buildings

MDPI

Article

Seismic Behaviour of EC8-Compliant Moment Resisting and Concentrically Braced Frames

Silvia Costanzo *, Roberto Tartaglia, Gianmaria Di Lorenzo and Attilio De Martino

University of Naples Federico II, Department of structures for Engineering and Architecture, Naples 80134, Italy

* Correspondence: silvia.costanzo@unina.it; Tel.: +39-081-25-38-917

Received: 25 July 2019; Accepted: 26 August 2019; Published: 28 August 2019

Abstract: The design procedure codified within current Eurocode 8 for dissipative moment resisting and concentrically braced frames have led to the design of massive systems characterized in the most of cases by poor energy dissipation capacity. The research activity presented in the current paper addresses the identification of the main criticisms and fallacies in the current EN 1998-1 for those seismic-resistant typologies. In this regard, the design provisions and codified rules for both moment resisting frames (MRFs) and chevron concentrically braced frames (CCBFs) are critically discussed and numerically investigated. Static and incremental dynamic analyses were performed on a set of 3 and 6-story frames designed compliant to EN 1998-1. The results from the numerical analyses are reported and discussed.

Keywords: Eurocode 8; Moment resisting frames; Concentrically braced frames; Dissipative systems; Seismic design; Earthquake

1. Introduction

In the framework of EN1998-1 (referred to in the text as EC8) [1], seismic resisting structures can be designed compliant with two different approaches: Low-dissipative behaviour and dissipative behaviour.

According to the former approach, the system is designed to merely withstand the seismic force in an elastic range and it is generally opted for in low seismicity areas or for structures of special importance. Indeed, guaranteeing that plastic deformations are totally prevented requires high lateral strength and stiffness, leading to uneconomical and massive structural systems in most of the common applications.

Conversely, the second design concept allows the achievement of a more economical design by dissipating seismic input energy thorough extended inelastic deformation restrained in specific zones (dissipative zones), while the remaining elements or parts of elements (non-dissipative zones) behave elastically. To satisfactorily guarantee seismic energy dissipation, capacity design criteria are applied that allow the significant ductility reserve to be exploited in dissipative zones.

During the last decade, Eurocode 8 has been widely used by structural designers, and several authors [2–24] have extensively investigated the seismic performance of steel dissipative frames designed according to EC8. Both experimental and numerical studies have revealed that the design procedure currently codified in EC8 is not adequate to guarantee global ductile failure and satisfactory energy dissipation capacity in the most of cases; indeed EC8-compliant detailing rules often entails significant effort in the design process and they lead to the design of massive and over-resistant structural systems, characterized by poor plastic engagement of dissipative zones and high constructional cost. These detrimental features are notably juxtaposed to the philosophy behind the second design concept, and numerous research [11–15,18,23] have been devoted to investigate and propose new design criteria to improve the seismic performance of steel structural systems. In these framework, special attention has been put on both moment resisting frames (MRFs) [2–9,21,22] and chevron concentrically braced frames (CCBFs) [11–23]; indeed, the design rules provided by EC8 for these structural typologies are affected by the largest number of criticisms and fallacies.

It is even worth mentioning that the Eurocode 8 is currently under revision [20]: The European six-year work program is currently ongoing to amend and revise all Eurocodes, also to include new findings and the advances in knowledge coming from the research and scientific community.

All the above-mentioned remarks have motivated the research addressed in this paper, which is merely devoted to identify the main criticisms affecting the design procedure currently codified in Eurocode 8 for these structural types. With this aim, the research was organized in two main parts: In Section 2, the current EC8-compliant design rules are summarized and critically discussed for both moment-resisting and concentrically-braced frames; and the current codified design rules were numerically validated by performing nonlinear dynamic and static pushover analyses on a set of reference frames, 3 and 6-storey high. Numerical results are presented and discussed in Section 3.

2. Seismic design of steel frames according to EN 1998-1: Critical discussion

2.1. Moment resistring frames (MRF)

EC8 assigns a reference behaviour factor equal to 4 and 5 α_u/α_y for ductility class medium (DCM) and high (DCH), respectively, where the ultimate-to-yielding capacity ratio $\alpha_u/\alpha_y \leq 1.6$ accounts for the redundancy of the systems; it can be evaluated by performing a pushover analysis or otherwise it can be assumed equal to 1.3 for multi-storey multi-bay MRFs.

In MRFs with full strength and full rigid joints, the dissipative zones are located at the ends of the beams, where plastic hinges form. At the beam ends, the following inequality should be met:

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1 \tag{1}$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0.15 \tag{2}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0.5 \tag{3}$$

where M_{Ed}, N_{Ed} and V_{Ed} are the design forces, and $M_{pl,Rd}$, $N_{pl,Rd}$ and $V_{pl,Rd}$ are design resistances in accordance with EN 1993:1-1 [25].

To achieve global ductile collapse, any plasticity should be avoided in the columns except at the base of the frame, at the top level of multi-storey buildings and for single storey buildings. This type of failure mode is generally referred as "weak beam/strong column" behaviour and it is guaranteed by meeting the following requirement:

$$\sum M_{Rc} \geq 1.3 \cdot \sum M_{Rb} \tag{4}$$

where, $\sum M_{Rc}$ and $\sum M_{Rb}$ are the sum of the design values of the moments of resistance of the columns and beams, respectively, framing at a joint. This requirement is waived at the base of the frame on the top level of multi-storey buildings and for single storey buildings.

For steel MRFs, Eurocode 8 even mandates the following specific capacity design requirements as follows:

$$\begin{aligned} M_{Ed,col} &= M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \\ N_{Ed,col} &= N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \\ V_{Ed,col} &= V_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot V_{Ed,E} \end{aligned} \tag{5}$$

where:

$N_{Ed,G}$, $M_{Ed,G}$, $V_{Ed,G}$ are the design forces in the column due to the non-seismic action included in the combination of actions for the seismic design situation;
$N_{Ed,E}$, $M_{Ed,E}$, $V_{Ed,E}$ are the seismic induced effects;
γ_{ov} is the overstrength factor accounting for randomness of yield strength according to EC8;
Ω is minimum of $\Omega_i = \frac{M_{pl,Rd,i}}{M_{Ed,E,i}}$ of all beams where dissipative zones are located;
$M_{pl,Rd,i}$ is the plastic bending resistance of the i-th beam;
$M_{Ed,E,i}$ is the bending moment due to the seismic loads in the i-th beam.

