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Review

State of the Art on Seismic Design of Steel Buildings in Europe

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Abstract: The seismic design of steel systems recently experienced profound changes and progress; in Europe, the research on this topic is very prolific in terms of importance and number of results achieved, even pushed by the recent process of the updating of Eurocode 8. The paper analyzes and discusses the scientific literature on the subject produced approximately over the last twenty years, focusing on both recent research and findings on traditional steel systems and innovative structural types and solutions. The review of the state of the art suggested that most of the authors are now concerned about the numerous criticisms widely encountered in the design of traditional systems according to current Eurocode 8, as well as the difficulty of the application of the relevant detailing rules. The scientific community is also aware of the need to include specific codified design procedures for innovative and promising structural types. Further investigations are needed to deepen the design of moderate-ductile systems and to extend the seismic European prequalification of beam-to-column joints to further typologies.

Keywords: seismic design; steel; earthquake; Eurocode 8; moment-resisting frames; concentrically braced frames; eccentrically braced frames; buckling-restrained braces; lightweight steel frames; ductility; dissipative; beam-to-column joints.

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1. Introduction

The benefits of using steel as a structural material for seismic applications are largely proven by experience (direct, experimental, and numerical) and the wide employment of steel systems in countries with very high seismic risk (e.g., California, Japan). Indeed, besides its high strength and ductility properties, there are also several ecological and environmental advantages, e.g., the high level of industrialization of the production and construction processes, the ease of transport and assembly, and the possibility of completely recycling the material.

Over the last twenty years, the market of constructions and its relevant industries have been affected by profound changes, i.e., technological advances and scientific discoveries; in this framework, the field of seismic-resistant steel buildings has been one of the most fruitful in terms of both novelties and recent findings, covering both global (structural systems) and local (members, connections) scales.

Particularly in Europe, the amount and importance of the results and investigations achieved have also been motivated and pushed by the recent process of the updating of current Eurocode 8 [1]which is the standard ruling the design of structures for earthquake resistance. The development of the second generation of Eurocode 8 represents an unmissable opportunity to achieve a better understanding of the seismic behavior of steel buildings designed according to the current Eurocode 8, and is even essential to a more effective, innovative, and safer design of steel systems without a cost increase.

Considering these considerations, this paper aims at providing a comprehensive review of the state of the art and the research findings in the field of seismic design of steel buildings in Europe. With this regard, the paper analyzes and discusses the scientific literature on the subject produced approximately over the last twenty years, focusing on both recent research and findings on traditional earthquake-resistant steel systems and innovative structural types and solutions.

2. Seismic Design of Steel Buildings in Europe

Numerous authors [2–143] investigated the seismic performance of steel buildings, both deepening the study of the traditional types [2–94] (moment-resisting frames, concentrically and eccentrically braced frames) and proposing innovative solutions and new structural systems [95–138] (buckling restrained bracings, lightweight steel frames with cold-formed profiles).

At the current stage in the framework of Eurocode 8 [1] steel buildings can be designed according to either non-dissipative (concept a) or dissipative (concept b) behavior [141]. The former are designed to resist seismic events largely in the elastic range and their design is generally limited to low-seismic areas or structures of special use. On the contrary, concept (b) accounts for the capability of parts of the structure to dissipate seismic input energy by undergoing plastic deformation and it should be adopted in moderate and high seismic zones [141].

The structural safety of dissipative structures depends on the ductility they can provide against earthquakes; the ductility is the capacity of a structural system (or its subcomponents) to exhibit large plastic engagement without a severe reduction in its bearing capacity. According to concept (b) specific ductile zones of the systems are responsible for energy dissipation, while the plastic deformations in the remaining structural elements should be prevented to guarantee that global ductile plastic mechanism occurs. This methodology is known as "capacity design" [140,141].

Since most applications in seismic areas consist of dissipative systems, this article focuses attention on the literature concerning this design methodology.

It is trivial to observe that most of the European scientific literature on the topic focuses attention on the application of the seismic design rules codified within the current Eurocode 8 [5–11,13,19,22,27,43–45,48–62,76–78,80–83,87,89,91–94,138,139], as well as on the seismic performance of the structural systems compliant with it. Numerous critical issues have been identified by the researchers both concerning the design of traditional systems and the need to include new contents (innovative systems and structural types).

Experimental, numerical, and theoretical investigations for the main types of structural systems are discussed in the following.

2.1. Traditional Types

2.1.1. Moment-Resisting Frames

Steel moment-resisting frames (MRFs) are very popular in seismic areas [3–33], despite their higher constructional cost than braced frames, mainly due to their characteristics of higher ductility and architectural functionality.

