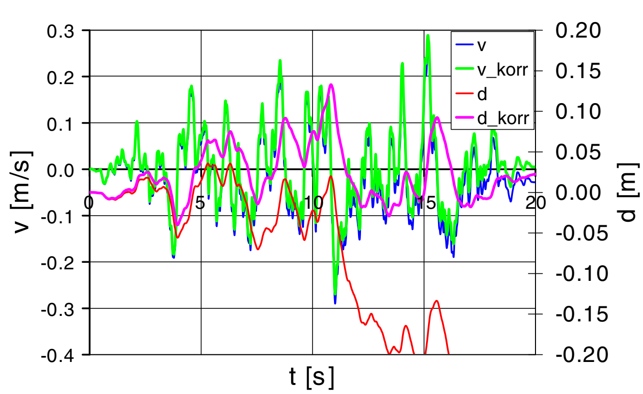

Fig. 12. Baseline correction for an artificial accelerogram (high seismicity).

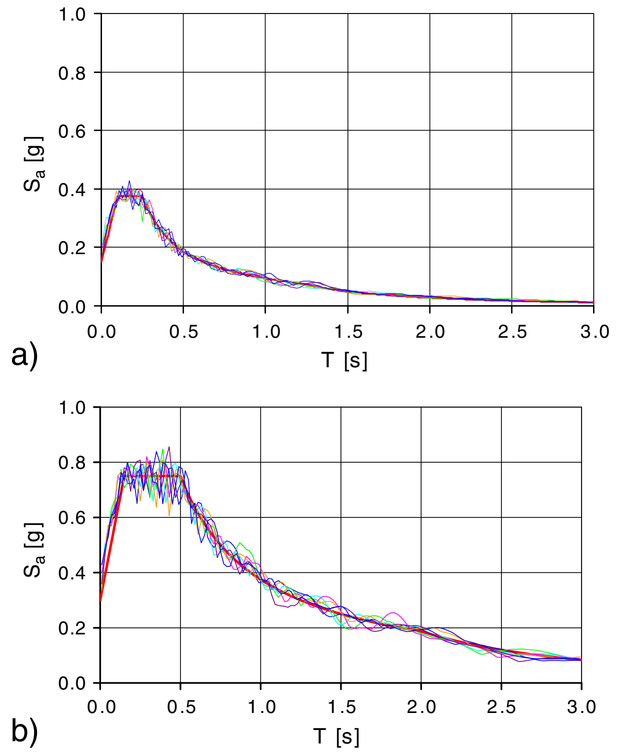

Fig. 13. Target and elastic response spectra of 7 artificial accelerograms: a) low and b) high seismicity.

4.5. Analysis of results and assessment of q-factor

In the following the assessment of behaviour factors is presented considering buildings in relation to their structural typology, as well as described in the Tables 3, 4 and 5.

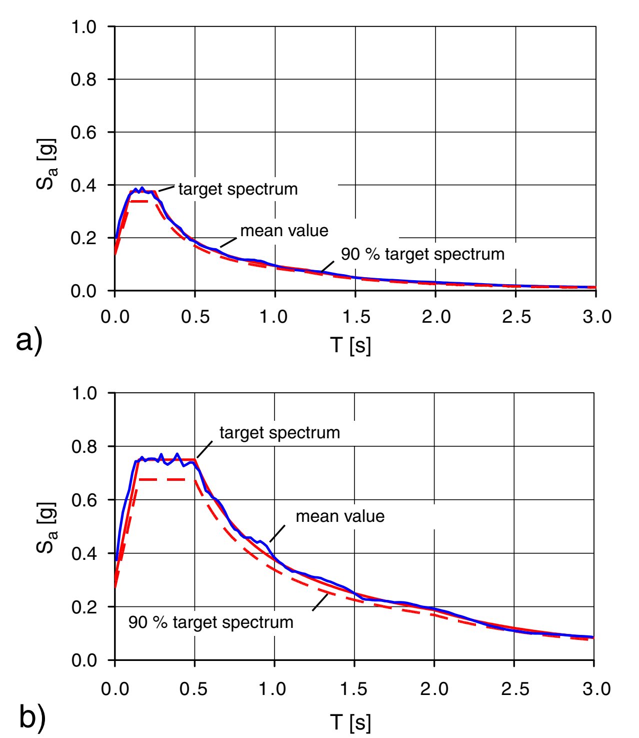

Fig. 14. Target and mean value of the elastic response spectra of 7 artificial accelerograms: a) low and b) high seismicity.