In addition, the column shear force V_{Ed} resulting from the structural analysis should satisfy the following expression:

$$\frac{V_{Ed,c}}{V_{pl,c,Rd}} \leq 0.5 \tag{6}$$

As observed by previous research [2,21,24], capacity design requirement expressed by Equation (5) is inaccurate in the most of cases; indeed the beam overstrength factor $\Omega_i = \frac{M_{pl,Rd,i}}{M_{Ed,E,i}}$ does not account for gravity loads. Therefore, the actual beam overstrength can be significantly larger than expected, even two or three times in gravity-ruled cases. Moreover, the minimum value of Ω evaluated according to Equation (5), considers the formation of the first plastic hinge and it does not correspond to the overall capacity of the structure. Depending on the frame redistribution capability, the column can be subjected to higher force than expected [2,21,24].

Beside the capacity design rules, the deformation-related requirements stated by EC8 for both the serviceability and ultimate limit states play key role in the seismic design of MRFs and thus deserve proper consideration.

The lateral displacement should be controlled at the serviceability limit state, according to the following inequality:

$$d_r \cdot \nu \leq K \cdot h \tag{7}$$

where d_r is the design interstorey drift, h is the story height, and v is a reduction factor accounting for the lower return period of the seismic action associated with the serviceability limit state, even depending on the importance class of the building. The factor K depends on the type of infill walls and it is equal to 0.05, 0.075, and 0.01 for brittle, ductile, and non-structural elements fixed in a way so as not to interfere with (or without) non-structural elements, respectively.

Moreover, EC8 stipulates that the influence of second order (namely, P-Δ) effect should be properly considered in the design at the ultimate limit state. With this regard, the Code defines an interstorey drift sensitivity coefficient as

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \tag{8}$$

where P_{tot} is the total gravity load, V_{tot} is the seismic shear at the storey under consideration, d_r is the interstorey drift (given by the elastic inter-storey drift by the behaviour factor), and h is the storey height. If $\theta \leq 0.1$, second order effects can be disregarded; conversely if $0.1 < \theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$. The value of the sensitivity coefficient θ should not exceed 0.3.

The deformation related requirements rule the sizing of MRFs in the most of cases; indeed, to fulfil the drift limitation and to assure enough stability against second order effects, the designer is forced to oversize the structural members with respect to the relevant required strength, leading to massive and uneconomical systems characterized by large lateral overstrength and poor energy dissipation capacity. For instance, Tenchini et al. [26] observed that for EC8 compliant MRFs, the overall overstrength factors are larger than the design behaviour factor owing to codified design procedure, which leads to increase member size to satisfy drift limitations. From nonlinear dynamic analyses, Tenchini et al. [26] showed that MRFs have a seismic demand (namely, transient and residual drift ratios, beam ductility) fairly below the proposed limit for the damage limitation (DL), significant damage (SD), and near collapse

(NC) limit states [27]. In particular, in an SD limit state, most of frames behave in an elastic field due to the design being oversized. In addition, the elastic response of MRFs can be responsible for large non-structural damage of acceleration-sensitive components, since they obtained median peak storey accelerations ranging from two to three times the design PGA.

More recently, Tartaglia et al. [2] investigated the influence of the P-Delta effect requirements in the design procedure of MRFs. In particular, they proposed to calculate the critical multiplier and the stability coefficients accounting for the design overstrength, thus using a secant stiffness of the non-linear equivalent structure larger than the value currently prescribed by EC8. This modification allowed the design of lighter and weaker MRFs that guaranteed an overall ductile response, while the EC8-compliant frames showed an elastic response up to a near collapse limit state.

In the light of these remarks and observations from previous studies, it can be argued that drift limitations and second order effect checks, as codified within EC8, are too stringent, even when compared with the corresponding rule provided by the US Code [28], which defines the stability coefficient based solely on the elastic inter-storey drift, thus using the elastic stiffness of the structure and resulting in less stringent requirements [21].

2.2. Chevron concentrically braced frames (CCBF)

The framework of EC8 bracings in chevron configuration are expected to provide limited ductility with respect to other concentric bracings types and a smaller value of behaviour factor is assigned. In detail, q equal to 2 and 2.5 is assumed in DCM and DCH, respectively, namely, smaller than value $q = 4$ is assumed for other bracing configurations.

The seismic design criteria codified within Eurocode 8 is aimed at guaranteeing an overall ductile performance with plastic deformations restrained into diagonal members, while the remaining structural members are kept in elastic range.

The required strength of diagonal members is evaluated by mean of elastic analysis and the bracings should be designed to fulfil the following inequality:

$$\chi \cdot N_{pl,br,Rd} \geq N_{Ed,br} \tag{9}$$

where $N_{pl,br,Rd}$ is the plastic axial strength, χ is the buckling reduction factor calculated according to EN 1993:1-1 [25], and $N_{Ed,br}$ is the axial force acting on the element.

To prevent too severe a deterioration of bracing response, EC8 limits the diagonal normalized slenderness $\overline{\lambda} = \sqrt{\frac{N_{pl,br}}{N_{cr,br}}}$, which should be smaller than 2.0. In addition, to avoid soft-storey mechanisms and to favour uniform distribution of plastic deformation along the building eight, the Code mandates to limit the variation of the diagonal capacity-to-demand ratios according to the following condition:

$$[(\Omega_i - \Omega)/\Omega] \leq 0.25 \tag{10}$$

where $\Omega = \min(\Omega_i)$ and $\Omega_i = \frac{N_{pl,br,Rd,i}}{N_{Ed,br,i}}$ is the overstrength ratio at the i-th storey. The rule expressed by Equation (10) leads to design systems characterized by significant lateral overstrength and very poor plastic engagement. Indeed, the seismic induced effect is generally lower at the roof storey, where the highest value of overstrength ratio is recognized due to need to contemporarily meet the maximum allowable slenderness ratio $\overline{\lambda} \leq 2$; as a consequence, to limit the capacity-to-demand variation, the designer is forced to oversize the bracing cross-sections even at the intermediate and lower storeys. Moreover, as numerous research have report, the requirement expressed by Equation (10) is not adequate to assure uniform distribution of plasticity along the frame height and to prevent soft-storey mechanisms.

To avoid any nonlinearity into non-dissipative members (namely, beams and columns), capacity design criteria are applied.

The columns belonging to the braced bays are designed to withstand the following action:

$$N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \quad (11)$$

where

$N_{pl,Rd}(M_{Ed})$ is the design resistance to axial force of the column calculated in accordance with EN 1993:1-1 [25], taking into account the interaction with the design value of bending moment, M_{Ed}, in the seismic design situation;
$N_{Ed,G}$ is the axial force in the column due to the non-seismic actions included in the combination of actions for the seismic design situation;
$N_{Ed,E}$ is the axial force in the column due to the design seismic action;
γ_{ov} is the material overstrength factor;
Ω is the minimum overstrength ratio $\Omega_i = N_{pl,bRd,i}/N_{Ed,br,i}$;

As highlighted by previous research [17–19], the force transfer mechanism in the post-buckling range differs significantly from the elastic behavior and the columns are likely be subjected to a force significantly larger than that calculated according to Equation (11).