MRFs resist the seismic force mainly in an essentially flexural manner, and the dissipative zones can be located at the beam ends and/or in the beam-to-column joints (strong column–weak beam mechanism) [1]; plastic hinges can even form in the columns at the base of the buildings and at the top of the roof level.

To assure global ductility, the columns should be designed to have sufficient overstrength (Figure 1), namely, to resist the most unfavorable condition of bending moment and axial force considering:

$$\begin{aligned} M_{Ed} &= M_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot M_{Ed,E} \\ N_{Ed} &= N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \end{aligned} \quad (1)$$

where $M_{Ed,G}$ and $N_{Ed,G}$ are the bending and axial forces respectively induced in the column due to the nonseismic actions included in the combination of actions for the seismic design situation; $M_{Ed,E}$ and $N_{Ed,E}$ are the bending and axial forces respectively due to the design seismic action; and γ_{ov} is the material randomness coefficient.

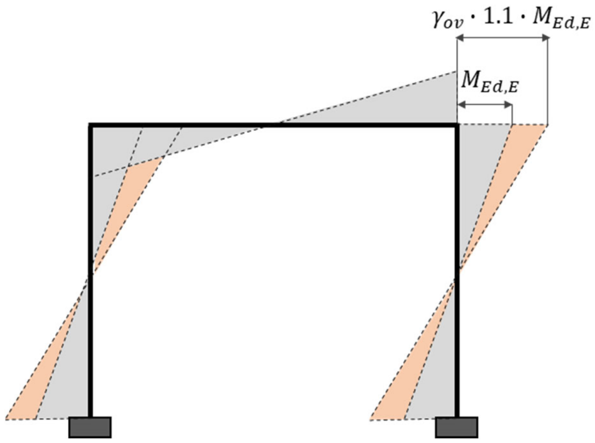

Figure 1. Moment-resisting frames: column capacity design criterion.

Concerning the criterion expressed by Equation (1), Elghazouli [5,6] observed that the beam overstrength ratio $\Omega = \min\left(\frac{M_{pl,b,Rd}}{M_{Ed,b}}\right)$, as adopted in EC8, does not account for the influence of gravity load ($M_{Ed,b}$ is the total demand seismic + gravity) and it could be not accurate, especially in case of gravity-dominated frames (e.g., large spans); the actual beam overstrength could be significantly larger (2 or 3 times) than one calculated according to EC8 when significant vertical loads are applied. Moreover, Equation 1 considers the minimum capacity-to-demand ratios within the beams of the frame, and it corresponds to the formation of the first plastic hinge; as a consequence, the redistribution capacity of the systems is not accounted for and the forces applied to the columns could be significantly larger than ones calculated according to Equation (1) [5,6].

However, the seismic design of MRFs compliant with Eurocode 8 is mainly ruled by the deformation-related requirements. The Code mandates two stiffness requirements for all buildings type, namely the control of second order (P-Δ) effects (associated with the ultimate limit state) and the control of interstory drift at the serviceability limit state. Several authors [5–11] recognized such requirements are too severe and they force the designer to oversize the structural members with respect to their relevant required strength, leading to massive structures characterized by large lateral ovestrength and poor energy dissipation capacity; an overstrength factor even larger than the behaviour factor assumed at the design stage can be recognized to fulfill the drift limitations and MRFs compliant to EC8 often elastically behave up to Near Collapse Limit State [8,11]. In detail, Elghazouli [5,6] highlighted that the drift-related requirements provided by EC8 are significantly more stringent in comparison with North American and other international provisions; according to [5,6] the too stringent drift and second-order effects rules, combined with the inherent sensitivity of moment-resisting frames to these effects, govern the sizing of structural members in the most of cases, leading to significant overstrength.

The influence of the P-Delta rule has even been deepened by [7,10,11]. These Authors agree that the rules stipulated by the European Code are more severe than those recommended by the US Code [142].

Both standards define a sensitivity coefficient θ, based on the application of the Horne method [143], to be controlled to assure the stability of the system against second-order effects. According to the Horne method, the critical buckling load of a plan multistory frame depends on its elastic stiffness. However, current EC8 relates the sensitivity coefficient to the secant stiffness of the frame to consider the stability condition of the system when a global mechanism is developed, resulting in very strict requirement, hard to be fulfilled; conversely, the ASCE 7 [142] in line with the elastic theory of the Horne's methods considers the elastic stiffness, resulting in less severe requirement, easier to be met. Tartaglia et al. [7,10] carried out a parametric numerical study to investigate the influence of the P-delta design rules on the seismic behaviour of ductile steel MRFs; they alternatively designed 48 structures according to EC8 [1] and US Codes [142,144], varying the number of stories, moment-resisting spans and seismic intensity. The results of non-linear static and dynamic analyses showed that the structures complaint with current Eurocode 8 massive and expensive, and they exhibit lateral overstrength larger than the corresponding frames designed according to US codes.