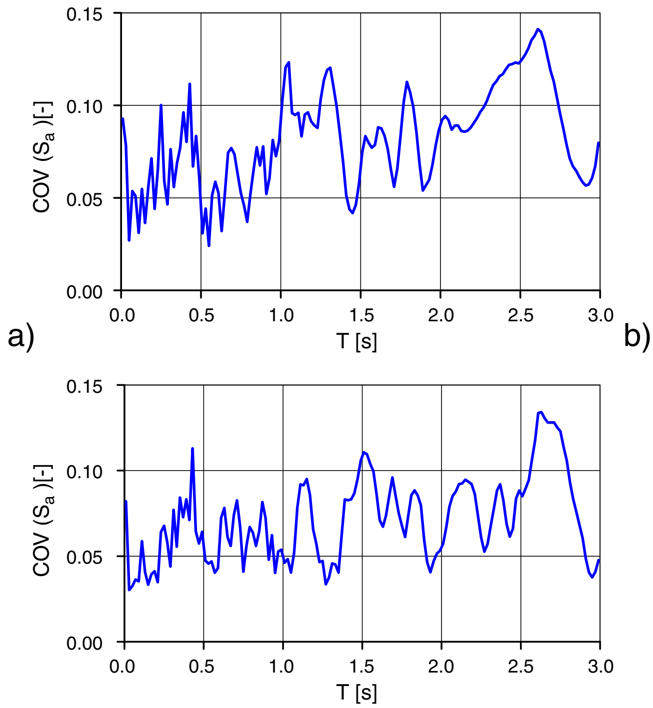

Fig. 15. COV of the elastic response spectra of 7 artificial accelerograms: a) low and b) high seismicity.

Data of the effective q factor were compared with the ones adopted in the design, according to what presented in Table 7. Results are presented for:

1. Steel buildings with combined MRF and CBF structure, i.e. buildings characterized by a different structural typology in relation to the two main direction of the building (in general MRF structure in the case of X-direction and CBF structure in Y-direction): buildings 1, 5, 12, 13, 14 and 15.
2. Steel structures with bracing system: EBF structures, i.e. buildings characterized by the presence of link elements working in shear or in bending with different positions inside frames in both X and Y direction – buildings 3, 4 and 16, and CBF structure, characterized by concentric braces in both X and Y direction – building 2.
3. Composite steel/concrete buildings, i.e. structures characterized by the presence of composite beams and steel columns (buildings 6, 8 and 10) or by the presence of composite beams and composite columns (buildings 7 and 11). In the case of composite buildings, the structural typology can vary among MRF (buildings 6, 7 and 8) and combined EBF (X-direction) and CBF (Y-direction) in the case of buildings 10 and 11.

4.5.1. Combined MRF-CBF structures

Buildings 1, 14, 15, 5, 12 and 13 are characterized by the presence of different structural typologies in the two main directions of the building: MRF in x-direction and CBF in y-direction, as already specified in Table 3. The values of the behaviour factors adopted in the design are presented in the Table 7.

Moreover, buildings 5, 12 and 13 were designed considering the adoption of two different steel grades (S355 and S460 or S235 and S275), as specified in Table 3, aiming to identify the influence of materials properties in the efficiency of the design procedure. The models and the analyses of these buildings were elaborated and executed using, respectively, Dynacs and Abaqus software for buildings 1, 14, 15 and 5, 12 and 13. The values of q-factors coming from PO(q_{PO}) and IDA(q_{IDA}) are presented in the Table 13, with reference to the ones assumed in the design. In the case of buildings 1 and 14 behaviour factor was evaluated only for X-direction (MRF).

The results of numerical simulations pointed out that MRF structures bear PGA levels higher than the ones adopted in the initial design process, usually executed using the lateral force method (Table 7). The values of q_{IDA} were generally higher than the ones assumed in the design process, with values equal to 6.7 in the case of building 1, 7.0 in the case of building 14 and rather equal to 9.3 in the case of building 15.

The high reserves of strength of those structures ware related to the fulfilment of some seismic design requirements leading to a general over-sizing of the structure: the resistance effectively required for the applied seismic design loads was lower respect to the resistance required by seismic design requirements (i.e. inter story drift). Such effects were usually more considerable in the case of structures designed for moderate seismic actions (i.e. buildings 1 and 15 with design PGA equal to 0.10g), since the same seismic design requirements were applied to ensure a sufficient performance of structures both for low and high seismic actions.

In general, for MRF structures the ultimate rotation ratio was the controlling failure criterion. In the offices, as well as in the industrial buildings, the columns were the critical elements, whose ultimate rotation capacity was often reduced as a consequence of the applied axial loads. The scattering of the ultimate rotation ratio between accelerograms was considerably high (80 – 140 % and 80 – 130 %). The other failure criteria, except for the interstorey drift ratio, were not particularly significant. The behaviour factors resulting from the simulations were consequently rather low in the case of monotonic pushover analysis, with values between 2.0 and 3.0, while higher values were obtained considering the modified Ballio-Setti procedure, with values around 6-7 for MRF of buildings 1 and 15 and equal to 3.2 in the case of building 14.