The seismic performance of chevron concentric bracings depends significantly on the behaviour of the brace-intercepted beam [17,29,30]. Indeed, after the buckling of the compression diagonal, an unbalanced vertical force is applied at the brace-intercepting section, inducing high bending demand. Flexural yielding of the beam would result in sudden drop of the storey lateral capacity; thereby, to prevent such detrimental behaviour the beam in chevron configuration should be designed to withstand: (i) All non-seismic loads without considering the intermediate support given by diagonals; (ii) the unbalanced vertical force as result of vertical components transmitted by tension and compression braces in the nonlinear range. In this regard, the tension brace is assumed to attain its nominal plastic capacity, while the compression diagonal exhibits its post buckling capacity, evaluated as

$$N_{pb,br} = \gamma_{pb} \cdot N_{pl,br,Rd} = 0.3 \cdot N_{pl,br,Rd} \quad (12)$$

It should be noted for diagonal members close to the maximum allowable slenderness ratio, the buckling reduction factor χ assumes values around 0.2 and thus the buckling capacity of the member results are smaller than the residual capacity evaluated according to Equation (12).

Several pieces of research have demonstrated that the capacity design requirements given by EC8 for brace-intercepted beams are not adequate to satisfactorily guarantee seismic behaviour [17,29,30]. Indeed, the detailing rules current codified in EC8 focus solely on the beam strength, disregarding the deformation-related aspects. Conversely, as highlighted by [17], the vertical deflection of the beam and the ductility demand on bracings are correlated phenomena, therefore frames with strong but flexible beams exhibit very poor seismic response. Shen et al. [29] observed that the inelastic deformation in the middle spans of brace-intersected beams substantially increases ductility demands on braces and beam-to-column connections. Indeed, under this condition, the rotational demand on beam-to-column connection may be larger than 0.06 radians at the 0.02 story drift ratio response. Shen et al. [29] also found that at 2% interstorey drift rotation, the ductility demand on diagonal bracings may significantly change from about eight times the yield displacement in CBF with stiff brace-intersected beams, up to more than 20 times the yield displacement in structures with weak and deformable brace-intersected beams.

3. Numerical assessment of Eurocode 8-compliant design rules

3.1. Design and modelling assumptions

The effectiveness of the EC8 seismic design rules was numerically investigated on a set of both moment resisting (MR) and chevron concentrically braced (CCB) frames.

3 and 6-storey 2D frames were extracted from the perimeter in X direction of the reference residential buildings reported in Figure 1, where the location of seismic resisting systems is highlighted with bold lines.

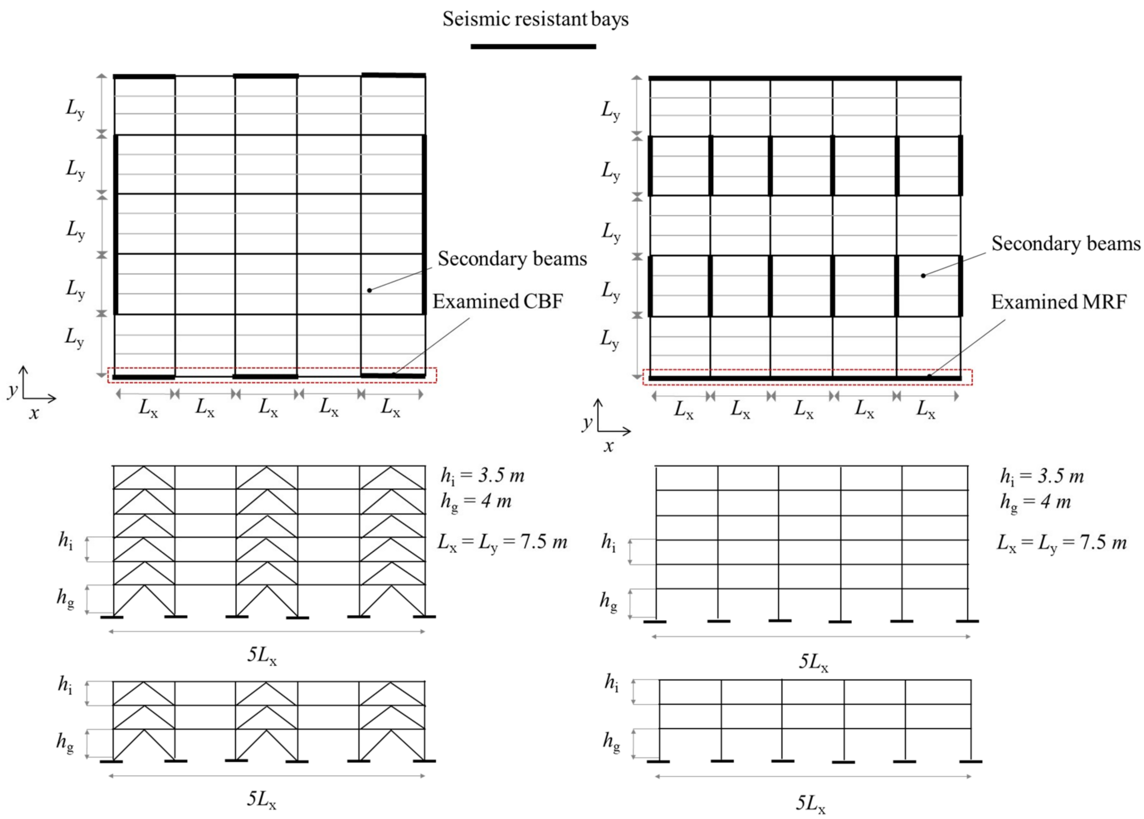

Figure 1. Reference buildings in plan and elevation.

At each floor, the rigid diaphragm was obtained by means of composite slabs with profiled steel sheeting supported by the hot-rolled steel beams. The structural design for gravity loads was developed compliant to EN 1993:1-1 [25]. Permanent loads (G_k) and live loads (Q_k) were assumed equal to 5.00 kN/m^2 and 3.00 kN/m^2, at each storey. The inertial effects in the seismic design situation were evaluated according to EC8. A reference peak ground acceleration equal to a_{gR} = 0.25g a type C soil, a type 1 spectral shape was assumed.

