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Review

Seismic Design of Bolted Connections in Steel Structures—A Critical Assessment of Practice and Research

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Abstract: The importance of connections in steel structures is paramount, not only because it greatly influences the cost of construction and provides room for innovations, but also due to the connections' impact on global structural behaviour. Therefore, research into innovative connections for seismic applications and related design criteria has significantly grown in recent years. However, it has been pursued mostly on local—connection or frame—levels, leaving the system analysis and code compliance levels with a meagre investigation. Moreover, less than 1% of published papers concerning steel connections and earthquake engineering are review articles. To overcome this gap, this systematic review of more than 240 references, including scientific contributions and design codes in the field aimed to cover both recent research and current shortcomings in practice and regulations. It has been found that European design rules updated to a fully performance-based design philosophy is imminent and is deemed to bring pre-qualified joints and increased complexity. Design rules have been systematized, and current hindrances have been highlighted. A deeper look into research needs and trends showed that investigations in connections for concentrically X braced frames are still a necessity, while developments in self-centring and replaceable connections as well as in simple solutions for increasing damping are expected to modify how joints are designed, as soon as semi-rigid and partial strength connections are more easily allowed by design codes.

Keywords: seismic design; bolted connections; joints; cover-plate; steel design; earthquake engineering; eurocode 8; structural engineering; structural design; review

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1. Introduction

Steel structures behaviour under seismic loading has long been regarded as very efficient [1] due to the material ductility, construction lightweight, and structural systems versatility [2,3]. Moreover, steel structures are expected to offer increasingly safe solutions, as innovations such as replaceable parts, self-centring systems, as well as easier supplemental damping, isolation devices, and economic loss control [4] have been unveiled and put into practice.

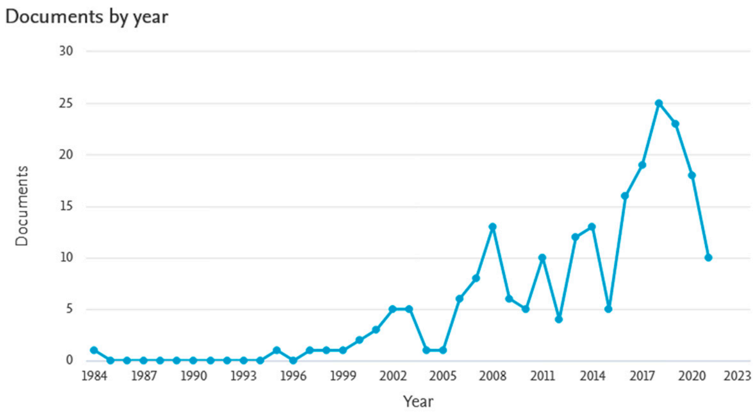
However, connections design plays a major role in structural detailing and fundamentally affects systems behaviour so that elements and connections design must occur simultaneously and iteratively. Such complexity is usually regarded as a hindrance for practitioners, especially under the encompassing legal framework imposed by the design standards. Nevertheless, it is also a critical research gap, as many developments and solutions, albeit promising, lack research on its applicability to system-level conditions and regulations framework.

With an eye on future developments in structural steel solutions for earthquake resistance, as much as on leveraging the employment of additively manufactured products for steel connections, this review is deemed to:

- Offer an overlook about seismic design philosophy, regulations, practice, and research problems;
- Systematically assess recent relevant research on seismic design of bolted connections;
- Critically analyse the state of current research, discuss how recent advances can be employed under current and forthcoming design standards and manufacturing capabilities, as well as assess prospective research needs.

The focus is set on bolted connections, considering both their extensive employment in steel structures designed to withstand seismic loading and their increased design complexity.

Performing a data analysis with the Scopus search tool (www.scopus.com) on 1 November 2021, it has been shown that the keyword string "*Seismic design*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" is associated with 215 documents. The counting started in 1984, but 213 out of those were published in 1997 or later (Figure 1a). The increasing trend in published documents annual output is evident, even if significant fluctuations are found.



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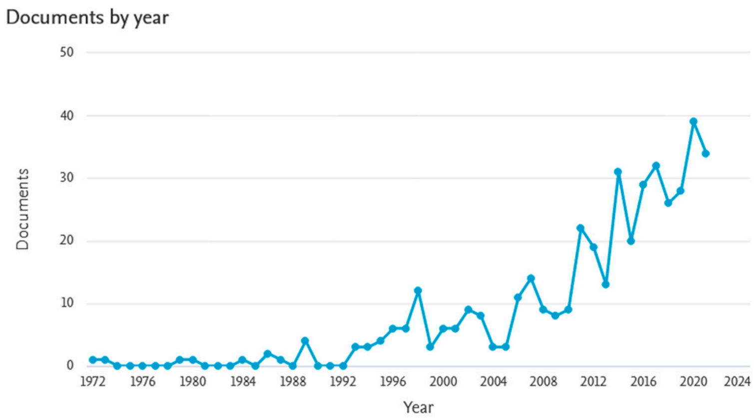

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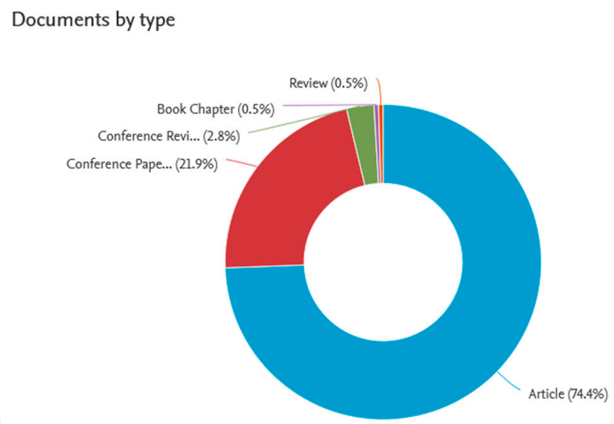
Figure 1. Scopus search data analysis for (**a**) "*Seismic design*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" and (**b**) "*Earthquake*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" in Title, Keywords or Abstract in Research, Review and Conference articles, on 1 November 2021.

Broadening the search into more general earthquake engineering allowed finding more works but that are less focused on connections design issues. Therefore, using the keyword string "*Earthquake*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted *connections*" OR "*Bolted joints*", one can find 428 documents, 416 of which were published in or after 1993, including some overlaps to the former search. Also, an increasing trend is found (Figure 1b), which demonstrates the lively and increasing interest in this research theme. Therefore, in Figure 1, images (a) and (b) differ in considering "*seismic design*" or "*earthquake*", respectively, to provide a comprehensive view of the research status.

Other keyword strings have been investigated, retrieving significantly fewer and partially repeated references. For instance, "*Eurocode 8*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" found no more than 12 documents.

The body-of-knowledge on this matter has been found to be constituted mainly by research articles in journals, accounting for almost 75% of the entries for the keyword string "*Seismic design*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" (Figure 1a) and over 68% of the literature for the keyword string "*Earthquake*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" (Figure 1b). Conference papers make up 22% to 28% of the published works, leaving only 0.5% to 09% for the review articles (Figure 2). The latter shows a meagre number of reviews for a consolidated scientific topic, highlighting the need for systematic review works. Assessing the difference between Figure 2a with a leading keyword of "*Seismic design*" and Figure 2b with "*Earthquake*" as the first keyword, one can observe that the former is associated with an increase in conference papers, at the expense of a similar decrease in journal articles.



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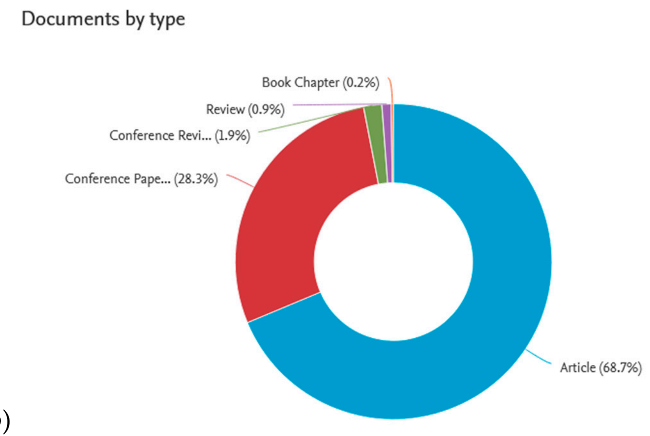


(**b**)

Figure 2. Document type distribution for (**a**) "*Seismic design*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" and (**b**) "*Earthquake*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*".

Assessing the most prolific authors list, the concentration of a significant share of research output in a few research groups is evident, suggesting that the topic is still a niche. Landolfo and D'Aniello are the most productive researchers in the field for both the aforementioned keyword strings (Figure 3). Comparing Figure 3a, where the "*Seismic design*" keyword is used in the string, with (b) where it was replaced by "*Earthquake*", we observe a difference in some of the enlisted authors, showing both regional lexical preferences and a different penchant for design applications or fundamental studies.

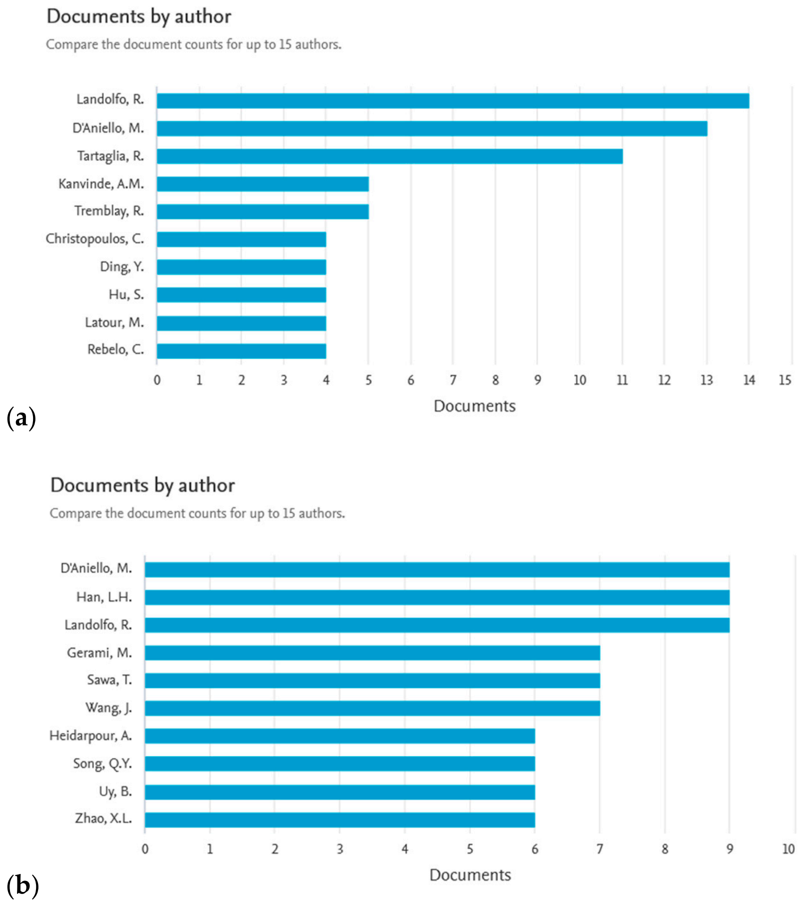
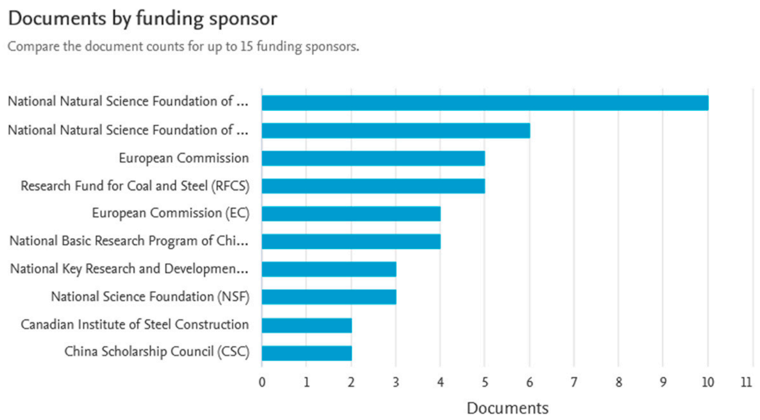


Figure 3. Documents by author for (**a**) "*Seismic design*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*" and (**b**) "*Earthquake*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*".