Looking at the behaviour of CBF frames (Y-direction of buildings 5, 12, 13 and 15), usually the ultimate deformation of braces in compression was the governing failure criterion, on the other hand neglected in the present analyses since buckling was directly included in the non linear models through the representation of imperfections according to Eurocode 3 [20]: as a consequence, the investigations were focused on the deformation in tension criterion.

The capacity ratios of the other failure criteria were rather low in the case of buildings 12, with values between 1.6 and 5.4 in relation to the steel grade adopted, respect to 4.0 assumed in the design. The scattering of the results adopting non linear static or dynamic analyses was evident also in the case of CBF structures, with generally higher values in the case of IDAs due to the high levels of the ratio $\lambda_u/\lambda_{e,static}$. Finally, the behaviour factors obtained do not vary in a significant way in relation to different steel grades adopted, with difference generally lower than 5%.

4.5.2. EBF and CBF steel structures

EBF buildings were designed adopting the prescriptions for High Ductility Class (DCH) in high seismicity (design PGA 0.25g) and using short shear links for buildings 3 and 16, and the ones for DCM in low seismicity for building 4 (design PGA 0.10g) with long bending links. The design q factors were equal to 6 (buildings 3 and 16) and 4 (building 4); the models and the analyses were elaborated and executed using OpenSees software [29].

In the case of buildings designed for PGA equal to 0.25g, the execution of numerical simulations (both PO and IDA) clearly showed the impossibility of activating the collapse criteria related to columns (i.e. instability and strength limit); at the same time, in the case of braces (buckling of members in compression, strength) the equivalent q factors were so high to make the exploration of the corresponding PGA level without technical meaning. Similarly, in the case of building 4, designed low seismicity area, the criteria related to columns and braces were the ones requiring the higher PGA levels to be activated. The values of behaviour factors obtained from simulations are briefly

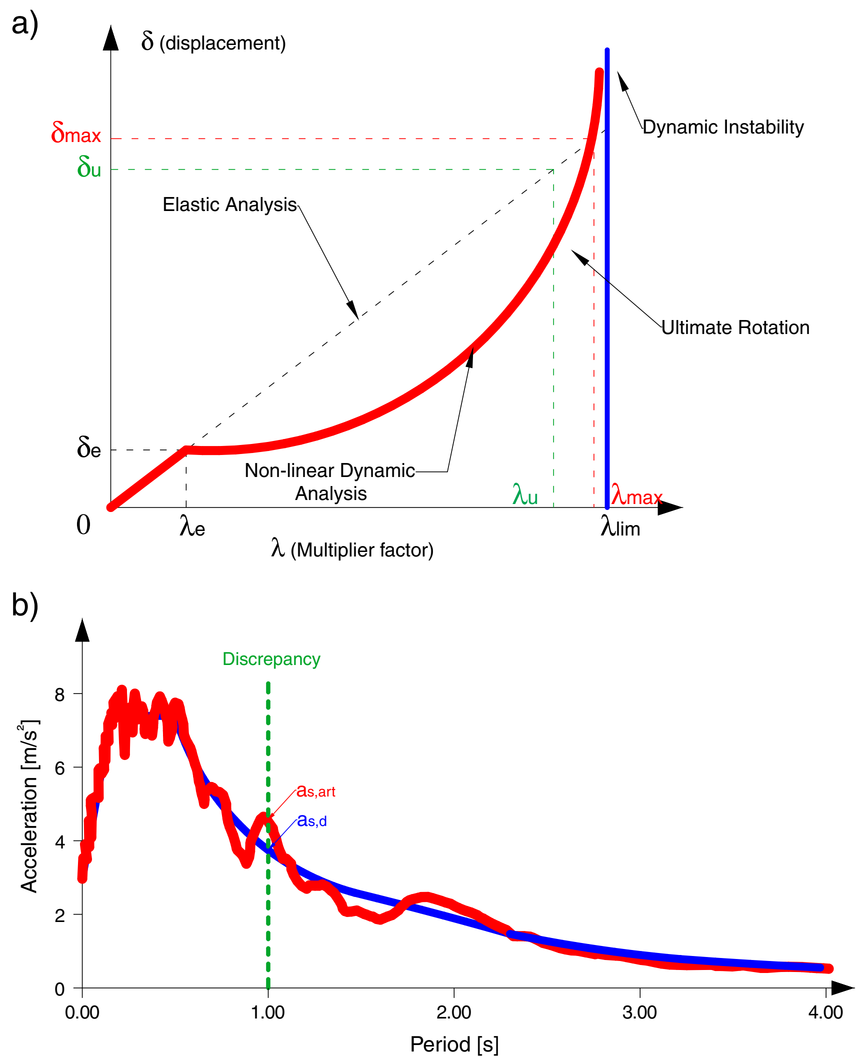

Fig. 16. a) Scheme of the Ballio-Setti procedure, b) discrepancy between target spectrum and real spectrum.

summarized in the Tables 14, 15 and 16 for each of the 7 time histories considered.