The cross sections of all steel members (i.e. beams, bracings and columns) were selected to satisfy the Class 1 requirements according to EN 1993:1-1 [25]. The geometrical and mechanical properties of structural members for the designed CCBFs and MRFs are ported in Tables 1–4.

Table 1. Structural members for 3-Story chevron concentrically braced frame (CCBF).

	Gravity Members		Concentrically Braced Bays		
Storey	**Column**	**Beam**	**Column**	**Beam**	**Brace (dxt)**
	S355	S355	S355	S355	S355
I	HEB240	IPE300	HEM300	HEB450	159 × 8
II	HEB200	IPE300	HEB300	HEB450	159 × 8
III	HEB200	IPE300	HEB300	HEB340	133 × 6.3

Table 2. Structural members for 6-Story CCBF.

	Gravity Members		Concentrically Braced Bays		
Storey	**Column**	**Beam**	**Column**	**Beam**	**Brace (dxt)**
	S355	S355	S355	S355	S355
I	HEB300	IPE300	HD_400 × 551	HEM500	193.7 × 12.5
II	HEB300	IPE300	HD_400 × 551	HEM500	177.8 × 14.2
III	HEB260	IPE300	HD_400 × 347	HEM500	177.8 × 12.5
IV	HEB260	IPE300	HD_400 × 347	HEB550	168.3 × 12.5
V	HEB220	IPE300	HEB400	HEB450	159 × 8
VI	HEB220	IPE300	HEB400	HEB360	139.7 × 6.3

Table 3. Structural members for 3-Story moment resisting frame (MRF).

Storey	**First Bay**		**Second Bay**		**Third Bay**	
	Column	**Beam**	**Column**	**Beam**	**Column**	**Beam**
	S355	S355	S355	S355	S355	S355
I	HE500B	IPE600	HE600B	IPE600	HE600B	IPE600
II	HE450B	IPE600	HE600B	IPE600	HE600B	IPE600
III	HE450B	IPE550	HE450B	IPE550	HE450B	IPE550

Table 4. Structural members for 6-Story MRF.

Storey	**First Bay**		**Second Bay**		**Third Bay**	
	Column	**Beam**	**Column**	**Beam**	**Column**	**Beam**
	S355	S355	S355	S355	S355	S355
I	HE650M	IPE750 × 147	HE700M	IPE750 × 147	HE700M	IPE750 × 147
II	HE650M	IPE750 × 147	HE700M	IPE750 × 147	HE700M	IPE750 × 147
III	HE650B	IPE750 × 137	HE650M	IPE750 × 137	HE650M	IPE750 × 137
IV	HE650B	IPE750 × 137	HE650M	IPE750 × 137	HE650M	IPE750 × 137
V	HE450B	IPE600	HE650B	IPE600	HE650B	IPE600
VI	HE450B	IPE600	HE650B	IPE600	HE650B	IPE600

The non-linear behaviour of the 2D examined frames was simulated by using the Seismostruct informatic platform [31]. Masses were lumped into a selected master joint at each storey, where rigid diaphragms were modelled. To account for the second order effects, the vertical loads that were not tributary on the examined 2D frames were assigned to a zero-stiffness leaning column connected to the frames by pinned rigid links.

The structural members were modelled using the force-based (FB) distributed inelasticity elements, which account for distributed inelasticity through integration of material response over the cross section and integration of the section response along the length of the element. The cross-section behaviour was reproduced by means of the fibre approach, by assigning a uniaxial stress–strain relationship at each fibre. The steel hysteretic behaviour was simulated by using the Menegotto–Pinto model [32]. For chevron concentrically braced frames, physical-theory model (PTM) was used to mimic the hysteretic response of bracing members as shown by Uriz et al. and D'Aniello et al. [33,34]. The bracing members were modelled as fixed in-plane of the frames and pinned out-of-plane. Columns were considered continuous through each floor beam and the beam-to-column connections were assumed pinned.

The MR-frames were modelled accounting for the strength and deformability of the beam-to-column joints. The column web panel was simulated using rigid links accounting for the geometric dimensions of the column, because the column web panels were designed with supplementary web plates to resist the required shear force without the contribution of the continuity plate. Hence, the panel zone may be considered as rigid and full strength and its contribution to the joint deformability can be neglected [35]. The connection behaviour and the plastic hinge behaviour were modelled by using a nonlinear spring model defined in accordance with the Ibarra–Medina–Krawinkler (IMK) modified model [36,37] (see Figure 2).

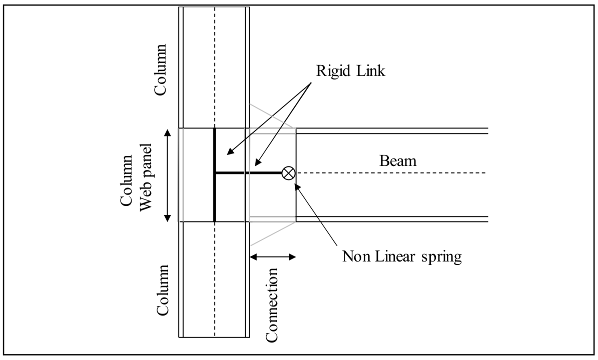

Figure 2. Beam-to-column joints modelling features introduced in the structures.

3.2. Seismic performance evaluation

Both static and incremental dynamic analyses (IDAs) were performed to evaluate the seismic performance of the examined frames. Static non-linear (pushover) analyses were performed considering two lateral loads distribution (as prescribed by EC8), namely according to the first vibration mode and proportional to the masses' distribution along the structure's height.

A suite of 14 natural earthquake acceleration records was considered to perform the non-linear dynamic analyses; the records were obtained from the RESORCE ground motion database [38] and selected according to [39] to match the elastic acceleration spectrum provided for by EC8 (see Figure 3). More details on the selection of the signals are reported in Table 5.

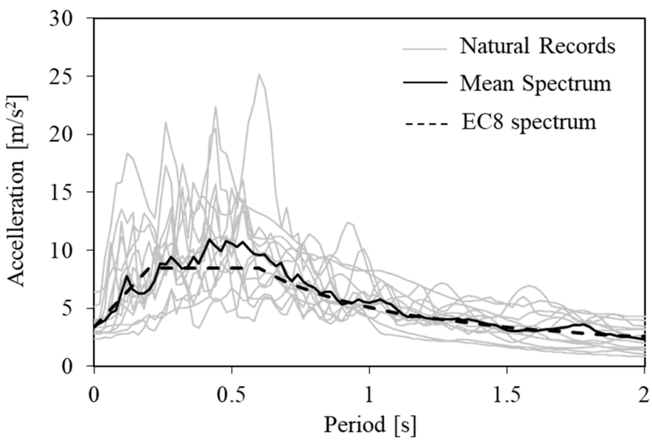

Figure 3. Selected earthquake acceleration records.