The National Natural Science Foundation of China is the primary funding agent for the research on seismic design of steel connections (Figure 4). This adds to several other Chinese funding agencies on the list to make China the leading investor in research within this topic. The European Commission and the European Research Funding for Coal and Steel also play an important role in funding research programmes in the field, followed by the U.S., Australian and Canada funders. Results can be regarded as consistent comparing Figure 4a with "*Seismic design*" and (b) with "*Earthquake*" in the keyword strings.



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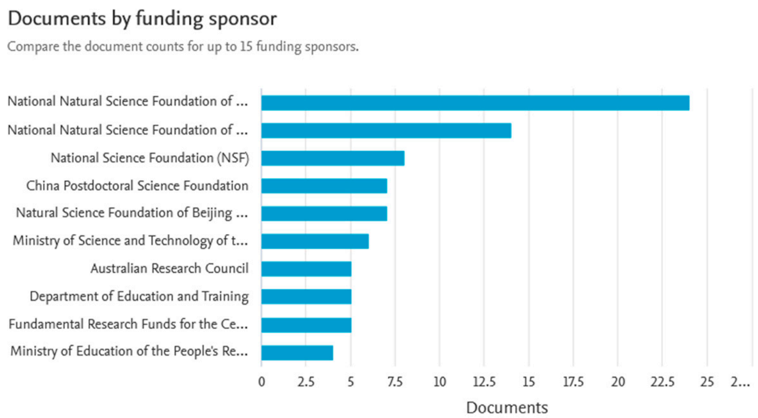

(**b**)

Figure 4. Funding sponsor for (**a**) *"Seismic design" AND "Steel joints" OR "Steel Connections" OR "Bolted connections" OR "Bolted joints"* and (**b**) "*Earthquake*" AND "*Steel joints*" OR "*Steel Connections*" OR "*Bolted connections*" OR "*Bolted joints*".

This work aims to fill the gap for a systematic review of very recent research on seismic design of steel bolted connections. To such an end, the 2016–2021 period has been set as the research focus, even if precursory relevant research could not be discarded to frame the field properly.

Moreover, an effort was made to bridge practice and research by investigating and systematising the current status of practice in code-compliant seismic design of bolted steel connections. While the European standards have been used to assist such an endeavour, it is hoped to unveil significant practical difficulties in current engineering practice, which demand answers from the research community.

The document structure includes a Methods section after this Introduction, followed by a section on Practical Seismic Design of Bolted Connections and a significant section on Recent Research within the same topic. Afterwards, Discussion and Conclusions sections are provided.

2. Methods

The literature research was conducted between February and March 2021, following the methodology thoroughly explained in [5]. Its steps include Identification, Screening, Sorting, Eligibility assessment, Information extraction, Qualitative synthesis, and Discussion stages, in a sequence illustrated in Figure 5.

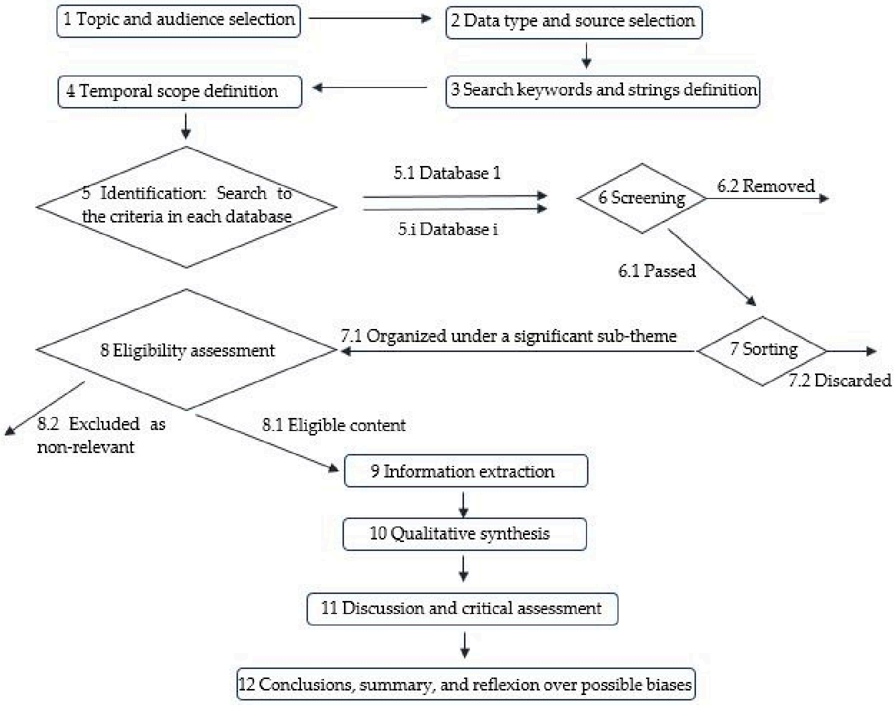

Figure 5. Search and systematic review methodology [5] reproduced under the Creative Commons Attribution 4.0 International License, (http://creativecommons.org/licenses/by/4.0/).

A choice has been made to consider all peer-reviewed items with value for the studied topic, including journal research articles, conference proceedings, review articles and peer-reviewed book chapters. Therefore, the use of technical books, which are not necessarily peer-reviewed, was limited to the Introduction and Section 3, where current design guidelines are alluded to.

The search for recent literature on the seismic design of bolted connections was set for the most recent five years (2016–2021). To such an end, journal articles and conference proceedings were redundantly searched in Mendeley Desktop and Scopus Online, search engines, even if literature finding through articles reading was a further important source. Table 1 quantifies the search dimensions.

Table 1. Search dimensions.

Stage	Included	Excluded
5 Identification	5.1 Mendeley (n = 26, of which 26 were eligible) 5.2 Scopus (n = 655, of which 214 were eligible) 5.3 References found in articles (n = 75, of which 43 were eligible)	
6 Screening	n = 270	n = 14
7 Sorting	n = 262	n = 8
8 Eligibility	n = 135	n = 127

A group of 10 keyword strings was equally used in all databases. Specifically, "*Seismic design*" AND "Steel connections", "*Seismic design*" AND "*Steel joints*", "*Seismic design*" AND "*Bolted connections*", "*Seismic design*" AND "*Bolted joints*", "*Seismic design*" AND "*Cover plates*", "*Seismic design*" AND "*Cover-plates*", "*Earthquake*" AND "*Steel connections*", "*Eurocode 8*" AND "*Steel connections*", "*Earthquake*" AND "*Steel joints*", and "*Eurocode 8*"

AND "*Steel joints*" were employed. These strings have been selected to match articles title, keywords or abstract where such option is explicitly available.

Further information on screening criteria can be found in [5].

3. Practical Seismic Design of Bolted Connections

3.1. Design Philosophy

Seismic design has added complexity and specificity to centuries-old structural engineering. Therefore, its most significant developments are mostly confined to the past 50 years. Its philosophy evolved from considering a lateral load, or pressure, with no relation to the construction properties, to considering a load only dependent on structural mass to consider structural dynamics up to incorporating modern principles of capacity design in advanced seismic design.

The first regulations answered seismic events with severe societal impacts and introduced the mass-related seismic load criteria. This included an early 20th-century Italian regulation, after the Messina–Reggio Calabria earthquake, for considering seismic loads valued between 8% and 12.5% of the mass over the floor [6]. Similar values, between 7.5% and 10% of the construction mass, were specified in the USA after 1925's Santa Barbara earthquake [6] and in Japan following 1923's Kanto earthquake [6].

This approach stood until the 1940s when design codes in the USA introduced structural flexibility as a parameter for the definition of seismic loading through static forces. Until the decade of seventy, this first generation of standards and recommendations for seismic loading and design had some developments but remained primarily dominated by elastic analyses and elastic response spectra with only residual accounting for material nonlinearity and ductility.

A second standards generation emerged in the 1970s, bearing innovative concepts such as ductility and energy dissipation. Following the work of Housner, Veletsos, and Newmark in the late 1950s ([7,8]), inelastic spectra were brought to seismic codes, introducing concepts such as energy balance, behaviour factor, amplification, and ductility requirements. Beyond collapse prevention, deformation and damage control became increasingly relevant in the subsequent seismic design practice.

Like most of its predecessors, codes of the third generation came as a result of seismic events with heavy economic and societal impacts on structures generally designed to the lawful requirements. In the mid-1990s, two extreme seismic events took place in the USA and Japan (Northridge and Hyogoken-Nambu, or Great Hanshin, earthquakes, respectively), imposing severe damage in structural systems designed to the current force-based design philosophy [6,9]. As a result, Performance-Based Seismic Design (PBSD) was introduced, first as a target philosophy in guidelines such as Vision 2000 [10] and after in design codes. PBSD allowed defining design objectives given the construction occupancy, importance, and cost of retrofitting and downtime. The focus was set on structural response and required performance at the global, system, and element levels, as well as in the associated uncertainty [6]. In parallel to the force-based design philosophy, an alternative direct displacement-based design philosophy was proposed [11] and incorporated in US standards such as FEMA 445 [12] and modern international seismic codes as clarified below.

At the same time, a significant development occurred in the materials prescriptive criteria, enabling the practical implementation of the capacity design (CD) principle, finding quantitative means to design for ductility.

Moreover, static and dynamic non-linear analyses emerged as paramount tools for the behavioural assessment of structures, even though its incompatibility with modal superposition and design spectra hindered broad employment as primary analysis and design tools.

Therefore, some researchers defined the current version of structural Eurocodes—the first generation, which succeeded the ENV pre-normative without significant changes of concepts—as a three-an-a-half design philosophy [6]. This is due to the fact that a full PBSD has yet to be implemented, design is still force-based, and analysis is generally elastic, employing design spectra. Displacement control is based on the equal displacement rule, suited for single degree of freedom (SDOF) systems with moderate-to-high fundamental periods but limited for the broader employment required by a comprehensive standard directed to multiple degrees of freedom (MDOF) systems. Displacement-based design (DBD) of structures [11], while well-known by the scientific community, is still negligible in current practice.

The seismic design philosophy of the fourth generation, on the other hand, envisages full employment of performance-based criteria. It includes using explicit performance estimates and thresholds for repair and retrofitting costs, life-cycle costs, casualties and downtime. It seeks a realistic assessment of the inelastic structural behaviour under seismic actions, using adequate parameters such as displacements, and adequately deals with uncertainty.

Currently, fourth-generation seismic design codes can be found in American practice, starting in 2006 [6] with FEMA 445, FEMA 695, and FEMA 750. Furthermore, the new Eurocode 8 generation, scheduled for release in the near future, is expected to endeavour into a fourth-generation seismic design approach.

While it has been accepted that steel structures showed a consistently good behaviour under seismic events through the last century [1], the literature pertaining to seismic design philosophy for structural steel, and especially for steel connections, is much scarcer compared with concrete structures [13] and detailing of these, for example.

Nevertheless, Teal's work [14] offered an excellent background for the seismic design philosophy of steel structures behind the 1970s American codes. However, it was limited to Californian regulations.

Concerning steel connections, discussion on design philosophy was not profuse before the Northridge event, being limited to lessons learnt with cyclic loading tests [15] and some seminal works, such as Popov et al. [16]. However, in the aftermath of many collapses due to inadequate seismic behaviour of steel connections, an exhaustive discussion was sparked post-Northridge earthquake. Popov et al. [17,18], Chen et al. [19], and Faridmehr et al. [20] provided a good understanding of the discussion.