It stands to reason that, from Table 15, EBFs in low seismicity area (building 4) were characterized by low values of the effective q-factor, especially when the Modified Ballio – Setti method was used, with values around 2.0-2.5 (Table 17). Taking into account the evaluation of q-factor using PO analysis and the methodology proposed by FEMA356 [23], the difference between the numerical and the design behaviour factor became smaller, leading to values very close to the design one, or in any case varying between 3.2 and 3.9 (q_{design} 4.0). This fact was mainly due to the factor $a_{s,art}/a_{s,d}$: the local difference between the spectrum corresponding to the considered accelerogram and the target adopted for the generation of time histories was higher than 5% (medium value considered by Eurocode for the definition of spectrum compatibility). Table 17 presents the evaluated real behaviour factors coming from PO analysis and from IDAs (mean values of 7 accelerograms, lowest values of the q factors in relation to different collapse criteria).

In the case of building 2, characterized by CBF structure in both the two main directions, the ultimate deformation of braces in compression was the governing failure criterion; considerations similar to the ones made for combined MRF-CBF frames (for Y direction) can be executed, since also in this case investigations were focused on the deformation in tension criterion since buckling of members in compression was directly introduced in the models. In particular, looking at the values obtained for the CBF frame in X-direction were respectively equal to 2.37 and 5.36 respectively in the case of PO and IDA and consequently respectively lower and higher than the value adopted in the design process (3.68).

4.5.3. Composite steel-concrete structures

Buildings 6, 7, 8, 10 and 11 are characterized by a composite steel-concrete structure: in particular, buildings 6 and 8 present composite beams and bare steel column while buildings 7 and 10 have both beams and columns in composite structure.

Buildings 6, 7 and 8 present a MRF structure both in x and y direction, while buildings 10 and 11 presents an EBF structure with short shear link in X-direction and a CBF structure in Y-direction. The PGA adopted in the design was, respectively, 0.10g for buildings 6, 7 and 10 and 0.25g in the case of buildings 8 and 11. The structures were designed with a behaviour factor equal to 4.0 and their seismic assessment was carried out using FineLG software.

In the case of building 6, the most conditioning criteria in the assessment of the building were related to beams' and columns' ductility and to the intensity of applied axial loads.

The evaluated q factor was around 7.0 using the modified Ballio-Setti method and around 8.0 using the classic method, higher than the one adopted in the design. Similar results were obtained for building 7, for which the estimated q factor was around 8.5 adopting the classical

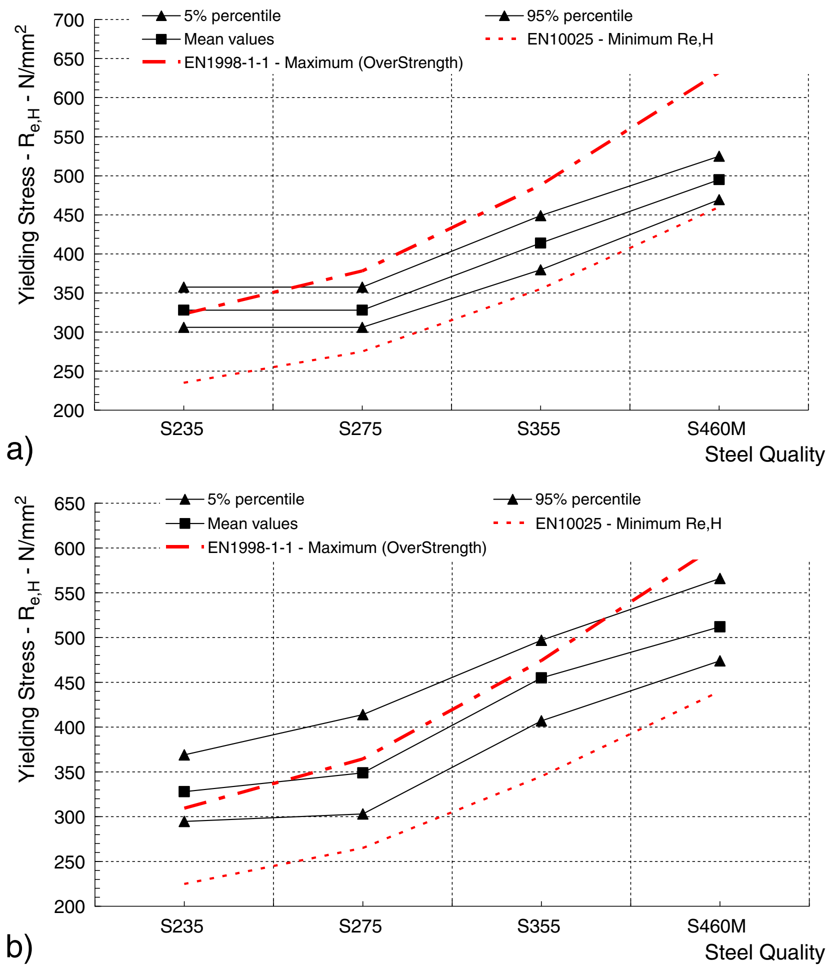

Fig. 17. a) Producer 1 – 7-16 mm thickness – $R_{e,H}$ – comparison between statistical values and EN10025/EN1998-1-1, b) Producer 1 – 16 – 40 mm thickness – $R_{e,H}$ – comparison between statistical values and EN10025/EN1998-1-1.

