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INFLUENCE OF STEEL MECHANICAL PROPERTIES ON EBF SEISMIC BEHAVIOUR

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Abstract*:* *Among the resisting steel types suitable for the design of high ductility structures, Eurocode 8 proposes MRFs and EBFs. Also if the formers are generally considered a more efficient structural solution for high-ductility design, they suffers a strong weakness in the lateral stiffness creating, during the design process, cumbersome procedures to avoid excessive lateral displacements maintaining quite high ductile behaviour under design seismic actions. In many cases, the design process produces not optimized structural members, oversized respect to the minimum seismic requirements due to lateral deformation limitations. On the contrary, EBF combines high lateral stiffness furnished by bracing elements and high dissipative capacities furnished by plastic hinges developed in links. Eurocode 8 proposes a design procedure for realizing high ductility EBF in which iterative checks are required to properly design the links assigning to every link a defined level resistance dependant on all other links resistance. The present paper investigates the seismic behaviour of EBFs using the Incremental Dynamic Analysis technique in order to explore their mechanical response under increasing seismic action levels. A set of steel structures is designed according to Eurocode 8. The numerical simulations are executed considering the variability of both steel mechanical properties and seismic input, aiming to a complete probabilistic characterization of mechanical response of the system and deeply analyzing the effective level of structural safety and the ability to internally redistribute plasticizations during the earthquake. Structural safety conditions will be defined according to a multi-level performance approach. The paper presents also some final suggestions for possible improvements/simplifications in EBF design.*

1 INTRODUCTION

In the last seventies the use of eccentrically braced frames (EBFs) as earthquake resistant structures in medium and high seismicity regions greatly increased; this was mainly due to the fact that, respect to other traditional structural typologies, EBFs are able to join good dissipative performances to high elastic stiffness [1], combining the plastic dissipative behaviour of moment resisting frames (MRFs) with the high lateral stiffness of concentrically braced frames (CBFs) [2].

The dissipative behaviour of EBFs is related to particular beam elements called “links”: during an earthquake the link, designed to plasticize while all the other elements remain in the elastic range, develops high plastic deformations and consequently dissipates seismic energy.

The behaviour of link elements, and afterwards the way they dissipate energy, is related to their length (e): short links (i.e. characterized by a ratio between the plastic shear and the plastic moment smaller than 1.6 times the link length) generally develop high shear deformations, while long links (i.e. characterized by a ratio between the plastic shear and the plastic moment higher than 2.5 times the link length) mainly dissipate energy trough the formation of flexural deformations.

The ability of EBFs in dissipating energy strictly depends on the criteria adopted in the design: the plastic deformations are essentially located on link elements, dimensioned for yielding before beams, braces and columns that, otherwise, are proportioned using the forces generated by the yielded and hardened links [3] in order to remain in the elastic field, according with the principles of capacity design. The overstrength of non dissipative elements is consequently related to the mechanical and geometrical characteristics of the links; the overstrength behaviour of elastic elements is expressed trough the factor Ω, defined as the ratio between the plastic design resistance and the effective action on the dissipative elements (shear for short links and bending for long links). According to actual standards, such as Eurocode 8, the distribution of the overstrength factor Ω should be quite uniform, not varying more than 25% respect to its minimum value: this is necessary for guaranteeing a uniform distribution of link plasticization on all the floors and the global dissipation of seismic energy.

Otherwise, recent studies [4] evidenced that frequently, especially in presence of a high number of storeys, EBFs underwent undesired collapse mechanisms, despite the presence of small plasticization of the links of some floor and the respect of the design criteria. This fact can be partially caused by the difference between the nominal design value and the real effective value of the mechanical properties of steel [1, 5]: this difference, generally taken into account trough the introduction of the material overstrength coefficient γ_{ov}, can lead to the alteration of the failure modes supposed in the design, causing premature local collapse phenomena of the structure and consequently avoiding the global dissipation of seismic energy. The present work aims at the investigation of the influence of variability material properties on the dissipative behaviour of EBFs and was developed in the framework of a European Research Project funded by the Research Found for Coal and Steel (OPUS – Optimizing the seismic Performance of steel and steel-composite concrete strUctures by Standardizing material quality control). To this purpose, different steel buildings were firstly designed according to actual European standards and then analyzed trough the execution of non linear incremental dynamic analyses (IDA), considering the variability of both mechanical properties of materials and seismic input, in order to achieve a complete probabilistic characterization of the mechanical response of the system. The results obtained using the nominal and the real values of mechanical material properties, provided by the European steel producers partners of the project, were compared in terms of activation of collapse criteria, analyzing the effective level of structural safety and the ability of the structure to redistribute the plastic demand imposed by the earthquake.

2 PROPOSED METHODOLOGY

Probabilistic techniques were adopted in order to assess structural response of case studies as function of seismic input and mechanical properties variability.

Indeed, even if in general seismic actions are time-variant variables (processes) and therefore reliability problems in earthquake engineering are time-variant reliability problems, for OPUS purposes, the problem was transformed in a time-invariant problem (i.e. looking only at extreme values) and a Monte Carlo simulation technique was applied in an efficient way; moreover, a probabilistic procedure able to furnish a good estimation of failure probability for all identified design points was also defined.

According to these final considerations, the research project adopted the following general approach devoted to the effective evaluation of seismic reliability for all structural case studies designed during the research:

- **step 1**. Deep knowledge of structural systems. The knowledge about the structural behaviour of the case studies was completed and determined thorough several numerical simulations, adopting non-linear static and dynamic analyses.
- **step 2**. Nonlinear modelling and collapse modalities assessment. Each structural system was described by accurate nonlinear models individuating the relevant collapse criteria.
- **step 3**. Characterization of seismic hazard. Seismic actions were modelled adopting parameters and hazard proposed by EN1998-1-1; in particular, hazard function (i.e. annual exceedance probability) for European seismicity is taken from EN1998-1-1 and calibrated according to design parameters associated to ultimate limit state verification. Seven seismic inputs to be adopted in the numerical simulations were artificially generated from response spectra adopted in the design.
- **step 4**. Probabilistic model of mechanical variables. Scattering of steel products was represented by a multi-variable model where yielding stress – $R_{e,H}$ (f_y) –, tensile strength – R_m (f_t) – and elongation at fracture – A (ε_u) were considered with their probabilistic interdependencies.
- **step 5**. Execution of nonlinear analyses and optimal planning of numerical simulations. The correlation between the seismic demand and the structural response of case studies was defined employing non-linear dynamic analyses; peak ground acceleration (PGA) of selected seismic inputs was varied according to appropriate levels chosen in order to activate collapse modes. In such a way, the number of simulations characterized by failures according to different modes was increased.
- **step 6**. Probabilistic procedure for P_{fail} estimation. Numerical results coming from dynamic analyses were analyzed employing a statistical procedure that furnishes fragility curves and yearly threshold exceedance probability of the relevant collapse modes for each case study.

The numerical simulations were executed using Incremental Dynamic Analysis techniques, suitable for the analysis of structural response at different PGA levels.

3 DESCRIPTION OF CASE STUDY

In the widest framework of OPUS project, fifteen different buildings in steel and steel – composite concrete structure were designed, in order to cover the most common geometrical and functional structural typologies in Europe: MRFs, CBFs and EBFs for offices, industrial buildings and car parks were analyzed.

The present paper deals with the seismic behaviour of EBF steel structures; three different buildings, so on called building 3, 4 and 16, were designed according to the criteria imposed by Eurocode 8. External EBFs were designed to resist the total seismic horizontal forces: stating the symmetry of geometrical properties and mass distribution, the design of the buildings was calibrated on single eccentrically braced frames referring to the two main directions of the structure. The consistency of the design was verified comparing the results so obtained with the ones coming from dynamic modal analyses of 3D global models of the buildings.