Over the last 20 years, many discussions about the seismic design philosophy of steel structures drove practice and research. Transversally, elements and its joints have been discussed together, as well as bearing systems and global constructions in many cases. For instance, Roeder analysed connections design philosophy within moment-resisting frames systems [21], Tremblay [22], Brandonisio et al. [23], Naqash [24] and Nip et al. [25] discussed the behaviour of braced frames and Bruneau et al. [26], Gioncu and Mazzolani [27] addressed the principles of ductile design for seismically loaded steel systems.

Looking ahead for future trends in seismic design of steel structures, one cannot disregard the fact that several technically excellent solutions, based on sound research, ended up not having a practical acceptance as broad as once thought, primarily due to economic efficiency, as well as site and structural specificity. That might have been the case of structural isolation and supplementary damping [6,28,29].

Therefore, assessing seismic behaviour more accurately and better capturing the true seismic response characteristics for different return periods of the earthquake excitation is regarded as a future trend for seismic design philosophy. Such an endeavour can be envisaged by pursuing fragility-based design, explicitly dealing with fragility or vulnerability functions [30,31]. The former quantifies the probability of exceeding a performance level as a function of seismic action intensity, while the latter quantifies an expected loss due to the seismic action intensity. Relevant work on fragility functions for steel systems with frail connections can be found in Ramirez et al. [32] work, impacting future design directions.

Innovative methods based on seismic reliability, such as [33,34], are expected to assist in the development of new concepts and solutions.

3.2. Current Standards

Structural Eurocodes comprise a group of 10 standards and fifty-eight parts, encompassing almost all significant issues in the design of civil engineering structures. Despite being issued by the European Committee for Standardisation (CEN) and, therefore, having the strength of law in the European Union (EU) countries, these standards are also adopted in European Free Trade Association (EFTA) states, and are becoming increasingly important for structural design in many other locations. The latter include Russia, Turkey, the United Kingdom, Balkans states, Ukraine, Belarus, Kazakhstan, Angola, Kenya, among others, in the progress of adoption, as well as countries that expressed interest in the Eurocodes such as India and China, according to EU's Joint Research Centre webpage as of May 2021.

Eurocodes release as EN standards succeeded the precursory ENV pre-standards, evolving important aspects [35], and was completed in 2007. Within the history of seismic design codes, Eurocode 8 has been defined as in-between the third and fourth code generations, in the terms expressed in the previous section [6]. The hands-on work towards the second Eurocodes package was initiated in 2015, amid concerns that its developments may hinder practitioners' readiness to employ it [36], and is expected to yield results between 2021 and 2023 [37]. Figure 6 synthesises a timeline with design philosophy generations and main design standards.

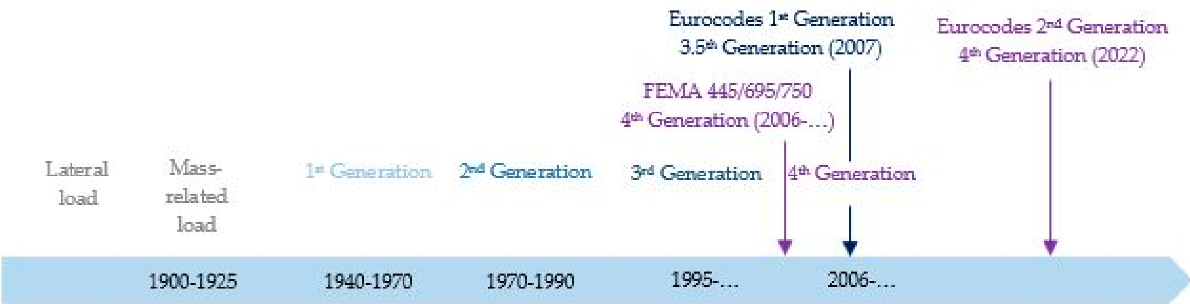

Figure 6. Design philosophy generations and main standards timeline.

Regarding the seismic design of steel connections, parts 1–1 [38] and 1–8 [39] of EN1993, and part 1 of EN1998 [40] govern the practice under the Eurocodes framework, even if various other parts are necessary to take into account.

Another widely used set of structural standards can be found in the North American framework. U.S. standards with an embracing scope and application to seismic design of steel structures include a plethora of documents, either with a global reach or a local jurisdiction, usually updated with a frequency not matched by structural design codes in other countries or regions [2].

The International Conference of Building Officials published the Uniform Building Code (UBC) until 1997, while the Building Officials and Code Administrators released the National Building Code (NBC) until 1999, and the Southern Building Code Congress International published the Standard Building Code (SBC) until 1999. These three organizations merged as the International Code Council in 1994 and, after a significant evolution to account for the Northridge earthquake lessons [41], emitted the International Building Code (IBC), with a triennial update frequency, currently with a 2021 edition. Competing standards include National Fire Prevention Association's NFPA 5000, currently in the 2021 edition.

However, these comprehensive codes hardly waive the use of load and material-specific standards. That is the case of the American Society of Civil Engineers' ASCE/SEI 7 for minimum loading with the latest edition of 2016, American Institute of Steel Construction, and American National Standards Institute's ANSI/AISC 360 2016's Specification for Structural Steel Buildings, as well as seismic design provisions. Regarding the latter, beyond the Los Angeles Tall Buildings Structural Design Council recommendations, ANSI/AISC

341 of 2016 provides design provisions for structural steel buildings and Federal Emergency Management Agency's National Earthquake Hazards Reduction Program regulations offer state-of-the-art seismic design criteria based on extensive research on steel connections brittle failure following the 1994 Northridge earthquake [41,42] as depicted in the 1997 Connection Test Summaries (FEMA 289) and in the 2000 state of the art report on connection performance (FEMA 355D), Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings (FEMA-350) and Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings (FEMA-351). Moreover, the American National Standards Institute and American Institute of Steel Construction's standard for prequalified connections for seismic applications—ANSI/AISC 358-16 [43]—has become paramount in assisting designers.

FEMA/NEHRP codes with significance for the seismic design of steel structures also include seismic provisions for new buildings: the 2003 FEMA/NEHRP 450, 2009's FEMA 750, and 2020's FEMA/NEHRP P-2082 as well as Performance-Based Seismic Design Guidelines for new and existing buildings in the 2006 FEMA 445 and in the 2009 FEMA 695.

North-American standards employment outreaches the U.S. territory and influences most of Central and Southern American practice, either as officially recommended practice or in addition to local codes.

Mostly constricted to national practice, but with modern and useful approaches to seismic design of steel structures, Japanese Building Standard Law (BSL), issued by the Building Center of Japan, was revised in 2000 to adopt performance-based seismic design [2,44–46] as an answer to the Hyogoken-Nambu Earthquake. Recently, in the aftermath of the Nigata Chuetsu and Great East Japan earthquakes, further amendments have been introduced to BSL, ensuring a state-of-the-art and experience-based regulation [47].

New Zealand's NZS 1170 code, issued in 2004 [2], and the Caribbean Model Code [48] should also be noted.

3.3. Design Rules and Procedures

Considering the formerly depicted international coverage and widespread use of Eurocodes, special attention shall be paid to this framework. Table 2 and Figure 7 summarize the steps of practical seismic design of bolted steel connections according to such standards (nomenclature can be found in this article's final section).

A first distinction is made between dissipative and non-dissipative connections. In fact, while joints in non-dissipative members, including columns, beams in concentrically braced frames, and braces in eccentrically braced frames, shall not bear dissipative connections, dissipative members could have dissipative or non-dissipative connections.

The other two fundamental differences lie in the type of internal forces to be resisted by the connection, either predominately axially loaded members (Figure 8) or members in bending (Figure 9), and in the solution type—with cover plates or endplates. In Figure 8 the connection components are, from left to right, gousset plate, two web cover-plates and the brace; a brace segment, two web cover-plates, two or four flange cover-plates and the brace second segment; a brace segment, two end-plates, two stiffeners and the brace second segment. In Figure 9 both arrangements have columns with web reinforcement plates and welded plates in-between flanges. The critical difference is, therefore, having end-plates connecting columns and beams in the left arrangement and web and flange cover-plates connecting beam segments—which are then welded to the columns—in the right arrangement.

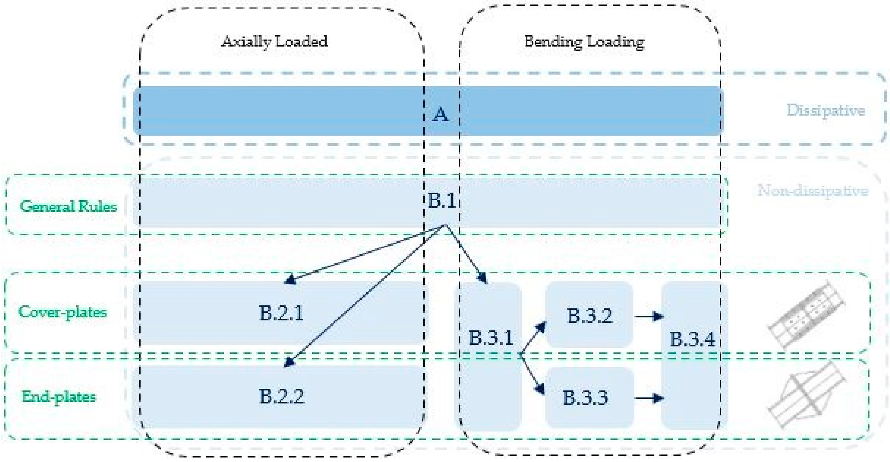

Figure 7. Steps of practical seismic design of bolted steel connections according to Eurocodes.

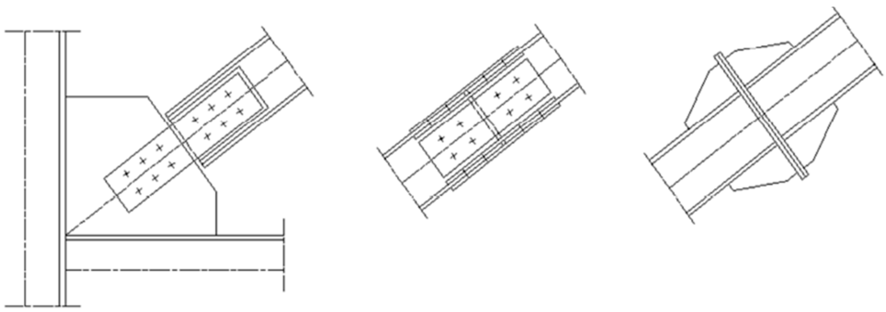

Figure 8. Connections in (predominantly) axially loaded members. From left to right: Web cover plates; Web and flanges cover plates; endplates.

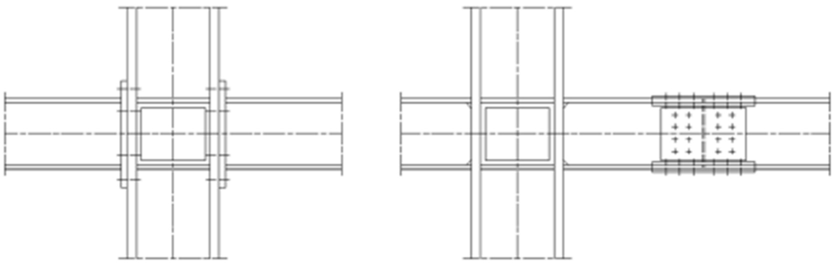

Figure 9. Connections in (predominantly) bent members. From left to right: end plates; web and flanges cover plates.

Table 2. Steps of practical seismic design of bolted steel connections according to Eurocodes.