procedure and around 8.0 using the modified Ballio-Setti method (Table 18). As a consequence of the results herein presented, the adoption of Medium Ductility Class (DCM) behaviour in low seismicity areas generally leaded to not optimized structural solutions, with actual q factors equal to twice the design values (Table 18). Beside these considerations, it's also necessary to underline that the design of these case studies was not guided by seismic action but by wind action: as a consequence, overstrength criteria to ensure the weak beam-strong column condition made the structure overdesigned.

In the case of building 8, designed for PGA equal to 0.25g, the significant collapse criteria were related to the reaching of the ultimate rotation of plastic hinges located at the ends of beams and at the base of columns. The values of ultimate rotations were fixed according to the limit values indicated by FEMA356 [23]. The minimal rotation capacity (i.e. 20 mrad according to EN1998-1:2005) leaded to values of the allowable ductility in terms of the q-factor equal to 2.5 or 2.6 respectively adopting the modified and the classical method, lower than the values adopted in the design process, equal to 4.0.

Table 11
Parameters of target spectra and filter function for low and high seismicity.

Seismicity	PGA	Spectrum	Soil	Total duration	Strong motion duration
Low	0.10 g	Type 2	Type C	15 s	5 s
High	0.25 g	Type 1	Type B	20 s	10 s

Results obtained for behaviour factors of buildings 6, 7 and 8 are presented in Table 18: with high seismic hazard and medium ductility, the q factors obtained from numerical analyses were lower than the ones adopted in the design. On the contrary for the structures designed with low seismic hazard actions, the q factor obtained from numerical simulations was systematically higher (at least 2 times) than the values adopted for the design.

In the case of buildings 10 and 11, with the same scheme in plan and elevation and designed for two different PGA levels, due to the rather stiff behaviour of the braced structure, non linear geometrical effects did not lead to the dynamic instability of the structure and, consequently, the non linear behaviour was essentially governed by material non linearities. This situation was observed for all the bi-dimensional frames studied, although less pronounced for CBF in high seismicity.

Behaviour factors were evaluated considering the reaching of two different limit state, i.e. Life Safety (LS) limit state and Collapse Prevention (CP), and the corresponding levels of multiplier activating the relevant collapse criteria. The values of the q factor so obtained were very high (Table 19), as a consequence of the fact that the assessments were made assuming an infinite deformation capacity of the structure. The behaviour factor was higher in the case of building 10, designed for PGA 0.10g, with values ranging between 5.6 (CBF) and 7.5 (EBF), that evidences the oversizing of the structure.

For EBF/CBF composite structures, the premature failure was triggered by an excessive demand on the ductile zones: i.e. excessive rotation of links for EBFs and excessive axial deformation of braces for CBFs, with limit values assumed in agreement to FEMA 356 [23] prescriptions.

Table 12
Mean real values for mechanical properties adopted in Opus project.

Steel grade	Main Characteristics	0-16	16-40	[mm]	0-16	16-40	[mm]	0-16	16-40	[mm]
		$R_{e,H}$			R_m			A		
S235	Mean [MPa]	310.0	320.0	[MPa]	410.0	420.0	[MPa]	25	28	[%]
	Standard deviation [MPa]	21.0	22.0	[MPa]	15.0	15.0	[MPa]	1.5	1.68	[%]
	Co.V	0.065	0.069	[-]	0.037	0.036	[-]	0.06	0.06	[-]
S275	Mean [MPa]	350.0	350.0	[MPa]	460.0	460.0	[MPa]	23	25	[%]
	Standard deviation [MPa]	31.0	32.0	[MPa]	18.0	21.0	[MPa]	1.38	1.75	[%]
	Co.V	0.069	0.094	[-]	0.039	0.046	[-]	0.06	0.07	[-]
S355	Mean [MPa]	410.0	430.0	[MPa]	565.0	550.0	[MPa]	25	25	[%]
	Standard deviation [MPa]	27.0	27.0	[MPa]	21.0	25.0	[MPa]	1.75	1.75	[%]
	Co.V	0.060	0.063	[-]	0.037	0.045	[-]	0.07	0.07	[-]
S460	Mean [MPa]	500.0	520.0	[MPa]	630.0	630.0	[MPa]	20	19	[%]
	Standard deviation [MPa]	27.0	28.0	[MPa]	25.0	31.0	[MPa]	1.2	1.33	[%]
	Co.V	0.058	0.054	[-]	0.04	0.049	[-]	0.06	0.07	[-]

Similar consideration were also executed for building 11, characterized by the same structural typology of building 10 and designed for PGA equal to 0.25g. Table 20 summarizes the values of real behaviour factors coming from PO and IDA for buildings 10 and 11 (LS condition).