Tartaglia et al. [10] even proposed an alternative calculation of the critical multiplier and of the stability coefficient specifically, according to which the secant stiffness of the structure is increased by considering the material and the design overstrength sources.

Mazzolani and Piluso [2], Montuori et al. [3], Sepahvad et al. [4], Elghazouli [6], and Tartaglia et al. [7] proposed and/or validated alternative design criteria to sidestep the criticisms above discussed.

The theory of plastic mechanism control (TPCM) was proposed as an alternative design approach by Mazzolani and Piluso [2] for steel moment frames and was more recently deepened and applied to common different structural types by Mazzolani and Piluso [2], Montuori et al. [3,12,84–86,88], Sepahvad et al. [4], Longo et al. [41], and Giugliano et al. [42].

The TPCM is based on the kinematic plastic collapse theory by performing the second-order plastic analysis to assure a global collapse mechanism occurs.

Among the recent research achievements, the one that most significantly affects the seismic design of MRFs is the development of European seismic prequalification of beam-to-column steel joints, carried out within the European projects founded by the Research Fund for Coal and Steel (RCFS), Equaljoints and Equaljoints PLUS projects (research and dissemination type, respectively) [15,16].

Current seismic Code in Europe [1] allows locating dissipative zones both in the joints and/or at the beam ends. However, few and incomplete tools and information are provided to evaluate the theoretical response of beam-to-column joints under cyclic loading, in terms of strength, stiffness, and rotation capacity. In the framework of Eurocodes, the flexural response of beam-to-column joints is calculated according to the components method, provided within Eurocode 3 (namely the nonseismic standard dealing with the design of steel structures). Such methodology consists in identifying the sources of strength and deformability (namely the joint components) that are modeled by means of elastic springs and rigid links and combined in a proper mechanical model, giving the response of the joint in terms of strength and stiffness, but solely for the monotonic case [16,17].

The seismic European prequalification introduced thanks to the research developed within [15,16] consists of a set of standard rules, design and execution criteria, and technological requirements developed through comprehensive analytical, numerical, and experimental investigations [15–30] to push larger and simpler use of dissipative joints in moment-resisting frames.

Three types of bolted joints most used in steel frames (namely extended endplate joints, stiffened extended endplate joints, and haunched joints) as well as a dog-bone welded joints were prequalified with reference to different performance objectives and for different seismic resistant systems (MRFs, CBFs, EBFs, DUALFs). Prenormative design guidelines were developed providing per joint type: (i) description of the joint type, (ii) step-by-step standard design procedure, (iii) list of systems for which the joint is qualified, (iv) limit of applications, (v) technological requirements, and (vi) calculation examples [16,32,33].

Besides the local ductility of beam-to-column joints, the ultimate behaviour of steel beams significantly affects the overall seismic performance of moment-resisting frames. Steel dissipative beams must develop ductile behaviour with high rotation capacity. With this regard, current Eurocode 8 provides local slenderness limitations based on the cross-section classification according to Eurocode 3; the latter subdivides the steel profiles into four classes depending on the material and slenderness properties of their compression elements. Eurocode 8 relates the cross-section classes defined by Eurocode 3 to the ductility class of the system and the relevant behaviour factor.

Some authors made certain objections to such classification criteria, as they neglect the influence of several parameters affecting the rotation capacity, e.g., the flange–web-interaction, the type of loading (cyclic or monotonic), the moment gradient, and the global member slenderness, and they proposed different classification criteria [34–39]. The studies developed by [34–39] provide a systematic review of current criteria to classify the steel beams depending on their flexural behaviour, and they present the results of a set of experimental tests performed on specimens with different profiles (I, H, circular, square and rectangular hollow sections) and global slenderness under both cyclic and monotonic loading. The results indicated that the loading condition has a deep influence on the rotation capacity, thus highlighting the need to determine ductility requirements specific to the seismic case. In detail, D'Aniello et al. [34], based on experimental results on steel cantilevers as well as a review of experimental data available in literature provided empirical equations to predict the flexural overstrength and the rotation capacity in monotonic and cyclic conditions. The aspect ratio of the cross section, the ratio between flange area on total gross section area, and the length of plastic hinge were recognized as the most influential parameters for the rotation capacity, while the flange and web slenderness, the shear length, and the steel post-yield hardening properties are the most representative for the flexural overstrength. Further analytical studies have been also carried out to investigate the possibility of deriving predictive equations of both rotation capacity and flexural overstrength of both wide flange and hollow profiles by means of genetic algorithms and neural network [35–38]. However, the accuracy of these equations is also significantly affected by the quality of the input data.