Table 5. Data of selected ground motion.

Earthquake Name	Date	Station Name	Station Country	Magnitude Mw	Fault Mechanism
Alkion	24.02.1981	Xylokastro-O.T.E.	Greece	6.6	Normal
Montenegro	24.05.1979	Bar-Skupstina Opstine	Montenegro	6.2	Reverse
Izmit	13.09.1999	Yarimca (Eri)	Turkey	5.8	Strike–Slip
Izmit	13.09.1999	Usgs Golden Station Kor	Turkey	5.8	Strike–Slip
Faial	09.07.1998	Horta	Portugal	6.1	Strike–Slip
L'Aquila	06.04.2009	L'Aquila - V. Aterno-Aquila Park In	Italy	6.3	Normal
Aigion	15.06.1995	Aigio-OTE	Greece	6.5	Normal
Alkion	24.02.1981	Korinthos-OTE Building	Greece	6.6	Normal
Umbria-Marche	26.09.1997	Castelnuovo-Assisi	Italy	6.0	Normal
Izmit	17.08.1999	Heybeliada-Senatoryum	Turkey	7.4	Strike–Slip
Izmit	17.08.1999	Istanbul-Zeytinburnu	Turkey	7.4	Strike–Slip
Ishakli	03.02.2002	Afyon-Bayindirlik ve Iskan	Turkey	5.8	Normal
Olfus	29.05.2008	Ljosafoss-Hydroelectric Power	Iceland	6.3	Strike–Slip
Olfus	29.05.2008	Selfoss-City Hall	Iceland	6.3	Strike–Slip

Figure 4 depicts the pushover response curves for the examined moment resisting frames. Both 3 and 6-story MRFs exhibit large lateral overstrength merely depending on two different sources: (i) The design overstrength, and (ii) the overstrength due to the redundancy of the system. To clarify this aspect, Table 6 reports the following data, evaluated from the pushover analyses:

- the plastic redistribution parameter α_u/α_y accounting for the system overstrength due to redundancy. The parameter α_y is the multiplier of the horizontal seismic design action to reach the first plastic resistance in the structure and α_u is the multiplier of the horizontal seismic design action necessary to form a global mechanism corresponding to the maximum shear capacity (V_{max}).
- The coefficient $\Omega_0 = V_y/V_d$ representing the design overstrength, namely the ratio between actual capacity (V_y) respect to the design shear (V_d).

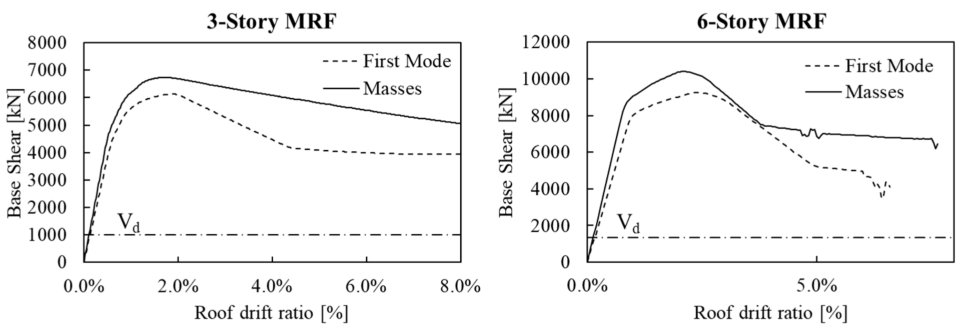

Figure 4. Pushover response curve for examined MRFs.

Table 6. Results from puhsover analyses performed on examined MRFs.

Structures		V_d	V_y	V_{max}	α_u/α_y	Ω_0	V_{max}/V_d
		kN	kN	kN	-	-	-
3-storey	First mode	995.58	3950.00	6136.66	1.55	3.97	6.16
	Masses	995.58	4200.00	6743.97	1.61	4.22	6.77
6-Storey	First mode	1361.79	7450.00	9247.89	1.24	5.47	6.79
	Masses	1361.79	8150.00	10406.35	1.28	5.98	7.64

It is worth noting that, even though the global lateral overstrength (V_{max}/V_d) is similar for both 3 and 6-storey frames, the larger influence of design overstrength Ω_0 can be recognized for the taller structure. This feature can be explained considering that the low-rise frame is less influenced by the deformation-related requirements and thus it exhibits limited design overstrength (smaller Ω_0 values) and better system redundancy (larger α_u/α_1) with respect to the taller frame.

Figure 5 shows interstorey drift ratios calculated by incremental dynamic analyses with reference to the three limit states: Damage limitation (DL), significant damage (SD) and at near collapse (NC), as defined by EN1998-3 [27]. Both 3 and 6-storey frames behave almost elastically up to NC limit state. Therefore, increasing the seismic acceleration up to 1.72 times, the interstorey drift remains almost under the limit of the 1.5% for 6-storey frames, implying that the structures remain in elastic range; while moderate plastic deformations were observed at the first storey of the 3-storey MRF with an interstorey drift of about 2% This result is consistent with the outcomes from the pushover analyses where the minimum difference overstrength (V_{max}/V_y) was equal to 6.16.

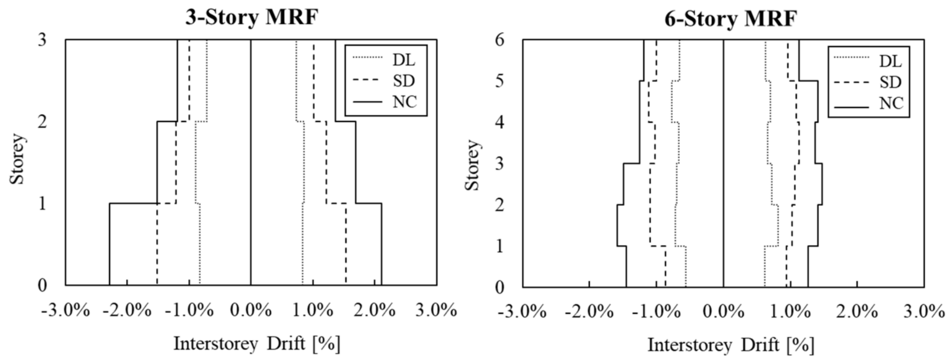

Figure 5. Results from IDAs: interstorey drift for examined MRFs.