According to the Table 20, EBFs were characterized by values of the allowable q-factor higher than the ones adopted in the design, equal to 4.0. In the particular case of a EBF designed for low seismicity (building 10, Y-direction), the homogeneity rules on the over-strength factor of the joints provided an inherent global over-strength of the building, leading to q-factors higher than 5.6.

Regarding CBFs, results strongly varied whether the limit state was considered as governed by compression or by tension collapse of the diagonal. When the limit state was assumed to be governed by the compression limit, the behaviour factors obtained in average for the seven accelerograms were in the range between 1.7 and 3.3. A critical investigation of results showed that the rather poor ductility of the system obtained under these assumptions was related to a concentration of deformation in correspondence of the top storey of the building, due to the high slenderness of the diagonal. In the design of presented case studies, in fact, the upper limit on the diagonal slenderness was released for the two upper levels due the quasi-impossibility to fulfil simultaneously all the design criteria and arguing that according to Eurocode 8, the limit was not mandatory for 2-storeys building. To confirm what already presented, if the potential collapse of the 5th storey was assumed to be governed by tension only, results became very close to the ones obtained considering tension only at all levels. Under this assumption, allowable q-factors became higher than the ones considered in the design for low-seismicity conditions and slightly lower for high-seismicity.

Table 13
Mean real values for mechanical properties adopted in Opus project.

Building ID/ Structural Typology			PGA design	q_{design}	q_{PO}	q_{IDA}
1	x-dir	MRF - S235	0.10g	2,35	1,96	6,66
14	x-dir	MRF - S355	0.25g	1,55	2,55	3,24
15	x-dir	MRF - S355	0.10g	1,96	2,81	7,04
	y-dir	CBF - S355		2,64	2,07	9,23
5	x-dir	MRF - S355	0.25g	4,00	1.98	2.80
	x-dir	MRF - S460			1.98	2.68
	y-dir	CBF - S355			3.11	6.28
	y-dir	CBF - S460			2.55	6.04
12	x-dir	MRF - S355	0.25g	2,33	1.71	9.97
	x-dir	MRF - S460		1,63	1.63	9.56
	y-dir	CBF - S355		4,00	2.33	2.54
	y-dir	CBF - S460		4,00	1.63	2.31
13	x-dir	MRF - S235	0.25g	4,00	2.02	3.74
	x-dir	MRF - S275			2.13	2.83
	y-dir	CBF - S235			4.05	6.45
	y-dir	CBF - S275			3.87	5.98

Table 14
Estimation of q_{IDA} for Building 3.

Acc n°	Building 3 EBF X-dir				Building 3 EBF Y-dir			
	Link	Column	Brace	Drift	Link	Column	Brace	Drift
1	11,8	-	-	7,9	8,3	-	13,8	9,2
2	9,3	-	-	9,3	8,3	-	11,6	9,1
3	9,8	-	-	11,8	8,5	-	11,0	9,3
4	7,4	-	-	7,4	7,0	-	10,1	7,7
5	10,9	-	-	7,9	6,6	-	9,9	6,6
6	7,0	-	-	7,7	9,5	-	11,2	10,3
7	9,1	-	-	10,9	8,8	-	11,2	8,8

Table 15
Estimation of q_{IDA} for Building 4.

Acc n°	Building 4 EBF X-dir				Building 4 EBF Y-dir			
	Link	Column	Brace	Drift	Link	Column	Brace	Drift
1	2,6	12,9	3,9	5,8	2,6	13,1	13,1	7,8
2	2,3	9,1	3,6	6,4	2,7	10,8	10,8	4,9
3	2,7	10,8	3,3	5,4	2,4	9,7	9,2	4,6
4	2,1	9,5	2,4	4,5	2,7	11,8	9,5	4,2
5	2,7	10,7	3,2	5,9	1,9	10,9	10,9	6,3
6	2,4	10,6	2,7	6,4	2,2	11,1	10,6	4,4
7	2,3	9,4	2,6	5,6	2,9	11,6	11,6	7,0

Table 16
Estimation of q_{IDA} for Building 16.