Building 3 and 4 are office buildings, while building 16 is a car park; buildings 3 and 16 are located in high seismicity region and present short shear links, while building 4, located in medium-low seismicity area, presents long bending links. Buildings 3 and 4 have a similar geometry, characterized by 5 storeys with an interstorey height of 3.50 m and a span length varying between 6 and 7 m; building 16 is characterized by a span length between 8 and 10.5 m and presents only two storeys with interstorey height equal to 4.0 m. The link disposition and length vary in relation to the typology (shear or bending) and to floor position.

A duplication of secondary beams was applied in buildings 3 and 4 in order to avoid the amount of vertical loads and connection of elements in correspondence of the dissipative zone of links (see Figure 1c). Pinned connections were used at the ends of non dissipative elements, such as braces and columns, and between beams and columns for K-brace frames (frame 3xz, 16xz and 16yz); welded connection were adopted for the beam to column joints in D-brace frames. The general geometrical properties of EBF buildings are presented in Table 1.

For all described buildings a floor type characterized by a concrete slab on prefabricated trussed slab for a global thickness of 23 cm was used; in the design of buildings 3 and 4 steel grade S355 (nominal yielding strength equal to 355 MPa) was used; building 16, on the other hand, was designed considering steel grade S275 (nominal yielding strength equal to 275 MPa). As regards seismic action, in buildings 3 and 16 a PGA equal to 0.25 g and a soil of category B were considered, while building 4 was dimensioned for a PGA equal to 0.10 g and a soft soil of type C; the response spectra adopted in the design are compatible with both Eurocode 8 [6] and actual Italian Standards for constructions NTC2008 [7]. Table 2 summarizes vertical and horizontal loads adopted in the design.

The general geometry of buildings is represented in Figures 1-4.

Building number	Height	Steel Quality	X direction		Y direction	
			Resisting system	Span [n° x L]	Resisting system	Span [n° x L]
3	5x3.5 m	S355	EBF shear	3x7m	EBF shear	4x6m
4	5x3.5 m	S355	EBF bending	3x7m	EBF bending	4x6m
16	2x4.0 m	S275	EBF shear	5x8m + 2x10m	EBF shear	6x10.5m

Table 1: Summary of geometric properties of EBF buildings.

According to Eurocode 8 and in relation to the location of the buildings, design factors respectively equal to 6 and 4 were adopted for high ductility class (HDC) buildings (3 and 16) and for low ductility class (LDC, building 4). All the EBF buildings so far described were designed to resist to vertical and horizontal forces provided by actual standards, both for seismic and static combination, without encountering global or local collapses. The design was optimized in order to have a uniform plasticization of links in all the floors: an accurate distribution of the overstrength factors Ω_i was pursued, obtaining variation smaller than 25% among the floors. The obtained values for overstrength factors are summarized in Table 3.

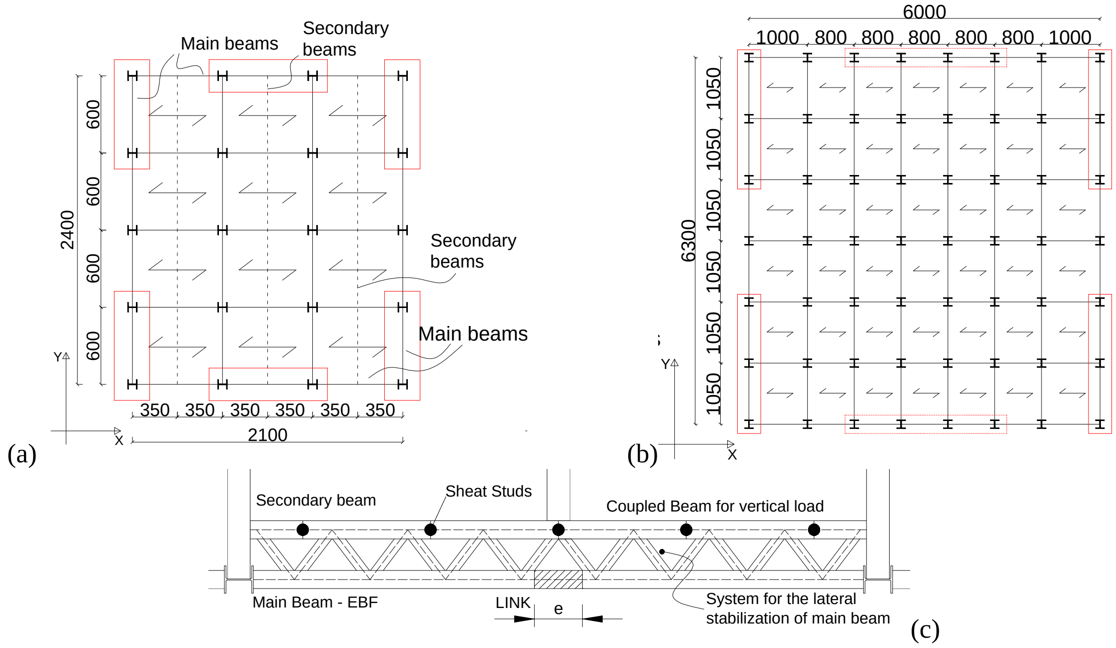


Figure 1: General plan of buildings a) office buildings 3 -4, b) car park 16 and c) beam duplication for decoupling vertical and seismic loads.

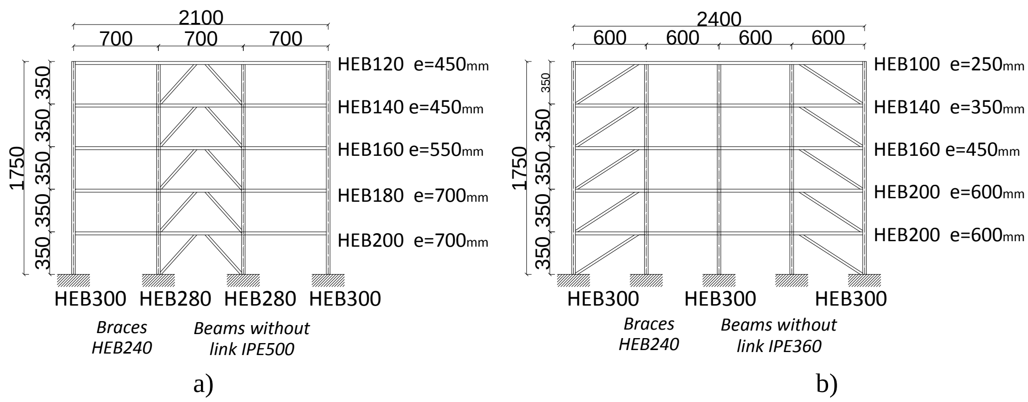


Figure 2: Building 3 (short links), geometry and elements: a) xz frame, b) yz frame.