Bosco et al. [39] recognized that current version of European seismic code provides limited rules and guidance to evaluate the ductility as well as the plastic rotation capacity of members with square hollow section (HSS). The research presented by [39] is aimed at developing analytical formulations to predict both deformation capacity and overstrength of cold formed HSS beams and columns, not requiring preliminary classification of members. The proposed equations are a function of the geometry of the cross-sections, the shear length, the half-wavelength of the buckled flange, and the axial load ratio. Finite element simulations confirmed the accuracy of proposed formulations within the ranges of variation considered in the study.

2.1.2. Concentrically Braced Frames

Concentric bracings are largely employed as seismic resisting systems in steel buildings in seismic areas. Indeed, even despite their relatively lower ductility compared to the MRFs, this type of system exhibits large lateral strength and stiffness even allowing easy fulfillment of deformation-related requirements (interstory drift limitations and second-order effects).

The seismic design of concentrically braced frames (CBFs) has been largely discussed within the scientific literature [40–63]; indeed, numerous researchers [40–45,48–60] recognized many inconsistencies and difficulties in the interpretation and application of the detailing rules currently codified within the Eurocode 8, as well as unsatisfactory seismic performance as highlighted by both experimental and numerical results.

Seismic design rules provided by the current European Code aim at restraining plastic deformations into diagonal members, which are responsible for energy dissipation, while beams and columns elastically behave. According to the current Eurocode 8 different behavior factors are assigned depending on the diagonal configurations, namely lower for V (VCBF) and chevron bracing (ΛCBF) than cross (XCBF) diagonal members. Indeed, under strong seismic action, large bending demand is imposed on the beam at the brace-intercepted section in the post-buckling range, leading to severe beam vertical deflection and thus limiting the elongation and the yielding of the brace under tension, while severe ductility demand is imposed to the brace under compression [46–53]. It is worth noting that differentiating the behavior factor depending on the bracings configuration is a prerogative of Eurocode 8 not shared by other seismic Codes [143,145]; moreover, recent studies [48,50,52,53,57,60] demonstrated that provided simple additional requirements are met, similar plastic engagement and seismic performance can be argued regardless of the bracings configuration and by assigning the same value of the behaviour factor.

Marino [44] even developed a unified approach for the seismic design of high ductile concentrically braced frames (whatever diagonal configuration is used). The method is based on design criteria previously developed by [40] for chevron bracings; a behaviour factor equal for X, V, and Λ configuration is assumed and the lateral resistance at each level is calculated assuming that the tension and compression bracings attain their full yielding and post-buckling strength, respectively.

The papers available in the scientific literature deepening the seismic performance of EC8-compliant CBFs mainly focus on the following aspects: (i) brace hysteretic behaviour, (ii) modeling aspects, (iii) capacity design criteria, and (iv) the role of the brace-intercepted beam in V (VCBF) and inverted V (ΛCBF) configurations.

Concerning the modeling aspects, it is worth noting that, for bracings arranged in X configuration, Eurocode 8 permits carrying out the required strength of diagonal members in a simple way, by tasking global elastic analysis using a tension-only (TO) model, in which solely the braces under tension are considered active while those under compression are omitted. The *ratio* of such a kind of model is based on the hypothesis that the compressed diagonal members experience a loss of strength and stiffness as they suffer buckling. Several researchers [58,59] observed this hypothesis can be deemed thorough for slender diagonal in the nonlinear range (i.e., after buckling); conversely, it is not accurate for stocky elements and especially at the beginning of the earthquake (in elastic range). In addition, the TO model may likely infer deceptive interpretation of structural behaviour, inducing the designer to neglect the diagonal-to-diagonal mutual restraint and thus to wrongly evaluate both in-plane and out-of-plane compression behaviour (diagonals result stockier than supposed). Current EC8 mandates the design of diagonal elements with a slenderness ratio ranging in [1.3–2.0]; the upper bound limit ($\overline{\lambda} \geq 2$) is conceived to control too-severe buckling at serviceability condition, while the minimum allowed slenderness ($\overline{\lambda} \leq 1.3$) is set to prevent overloading of columns and joints, transferred by the diagonals under compression (omitted in the model). It was widely noted [43,45,56,58,59] that the need to observe the lower bound limit implicates noteworthy efforts in sizing of bracings, often forcing the designer to increase the number of bays equipped with diagonals, thus leading to a cost increase due to the larger number of members and connections.