Acc n°	q- factor for 16 EBF X				q- factor for 16 EBF Y			
	Link	Column	Brace	Drift	Link	Column	Brace	Drift
1	12,0	-	11,0	13,9	15,3	-	18,5	17,4
2	12,0	-	11,0	13,9	12,3	-	15,0	13,2
3	12,7	-	11,6	14,8	11,2	-	16,0	12,8
4	9,9	-	9,9	10,9	9,6	-	13,1	12,3
5	11,8	-	9,9	12,8	10,3	-	12,9	12,9
6	10,4	-	10,4	12,2	9,4	-	12,8	12,8
7	12,7	-	10,9	14,5	12,4	-	17,7	14,2

Table 17
Behaviour factors evaluated for EBF buildings with static and dynamic analyses.

Building ID		q_{PO}	q_{IDA}
3	EBF - X	5,8	9,0 (drift)
	EBF - Y	7,7	8,1 (Link)
4	EBF - X	3,2	2,4 (Link)
	EBF - Y	3,9	2,5 (Link)
16	EBF - X	9,2	10,7 (Brace)
	EBF - Y	11,1	11,5 (Link)

Table 18
q factor of the different buildings calculated using different methods.

Building ID	q_{IDA}		q_{design}
	Modified Ballio-Setti method	Classic method	
6	7.0	8.0	4.0
7	8.0	8.5	4.0
8	3.0	3.75	4.0

Table 19
Behaviour factors. (*) CP level of the criterion is never reach by any of the 7 accelerograms even for a multiplier equal to 15; (**) Only one out of the 7 ground motion time-history is able to trigger the collapse criterion. For all other six, collapse is not reach even for an accelerogram multiplier equal to 15.

Accelerogram	Building 10 (PGA 0.10g)				Building 11 (PGA 0.25g)			
	EBF (X-direction)		CBF (Y-direction)		EBF (X-direction)		CBF (Y-direction)	
	LS	CP	LS	CP	LS	CP	LS	CP
1	7.5	7.5	4.6	--	5.4	6.9	--	--
2	--	--	5.4	--	3.5	3.8	6.2	6.5
3	--	--	6.2	--	6.2	6.9	--	--
4	--	--	5.4	--	4.6	4.6	2	2
5	--	--	6.2	--	6.2	--	--	--
6	--	--	5.4	--	--	8.5	2.75	2.75
7	--	--	6.2	--	5.4	6.9	3	3
Mean Values	7.5 (**)	7.5 (**)	5.6	-- (*)	5.2	6.3	3.5	3.6

Table 20
Real behaviour factors coming from pushover and IDA analyses for buildings 10 and 11.

Building ID		q_{PO}	q_{IDA}	q_{design}
10	EBF - X direction	6.8	5.6	4.0
	CBF - Y direction	4.3	7.5	4.0
11	EBF - X direction	6.6	5.2	4.0
	CBF - Y direction	4.0	3.5	4.0

5. Comparisons and conclusions

Table 21 briefly summarizes the complete set of the q-factor values obtained from PO and IDA analyses carried out using the mean values of the real mechanical properties of materials in contrast to the design procedure, executed considering the nominal ones.

For the majority of the designed buildings for both their main directions, the values of the q_{IDA} were higher, or not much lower, than the ones adopted in the design, pointing out a substantial over-strength of buildings designed to bear high seismicity actions.

Looking at the results presented in Table 21, buildings designed for bearing low seismicity actions (characterized by a design PGA equal to 0.10 g like buildings 1, 2, 6, 7, 10 and 15) were associated to very high values of the q_{IDA} respect to q_{design}, highlighting the oversizing of structural members.

For instance, the design of building 1 (MRF) adopted a q-factor equal to 3.99 whereas the q-factor obtained from the "modified Ballio-Setti" procedure was equal to 6.66. A similar condition was revealed also for building 2 (frame in X direction - CBF) with design value of q-factor equal to 4.00 towards an evaluated one up to 5.37, and in the case of building 7 (MRF, composite structure) with q_{design} 4.00 versus an obtained q_{IDA} equal to 7.00.

On the other hand, considering the case of EBF structure with long bending link (building 4) the behaviour factor adopted for the design according to EN1998-1:2005 [1] seems to be over-estimated, with design values equal to 4.00 for both the two main directions of the building and obtained values from IDAs respectively equal to 2.45 and 2.48 for X and Y directions.

Otherwise, considering the structural behaviour of buildings designed for high seismicity (PGA equal to 0.25g), the values of q_{design} are generally lower (or at least equal) than the ones evaluated using the modified Ballio-Setti method, pointing out the correct approach of Eurocode 8 [1] for the design. For instance, the design of the building 3 (EBF with shear link) was made using q_{design} equal to 6 in both X and Y directions, whereas the q_{IDA} were respectively equal to 8.32 and 8.11. Similar considerations can be made for building 12 (MRF X-direction) with q_{design} equal to 4.0 and q_{IDA} around 9 and for building 10 with higher values of the q_{IDA} respect to q_{design} both in X and Y directions.

Looking at the results summarized in Table 21 it can be observed that results coming from PO analyses are in general lower than the ones coming from IDAs.