Clauses [(1)]	Step	References
	A. Dissipative connections	
6.5.2 (3), 6.5.5 (6), and 6.5.5 (7)	**A.1** Connections' design should be either supported by experimental testing under cyclic loading or based on existing data (which exactly matches the designed connections).	[2,40,49]
6.6.4 (2) and 6.7.3 (9)	**A.2** Global structural analysis shall be non-linear (pushover or time history). This leads to waiving the response spectra approach or using both (response spectra and non-linear) approaches. Since national annexes usually only provide site-specific spectra, this is usually an issue.	[2,40,49]
	B. Non-dissipative connections	
	B.1 General rules	
6.2 (3), 6.5.2 (4) and 6.5.5 (3)	**B.1.1** Connection resistance R_d, as computed to EN 1993-1-8, shall account for an overstrength requirement: $R_d \geq 1.1\ \gamma_{ov}\ R_{fy}$ Where γ_{ov} is the overstrength factor, recommended as 1.25 but subjected to national specification R_{fy} is the plastic resistance of the connected member Hence, $R_d \geq 1.375\ R_{fy}$	[2,40,49]
6.5.5 (5)	**B.1.2** Bolts shear resistance shall be 20% higher than its plate bearing resistance. Using EN 1993-1-8 nomenclature: $F_{v,Rd} \geq 1.2\ F_{b,Rd}$ Where $F_{v,Rd}$ is the design shear resistance $F_{b,Rd}$ is the design bearing resistance (This leads to $F_{v,Rd} \geq 1.65\ R_{fy}$, whereas the design of bolts in shear out of the EN 1998 framework is usually shaped by the inequality $F_{v,Rd} \geq F_{v,Ed}$, with the design shear force $F_{v,Ed}, \leq R_{fy}$)	[2,39,40,49]
	Conditions **B.1.1** and **B.1.2** apply to the design of bolts to shear ($F_{v,Rd}$), tension ($F_{t,Rd}$) and its interaction as well as the design of plates and members' parts (flanges, webs, etc.) to bearing ($F_{b,Rd}$), punching ($B_{p,Rd}$), block tearing ($V_{eff,1,Rd}$), shear tearing ($V_{eff,2,Rd}$), gross cross-sectional tension ($N_{pl,Rd}$), net cross-sectional tension at the perforated sections ($N_{u,Rd}$) and local instability in parts subjected to compression or shear.	
6.2 (9) and 6.5.5 (4)	**B.1.3** Only categories B, C and E preloaded connections are allowed. This limits bolts classes to 8.8 and 10.9 and excludes non-preloaded connections. For categories B and C, slip-resistance is defined in EN 1993-1-8 to serviceability and ultimate, respectively. However, these safety checking conditions are defined to service shear force ($F_{v,Ed.ser}$) and design shear force ($F_{v,Ed}$). Thus, slip-resistance must be checked to service load combinations or ultimate (including seismic) load combinations and not to connected parts resistance, with overstrength, as in EN 1998-1 clauses. Surfaces' friction classes should be A or B, in compliance with EN1990-2. Thus, friction coefficient to EN1990-2's annex G shall not be less than 0.40.	[2,39,40,49,50]
6.5.5 (6)	**B.1.4** Even non-dissipative connections shall be experimentally validated if located in dissipative zones or zones adjacent to dissipative ones.	[2,40,49]

Table 2. *Cont.*

Clauses (1)	Step	References
	B.2 Rules for axially loaded connections	
	B.2.1 Connections with cover-plates with in-plane loading (tension, compression or shear)	
6.5.4 (1) and EN 1993-1-1 6.2.3 (3)	**B.2.1.1** The net cross-sectional tension resistance at the perforated sections ($N_{u,Rd}$) shall exceed the gross cross-sectional tension resistance ($N_{pl,Rd}$), therefore: $N_{u,Rd} > N_{pl,Rd}$ Hence, $0.9\ A_{net}\ f_u/\gamma_{M2} > A\ f_y/\gamma_{M0}$ With: A_{net} the net cross-sectional area f_u ultimate stress f_y yield stress γ_{M0} is a safety partial factor, recommended as 1.00 but subjected to national specification γ_{M2} is a safety partial factor, recommended as 1.25 but subjected to national specification Therefore, $0.72\ A_{net}\ f_u > A\ f_y$, or $A_{net}/A > f_y/(0.72\ f_u)$	[2,38,40,49,51–53]
	B.2.1.2 The relation between flanges and webs' cover-plates sectional area should be kept proportional to the relation between flanges and webs sectional area. Moreover, cover-plates should not have very disproportionate sectional areas when connecting the same part (flange, web, etc.)	[2]
EC3-1-8 Table 3.4	**B.2.1.3** Bearing resistance per bolt, $F_{b,Rd}$, shall be computed according to EC3-1-8. This capacity should comply with the condition defined in B.1.1, which is much more demanding than simply assuring that resistance exceeds the bearing capacity, as defined in EC3-1-8. As defined in the standard $F_{b,Rd} = k_1\ a_b\ f_u\ d\ t/\gamma_{M2}$ with the design factors k_1 and a_b, f_u the plate ultimate stress, d the hole diameter and t the plate thickness. Oversized and slotted holes may be used, at the cost of a capacity reduction.	[39,54,55]
EC3-1-8 Table 3.4, EC3-1-8 3.6.1 (12), EC3-1-8 3.8 (1)	**B.2.1.4** Shear resistance per bolt, $F_{v,Rd}$, shall be computed according to EC3-1-8. Conditions in B.1.1 and B.1.2 shall be accounted for and determine the required shear resistance. As defined in the standard $F_{v,Rd} = \alpha_v\ f_{ub}\ A/\gamma_{M2}$ with the design factor α_v, f_{ub} the bolt ultimate stress and A the bolt net area. The following capacity reductions apply: 1. If packing plates with a thickness (t_p) greater than one third of the bolt diameter (d) are used, the reduction factor $\beta_p = 9d/(8d + 3\ t_p)$ shall be used. Packing plates are usually employed only for geometry constrains and tend to be thin. However, given the need for controlling $F_{b,Rd}$ under the Eurocode 8 framework, as expressed in B.1.2, it has been regarded as a possible practical solution. Hence, this capacity reduction is especially relevant. 2. Long joints—those whose length between extreme holes centres, L_j, exceeds 15d—shall have its shear resistance multiplied by $\beta_{Lf} = 1 - (L_j - 15d)/(200d)$, but $0.75 \leq \beta_{Lf} \leq 1.00$.	[39,54,55]

Table 2. *Cont.*

Clauses (1)	Step	References
EC3-1-8 Table 3.4, EC3-1-8 Table 3.7, EC3-1-8 3.9.1 (1), EC3-1-8 3.9.1 (2)	**B.2.1.5** As expressed in B.1.3 service shear force, $F_{v,Ed.ser}$, and design shear force, $F_{v,Ed}$, shall comply with the inequality $F_{v,Ed,Ser} < F_{s,RD,ser}$ for Class B connections and $F_{v,Ed} < F_{s,Rd}$ for class C connections. For this particular case, internal forces can be attained from serviceability and design load combinations, according to the designers' choice for class B or C connections. Resistance formulae can be found in EC3-1-8, as: $F_{s,Rd,ser} = k_s\ n\ \mu\ 0.7\ f_{ub}\ A_s / \gamma_{M3,ser}$ $F_{s,Rd} = k_s\ n\ \mu\ 0.7\ f_{ub}\ A_s / \gamma_{M3}$ Where k_s is a design factor of 1.00 for normal holes, μ is the slip factor, n is the number of friction surfaces, A_s is the bolt area, and γ_{M3} and $\gamma_{M3,ser}$ are safety factors. The latter must be nationally defined, but recommended as 1.25 and 1.10, respectively.	[39,54,55]
EC3-1-8 3.10.2 (2), EC3-1-8 3.10.2 (3),	**B.2.1.6** Block tearing, $V_{eff,1,Rd}$, and shear tearing, $V_{eff,2,Rd}$, resistances should be assessed according to EC3-1-8 formulae and compared with the plates required resistance according to B.1.1. The following expression should be used: $V_{eff,1,Rd} = f_u\ A_{nt} / \gamma_{M2} + (1/\sqrt{3})\ f_y\ A_{nv} / \gamma_{M0}$ $V_{eff,2,Rd} = 0.5\ f_u\ A_{nt} / \gamma_{M2} + (1/\sqrt{3})\ f_y\ A_{nv} / \gamma_{M0}$ With A_{nt} the net area under tension and A_{nv} the net area under shear forces.	[39,54,55]
EC3-1-8 Table 3.3, EC3-1-1 6.3.1 and EC3-1-5 5	**B.2.1.7** Plates under compression and shearing stresses should be analysed for its stability. EC3-1-8 specifies prescriptive measures to avoid local buckling. Those can be found in Table 3.3 maximum distances for bolted plates and slenderness relations for avoiding local buckling. For slender parts and plates, and especially for gousset plates in connections for axially loaded elements, buckling shall be computed to EC3-1-1 and EC3-1-5 formulae. However, the case of gousset plates frequently requires more than following the Eurocodes. For such an end, the modified Thornton method can be employed, and its application assisted by [56–61].	[38,39,53–62]
6.5.5 (2), EC3-1-8 4.5, EC3-1-8 4.7, EC3-1-8 4.10, EC3-1-8 4.11 and EC3-1-8 4.12	**B.2.1.8** Despite the strength requirements systematized in B.1.1, Eurocode 8 recognizes full penetration butt welds as sufficient for fulfilling the overstrength criteria. For the remaining cases, EC3-1-8 rules should be followed for welds design. Welds might not be needed for the most straightforward cover-plates solutions but will be needed if members or plates reinforcement is needed, even in bolted solutions.	[39,40,54,55]
	B.2.2 Connections with end-plates with out-of-plane loading (shear and bending)	
EC3-1-8 Table 6.1, EC3-1-8 6.2.4, EC3-1-8 6.2.5, EC3-1-8 6.2.6, EC3-1-8 6.2.7	**B.2.2.1** End plate design shall comply with resistance criteria depicted in B.1.1. Yet, its computation shall encompass the verification of each individual resistance component defined in the EC3-1-8 components method (Table 6.1). The following conditions—B.2.2.2 to B.2.2.6—add to those mentioned herein, for comprehensive formulae.	[39,54,55,63,64]
EC3-1-8 Table 3.4	**B.2.2.2** Bolts tension resistance, $F_{t,Rd}$, should be assessed according to the following EC3-1-8 expression: $F_{t,Rd} = k_2\ f_{ub}\ A_s / \gamma_{M2}$ with $k_2 = 0.9$, except for countersunk bolts, for which $k_2 = 0.63$ applies.	[39,54,55]
EC3-1-8 Table 3.4	**B.2.2.3** Punching shear resistance, $B_{p,Rd}$, is also to be determined according to EC3-1-8 and compared with the tension force at each bolt, accounting for the Eurocode 8 resistance requirements. The following expression must be used: $B_{p,Rd} = 0.6\ \pi\ d_m\ t_p\ f_u / \gamma_{M2}$, where d_m is the smaller value among bolt head and nut average points and flats diameters.	[39,54,55]