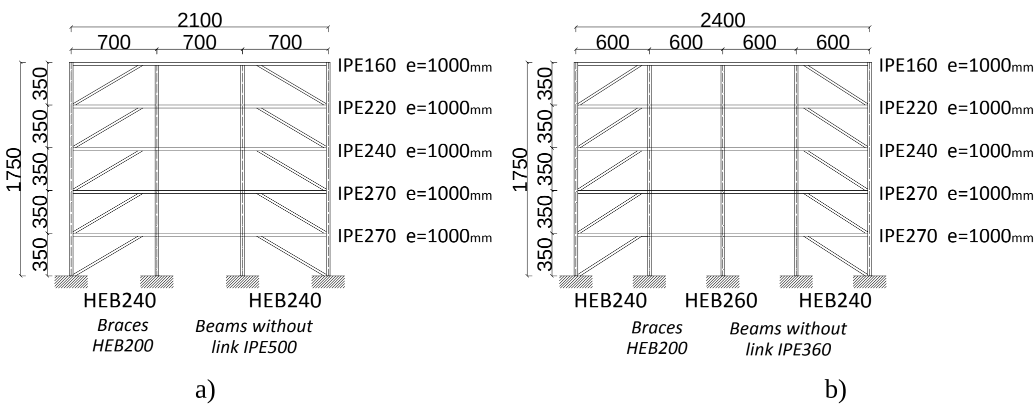


Figure 3: Building 4 (long links), geometry and elements: a) xz frame, b) yz frame.

a)

b)

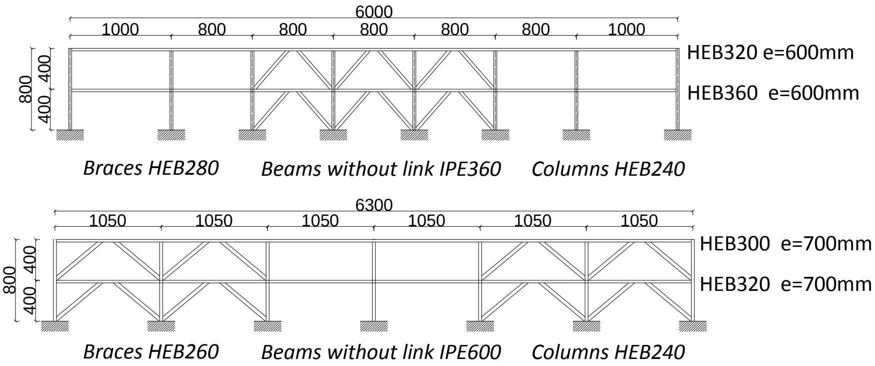


Figure 4: Building 16 (short links), geometry and elements: a) xz frame and b) yz frame.

Building	Type	Typology	Live Load	Snow Load	Wind Load	Soil type	Seismic Action	Seismic mass Floor	Seismic mass roof
-	-	-	kN/m^2	kN/m^2	kN/m^2	-	-	kN	kN
3	Office	EBF	3.00	1.00	1.10	B	0.25 g	3480	3220
4	Office	EBF	3.00	1.00	1.10	C	0.10 g	3480	3220
16	Car Park	EBF	2.50	1.00	1.10	B	0.25 g	27700	28820

Table 2: Summary of vertical and horizontal loads acting on buildings.

Building	Storey	X direction	Y direction
		Ωi	Ωi
3	Storey 1	1.66	2.12
	Storey 2	1.54	2.47
	Storey 3	1.53	2.00
	Storey 4	1.62	2.03
	Roof 5	1.86	2.24
4	Storey 1	1.68	1.99
	Storey 2	1.87	1.74
	Storey 3	1.63	1.78
	Storey 4	1.66	1.76
	Roof 5	1.51	1.61
16	Storey 1	1.53	1.57
	Roof 2	1.88	1.91

Table 3: Overstrength factors for each building.

The sizing of the links with actions coming from the linear analysis was the base for the proportioning of the other overstrengthening elements such as beams, braces and columns, according to the principles of capacity design; buckling phenomena of elements in compression and interstorey drift limits, i.e. lateral stiffness requirements, were also conditioning for the definition of brace and columns profiles.

Typically, HEB sections were used for columns and braces in all the buildings; otherwise, HEB or IPE sections were adopted for links: HDC buildings with short shear links present HEB section for dissipative elements (e varies between 250 and 700 mm), while LDC building employs IPE section for long bending links (e equal to 1000 mm), as presented in Table 4.

Building	Storey	X direction		Y direction	
		Link profile	Link length (mm)	Link profile	Link length (mm)
3	Storey 1	HEB200	700	HEB200	600
	Storey 2	HEB180	700	HEB200	600
	Storey 3	HEB160	550	HEB160	450
	Storey 4	HEB140	450	HEB140	350
	Roof 5	HEB120	450	HEB100	250
4	Storey 1	IPE270	1000	IPE270	1000
	Storey 2	IPE270	1000	IPE270	1000
	Storey 3	IPE240	1000	IPE240	1000
	Storey 4	IPE220	1000	IPE220	1000
	Roof 5	IPE160	1000	IPE160	1000
16	Storey 1	HEB320	600	HEB300	700
	Roof 2	HEB360	600	HEB280	700

Table 4: Link profile and length for each building.

4 DESCRIPTION OF NON LINEAR MODELS

4.1 Numerical Non linear models

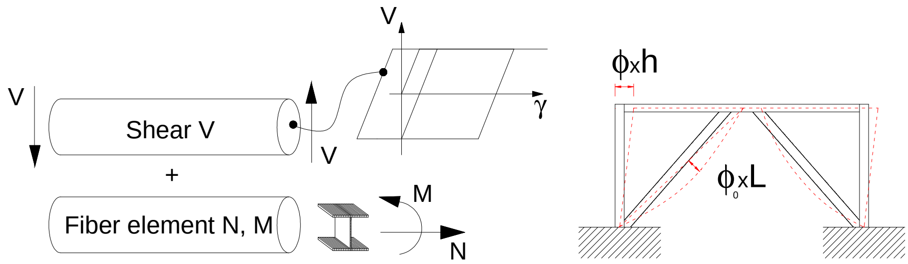
In order to evaluate the influence of the variability of material properties on the effective seismic behaviour of EBF structures, non linear Incremental Dynamic Analyses (IDA) were executed on plane frame models of the buildings previously described.

As many past works evidenced [8, 9] the modelling of link elements should be very accurate for obtaining numerical outcomes consistent with the EBF response prediction; both short and long links, despite the different mechanism they use for dissipating seismic energy, develop flexural forces combined with shear ones: the model of link should consequently be able to reproduce both the two effects.

Many numerical models were proposed in literature to represent the behaviour of link elements, for example one component models with concentrated plastic hinges at the ends of the element [10] or two component models constituted by beams working in parallel [8]; nevertheless, only more recent models are able to encounter the shear behaviour of the dissipative link element [8].

In the present work, bi-dimensional models of the main frames of the buildings were realized using the numerical software OpenSees [11]. The dissipative behaviour of link elements and the combined effect of shear forces and bending moments were directly taken into account modelling all the elements as “fiber section elements” (Figure 5a). The calibration of the software was executed comparing the outcomes from cyclic loading histories on single components (for example the braces) with literature results [12].

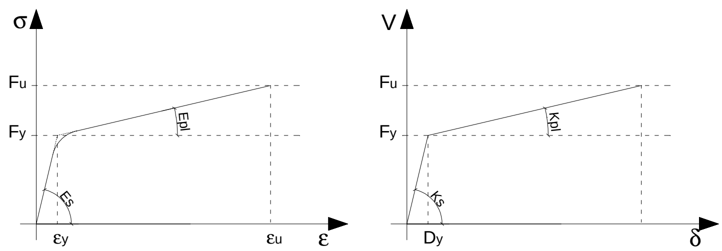
Inelastic fiber elements were used for representing columns, beams without links and short shear links; on the other hand, two elements were used for modelling each long bending link and four elements were employed for each brace. Buckling phenomena of braces were directly taken into account giving an initial imperfection equal to 1/500 of the brace length to the middle point of the brace, as represented in Figure 5b; a similar imperfection was also assigned to the top of columns in order to include in the analysis P-Δ effect (Figure 5b). The value adopted for the imperfection was evaluated from the calibration with literature results.



a) b)

Figure 5: a) General scheme of fiber elements and b) model of imperfections of braces and columns.