Costanzo et al. [58,59] specifically investigated this aspect and found that using a TC scheme in which both diagonal members are specifically accounted for both in tension and compression does not entail significant computational efforts; on the other hand, it allows for estimating the force-transfer mechanism induced by seismic action more effectively than TO model and removing the lower bound limit on normalized slenderness, thus simplifying the [59] sizing process of bracings (less iteration, less overstrength).

In this regard, it is worth noting that the current EC8 does not provide specific instruction for the calculation of brace slenderness in X configuration depending on the brace-to-brace connection type (continuous/discontinuous), while several authors [64–75] suggest that considering the actual brace-to-brace restraint to evaluate the buckling length (i.e., half-length of the diagonal for restrained braces) leads to more efficient structures.

Another aspect largely discussed within the scientific literature regards the evaluation of diagonal overstrength (namely capacity-to-demand ratio) and the relevant variation limits. To mitigate the tendency to form soft-story mechanisms and to favor uniform distribution of plastic deformation along the building height, the current standards mandate limiting the capacity-to-demand ratio $\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}}$ (being $N_{pl,br,Rd,i}$ the brace yield strength and $N_{Ed,br,i}$ the design forces due to the seismic actions) to the range $[\Omega, 1.25 \cdot \Omega]$ (being $\Omega = \min{(\Omega_i)}$).

Numerous authors [45,49–62] share the strong belief that this rule is not sufficient either to assure the plastic engagement of most of the dissipative members or to prevent soft-story mechanisms. The global slenderness upper bound limit ($\bar{\lambda} \leq 2$) and the requirements on the overstrength variation are very difficult to be contemporarily meet, being interrelated and counterposed; indeed, the brace at the roof level is generally characterized by the largest overstrength ratio, being the selection of the cross-section generally ruled by the slenderness limit; to meet the requirement on the overstrength homogeneity, the designer is then forced to oversize the element at the lower and intermediate storeys, leading to cost-ineffective and massive structural systems characterized by large lateral overstrength (Ω even significantly overcomes the unit) which exhibit poor plastic engagement and energy dissipation capacity [52–54].

Costanzo et al. specifically investigated this feature in [53] and they proposed and numerically validated the potential upgrading of the current rule; they found that using a compression-based approach in the definition of capacity-to-demand ratio (namely considering the buckling of brace under compression as the first nonlinear event rather than yielding of brace under tension) is more effective to control the activation of buckling along the building height and to assure uniform distribution of plasticity.

On the other hand, Bosco et al. [56] formulated for X-CBF the "Ω* method" which consists in relaxing the design rule given by EC8 to avoid oversizing of diagonals and non-feasible structural solutions.

The earthquake-induced effects in nondissipative components (namely beams, columns, and connections) are estimated by magnifying by the overstrength factor Ω the internal forces calculated using elastic analysis. Solely for beams in V and Λ configurations a plastic mechanism analysis accounting for the force-transfer mechanism peculiar of the nonlinear range is specifically considered to evaluate the seismic-induced effects on the brace-intercepted beam.

The design of the beam in V and Λ CBFs is a key aspect, and its flexural yielding causes significant deterioration of the overall force-displacement curve [46,47]. However, numerical results available in the literature [48,53,54] showed the capacity design rules currently codified in Eurocode 8 are not conservative in numerous cases.

The evaluation of buckling and post-buckling compression strength of diagonals is a key aspect in the seismic design of chevron concentrically braced frames, because it directly affects the design of the brace-intercepted beam.

Recently, Poursadrollah et al. [76] carried out experimental tests and numerical simulations aimed at investigating the buckling response of cold-formed square (SHS) and rectangular (RHS) hollow sections, widely used as diagonals in concentrically braced frames; results on material tests by [76] clearly shows that tested specimens do not experience adequate ductility to guarantee the development of a ductile plastic hinge, thus highlighting the need to further investigate the use of this type of profiles as dissipative parts in CBFs. Moreover, the results of buckling tests performed on 21 specimens with different slenderness ratios showed that the buckling curves recommended by current EN 1993-1-1 [146] for CFHS members have a nonuniform safety margin, leading to uneconomic design in the range of slenderness commonly used for diagonal members.