Table 2. *Cont.*

Clauses [(1)]	Step	References
	B.2.2.4 Similar to B.2.1.4	
EC3-1-8 Table 3.4, EC3-1-8 3.9.2 (1)	**B.2.2.5** Tension–shear interaction should be assessed both for ultimate and slippage conditions. The former can be evaluated with the assistance of the following equation: $F_{v,Ed}/F_{v,Rd} + F_{t,Ed}/(1.4\ F_{t,Rd}) \leq 1$ Concerning slippage subjected to tension forces, $F_{s,Rd,ser}$ and $F_{s,Rd}$ can be re-written in the following form: $F_{s,Rd,ser} = k_s\ n\ \mu\ (0.7 f_{ub}\ A_s - 0.8\ F_{t,Ed,ser})/\gamma_{M3,ser}$ for class B connections $F_{s,Rd} = k_s\ n\ \mu\ (0.7 f_{ub}\ A_s - 0.8\ F_{t,Ed})/\gamma_{M3}$ for class C connections	[39,54,55]
	B.2.2.6 Similar to B.2.1.8	
	B.3 Rules for connections subjected to bending	
	B.3.1 General rules for connections in beams	
6.6.4 (3), 6.6.4 (4) and 6.6.4 (5)	**B.3.1.1** The influence of connections behaviour upon the beam must be controled so that beams plastic rotation, θ_p, shall not be less than 0.035 radians for DCH structures or 0.025 radians for DCM structures. Rotation capacity is defined as $\theta_p = 2\ \delta/L$, with δ the beams' mid-span deflection and L its span. Moreover, the deflection term shall be computed in such a way that columns elastic deformation contribution is not taken into account, columns web panel shear deformation impact upon the deflection value does not exceed 30% of the total deflection and stiffness degradation due to cyclic loading-induced damage does not exceed 20% of the computed deflection.	[40,49]
	B.3.2 Connections with cover-plates with in-plane loading (tension, compression or shear)	
	The same as B2.1	
	B.3.3 Connections with end-plates with out-of-plane loading (shear and bending)	
	The same as B2.2	
	B.3.4 Columns' web panels	
6.6.3 (6), 6.6.3 (7) EN1993-1-8 and EC3-1-5 5	**B.3.4.1** In each column-beam node, columns' web panel shear resistance $V_{wp,Rd}$ (as defined in EN1993-1-8), as well as its buckling resistance $V_{wb,Rd}$ (as defined in EN1993-1-5) shall exceed the design shear force in the web panel, $V_{wp,Ed}$. The latter shall be defined in compliance with the adjoining beam or connection plastic bending capacity but need not to account axial and bending stresses in the web panel. This condition is frequently impossible to meet without reinforcing the web panel. Beyond a deeper discussion in [65], Figure 10 shows some practical options to solve the issue. In Figure 10 different arrangements are displayed. All but the second have reinforcement plates in between the columns' flanges, the second and fourth have welded web panel reinforcement plates, the third option includes "Z" diagonal welded bars and the fifth arrangement has its two node reinforcement plates welded by the tip of column flanges.	[39,40,49,54,55,62,65]

[(1)] When not specified otherwise, clauses are from EN 1998-1.

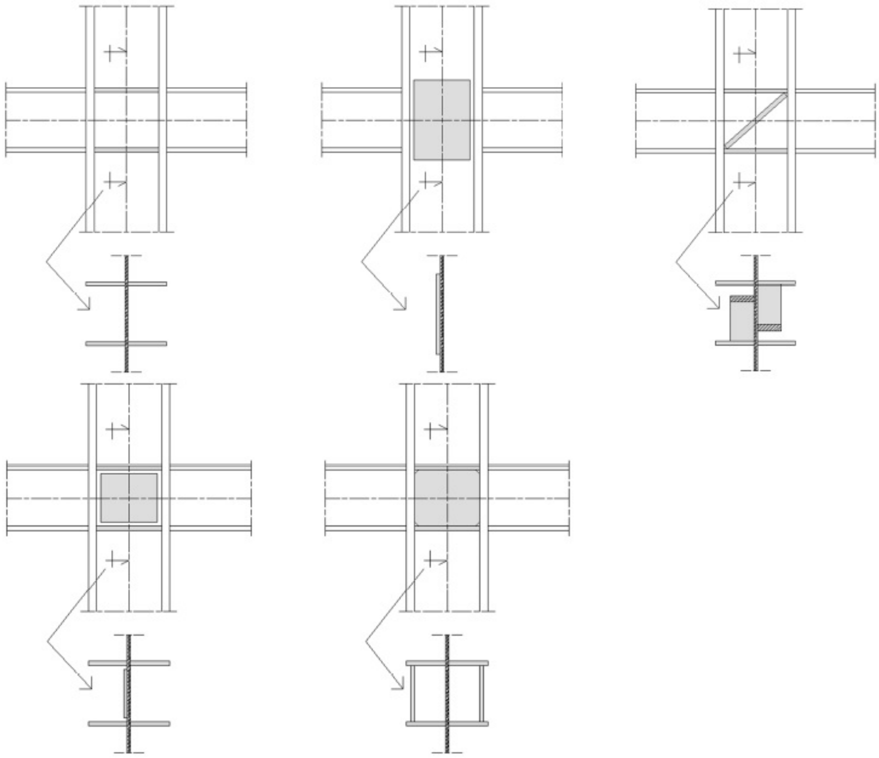

Figure 10. Web panels reinforcement options.

Further discussions of interest for establishing design procedures include the assessment of the cover-plate solution benefits and limitations in moment-resisting frames by Engelhardt and Sabol [66], as well as the investigation of Johnson et al. [67] of concentrically braced frames (CBF) connections seismic behaviour. The usually challenging case of column base connections can be assisted by the works of Latour and Rizzano [68] and Kanvinde et al. [69]. Connections design for partial strength is very limited under the Eurocode 8 framework, but can be better understood with studies by Lee and Kim in reduced beam section (RBS) moment connections [70], as well as Wang et al. work in low strength connection plates [71]. Other references can give valuable information concerning RBS behaviour [72].

However, the complexity of seismic design goes far beyond the practical steps for designing and safety checking elements and its connections. Complying with the strength and ductility requirements and accurately predicting its flexibility is paramount for connections designed to harmoniously merge with general design philosophy and procedures.

General design procedures include Castro et al. improved force-based design (IFBD) for moment resisting frames (MRF) [73], procedures and analyses for CBF design [74,75], as well as the influence of connections design upon the system performance [76]. Studies on the eccentrically braced frames (EBF) design can be found in [77]. Figure 11, Figure 12, Figure 13, Figure 14 may assist the reader with the most common nomenclature for structural systems classification regarding lateral load resistance.

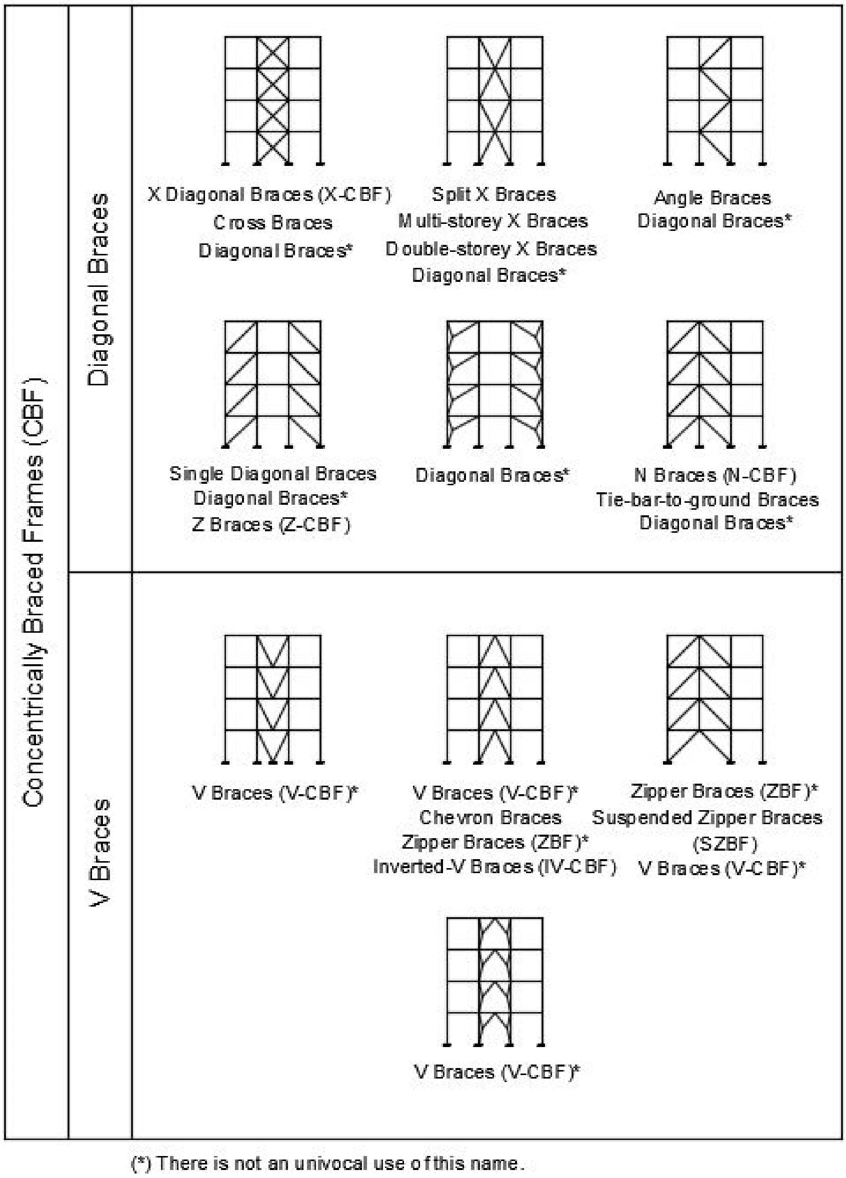

Figure 11. Nomenclature for the main types of concentrically braced frames.

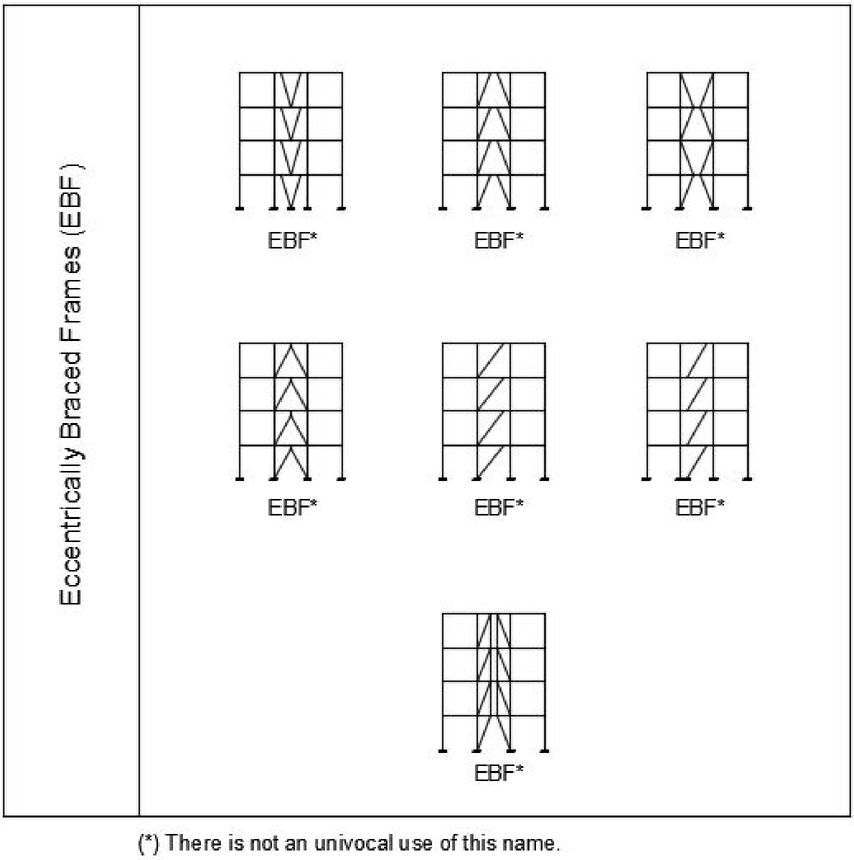

Figure 12. Nomenclature for the main types of eccentrically braced frames.