For modelling the flexural behaviour of steel members – beams, braces and columns – , the Menegotto-Pinto law [13], characterized by bilinear elastic-plastic stress-strain curve with kinematic hardening, accurately calibrated in order to agree with literature results, was used (Figure 6a); moreover, as regards the force-distortion law used for representing the shear behaviour of elements, a bilinear elastic-plastic law with kinematic hardening was used for links (Figure 6b).



a) b)

Figure 6: Constitutive law adopted for a) flexural behaviour, b) shear behaviour of dissipative elements.

4.2 Definition of collapse criteria for EBFs

For evaluating the global ductile behaviour of the structures under seismic action all the possible collapse mechanism for EBFs were analyzed, referring to what prescribed by actual standards, such as EN 1993-1 [15] and FEMA [16]; the collapse limits individuate the point at which IDA simulation should be stopped, since for higher levels of PGA it's not important to consider the behaviour of the structure. The collapse limitations adopted are strictly related to different limit states, assessing the structural performances both at ultimate and serviceability limit state. One of the most conditioning collapse criteria for eccentrically braced frames is obviously the failure of link elements, in which plastic deformation are concentrated according to the design principles. The plastic rotation is calculated as the ratio between the relative vertical displacement (δ) and the link length (e), see expression (1): for shear short link δ is evaluated as the relative vertical displacement between the two ends of the link (Figure 7a), for long bending links δ refers to the mid length of the element (Figure 7b). The limits assumed according to the standards are presented in Table 5.

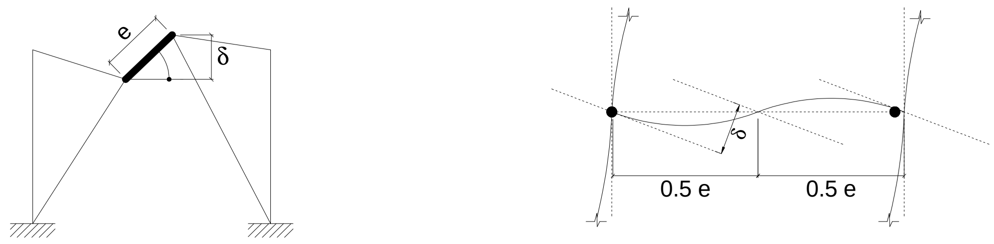


Figure 7: Evaluation of link plastic rotation a) for short shear links, b) for long bending links.

$$\frac{v_1 - v_2}{e} = \frac{\delta}{e} = \gamma_{LINK} \tag{1}$$

The limit axial load for the buckling of steel members in compression (columns and braces) is evaluated according to Eurocode 3 [15] with expression (2), and is consequently strongly influenced by the mechanical properties of materials (yielding strength f_y):

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \tag{2}$$

The limitation imposed to interstorey drift should be assessed; the respect of the lateral stiffness requirements strongly influenced the sizing of steel members such as braces, especially in building 4, designed for low-ductility class. For EBFs, the collapse criteria to be taken into account are summarized in Table 5.

Collapse Criteria	Reference code	Limit value
Ultimate plastic rotation	EC8, FEMA 356	110 mrad (shear), 20 mrad (bending)
Global buckling	EN 1993-1	-
Interstorey drift ratio	EN 1998-1	1.5% x Interstorey height

Table 5: Summarizing table of collapse criteria for EBFs.

5 SEISMIC HAZARD vs. SEISMIC INPUT

5.1 Seismic hazard

Seismic hazard of a particular site expresses its natural exposure to severity of possible earthquake. Seismic hazard analysis characterize the maximal amplitude of ground shaking during the earthquake by chosen design ground motion parameter in the specified level of probability and time of occurrence of the event.

According to EN1998-1 guidelines, it is possible to assume that the annual rate of exceedance of the reference peak ground acceleration a_{gR} may be taken to vary with a_{gR} as:

$$H(a_{gR}) = k_0 \cdot a_{gR}^{-k} \tag{3}$$

Moreover, EN1998-1 suggests that exponent k, depending on seismicity, can be generally taken equal to 3. The value of k_0 was fixed according to basic performance requirements imposed by EN1998-1: the design seismic action should have a exceeding probability of 10% (P_{NCR}, probability of non collapse requirement) in 50 years (T_L, exposition period of the structure) for the non-collapse requirements. The return period of seismic action, T_R, is correlated with P_{NCR} and T_L by the following formula

$$T_R = \frac{-T_L}{\ln(1 - P_{NCR})} \tag{4}$$

that gives a return period of 475 years for the design PGA. According to PGA levels assumed during seismic design, 475 years of return period corresponded, respectively, to 0.25g in high seismicity areas and 0.10g in low seismicity areas and fixed k_0 parameter in eqn. (3) equal to 3.29×10^{-5} for high seismicity areas and 2.10×10^{-6}. Resulting hazard functions for high and low seismicity are presented in Figure 8.

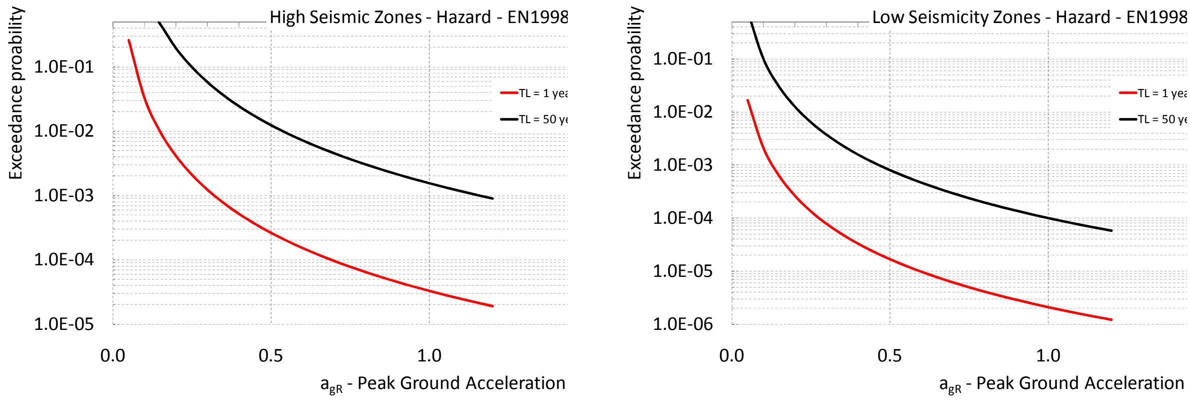


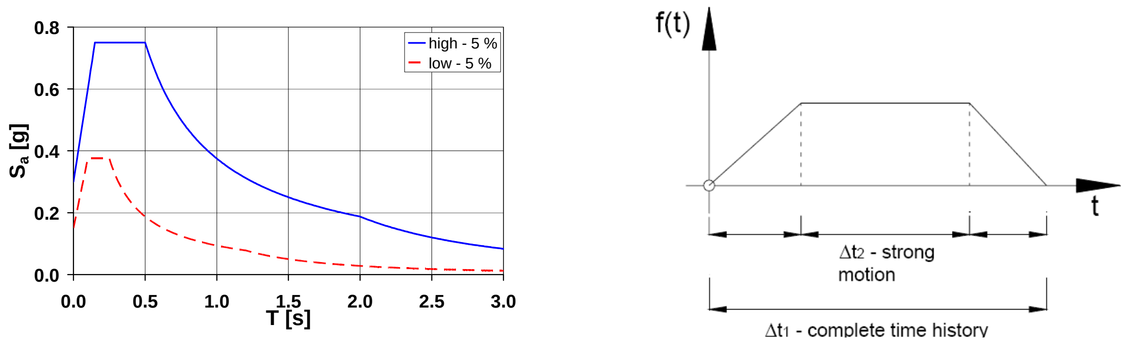
Figure 8: Hazard function according to EN1998-1 prescriptions: (a) high seismic hazard; (b) low seismic hazard.