Moreover, D'Aniello et al. [48] found that besides the capacity design requirements, the flexural stiffness of the brace-intercepted beam should be controlled to guarantee satisfactory ductility and energy-dissipation capacity. Indeed, beam deflection and brace ductility demand are correlated phenomena (see Figure 2), and stiffer beams allow avoiding severe deterioration of strength and stiffness in the post-buckling range while the yielding of the diagonal under tension is favored [48,50,53,57].

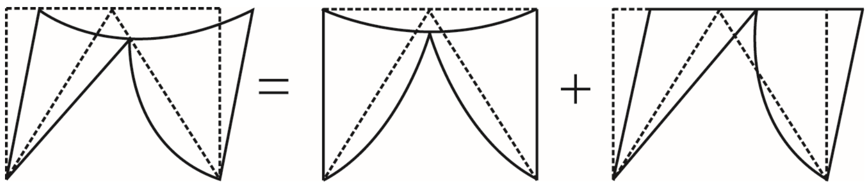

Figure 2. Concentrically braced frames: beam deflection and braces ductility demand are correlated phenomena [48].

The lack of specific provisions and technological requirements for diagonal-to-frame members connections constitutes one of the main impediments to a rational and efficient design of steel frames equipped with concentric braces. The capacity design rule currently provided by EC8 accounts only for the plastic axial strength transferred by the diagonal under tension while it totally neglects the need to arrange the brace buckling; in addition, the current codified rule does not consider the flexural capacity required to the connection in case of diagonals fixed at both ends [140]. The AISC 341-16 [144] supplies thorough and exhaustive detailing rules for gusset plate connections based on [147–149]. Consistent with the results and the recommendation developed by [147–149], single gusset plate connections can be properly designed to accommodate the buckling of compressed diagonals thanks to the formation of a weak axis (yield line) in the gusset plate itself, causing it to act as an equivalent pin connection.

2.1.3. Eccentrically Braced Frames

Eccentrically braced frames (EBF) combine the advantages of moment-resisting frames (i.e., high energy dissipation capacity under severe seismic ground motion) and concentrically braced frames (i.e., high lateral stiffness to avoid severe damage at serviceability limit state) [77]. According to Eurocode 8, EBFs are designed to dissipate seismic input energy by flexural or shear plastic deformation of the link, while any yielding and/or buckling should be prevented for the diagonals, columns, and beam segments outside the links.

Based on the type of plastic mechanisms, the seismic links can be classified as: (i) short links (dissipating by yield in shear), (ii) long links (dissipating by yield in bending), and (iii) intermediate links (which experience plastic deformations in both shear and bending). The length of the link "e" is the mechanical parameter influencing the type of plastic mechanism, and it is related to the plastic shear-to-plastic bending ratio of the link.

Several objections [77–94] have been made by the scientific community to the design procedure currently codified in Eurocode 8, and many authors investigated the nonlinear response of both EBFs and Dual-EBFs [80–83,85,88,93] even providing recommendations to improve their seismic performance.

Most of the authors' concerns are related to the evaluation of the link overstrength and the effectiveness of Eurocode 8-compliant design procedure for frames with intermediate and long links. Many researchers recognized that current codified rules are not efficient in avoiding the plastic engagement of non-dissipative members. Popov et al. [150] recommended assuring uniform distribution of link overstrength (calculated as the ratio link yield shear strength-to-design shear force) along the building height to assure that the yielding of links contemporarily occurs at all levels; however, according to current EC8, the link overstrength factor is defined considering the ultimate internal force as:

$$\Omega_i = 1.5\frac{V_{pl,\mathrm{i}}}{V_{Ed,\mathrm{i}}} \text{ for short links and} \tag{2}$$

$$\Omega_i = 1.5\frac{M_{pl,\mathrm{i}}}{M_{Ed,\mathrm{i}}} \text{ for long and intermediate links} \tag{3}$$

and it should be kept in the range $[\Omega, 1.25 \cdot \Omega]$, with $\Omega = \min(\Omega_i)$).

$V_{Ed,i}$ and $M_{Ed,i}$ are the design values of the shear force and the bending moment in link "*i*th" in the seismic design situation, while $V_{pl,i}$ and $M_{pl,i}$ are the shear and bending plastic design resistances of "*i*th" link.

It should be noted that the amount 1.5 V_{pl} and 1.5 M_{pl} are an approximate evaluation of ultimate link capacities; moreover, the overstrength factor defined by Equations (2)–(3) is discontinuous at specific link mechanical lengths due to a discontinuity in the ultimate shear force [83]. Elghazouli [6] even noticed that the influence of gravity loads is neglected.

Bosco et al. [83] proposed some modifications to the EC8 design rules. The modified design procedure is basically consistent with the currently codified one, but some upgrades are proposed concerning the evaluation of link overstrength and the behaviour factor assumed at design stage (that should be reduced in the opinion of [83]).