Table 21 presents the results obtained from buildings (indicated by the adoption of the asterisk) that were re-designed using lower q-factor values according to a preliminary evaluation using the modified Ballio-Setti procedure: this methodology was adopted mainly for buildings with MRF structure, in which the use of design q-factors according to actual standards' procedure is not generally able to obtain optimized

Table 21
Summarizing table of q_{IDA} and q_{PO} with respect to q_{design}. (*) frames re-designed adopting lower q factors - values under re-evaluation. (**) no material over-strength ($V_{y,real}/V_{y,design}$).

Building ID	X – direction					Y – direction					PGA
	system		q_{static}	q_{IDA}	q_{design}	system		q_{static}	q_{IDA}	q_{design}	[g]
1	MRF		1,96(**)	6,66	2,35	Not des.		-	-	3,99	0,10
2	CBF		2,36(**)	5,37	3,68	CBF		-	-	4,00	0,10
3	EBF shear		5,78	8,32	6,00	EBF shear		7,66	8,11	6,00	0,25
4	EBF bending		3,18	2,45	4,00	EBF bending		3,90	2,48	4,00	0,15
5	MRF	S355	1,98	2,80	4,00	CBF	S355	3,11	6,28	4,00	0,25
		S460	1,98	2,68			S460	2,55	6,04		
6	MRF		2,65(**)	7,00	4,00	Not des.		-	-	4,00	0,10
7	MRF		2,60(**)	8,00	4,00	Not des.		-	-	4,00	0,10
8	MRF		1,80(**)	3,00	4,00	Not des.		-	-	4,00	0,25
9	MRF		1,75(**)	-	4,00	Not des.		-	-	4,00	0,25
10	EBF vertical shear		6,78	7,50(*)	4,00	CBF		4,30	5,60	4,00	0,10
11	EBF vertical shear		6,62	5,20	4,00	CBF		4,00	3,50	4,00	0,25
12	MRF	S355	1,71	9,97	4,00	CBF	S355	2,33	2,54	4,00	0,25
		S460	1,63	9,56			S460	1,63	2,31		
13	MRF	S235	2,02	3,74	4,00	CBF	S235	4,05	6,45	4,00	0,25
		S275	2,13	2,83			S275	3,87	5,98		
14	MRF truss girder		2,55	3,24	1,55	CBF		-	-	3,80	0,25
15	MRF		11,5	12,32	1,96	CBF		4,66	7,47	2,64	0,10
16	EBF shear		2,81	7,04	6,00	EBF (shear)		2,07	9,23	6,00	0,25

solutions, that were consequently re-designed with more efficient values of behaviour factors. In this sense, the above mentioned procedure can be used in order to optimize the design of structures.

According to what herein presented, it's possible to state that the behaviour factors suggested by EN 1998-1:2005 [1] for steel and steel-concrete structures can be considered conservative for buildings designed with high level of PGA (i.e. 0.25g), with real values coming from IDAs higher than the ones suggested in Eurocode 8. Buildings designed for PGA 0.10g (low seismicity) generally evidenced an oversizing of structural elements, with following behaviour factors higher than the ones adopted in the design, being Eurocode 8 conservative respect to the effective condition, even if some exceptions can be observed.

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A. Braconi[a], S. Caprili[b,*], H. Degee[c], M. Guendel[h], M. Hjiaj[e], B. Hoffmeister[d], S.A. Karamanos[f], V. Rinaldi[g], W. Salvatore[b], H. Somja[e]

[a] Thy Marcinelle Rivagroup, Rue de l'Acier, BP 1002 Charleroi, Belgium
[b] Department of Civil and Industrial Engineering University of Pisa, Italy
[c] Faculty of Engineering Technology, University of Hasselt, Belgium
[d] RWTH Aachen University, Institut für Stahlbau, Aachen, Germany
[e] Institut National des Sciences Appliquées de Rennes, France
[f] University of Thessaly, Volos, Greece
[g] Arcelor Mittal S.A., Luxembourg
[h] Wölfel Beratende Ingenieure GmbH + Co. KG, Max-Planck-Str. 15, 97204 Höchberg, Germany

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We regret that an error appeared in the name of the author M. Hjaij and to add Dr. Hugues Somja as one of the co-author for his contribution towards the research.

Whereas it should be:

- A. Braconi[a],
- S. Caprili[b,*],
- H. Degee[c],
- M. Guendel[h],
- M. Hjiaj[e],
- B. Hoffmeister[d],
- S.A. Karamanos[f],
- V. Rinaldi[g],
- W. Salvatore[b]
- H. Somja

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* Corresponding author at: University of Pisa, Dept. of Civil and Industrial Engineering, Largo L. Lazzarino 1, Pisa 56122, Italy.
E-mail addresses: silvia.caprili@ing.unipi.it, silvia.caprili@gmail.com (S. Caprili).

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