5.2 Definition of accelerograms

According to EN1998-1-1 prescription, seven earthquake time-histories of natural earthquakes or artificially generated time histories can be used. To obtain results representative for any seismic area in Europe, it reasonable to use artificial accelerograms, which meet the elastic response spectra in EN 1998-1 and are so consistent with chosen hazard model. In order to generate artificial earthquake time histories, the program SIMQKE [14], developed by Gasparini and Vanmarcke (1976), was used.

Two types of seismic intensities were considered: for high seismicity the PGA level was 0.25 g, while the type 1 spectrum for soil type B was used; for low seismicity the PGA was fixed 0.10 g, while the type 2 spectrum for soil type C was applied, Figure 9a. The filter function was defined by a trapezoidal shape, where the time intervals for the initial and ending ramp were 5 s and the strong motion duration was 10 s for high and 5 s for low seismicity, Figure 9b. The relevant Eigen-periods were assumed to be in a range between 0.1 s and 3.0 s. The chosen sampling interval of $\Delta t = 0.01$ s allowed a sufficient accurate calculation for Eigen-frequencies up to 20Hz (5 points for each period).

The verification of the accelerograms by determining the velocity and displacement time histories showed that the displacements were running out (Figure 10). Hence, a baseline correction was applied 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, Figure 11. For periods lower than T_B the spectrum value S_a was slightly too high, Figure 12; however, the target spectrum is sufficiently met and the requirements defined in EN1998-1 were met.



a) b)

Figure 9: Target spectra (a) and filter function (b) for the generation of artificial time histories

Seismicity	p.g.a.	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

Table 6: Parameters of target spectra and filter function for low and high seismicity

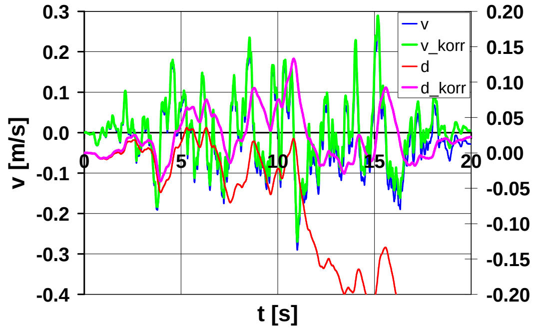
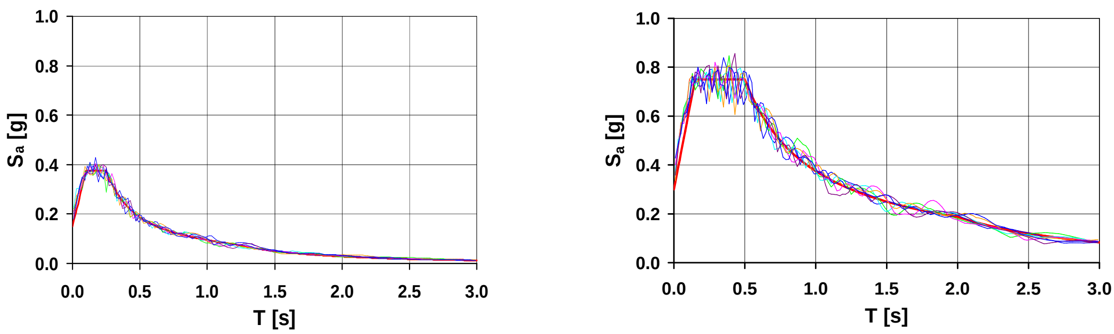


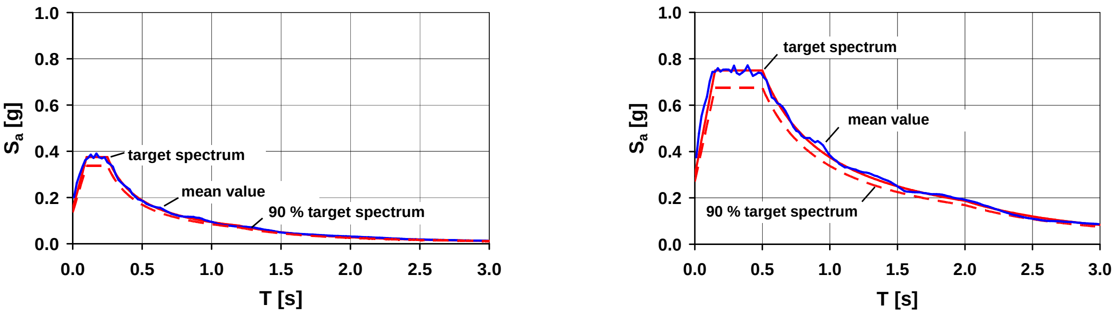
Figure 10: Baseline correction for an artificial accelerogram (high seismicity)

The COV of the spectral values for the 7 accelerograms is between 0.04 and 0.12, Figure 13. It should be noted, that the energy density of artificial accelerograms is much higher than of natural accelerograms, as all frequencies of interest are included.



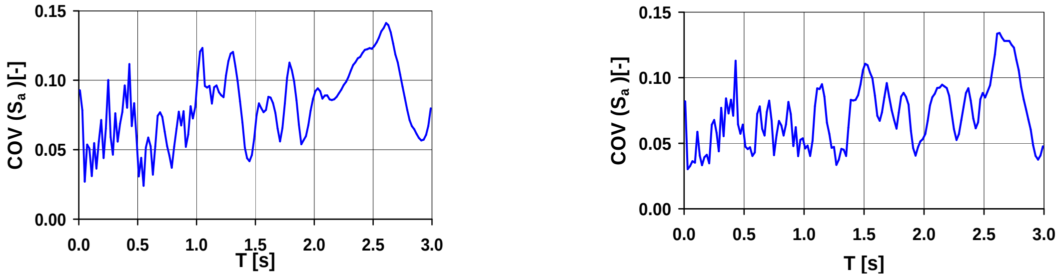
a) b)

Figure 11: Target spectrum and elastic response spectra of 7 artificial accelerograms: low seismicity (a) and high seismicity (b).



a) b)

Figure 12: Target spectrum and mean value of the elastic response spectra of 7 artificial accelerograms: low seismicity (a) and high seismicity (b).



a) b)

Figure 13: COV of the elastic response spectra of 7 artificial accelerograms: low seismicity (a) and high seismicity (b).