Moreover, in order to investigate the ultimate structures capacity, the IDAs were performed, scaling the intensity of each signal from 0.25 up to 3.5 times. The results of IDAs are reported in terms of spectral acceleration, corresponding to the first vibration period (which represents the ground motion intensity) against the maximum interstorey drift ration (which represents the engineering demand parameter) [40].

Figure 6 shows the average of all the 14 IDA curves for the analysed MRFs, where the activation of global dynamic instability of the structure corresponds to the part of each curve that becomes flat. However, it can be observed that the collapse corresponds to very large values of ground motion intensity, namely, 11.6 and 5.7 times the design values, respectively, for the 3 and 6-storey structures.

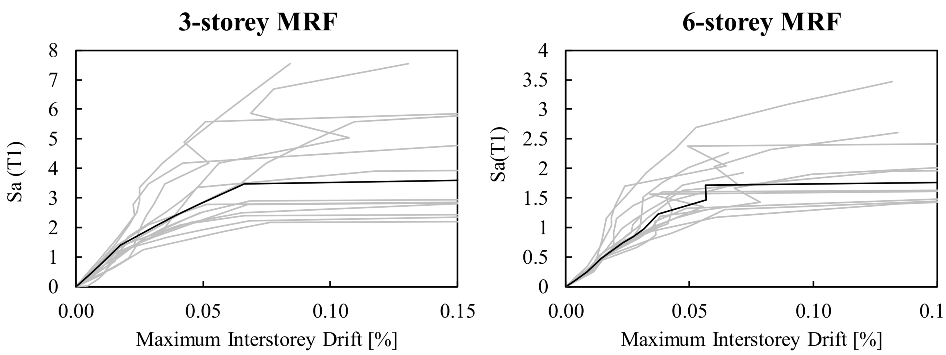

Figure 6. IDAs curves for examined MRFs.

Pushover response curves for the examined CBFs are depicted in Figure 7, while the relevant data are reported in Table 7. Both 3 and 6-storey frames exhibit large capacity-to-design base shear ratios

Ω_0 (ranging within [3.05, 4.41]), thus confirming large lateral overstrength due to the application of the codified design rules. Equipping multiple bays with chevron concentric bracing provides satisfactorily redundancy with values of plastic redistribution parameter α_u/α_1 ranges within [1.34, 1.66].

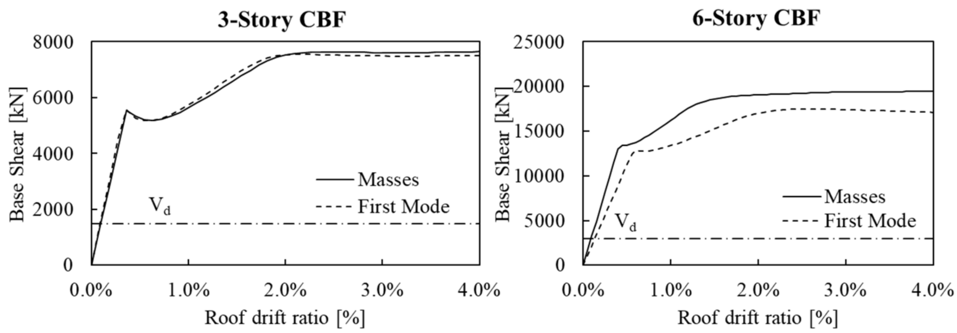

Figure 7. Pushover curves for examined concentrically braced frames (CBFs).

Table 7. Results from puhsover analyses performed on examined CBFs.

Structures		V_d	V_y	$\mathrm{V_{max}}$	α_u/α_1	Ω_0	$\mathrm{V_{max}}/V_d$
		kN	**kN**	**kN**	-	-	-
3-storey	First mode	1484.99	4530.00	7510.00	1.66	3.05	5.06
	Masses	1484.99	5570.00	7640.00	1.37	3.75	5.14
6-Storey	First mode	2945.86	12800.00	17100.00	1.34	4.35	5.80
	Masses	2945.86	13000.00	19500.00	1.50	4.41	7.64

Results from incremental dynamic analyses are reported in Figure 8 in terms of both interstorey drift ratio (see Figure 8a) and brace ductility demand (see Figure 8b) ($\mu = \frac{d}{d_y}$, being d_y the axial deformation corresponding to the yielding), with reference to the three limit states DL, SD, and NC. Results depicted in Figure 8a confirm the requirement devoted to control capacity-to-demand ratio is not adequate to avoid a soft-storey mechanism and to assure the uniform distribution of plastic deformation along the building height. Indeed, a cantilever-like displacement profile is recognized for both the 3 and 6-storey buildings, with severe damage concentration solely at the roof (see Figure 7b). Very poor plastic engagement of braces under tension can be recognized up to the NC limit states.

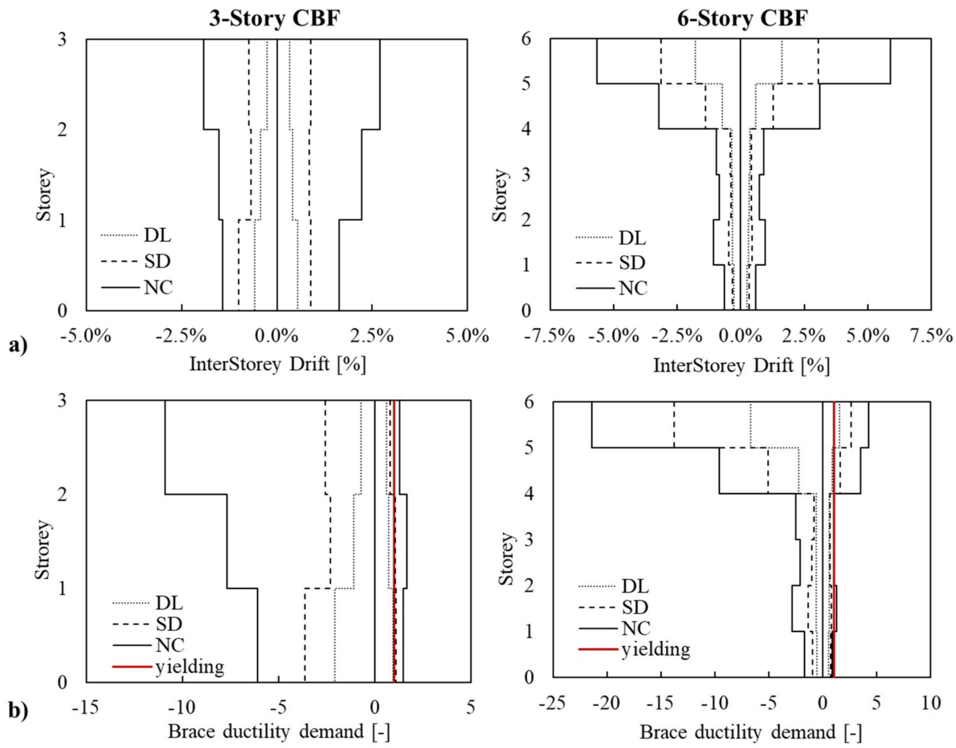

Figure 8. Results from IDAs for examined CBFs: (**a**) Interstorey drift ratio; (**b**) braces ductility demand.