The repair and retrofit of frames equipped with eccentric bracings became a felt topic after the 2010 and 2011 New Zealand earthquakes. Replaceable horizontal links were specifically investigated by Stratan et al. [151,152] who proposed using bolted flush endplate connections to connect the link to the beam segments. More recently, the effectiveness of combining bolted detachable links with the recentering capacity of the systems within dual-eccentrically braced frames to reduce repair cost and efforts was deepened by [153–157]. The repair of a structure after earthquakes is often inhibited by the permanent residual drift. In [153-157 the bolted links are intended to be responsible for the energy dissipation capacity and they are designed to be easily replaceable, while the moment-resisting system is expected to elastically behave during the seismic event, thus providing the recentering capability. This strategy was also experimentally validated on a full-scale eccentrically braced frame tested within the DUAREM project [158]; pseudo-dynamic experimental tests with three levels of seismic intensity were carried out on a three-level three-to-one-bay full-scale dual-EBF with bolted replaceable links. The outcomes from the DUAREM project confirmed Dual-EBFs with dissipative removable links represent a very competitive lateral load-resisting system in terms of both seismic performance and sustainability with low costs of repair.

2.1.4. New Structural Types and Solutions

Besides the numerous works focused on the study of the seismic performance of traditional steel systems, some innovative solutions and systems are addressed within the scientific literature, deserving to be mentioned and considered both as significant achievements as well as a topic for future challenges.

Buckling restrained braces (Figure 3) have been relatively recently developed as a response to the strength and stiffness degradation typically observed in diagonal members subjected to repeated buckling under compression [95]. A buckling restrained brace can dissipate the seismic input energy both under tension and compression, thanks to the nonbuckling behaviour obtained by encasing a steel brace within an external case, made by a mortar-filled tube or by a wholly-steel buckling-inhibiting system [96,97], separated by a small void from the steel core.

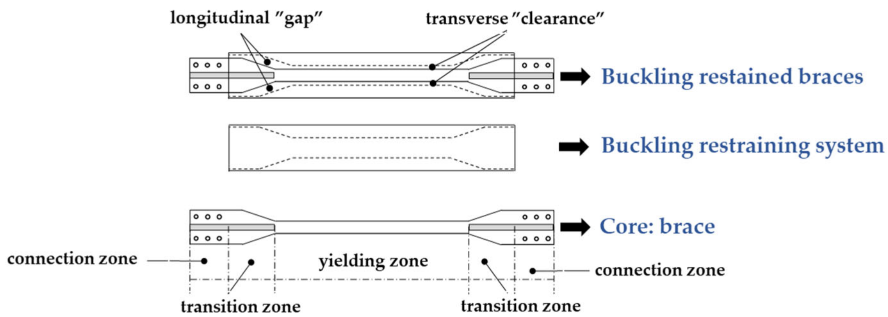

Figure 3. Buckling braced frames.

The earliest proposals to restrain the brace buckling can be recognized in Japan by [98–101]. Numerous theoretical, experimental, and numerical studies on BRBs can be recognized in the extra-European framework, deepening the behaviour of components, subassemblies, and global-scale systems, while few European researchers mainly focus on the use of buckling restrained braces to the seismic retrofit and upgrading of existing buildings [102,103]. Some foreign countries (Japan, USA, Canada, Taiwan) already have introduced the use of BRBs within their seismic design codes, while their use is not specifically addressed within the current Eurocode 8; a set of requirements regarding testing and manufacturing of BRBs is given by EN 15129 (2010) [104]. Thereby, despite their structural efficiency, widespread use of these systems is hampered by the lack of a specific codified design procedure and the consequent need for experimental qualification. With this regard Statan et al. and Zub et al. [105–107] carried out a comprehensive campaign of experimental tests and numerical simulations on a set of BRBs with capacities corresponding to typical steel multi-story buildings in Europe, specifically in view to develop a proper prequalification procedure.

Growing attention was put during the last decade on the so-called "Lightweight Steel Framed Systems" (LWSF) due to their inherent features of lightness, structural efficiency, economy, and sustainability [108,109]. The LWSF are low- to mid-rise buildings made of cold-formed steel profiles (CFS) sheathed to different kinds of panels (gypsum, wood, cement-based) to form the envelope of the building. LWSF can be employed as both bearing structures and nonstructural elements [110–112] such as drywall partitions, suspended ceilings, and façades.