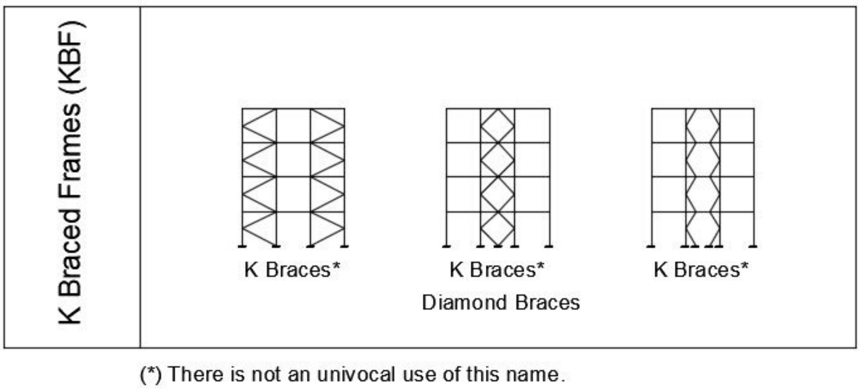

Figure 13. Nomenclature for the main types of K braced frames.

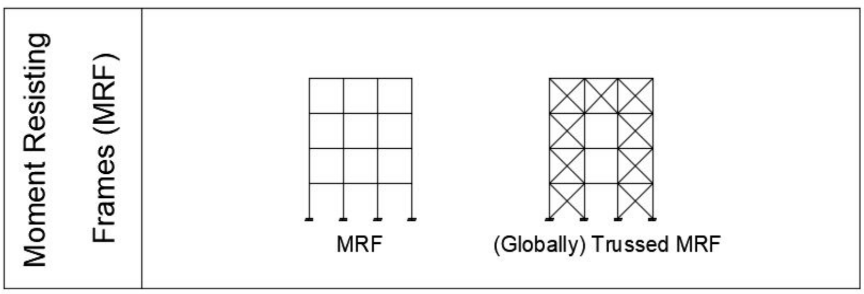

Figure 14. Nomenclature for the main types of moment resisting frames.

While modern design philosophy for seismic conditions already puts ductility in the centre of structural design, ensuring the fulfilment of certain code expressions may not be sufficient to assure an adequate hysteretic behaviour. Therefore, hysteretic models development is advised, especially for uncommon connections. Guidance for such an endeavour can be found in [78–84].

Weld design is also particularly important to ensure the adequate ductility of steel connections under seismic actions, even for bolted connections that inevitably have welded parts. Some interesting practical insights for seismic applications can be found in [85–89].

Adding to the aforementioned aspects, some issues not subjected to specific code prescriptions are still not negligible for the structural behaviour under seismic loading.

That is the case of the low cycle and ultra-low cycle fatigue of steel members and connections, on which design assistance can be found in [90–92].

Other cases include designing connections in braces for moment–shear interaction [93], accounting for geometry defects [94], or analysing innovative solutions, partially or totally uncovered by the standards [95–97].

The Eurocodes framework importance cannot be disregarded when an innovation or methodology is developed for the design or fabrication of connections in steel construction as, without assuring compliance to it, impracticability is unavoidable.

3.4. Idiosyncrasies and Research Gaps

Albeit comprehensive and based on scientific consensus, the Eurocodes approach for the design of steel structures includes some well-known idiosyncrasies from the practitioners' point-of-view, which require further research from scholars. This is especially true for the Eurocode 8 provisions to steel design of connections.

As well synthesized in the literature [6,98], some emergent needs for improvement in the Eurocode 8 include adopting a common hazard evaluation methodology for all the European regions, as well as codifying more economical, safer and less intrusive approaches for assessing and strengthening existing structures.

Specifically concerning the seismic design of steel structures to the Eurocode 8, one can find the European Convention for Constructional Steelwork's assessment of EC8 provisions for seismic design of steel structures [6]. This document highlights the need for improving material choice, concerning its toughness and overstrength, finding suitable approaches for sectional ductility, rather than prohibiting the use of class 4 sections, prequalifying dissipative connections, revising the design criteria for structures in low seismicity regions, and widening the portfolio of allowed structural typologies.

However, practitioners and researchers have discussed several other critical issues pertaining to the design of steel connections to the Eurocode 8.

As reported in [52], the conjunction of EN1998-1's clause 6.5.4 (1), EN1993-1-1's clause 6.2.3. (3) and EN1993-1-1's safety partial factors and yield and ultimate stresses for the defined structural steels lead to the circumstance that no holes can be made in many profiles, hindering the design of bolted connections and their feasibility in practical circumstances.

These clauses impose the inequality in Equation (1), developed in Equation (2) and leading to Equation (3).

$$N_{pl,Rd} < N_{u,Rd} \tag{1}$$

$$A \times f_y/\gamma_{M0} < 0.9 \times A_{net} \times f_u/\gamma_{M2} \tag{2}$$

$$A_{net}/A > \gamma_{M2} \times f_y/(0.9 \times f_u \times \gamma_{M0}) \tag{3}$$

Considering γ_{M0} = 1,00 and γ_{M2} = 1.25, as specified in EN1993-1-1's Table 3.1, and material properties in the same standard the minimum allowable A_{net}/A ratio depicted in Table 3 is found.

Table 3. Allowable net/gross area ratio for bolted steel connections under the Eurocode 8 framework, considering EN1993-1-1 steel properties.

Steel		Thickness Range	$A_{net}/A \geq$	Allowable Holes ≤ (%)
EN 10025-2	S 235	t ≤ 40 mm	0.907	9.3
		40 mm < t ≤ 80 mm	0.829	17.1
	S 275	t ≤ 40 mm	0.888	11.2
		40 mm < t ≤ 80 mm	0.864	13.6
	S 355	t ≤ 40 mm	0.967	3.3
		40 mm < t ≤ 80 mm	0.990	1.0
	S 450	t ≤ 40 mm	1.111	-
		40 mm < t ≤ 80 mm	1.035	-
EN 10025-3	S 275 N/NL	t ≤ 40 mm	0.979	2.1
		40 mm < t ≤ 80 mm	0.957	4.3
	S 355 N/NL	t ≤ 40 mm	1.006	-
		40 mm < t ≤ 80 mm	0.990	1.0
	S 420 N/NL	t ≤ 40 mm	1.122	-
		40 mm < t ≤ 80 mm	1.042	-
	S 460 N/NL	t ≤ 40 mm	1.183	-
		40 mm < t ≤ 80 mm	1.106	-
EN 10025-4	S 275 M/ML	t ≤ 40 mm	1.032	-
		40 mm < t ≤ 80 mm	0.984	1.6
	S 355 M/ML	t ≤ 40 mm	1.049	-
		40 mm < t ≤ 80 mm	1.034	-
	S 420 M/ML	t ≤ 40 mm	1.122	-
		40 mm < t ≤ 80 mm	1.083	-
	S 460 M/ML	t ≤ 40 mm	1.183	-
		40 mm < t ≤ 80 mm	1.127	-
EN 10025-5	S 235 W	t ≤ 40 mm	0.907	9.3
		40 mm < t ≤ 80 mm	0.878	12.2
	S 355 W	t ≤ 40 mm	0.967	3.3
		40 mm < t ≤ 80 mm	0.950	5.0
EN 10025-6	S 460 Q/QL/QL1	t ≤ 40 mm	1.121	-
		40 mm < t ≤ 80 mm	1.111	-
EN 10210-1	S 235 H	t ≤ 40 mm	0.907	9.3
		40 mm < t ≤ 80 mm	0.878	12.2
	S 275 H	t ≤ 40 mm	0.888	11.2
		40 mm < t ≤ 80 mm	0.864	13.6
	S 355 H	t ≤ 40 mm	0.967	3.3
		40 mm < t ≤ 80 mm	0.950	5.0
	S 275 NH/NLH	t ≤ 40 mm	0.979	2.1
		40 mm < t ≤ 80 mm	0.957	4.3
	S 355 NH/NLH	t ≤ 40 mm	1.006	-
		40 mm < t ≤ 80 mm	0.990	1.0
	S 420 NH/NHL	t ≤ 40 mm	1.080	-
		40 mm < t ≤ 80 mm	1.042	-
	S 460 NH/NLH	t ≤ 40 mm	1.141	-
		40 mm < t ≤ 80 mm	1.086	-
EN 10219-1	S 235 H	t ≤ 40 mm	0.907	9.3
	S 275 H	t ≤ 40 mm	0.888	11.2
	S 355 H	t ≤ 40 mm	0.967	3.3
	S 275 NH/NLH	t ≤ 40 mm	1.032	-
	S 355 NH/NLH	t ≤ 40 mm	1.049	-
	S 460 NH/NLH	t ≤ 40 mm	1.162	-
	S 275 MH/MLH	t ≤ 40 mm	1.061	-
	S 355 MH/MLH	t ≤ 40 mm	1.049	-
	S 420 MH/MLH	t ≤ 40 mm	1.167	-
	S 460 MH/MLH	t ≤ 40 mm	1.205	-

Beyond the cases where code compliance is not achieved even without holes, one should note that allowable hole areas under 10%, for example, would lead to the need to have plate widths over 200 mm for one single 20 mm hole, assuming constant thickness. Even with quincunx layouts, that would not be reasonable for most, if not all, practical cases.

Two possible options for overcoming this hindrance are employing steel grades with a higher f_u/f_y relation and detailing locally reinforced joints as proposed in [52]. While the latter option requires further research and has a significant hindrance in the fact that EN1998-1 requires that bolts shear resistance exceeds a connection plate bearing resistance by, at least, 20%, the former is limited to EN1993-1-1's clause 3.2.1 (1) constraints. Such a clause constrains steel yield and ultimate stresses to product standards or nationally defined annexes to the Eurocode, beyond the aforementioned comparison values.

However, even if using EN10025, EN10210, and EN10219 standards for material properties, similar A_{net}/A ratios would apply, as exemplified in Table 4 for EN10025-2 steels.

Table 4. Allowable net/gross area ratio for bolted steel connections under the Eurocode 8 framework, considering EN10025-2 steel properties.

Steel		Thickness Range	A_{net}/A	Allowable Holes (%)
EN 10025-2	S 235	3 mm < t ≤ 16 mm	0.907	9.3
		16 mm < t ≤ 40 mm	0.868	13.2
		40 mm < t ≤ 63 mm	0.829	17.1
		63 mm < t ≤ 80 mm	0.829	17.1
	S 275	3 mm < t ≤ 16 mm	0.932	6.8
		16 mm < t ≤ 40 mm	0.898	10.2
		40 mm < t ≤ 63 mm	0.864	13.6
		63 mm < t ≤ 80 mm	0.830	17.0
	S 355	3 mm < t ≤ 16 mm	1.049	-
		16 mm < t ≤ 40 mm	1.020	-
		40 mm < t ≤ 63 mm	0.990	1.0
		63 mm < t ≤ 80 mm	0.960	4.0
	S 450	3 mm < t ≤ 16 mm	1.136	-
		16 mm < t ≤ 40 mm	1.086	-
		40 mm < t ≤ 63 mm	1.035	-
		63 mm < t ≤ 80 mm	0.985	1.5

Another interesting issue lies in the failure modes hierarchy prescribed in EN1998-1's clause 6.5.5 (5). By specifying that bolts' shear resistance should exceed at least 20% of the plate bearing resistance, a significant constraint is imposed to the design since EN1998-1's clause 6.5.5 (3) also requires non-dissipative connection resistance should exceed the resistance of the connecting element by no less than 37.5%. Cumulatively, bolts shear resistance design criterion is frequently challenging to fulfil.

Moreover, as connection resistance is already significantly greater than the element's, the impact of establishing a failure modes hierarchy for connection plates and bolts may be unclear.

Still, in EN1998-1's clause 6.5.5, bolted connections in shear are limited to categories B or C, making connections with pins out of the code's scope. This is usually a practical problem for designing many special structures, even when it is not governed by seismic loading.