The IDA curves of examined CBFs are given in Figure 9, in terms of maximum interstorey drift against structural acceleration at the period of first mode of vibration. As observed for the MRFs, also in this case, the structural collapse (defined as the flat branch of the IDAs curve) occurs only at very large interstorey drift ratios, namely 12 and 14.7 (respectively, for the 3 and 6-storey structures) times the design values.

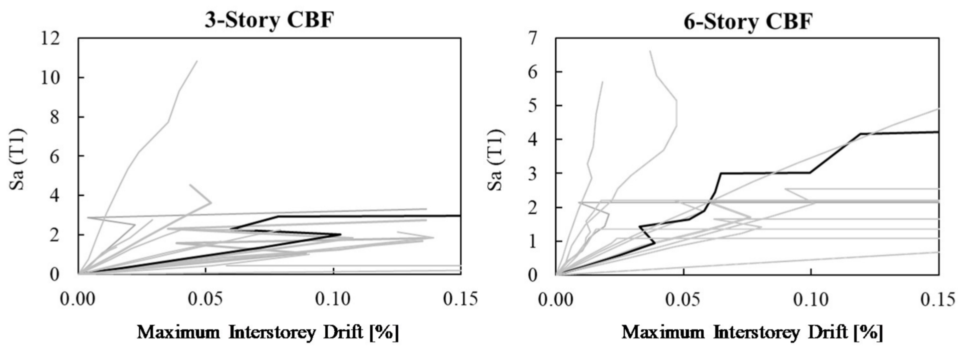

Figure 9. IDAs curves for examined CBFs.

4. Conclusive Remarks

The design procedure codified within the current Eurocode 8 for dissipative moment resisting and concentrically braced frames have led to the design of massive systems characterized in most of the cases by a poor energy dissipation capacity.

The research activity presented in the current paper addressed the identification of the main criticisms and fallacies in the current EC8 for those seismic-resistant typologies. In this regard, the design provisions and codified rules for both MRFs and CBFs were critically discussed and numerically investigated. Both static and incremental dynamic analyses were performed on a set of 3 and 6-storey frames designed compliant to EC8.

The interpretation of numerical results inferred the following remarks:

- Both moment resisting and concentrically braced frames designed in compliance to EC8 are characterized by large lateral overstrength due to the application of codified design requirements.
- The design process of MRFs is merely ruled by the deformation-related requirements: The need to fulfil the drift limitations at the serviceability limit state and the stability requirement (against second order effects) at the ultimate limit state forces the designer to select massive and over-resistant structural members, as shown in this study for both 3- and 6-storey frames.
- The design overstrength of CBFs is mostly derived from the interrelation and juxtaposition of the capacity-to-demand variation requirement and the maximum allowable brace slenderness ratio; to contemporarily fulfil these rules, the designer is forced to significantly oversize diagonals along the building height, as shown for both 3- and 6-storey frames.
- Results from numerical analyses confirm that the examined MRFs and CBFs behave almost elastically up to the NC limit state. Very poor plastic engagement of dissipative zones and energy dissipation capacity is recognized for both 3- and 6-storey frames.
- The overstrength variation requirement is not adequate to avoid a soft-storey mechanism in CBFs. The examined frames exhibited a cantilever displacement shape profile with severe damage concentration solely in the diagonals at upper stories.
- The obtained numerical results for both MRFs and CCBFs are consistent with what was observed in recent literature, confirming the need to amend EC8. However, a larger number of cases and parameters should be investigated to validate the rules for the next code.

Author Contributions: All the Authors equally contributed to the conception and the design of the study. The final draft was critically revised and accepted by all the Authors.

Funding: This research received no external funding.

Conflicts of Interest: The authors declare no conflict of interest.

References

1. EN 1998-1. *Eurocode 8: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings*; CEN: Brussels, Belgium, 2005.
2. Tartaglia, R.; D'Aniello, M.; Di Lorenzo, G.; De Martino, A. Influence of EC8 rules on P-delta effects on the design and response of steel MRF. *Ing. Sismica* **2018**, *35*, 104–120.
3. Tartaglia, R.; D'Aniello, M.; Zimbru, M.; Landolfo, R. Finite element simulations on the ultimate response of extended stiffened end-plate joints. *Steel Compos. Struct.* **2018**, *27*, 727–745.
4. Tartaglia, R.; D'Aniello, M.; Rassati, G.A.; Swanson, J.; Landolfo, R. Influence of composite slab on the nonlinear response of extended end-plate beam-to-column joints. *Key Eng.* **2018**, *763*, 818–825. [CrossRef]
5. Medina, A.R.; Krawinkler, H.J. Evaluation of drift demands for the seismic performance assessment of frames. *J. Struct. Eng.* **2005**, *131*, 1003–1013. [CrossRef]
6. Bravo-Haro, M.A.; Tsitos, A.; Elghzouli, A. Drift and rotation demands in steel frames incorporating degradation effects. *Bull. Earthq. Eng.* **2017**, *16*, 4919–4950. [CrossRef]
7. Montuori, R.; Nastri, E.; Piluso, V.; Troisi, M. Influence of the cyclic behaviour of beam-to-column connection on the seismic response of regular steel frames. *Ing. Sismica* **2016**, *33*, 91–105.
8. Tsitos, A.; Bravo-Haro, M.A.; Elghzouli, A. Influence of deterioration modelling on the seismic response of steel moment frames designed to Eurocode 8. *Earthq. Eng. Struct.* **2017**, *47*, 356–376. [CrossRef]
9. Piluso, V.; Montuori, R.; Nastri, E.; Paciello, A. Seismic response of MRF-CBF dual systems equipped with low damage friction connections. *J. Constr. Steel Res.* **2019**, *154*, 263–277. [CrossRef]
10. Faggiano, B.; Formisano, A.; Castaldo, C.; Fiorino, L.; Macillo, V.; Mazzolani, F.M. Appraisal of seismic design criteria for concentric bracing steel structures according to Italian and European codes. *Ing. Sismica* **2016**, *33*, 42–50.
11. Marino, E. A unified approach for the design of high ductility steel frames with concentric braces in the framework of Eurocode 8. *Earthq. Eng. Struct.* **2014**, *43*, 97–118. [CrossRef]