A large amount of experimental and numerical investigations can be recognized within European scientific literature focusing on both the global behaviour [113–119] of LWSFs and their components (e.g., shear walls, members, connections, diagrams) [120–124]. The University of Naples Federico II has been particularly active on this issue in the last two decades. A huge amount of studies signed by the researchers of this institution are available within the scientific literature, including (i) experimental studies on shear walls sheathed with different panels [125–127] and strap-braced walls [129–131]; (ii) shaking table tests on whole structure [128]; (iii) experimental tests on different type of panel-to-members connections [131–134]; and (iv) theoretical and numerical studies [135–137].

It should be noted that at the current stage, Eurocode 8 does not specifically address the seismic design of LWSF; results from such extensive research [125–137] constituted a solid background to develop (and numerically validate) a proposal of codified seismic design rules and procedure [119,138] to be included in the next version of Eurocode 8 [159,160].

3. Conclusions and Future Directions

The seismic resilience of steel structures is largely proved by direct, experimental, and numerical experiences. During the last twenty years the seismic design of steel systems experienced profound change and progress, which made the standard currently in force in Europe incomplete and obsolete.

The current paper has presented a systematic review of the most recent and representative investigations on the seismic design of steel buildings in the European framework. In detail, the European scientific literature on the subject produced approximately over the last two decades has been considered and discussed.

Based on the examined studies, it can be clearly recognized that the research community is now concerned about the numerous criticisms widely encountered in the design of traditional systems according to the current Eurocode 8, as well as about the difficulty of application of the relevant detailing rules. The scientific community is also aware of the significant limitation of application of innovative systems and types which, despite very efficient and promising, cannot find wide employment in seismic areas due to the lack of specific standards.

The achievement and future needs in the field of seismic design of steel structures in Europe can be summarized as follows:

Moment-resisting frames: these systems are very popular, despite their higher constructional cost than braced frames, thanks to their characteristics of higher ductility and architectural functionality. The main concerns of researchers are basically focused on the effectiveness of the column capacity design criterion, as well as on the deformation-related requirements. The former is widely considered inaccurate, since it does not consider the redistribution capacity of the systems and because the beam overstrength ratio does not account for the influence of gravity loads; the latter is considered too stringent as compared with the corresponding rules provided by other seismic codes, thus ruling the sizing of structural members and leading to massive and overstrong systems with poor energy dissipation capacity.

Concentrically braced frames: they are largely employed as seismic resisting systems in steel buildings in seismic areas due to their large lateral strength and stiffness. The seismic design of CBFs according to Eurocode 8 is likely one of the most debated topics due to the large amount of inconsistencies and difficulties in interpretation and application of the detailing rules, leading to unsatisfactory seismic performance.

The papers available in the scientific literature deepening the seismic performance of EC8-compliant CBFs mainly focus on the following aspects: (i) brace hysteretic behaviour, (ii) modeling aspects, (iii) capacity design criteria, and (iv) the role of the brace-intercepted beam in V (VCBF) and inverted V (ΛCBF) configurations.

Eccentrically braced frames: these structures combine the high energy dissipation capacity of moment-resisting frames and the high lateral stiffness of concentrically braced frames. Within most of the examined papers, authors made objections to the evaluation of the link overstrength, and they recognized that current codified rules are not adequate to prevent yielding of nondissipative members. Replaceable horizontal links have been recently investigated and proposed as a competitive solution, even combined with recentering dual-eccentrically braced frames to reduce repair cost and efforts.

Innovative structural types: the most investigated new structural type are the buckling restrained braces and the lightweight steel frames (LWSF) made of cold formed profiles. The former have been experimentally and numerically investigated especially in extra-European frameworks, while few European researchers mainly focus on the use of buckling restrained braces for the seismic retrofitting and upgrading of existing buildings. Conversely, a very large amount of experimental, numerical, and analytical investigations on LWSFs are available within European scientific literature, focusing on both global and local behaviour. Seismic design rules and procedures have been developed and proposed in the framework of Eurocode 8.

Future directions*:* the scientific literature examined has underlined the need to upgrade the current Eurocode 8 both to improve the seismic efficiency of traditional systems as well as to include new contents. However, it is trivial to observe that the scientific community paid significant attention to the behaviour of ductile seismic-resisting steel systems, while the scientific literature is lacking regarding the seismic design and performance of steel structural systems designed for medium ductility class; the review of the state of the art has thus inferred the need to deepen this aspect by setting up simplified design rules specific for moderate ductility. Moreover, even though the seismic European prequalification of beam-to-column joints represents a very important innovation and a turning point for the design of steel structures in seismic applications, further joint types need to be prequalified. As well as this, steel–concrete composite beam-to-column connections deserve additional study.

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