Plastic hinge formation at columns bottom can be very difficult to attain, albeit depicted in EN1998-1's clause 6.3.1 (5) illustration of moment resisting frames. In fact, if designed for resisting bending moments, columns base connections usually require thick base plates and stiffeners to transfer loads to anchor bolts. Such a layout can usually only be compatible with plastic hinges above the stiffeners and not at the base of the columns.

A capital issue concerning seismic overstrength demand in concentrically braced frames designed to the Eurocode 8 is plainly depicted in a seminal work by Málaga-Chuquitaype and Elghazouli [99]. In this work, the authors show that the European option for tension-only design, contrary to the usual American approach, may lead to an overstrength demand upon columns, as compressed braces can withstand significant loading either in pre or post-buckling behaviour. Eurocode 8 slenderness prescriptions only partially address this issue, mitigating, but not fully avoiding, the problem.

One other issue resulting from EN1998-1's clauses 6.5.5 (6) and (7), 6.6.4 (2), and 6.7.3 (9) is the request for experimental evidence for dissipative connections, or connections into dissipative zones. Structural designers cannot fulfil such a request. Even when some experimental evidence is available, it will hardly comply with each designed connection exact geometry, scale, materials and loading conditions. Moreover, the relationship between even the simpler partial strength and stiffness connections and global flexibility and rotation demands may be difficult to accurately assess without experimental or advanced numerical methods [100], subtracting the possibility of regular designers comply with regulatory prescriptions.

To overcome these difficulties, European codes may follow Japanese and American experience in defining pre-qualified joints [43,101]. Recent research in Eurocodes compliant solutions, as reported in [102–104] may emerge as an important asset. Nevertheless, it should be noted that prequalified solutions, if not sufficiently broad but agile, may face the challenges as mentioned for the current use of literature-based joints.

Finally, one should note that several solutions for enhancing the seismic behaviour of structures are executed at the connections level and may be difficult to frame under the Eurocode 8 framework. That may be the case for the RBS solution, if materialized on connection plates rather than in the element, for the similar strategy of replacing the element or the connection plates material by one with lower yield stress to foster the occurrence of a controlled plastic hinge [105], as well as the use of slotted bolted connections (SBC), shear slotted bolted connections (SSBC) and rotational slotted bolted connections (RSBC) as friction damping devices [106,107].

4. Recent Research on Seismic Design of Bolted Connections

4.1. Bolted Connections in Concentrically X Braced Frames

Experimental investigation in connections within CBFs has found some recent developments with the work of Sen et al. [108], which shed light on the detrimental effects upon seismic behaviour of deficiencies in older CBF connections. This included local slenderness and weld quality in gusset plates which hinder drift capacity and increase brittleness [108], respectively. Moreover, Rongqian and Xuejun recently presented more evidence that connections fracture is responsible for lower ductility and energy dissipation in X concentric braces compared with V braced frames [109].

On the other hand, by testing realistic joints in CBFs under different levels of seismic excitation, Goggins et al. [110] found that brace fracture occurred under high-intensity excitations whereas gusset plates remained unfractured if designed with standard linear clearance (SLC) or elliptical clearance (EC) methods. Complementarily, Kanyilmaz assessed the ductility of bolted joints with gusset plates with angle braces [111]. Reinforcement strategies for such members and connections have recently been assessed by Kishiki et al. [112].

As highlighted by Campiche and Costanzo [113], the design of brace-to-frame joints has brought significant criticism to the Eurocode 8 and will be subjected to a modification in the forthcoming issue. According to the same authors, the previous capacity design condition will now be replaced by a group of three inequalities deemed to assure an adequate behaviour, both in compression and in tension, in and out of plane [113].

Another frequently disputed prescription in the current European practice is the waiving of compression braces—and their connections—in N and X CBFs. A recent study by Kanyilmaz supports the current trend to consider the role of such compression elements in analysis and design [114].

Further recent research on the impact of the connections on CBFs actual behaviour cannot disregard Silva et al. contribution. In [115,116], the dichotomy between design assumptions and gusset connections' real performance was assessed, to the point of suggesting a new slenderness factor for a more realistic design. A further step allowed comparing the use of simplified and code-compliant, pinned boundary conditions for braces with the realistic modelling of gusset plates and attaining conclusions on expected behaviour and costs for both approaches [117]. Other cost investigations on Eurocode 8-designed CBFs can be found in del Gobbo el al. recent work [118]. In [119] other alternatives to Eurocode 8 current rules are assessed, and it has been found that the relaxation of some current prescriptions may benefit the design cost-effectiveness while ensuring a similar performance.

Significant proposals of alternative methods to Eurocode 8 rules for concentrically braced frames can be found in Costanzo et al.'s recommendations for simpler and more ductile design rules [120] and in Bosco et al. Ω* method [121].

4.2. Bolted Connections in V Braced Frames

V concentrically braced frames, also referred to as chevron braced frames and zipper braced frames (ZBF), are well-established in the worldwide engineering practice, despite a frequently more constrained dissipative behaviour and several standards prescriptions difficult to comply with. Yet, another important issue lies in the size of brace-to-beam connection plates. A long plate not only poses cost-efficiency issues but also may have an underestimated impact upon the system behaviour.

Nevertheless, recent research in connections within V CBFs is not profuse. Rahimi et al. [122] investigated the cyclic behaviour of these systems, as well as of its suspended variant—suspended zipper braced frames (SZBF)—accounting for the joints details to find that SZBFs tend to outperform ZBFs if gusset plates behaviour and accurately modelled buckling are considered.

In [123], Costanzo et al. discuss the current European rules for V CBFs, accounting for the joints behaviour, and suggest some improvements.

4.3. Bolted Connections in Eccentrically Braced Frames

Eccentrically Braced Frames have some important advantages over CBFs due to its energy dissipation capacity and contribution towards system ductility. On the other hand, its stiffness, bounded in an inferior aspect by MRF and in a superior aspect by CBF, its architectural impact, and the need for voluminous shear link connections (and associated supplemental costs) lead to its scarce employment in regular practice, even though some seismic design standards, such as Eurocode 8, provide a favourable context for its use.

Even though research in EBF behaviour and design has found significant advances in recent years, as reported by Hu et al. [124], innovations in EBF connections are still limited. Valuable exceptions can be found in Hu et al. new shear links proposals [125–127]. As depicted in those articles, a short shear link with shear slotted bolted connection (SSL-SSBC) was proven effective in enhancing the system ductility and energy dissipation with limited damage for medium intensity earthquakes and concentrated damage for large intensity earthquakes in Y shaped EBFs. Moreover, a decisive contribution towards the understanding of detachable short links was provided by Zimbru et al. [128].

4.4. Bolted Connections in Moment-Resisting Frames

The research on MRF connections is the most common among the initiatives deemed to develop joint systems in steel structures under seismic loading.

Several researchers and practitioners have mentioned the limited energy dissipation associated with the design of massive connections in compliance with seismic standards [52]. However, Costanzo et al. recently endeavoured a deep analysis on the matter, considering the connections effect in MRF by employing the Ibarra–Medina–Krawinkler (IMK) non-linear modified model, and showed that MRFs typically behave in the elastic range almost until the near collapse (NC) threshold, being governed essentially by the code's displacement constraints [129].

Not long before, D'Aniello et al. had analysed the shortcomings in Eurocode compliant design of end-plate (EP) bolted connections in MRFs and proposed design criteria for stiffened connections of such a kind [130], while Tartaglia et al. compared its design to its American pre-qualified counterpart [102,131]. As a result, it has been shown that both AISC 358-16 pre-qualified and Eurocode compliant pre-qualification proposed (in EQUALJOINT project [132]) approaches for extended stiffened EP bolted joint assure the formation of plastic hinges in beams subjected to cyclic loading, despite following different design criteria [102].

Further experimental investigations in MRFs with EP connections led Francavilla et al. to conclude that partial strength connections can guarantee an adequate performance under seismic loading [133]. Meanwhile, the quest for innovative solutions was pursued by Saberi et al. with the proposal of a rehabilitation scheme for end-plate connections with haunches [134], and Lin et al. used EP connections in a structural solution for built-up columns for MRFs, which allows a significant level of functionality after withstanding major seismic actions [135].

A seminal work by Radmehr and Homami addressed the uncertainty in EP connections capacity (along with the uncertainty in seismic demand) with the so-called simplified response surface method (SRSM). With this approach, which includes a Latin hypercube sampling method for determining the probabilistic moment-rotation curve, the Monte Carlo sampling technique, and accounts for the most significant random parameters such as geometry, material properties and construction errors, reliability indices and probability of failure can be attained [136].

Bolted splice connections, or cover-plate connections—the other embracing group of connections in MRFs—have also experienced significant research advances concerning its behaviour under seismic actions.

Within this group, prefabricated beam-column steel joints (PBCSJ) [137,138] or prefabricated beam-column steel connections (PBCSC) [139] are a very active research topic, with sensible recent developments. Namely, Jiang et al. assessed the flange cover plates (FCP) behaviour, with simple and double cover-plates layout, and depicted its adequate energy dissipation and post-earthquake reparability [137,138], while pointing out the eventual insufficient beams gap as a source of local damage [137]. Previously, similar results concerning satisfactory PBCSC energy dissipation capacity had been attained by Ai-lin et al. [139], who also concluded that slip friction could provide a further significant source for dissipative behaviour.

Flange cover plate joints were also employed by Liu et al. to connect trussed beams to columns. By doing so, an arrangement that can be classified as rigid for weak seismic motion and exhibits flexibility and energy dissipation due to slippage for strong motion was attained [140].

In the context of the FREEDAM project, D'Antimo et al. further developed a slippage-based joint exclusively connected at flanges [141], deemed to answer to earthquake and explosion-originated strong actions. Previously, Faridmehr et al. had developed a different concept for cyclic and explosive loading suitable connection, with welded flanges and bolted web with adequate results for rotation capacity, plastic behaviour of the appended beam, and catenary behaviour [142].

A very different approach to flange bolted connections allowed Sarvestani to attain adequate strength and local stability in connecting a castellated beam in an MRF simply with angle connections added to post-tensioned strands [143]. This solution is also suitable for employment in a self-centring system.

A very interesting concept idealized by Qian and Astaneh-Asl relies on designing a gusset plate connection for MRFs. As the beam length does not extend to the column, the gap is filled with a plate which will foster plastic behaviour due to in-plane bending [144]. This concept can be suited for fully welded connections, as well as both to end-plate or flange cover-plate connections. The connection ductility was proven for the cases where the beam moment capacity exceeds the gusset plate one [144].

In recent years another topic drawing increasing attention from the research community relates to partially restrained (PR) connections in MRFs due to field evidence after seismic events. A recent study [145] assessed this typology performing incremental dynamic analyses (IDA) to attain fragility models and, by doing so, created an advanced design and analysis tool for this joints concept.

4.5. Partial Strength and Semi-Rigid Moment Connections

As reported in some of the aforementioned research articles, an efficient dissipative behaviour in MRFs is frequently achieved by using partial strength and/or semi-rigid moment connections. Further insight on these specific solutions can be attained by scrutinizing the investigations by Zhao et al. [146], Tartaglia and D'Aniello [147], and Pal et al. [148], who carried out parametric analyses concerning plates' and bolts' geometry, the influence of transverse beams, and the issue of bolts loosening, respectively.

Moreover, modelling of semi-rigid connections has been studied by Movaghati, who suggested the use of three-dimensional beam or rigid elements for simulating bolts shank, for more efficient computation [149].

Innovative solutions for semi-rigid joints have recently been studied by Saberi et al., with a rehabilitation approach using post-tensioned tendons in a weak EP connection [150] and by Gaxiola-Camacho et al., who assessed an economical, flexible joint using a PBSD procedure [151]. However, we must note that while the latter is proposed by its authors as a daily approach for practicing engineers, the realistic assessment of joints' rigidity is paramount and may be difficult to attain for the complete set of connections within a design endeavour.