12. Marino, E.M.; Nakashima, M. Seismic performance and new procedure for chevron braced frames. *Earthq. Eng. Struct.* **2006**, *35*, 422–452. [CrossRef]
13. Bosco, M.; Ghersi, A.; Marino, E.M.; Rossi, P.P. A capacity design procedure for columns of steel structures with diagonals braces. *Open Constr. Build. Technol. J.* **2014**, *8*, 196–207. [CrossRef]
14. Rossi, P.P.; Quaceci, G. A design procedure for suspended zipper-braced frames in the framework of Eurocode 8. *Earthq. Eng. Struct.* **2017**, *46*, 1963–1985. [CrossRef]
15. Longo, A.; Montuori, R.; Piluso, V. Plastic design of seismic Resistant V-Braced frames. *J. Earthq. Eng.* **2008**, *12*, 1246–1266. [CrossRef]
16. Longo, A.; Montuori, R.; Piluso, V. Seismic reliability of V-braced frames: Influence of design methodologies. *Earthq. Eng. Struct.* **2009**, *38*, 1587–1608. [CrossRef]
17. Costanzo, S.; D'Aniello, M.; Landolfo, R.; De Martino, A. Remarks on seismic design rules of EC8 for inverted-V CBFs. *Key Eng.* **2018**, *763*, 406–413. [CrossRef]
18. Costanzo, S.; D'Aniello, M.; Landolfo, R. Proposal of design rules for ductile X-CBFS in the framework of EUROCODE 8. *Earthq. Eng. Struct.* **2019**, *48*, 124–151. [CrossRef]
19. Costanzo, S.; D'Aniello, M.; Landolfo, R.; De Martino, A. Critical discussion on seismic design criteria for cross concentrically braced frames. *Ing. Sismica* **2018**, *35*, 23–26.
20. Landolfo, R. (Ed.) Assessment of EC8 provisions for seismic design of steel structures. In *Technical Committee 13—Seismic Design, No 131/2013*; ECCS—European Convention for Constructional Steelwork: Brussels, Belgium, 2013.
21. Elghazouli, A.Y. Assessment of European seismic design procedures for steel framed structures. *Bull. Earthq. Eng.* **2010**, *8*, 65–89. [CrossRef]
22. Barbagallo, F.; Bosco, M.; Ghersi, A.; Marino, E.M.; Rossi, P.P. Seismic assessment of steel MRFs by cyclic pushover analysis. *Open Constr. Build. Technol. J.* **2019**, *13*, 12–26. [CrossRef]
23. Tenchini, A.; D'Aniello, M.; Rebelo, C.; Landolfo, R.; da Silva, L.; Lima, L. High strength steel in chevron concentrically braced frames designed according to Eurocode 8. *Eng. Struct.* **2016**, *124*, 167–185. [CrossRef]
24. Elghazouli, A.Y. Seismic Design of Steel Structures to Eurocode 8. *J. Struct. Eng. ASCE* **2007**, *85*, 26–31.
25. EN 1993:1-1. *Eurocode 3: Design of Steel Structures—Part 1-1: General Rules and Rules for Buildings*; CEN: Brussels, Belgium, 2005.
26. Tenchini, A.; D'Aniello, M.; Rebelo, C.; Landolfo, R.; da Silva, L.; Lima, L. Seismic performance of dual-steel moment resisting frames. *J. Constr. Steel Res.* **2014**, *101*, 437–454. [CrossRef]
27. EN 1998-3. *Eurocode 8: Design of Structures for Earthquake Resistance—Part 3: Assessment and Retrofitting of Buildings*; CEN: Brussels, Belgium, 2005.
28. American Institute of Steel Construction, Inc. (AISC). *Seismic Provisions for Structural Steel Buildings*; ANSI/AISC Standard 341-16; AISC: Chicago, IL, USA, 2016.
29. Shen, J.; Wen, R.; Akbas, B.; Doran, B.; Uckan, E. Seismic demand on brace-intersected beams in two-story X-braced frames. *Eng. Struct.* **2014**, *76*, 295–312. [CrossRef]
30. Shen, J.; Wen, R.; Akbas, B. Mechanisms in two-story X-braced frames. *J. Constr. Steel Res.* **2015**, *106*, 258–277. [CrossRef]
31. Seismosoft. SeismoStruct—A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures. 2011. Available online: www.seismosoft.com (accessed on 6 May 2019).
32. Menegotto, M.; Pinto, P.E. Method of analysis for cyclically loaded R.C. plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending. In Proceedings of the Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well Defined Repeated Loads, International Association for Bridge and Structural Engineering, Zurich, Switzerland, 1973; Volume 11, pp. 15–22.
33. Uriz, P.; Filippou, F.C.; Mahin, S.A. Model for Cyclic Inelastic Buckling of Steel Braces. *J. Struct. Eng. ASCE* **2008**, *134*, 619–628. [CrossRef]
34. D'Aniello, M.; La Manna Ambrosino, G.; Portioli, F.; Landolfo, R. The influence of out-of-straightness imperfection in Physical-Theory models of bracing members on seismic performance assessment of concentric braced structures. *Struct. Des. Tall Spec.* **2015**, *24*, 176–197. [CrossRef]
35. Castro, J.; Davila-Arbona, F.; Elghazouli, A.Y. Seismic design approaches for panel zones in steel moment frames. *J. Earthq. Eng.* **2008**, *12*, 34–51. [CrossRef]

36. Lignos, D.G.; Krawinkler, H. Deterioration modelling of steel components in support of collapse prediction of steel moment frames under earthquake loading. *J. Struct. Eng. ASCE* **2011**, *137*, 1291–1302. [CrossRef]
37. Lignos, D.G.; Hikino, T.; Matsuoka, Y.; Nakashima, M. Collapse assessment of steel moment frames based on E-defense full-scale shake table collapse tests. *J. Struct. Eng.* **2012**, *139*, 120–132. [CrossRef]
38. Akkar, S.; Sandıkkaya, M.A.; Şenyurt, M. Reference database for seismic ground-motion in Europe (RESORCE). *Bull. Earthq. Eng.* **2014**, *12*, 311–339. [CrossRef]
39. Fulop, L. Selection of earthquake records for the parametric analysis. In *Research Report VTT-R-03238-10*; VTT: Espoo, Finland, 2010.
40. Vamvatsikos, D.; Cornell, C.A. Incremental dynamic analysis. *Earthq. Eng. Struct.* **2002**, *31*, 491–514. [CrossRef]