5.3 Seismic input for nonlinear analyses

IDA simulations were executed scaling generated seismic inputs at different PGA levels; PGA levels, considered as significant for the probabilistic assessment of seismic response of designed structure, were previously determined studying structural response of case studies. In particular, for each plain structure (see Figures 2, 3 and 4), different excitation levels were individuated according to the collapse modes that can be activated increasing PGA, see table 7. It is worth noting that collapse of columns was a mode that cannot be activated for PGA levels lower than 1.50g: hence, PGA levels corresponding to column collapse were not considered; concerning 3EBFX also collapse mode associated to braces was not activated. This high level of PGA for these two collapse modes was correlated to the frame design: columns are sized through column verification under full vertical static actions at Ultimate Limit State; braces for 3EBFX were over-sized due to the extreme sensitivity of structural configuration to second order effects.

a)

	Frame 3X				Frame 3Y			
Acc	Link	Col.	Brace	Drift	Link	Col.	Brace	Drift
-	[g]	[g]	[g]	[g]	[g]	[g]	[g]	[g]
1	0.60	2.00	2.00	0.40	0.45	2.00	0.75	0.50
2	0.50	2.00	2.00	0.55	0.50	2.00	0.70	0.55
3	0.50	2.00	2.00	0.60	0.50	2.00	0.65	0.55
4	0.45	2.00	2.00	0.45	0.45	2.00	0.65	0.50
5	0.55	2.00	2.00	0.40	0.40	2.00	0.65	0.40
6	0.45	2.00	2.00	0.50	0.55	2.00	0.70	0.60
7	0.50	2.00	2.00	0.60	0.55	2.00	0.70	0.55

b)

	Frame 4X				Frame 4Y			
Acc.	Link	Col.	Brace	Drift	Link	Col.	Brace	Drift
-	[g]	[g]	[g]	[g]	[g]	[g]	[g]	[g]
1	0.40	2.00	0.60	0.90	0.40	2.00	2.00	1.20
2	0.50	2.00	0.80	1.40	0.50	2.00	2.00	0.90
3	0.50	2.00	0.60	1.00	0.50	2.00	1.90	0.95
4	0.45	2.00	0.50	0.95	0.40	2.00	1.60	0.70
5	0.50	2.00	0.50	1.10	0.50	2.00	2.00	1.15
6	0.45	2.00	0.50	1.20	0.40	2.00	1.90	0.80
7	0.50	2.00	0.55	1.20	0.50	2.00	2.00	1.20

c)

212	Frame 16X				Frame 16Y			
Acc.	Link	Col.	Brace	Drift	Link	Col.	Brace	Drift
-	[g]	[g]	[g]	[g]	[g]	[g]	[g]	[g]
1	0.60	1.70	0.55	0.70	0.70	2.00	0.50	0.65
2	0.60	1.70	0.55	0.70	0.70	2.00	0.40	0.80
3	0.60	2.00	0.55	0.70	0.75	2.00	0.35	0.80
4	0.50	1.50	0.50	0.55	0.60	2.00	0.35	0.70
5	0.60	1.60	0.50	0.65	0.65	2.00	0.35	0.75
6	0.55	1.70	0.55	0.65	0.60	2.00	0.35	0.75
7	0.70	2.00	0.60	0.80	0.70	2.00	0.35	0.80

Table 7: PGA levels determined according to relevant collapse modes, for a) building 3, b) building 4 and c) building 16.

6 EXECUTION OF NONLINEAR ANALYSES

For each case study, collapse criteria were analysed for each considered PGA level, executing incremental dynamic analyses adopting alternatively the 7 artificially generated accelerograms.

Monte Carlo Method was applied to each analysis generating 500 samples of mechanical variables and running IDA for each of them. In particular, to be adherent to the real assembling of steel structures, all beams and braces members were considered as probabilistically not dependant (generating independent sets of mechanical variables) while columns of two subsequent floors were considered as characterized by the same probabilistic variables, see generation scheme in Figure 14c.

In order to generate samples of mechanical properties, a log-normal model was assumed for each of them – yield strength $R_{e,H}$, ultimate strength R_m and elongation A – so that their distribution resulted multivariate in which the three variable were inter-correlated.

The correlation matrix of the adopted model was determined from statistical parameters derived from industrial steel production [15], summarized in Table 8.

The generation procedure was based on the adoption of an equivalent multi-normal probabilistic distribution [17] obtained from the original multivariate log-normal model.

In such a way, for each case study 3500 numerical simulations were carried out (i.e. 7 quakes × 500 material samples) for each considered PGA level and each considered collapse criterion.

Defining, for each collapse criterion, the damage measure (DM) for the relevant engineering demand parameter (EDP) stated in the Table 5, nonlinear analyses explored structural responses using a strip method as depicted in Figures 14a and 14b (Figure 14a includes seismic input and material variability, 3500 results for each PGA level; Figure 14b only shows material variability, 500 results for each PGA level).

Moreover each set of 500 nonlinear analyses, results (related to a single collapse criterion, a PGA level and accelerogram) were suitably standardised referring to the values $Y_i = 100 \cdot DM_i / DM_u$ being, for the specified collapse criterion, DM_i the damage measure assumed by the EDP in the i-th analysis and DM_u its limit value corresponding to collapse.

The so obtained new set of data was statistically analysed evaluating the basic parameters (maximum, minimum and mean values and standard deviation) and executing the χ^2 test to check the hypothesis of Normal or Log-Normal distributions. When the χ^2 test was not negative a Normal or Log-Normal distribution was assumed; alternatively the statistical cumulative density function was built, completed in correspondence of tails by suitable exponential functions [18].

The probability of failure related to each set of 500 data (related to a single collapse criterion, a PGA level and accelerogram) was so simply evaluated using its cumulative density function, being $P_f = P[Y > 100]$. Clearly, for each collapse criterion and each PGA level, 7 values of P_f, and so 7 fragility curves, were obtained, one for each accelerogram. The averaged of 7 fragility curves was assumed as the fragility curve related to that specific collapse criterion (see Figure 15).

Fragility of case studies referred to a collapse mode, was finally integrated with European Seismic Hazard function, as described in [6], furnishing annual probability of failure for relevant collapse criteria of all case studies, shortly presented in Tables from 9a to 9f.

Grade		Mean μ N/mm^2	Std. Dev.σ N/mm^2	Model	Correlation Matrix f_y	f_t	ε_u
S275	f_y	350	32	f_y	1	0.74	-0.276
S275	f_t	460	21	f_t	0.736	1	-0.402
S275	ε_u	25	1.75	ε_u	-0.276	-0.4	1
S355	f_y	430	27	f_y	1	0.85	-0.382
S355	f_t	550	25	f_t	0.851	1	-0.577
S355	ε_u	25	1.75	ε_u	-0.382	-0.6	1

Table 8: Statistical parameters assumed for samples generation

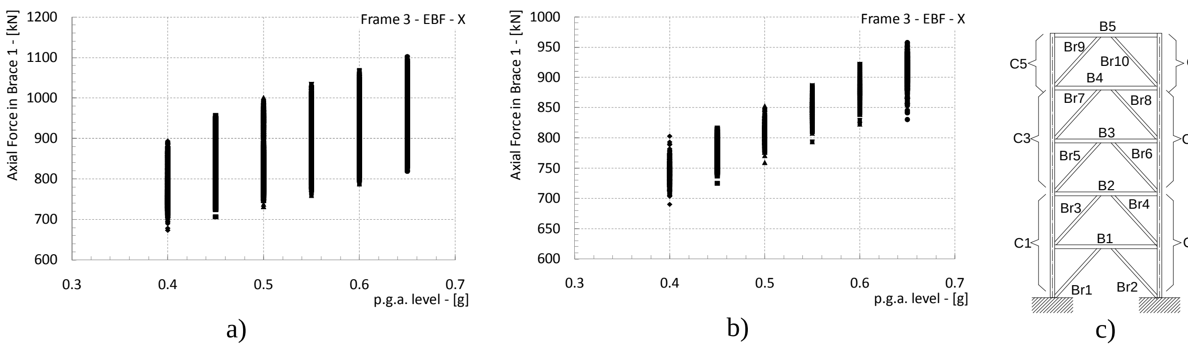
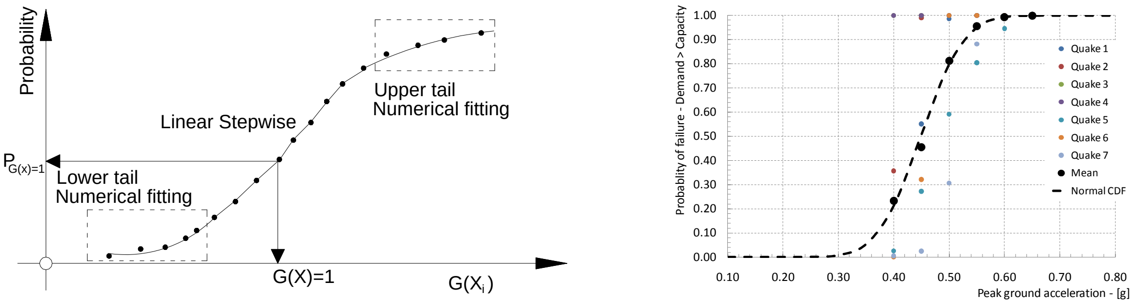


Figure 14: IDA results in terms of Br1 force – a) material and seismic input variability; b) material variability; c) distribution of independent variables inside 3EBFX.



a) b)

Figure 15: a) numerical CDF directly derived from IDA results (when χ^2 failed); b) fragility of 3EBFX for ultimate plastic rotation of the link B1.

7 ANALYSIS OF RESULTS

The statistical procedure defined for a comprehensive analysis of IDA outputs was extensively applied to most part of structural members; in particular, all collapse modes presented in Table 5 were analyzed. The results in terms of annual seismic risk are reported in Tables 9a and 9b for frame 3, in Tables 9c and 9d for frame 4 and in Tables 9e and 9f for frame 16.

Comparing failure probability of braces in frame 3, it is clear which checks had conditioned the design: risk associated to braces in X direction is 5 times less than risk associated to Y frame, confirming the over-sizing required during the design for braces in X direction. The accurate design followed during the sizing of links in order to reduce as much as possible Ω factor and its differences between dissipative members it is confirmed by comparable failure probabilities for all links. The comparison of Ω factor derived from the elastic design, see Table 3, with link failure probability confirms that higher Ω are related to lower failure probability.

Element	B1	B2	B3	B4	B5	Br1	Br2
Seismic Risk	4.10E-04	4.20E-04	4.10E-04	2.80E-04	2.00E-04	4.50E-05	4.30E-05
Element	Drift 1	Drift 2	Drift 3	Drift 4	Drift 5		
Seismic Risk	2.80E-04	3.00E-04	1.60E-04	1.00E-04	9.30E-05		

Table 9a: Annual exceedance probability (Seismic risks) associated to 3EBFX collapse modes.

Element	B1	B4	B5	B8	B9	B12	B13
Seismic Risk	2.90E-04	3.10E-04	9.30E-05	1.30E-04	8.80E-05	1.30E-04	5.70E-05
Element	B16	B17	B20	Drift 1	Drift 2	Drift 3	Drift 4
Seismic Risk	8.20E-05	3.60E-05	5.3-05	2.80E-04	7.10E-05	5.30E-05	3.30E-08
Element	Drift 5	Br1	Br2	C1	C2	C4	
Seismic Risk	3.30E-08	2.80E-04	2.40E-04	1.50E-06	5.90E-06	1.50E-06	

Table 9b: Annual exceedance probability (Seismic risks) associated to 3EBFY collapse modes.

Failure probability associated to columns are really low, confirming that for such kind of structural systems, static combinations with complete factorized set of vertical loads represents for the column the most demanding check in many cases. The annual exceedance probability is often zero or 10^{-5}, largely lower than accepted threshold fixed between 10^{-3} and 10^{-4} for this type of structure accurately designed [19] and belonging to a standard (common) use category. It is worth noting that this trend is confirmed also for the columns in the frame 16: columns have a (annual) failure probability lower than 10-5 also if in such a case vertical loads are not relevant. This effect is related to the capacity design and, in particular, to the contribution of Ω factor: this coefficient is equal to 1.5 in the most optimized design and higher in the common practice, increasing of 50% seismic solicitation without material over-strength factor.

Element	B1	B3	B4	B6	B7	B9	B10
Seismic Risk	1.20E-05	1.10E-05	3.80E-06	3.70E-06	1.10E-06	9.60E-07	1.10E-07
Element	B12	B13	B15	Drift 1	Drift 2	Drift 3	Drift 4
Seismic Risk	3.50E-07	8.30E-06	9.50E-06	5.50E-06	1.70E-07	4.20E-09	5.50E-08
Element	Drift 5	Br1	Br2	C1	C2	C3	C4
Seismic Risk	5.50E-08	1.10E-05	1.00E-08	2.40E-15	2.00E-14	2.40E-15	2.40E-15

Table 9c: Annual exceedance probability (Seismic risk) associated to 4EBFX collapse modes.

Element	B1	B4	B5	B8	B9	B12	B13
Seismic Risk	1.20E-05	1.30E-05	2.70E-06	2.80E-06	4.20E-07	4.40E-07	2.00E-06
Element	B16	B17	B20	Drift 1	Drift 2	Br1	Br2
Seismic Risk	2.40E-06	2.30E-02	2.90E-02	4.50E-06	1.30E-06	1.20E-05	2.30E-05

Table 9d: Annual exceedance probability (Seismic risk) associated to 4EBFY collapse modes.

Seismic links contained in the frame 3 and 16 were shear links designed for high seismicity zones and their annual failure probability are fixed about 10^{-4}, while for bending links – frame 4 – failure probability is set about 10^{-5}, giving in such a case a more conservative design respect those executed in high seismic zones and with higher behaviour factor. This could suggest that EN1998 design procedure cannot allow the designers to optimize structural solutions designed for low seismic loads or with low behaviour factors.

Element	B1	B2	B3	B4	B5	B6	Br1
Seismic Risk	1.90E-04	2.00E-04	2.10E-04	3.40E-05	3.60E-05	3.40E-05	1.60E-04
Element	Br2	Br3	Br4	Br5	Br6	Drift 1	Drift 2
Seismic Risk	1.50E-04	1.50E-04	1.50E-04	1.60E-04	1.60E-04	2.40E-04	3.90E-05
Element	C1	C2	C3	C4			
Seismic Risk	3.50E-06	1.50E-05	1.50E-05	4.50E-06			

Table 9e: Annual exceedance probability (Seismic risk) associated to 16EBFX collapse modes.

Element	B1	B2	B5	B6	B7	B8	B11
Seismic Risk	2.60E-04	2.60E-04	2.60E-04	2.60E-04	3.60E-06	3.6E-060	3.6E-06
Element	B12	Br1	Br2	Br3	Br4	Br6	
Seismic Risk	3.60E-06	5.00E-06	5.10E-06	9.80E-06	5.40E-06	4.70E-06	
Element	C1	C2	C3	C5	C6	C7	
Seismic Risk	7.60E-08	7.60E-08	7.60E-08	7.60E-08	3.00E-07	3.00E-07	

Table 9f: Annual exceedance probability (Seismic risk) associated to 16EBFY collapse modes.

It is also important to underline that all case studies furnished annual failure probability in-line with the limit proposed by Melcher of 10^{-4} for such structures subjected to exceptional loading conditions [19]. This confirms that control measures considered inside capacity design approach, as material over-strength factor – γ_{OV} – and structural over-strength – Ω, can guarantee an adequate protection level to braces and columns. It seems, moreover, that this protection is too pronounced in the columns and so probably capacity design rules could be relaxed for this structural member.

7.1 Upper limitation on yielding stress: influence on IDA results

The probabilistic procedure was newly applied imposing a preconditioning of material input variables: the f_y of dissipative members was limited imposing a fictitious upper limit equal to 1.375, 1.35, 1.30 and 1.25 time the nominal yielding of the steel quality; in such a way, all results coming from simulations characterized by seismic link yielding higher than fixed limits were not considered. These limits were equivalent to impose a fictitious quality control for the seismic qualification, according to [6] or more severe limits, of steel profiles produced according to EN10025 [20]. Upper yielding definition reduced the number of useful material samples (variables) employable in the failure probability estimation; this reduction was more marked, as expected, for S275 quality being a steel quality less controlled than S355. In the Figure 16 it has been reported the effect of imposing an upper limit of 1.375 times the nominal yielding on link 1 properties for frame 3X and 16X: the upper yielding limit has no effect on generated samples while the effect is stronger for S275.

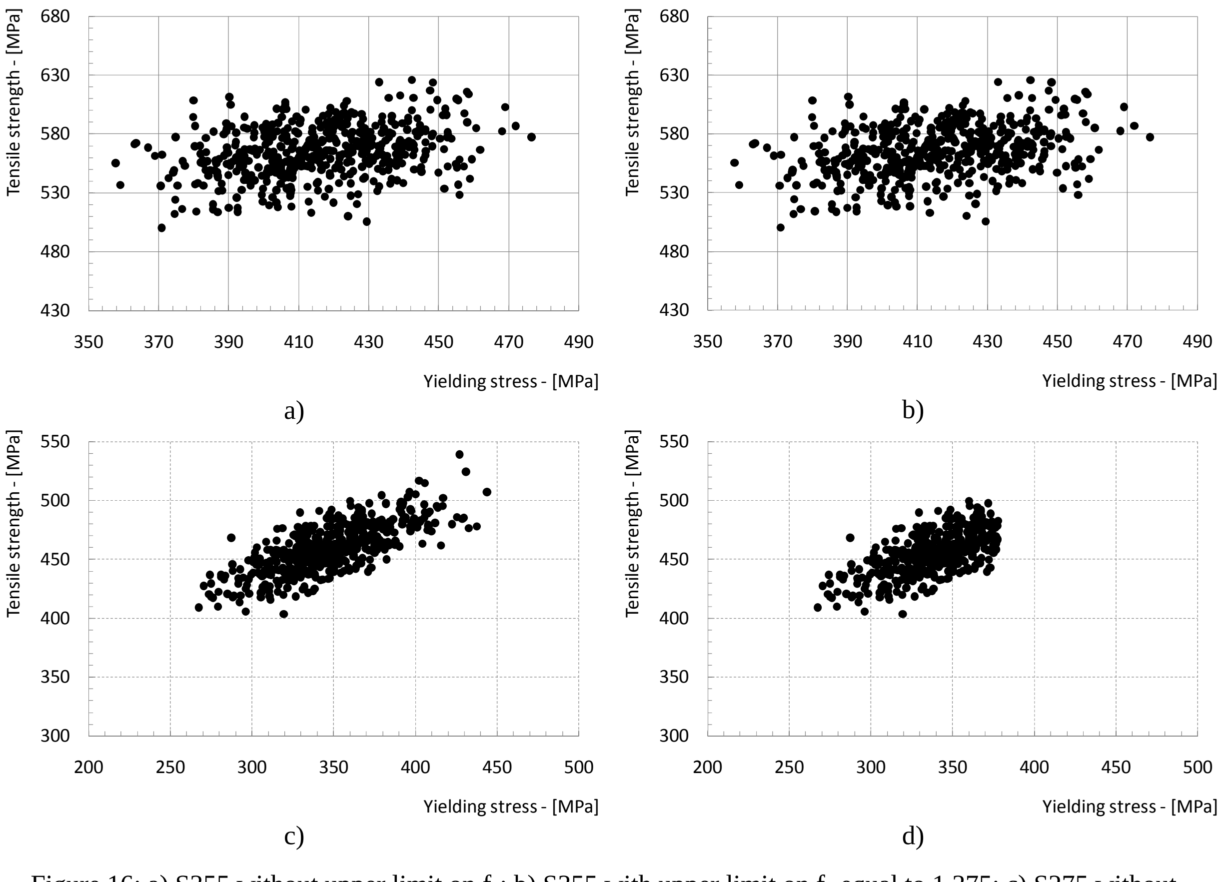
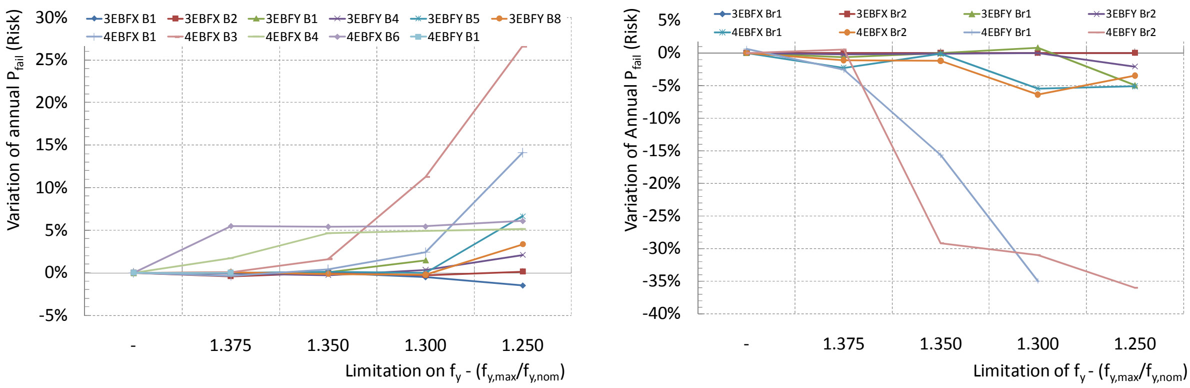


Figure 16: a) S355 without upper limit on f_y; b) S355 with upper limit on f_y equal to 1.375; c) S275 without upper limit on f_y; d) S275 with upper limit on f_y equal to 1.375.

Results in terms of annual failure probabilities (risk) calculated with the previous procedure are presented in Figures 17a and 17b.



a) b)

Figure 17: a) variation of risk associated to ultimate plastic rotation of links; b) variation of risk associated to buckling of first storey braces.

The assignation of upper fy limits produced, as expected, a variation in the risk associated to link rotation and brace buckling; in particular, the annual probability of the link failure increased from 2% to 25% also while probability associated to braces failure decreased from 1% to 35%. According to these results it is clear that the definition of upper f_y limits must be accurately evaluated in order to do not unbalance too much the design in the exploitation of link plastic resources over its failure. At the same time it is also clear that the big decreasing of risk associated to brace failure is related to 4EBFY only, in which braces were really optimized (i.e. Capacity Design=Buckling Strength); in the other cases, more adherent to day-to-day practice, a little over-sizing (i.e. C.D.=0.92B.S.) furnished maximum variations of risk about -6%, mitigating strongly the effects of upper fy limit. It is worth noting that the benefits and the safety increment associated to additional controls for the seismic qualification of steel profiles must be carefully evaluated because structural safety herein estimated, considering both seismic input and material variability – Tables 9a-9f, is in-line with nominal values proposed by experts for the structural cases – 3EBFX, 3EBFY, 4EBFX, 4EBFY, 16EBFX and 16EBFY – under exceptional loading situation as earthquake.

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