4.6. Slotted-Holes or Slipping Bolted Connections

Several of the aforementioned research articles reported that slippage is currently and increasingly regarded as an efficient means for ensuring a dissipative behaviour in bolted cover-plate connections under low-to-medium seismic motions. Nonetheless, for controlled slippage to occur, slotted holes are needed.

A comprehensive depiction of the phenomenon can be found in [152]. There, Shu et al. explain the purpose of slotted holes in slotted bolted connections, aided by experimental and numerical tests on case-studies [152], which led to design recommendations [153].

Still within the group of recent research on the matter, Liu et al. developed a connection system to link trussed beams to columns with the particular feature of ensuring the dissipative behaviour under seismic actions by slippage between cover plates and chord members [154].

4.7. Bolted Connections in Dual Systems

Merging braced frames and moment-resisting frames in one single structural system increases heterogeneity and uncertainty in its behaviour under seismic actions and faces significant shortcomings in most design standards. However, those dual systems frequently offer valuable solutions for practical problems and may also be used for increasing redundancy if properly studied and nest interesting innovations.

Despite the scarcity of research in these systems and mostly in its connections, interesting research has been conducted in friction connections of braces to be added to MRFs. As reported by Piluso et al., friction connections can be added to those stiffer elements so that damage and displacement can be concentrated in such replaceable joints [155].

4.8. Bolted Connections for Hollow Sections

The use of hollow structural sections (HSS) as members in structural systems subjected to earthquake actions has numerous advantages. These include its suitability for withstanding significant bending moments in two orthogonal directions and its enhanced stability under compression. Moreover, some of those sections may be designed for a ductile post-(local)-buckling behaviour or even be filled with other materials, such as concrete, attaining increased resistance.

However, the employment of HSS in building structures is still rare. Such an outcome is partially due to the sections' usual higher cost-to-strength ratio, to the lesser benefit in terms of buckling mitigation for regular low-height columns and especially to the difficulty in endeavouring practical and economic joints. In fact, not only do such connections usually require more steel, more cutting and welding operations in the steel profiles and more complex installation, but also face a more complex and less standardized design procedure, including profuse finite element analyses (FEA) and an increased difficulty to comply with some code prescriptions, namely the Eurocode 8 ones.

Under these circumstances, the research in the field of seismic behaviour and energy dissipation of HSS and especially in HSS connections and HSS members behaviour accounting for the usual connection types is paramount both for code development and for leveraging the practical applicability of those solutions [156].

Recent research on the matter is not profuse due both to its specificity and the scarcity of the current use of HSS in building structures. However, one can highlight interesting recent experimental and numerical studies.

Among the former, one can highlight the recent work of Nuñez et al. assessing the cyclic behaviour and seismic performance of end-plate connections to square HSS columns (EP-HSS) with two-level reinforcement. In [157], they found that such moment connection type, with an I beam and HSS column was able to concentrate the plastic behaviour in the beam with significant energy dissipation, thus avoiding stress concentrations in the column. This allowed them to suggest the prequalification of such a joint under the American ANSI/AISC framework. Later, Nuñez et al. also found that EP-HSS joints with HSS columns and HSS beams, depicted in Figure 15, show a non-ductile behaviour due to the beam buckling early [158].

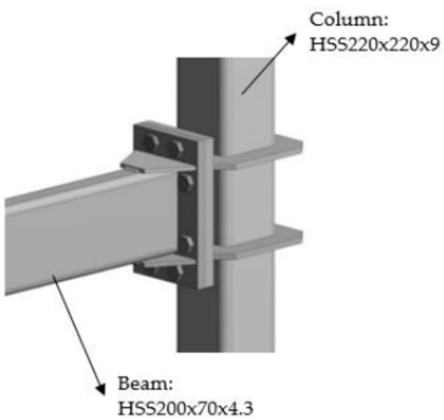

Figure 15. End-plate connections to square hollow structural section columns (EP-HSS) connection with HSS beam and HSS column showing a non-ductile behaviour [158] reproduced under the Creative Commons Attribution 4.0 International License, (http://creativecommons.org/licenses/by/4.0/).

The experimental work of Song et al. assessed the behaviour of a simpler joint layout, using two angles to connect I beams to square HSS columns [159]. Beyond an extensive parametric analysis, including very different failure modes under different tested parameters, a general conclusion could be drawn that bolted setups showed better energy dissipation [159]. On the other hand, several setups have been shown to fail under fracture modes [159]. The work was then extended to analyse such connections behaviour considering the post-earthquake fire (PEF) scenario. Prior damage due to cyclic loading has been found to severely limit the resistance of the connections under fire actions [160,161], suggesting that this usually overlooked issue may be critical for structural systems safety, at least if employing the assessed connection concepts.

In opposition to EP-HSS, another typical layout for connecting I-section beams to HSS columns can be attained with bolted flange plates. This concept has recently been studied by the Liu, Zhan and Yu group. Their experimental findings revealed that such an arrangement could be designed to attain high ductility and substantial energy dissipation by allowing cover-plates slippage under severe cyclic actions [162]. This is an interesting result, if analysed under the design constraints to semi-rigid connections posed by several seismic design codes. As the study evolved to a double cover-plate connection, a full-strength—rigid before slippage and semi-rigid after slippage—the design was tested, showing adequate rotation capacity and the possibility of endeavouring a repositionable design, with flange bolts retightening after severe earthquakes [163].

An interesting joint proposal was brought forward by Liu et al., as end-plate, flanges plates and column splice were merged together in a successful attempt to reduce the joint overall complexity while ensuring a satisfactory dynamic behaviour [164].

A different perspective of connections in HSS can be found in flange joints of pipes deemed to bear significant internal pressures. Those connections must also withstand earthquake actions and, therefore, experimental and numerical research on innovative solutions is critical. In this context, Sato et al. research [165] is fundamental, by assessing possible leakages in bolted flange connections with polytetrafluoroethylene (PTFE) gaskets.

Exclusively numerical studies also have a role in the development of solutions for HSS connections. That is the case of Gallegos et al. investigation of the aforementioned beam failure conditions in EP-HSS connections between I-section beams and square hollow columns (Figure 16) [166], as well as the Li et al. early-stage proposal of a wedge shape connection [167].

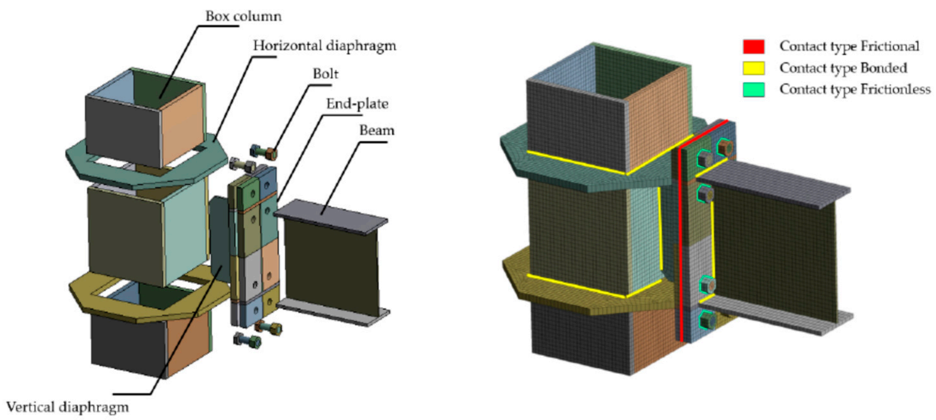

Figure 16. Numerical study of an EP-HSS connection with HSS beam and HSS column [166] reproduced under the Creative Commons Attribution 4.0 International License, (http://creativecommons.org/licenses/by/4.0/).

4.9. Connections in Systems with Supplementary Damping

Dampers have long been used for radically increasing structural systems' energy dissipation capacity. There is a plethora of damping devices of many types, with a common feature of being inserted into structural elements exposed mostly to axial forces but also bending or shear. Therefore, inserting dampers in connections is a less common approach, also because it is usually not suitable for the employment of commercial devices.

However, the interest for case-specific dampers to be harmonized in steel connections has grown in the latest years, mostly due to its potential for ensuring efficient and economical solutions. Hence, recent research in connections with supplementary damping for steel systems has brought significant developments in the fields of friction dampers, slit dampers, viscous dampers, and post-tensioned strands.

Concerning friction dampers in beam flange cover-plates, Cavallaro et al. assessed bolts preload and deferred effects in time to find that the most significant loss of preload, with an impact on the damping capacity, occurs in the first 30 days after tightening [168] and Chan and Hu [169] investigated the behaviour of friction dampers installed in the interface of steel beams and concrete columns.

Slit dampers are a potentially inexpensive solution to ensure ductile behaviour in MRF connections. To such an end, Lor et al. [170] and Shahri and Mousavi [171] recently proposed and tested two layouts.

Velocity-dependent viscous dampers have been widely employed for seismic retrofitting of existing substandard steel frames due to their advantageous capability of providing supplemental energy dissipation without significantly altering the stiffness properties of the structure. Optimization procedures of viscous dampers for applications to substandard braced steel frames were comprehensively discussed by Del Gobbo et al. [172] and by De Domenico and Hajirasouliha [173] with reference to linear and nonlinear fluid viscous dampers, respectively.

Post-tensioned strands emerged as an arguably simpler and more economical technique to enhance damping in steel frames connections. Not only does this solution increase structural damping, but also it may play a pivotal role in the making of self-centring systems and replaceable connections.

Recently, post-tensioned (PT) high-strength strands (HSS) modelling procedures have been revisited by Moradi and Alam [174], improved for including local buckling and fracture phenomena by Al Kajbaf et al. [175] and investigated in the context of multi-criteria optimization with a response surface methodology (RSM) by Moradi and Alam [176].

The use of shape memory alloys (SMA) with super elastic properties in PTHSS solutions was recently studied by Rahmzadeh and Alam [177], Toghroli et al. [178], Torabipour et al. [179] and Chowdhury et al. [180]. Transversely, an adequate hysteretic transverse behaviour has been found, while the latter concluded that using shorter PTHSS would increase the energy absorption but also the residual drifts. To overcome this hindrance, hybrid HSS with SMA devices is proposed [180].

Post-tensioned strands have also been suggested for the retrofitting of weak bolted T-stub connections, as well as for modifying pinned connections into moment ones [181] and for avoiding the progressive collapse of frames [182].

In a broader field of application, damping devices embedded in steel connections have also been studied for the use on concrete frames. That is the case of the prefabricated steel joint (PSJ) proposed by Qi et al. [183] and the Manfredi et al. high-performance dissipating frame (HPDF) with shear damping connections [184].

4.10. Bolted Connections in Reduced Beam Sections

Reduced Beam Section is the term usually employed in American practice and research to depict the cutting of beam flanges with the objective of locally limiting its moment resisting capacity and, therefore, concentrating the plastic hinge in the desired position. Its most significant advantage is fostering a more predictable and ductile behaviour in the MRF system. However, it is also useful in ensuring that bolted connections between beams and columns are not subjected to significant moments and remain in the elastic domain. This protective effect has recently been studied by Antoo and Joseph [185,186]. Other studies found the pivotal role of RBS in limiting other connections parts' damage [187,188].

In Europe, on the other hand, RBS has not been a prevalent field of study until the last decade [189], even if it had some of the earlier, pre-Northridge, applications [190]. To assess the application of recent RBS developments in European practice, Sofias and Pachoumis performed an experimental and numerical investigation of end-plate bolted connections adjoining RBS beams with European profiles [189] to find its ductile behaviour. Nevertheless, local buckling and fracture were observed.

4.11. Replaceable Connections

Reparability with non-disproportionate costs after a major earthquake emerged as a condition for seismic design as assured by performance-based seismic design codes and practice, with increasing efficiency, the no-collapse, and significant damage limitations. As a result, there is a need to concentrate damage in structural parts that can be easily replaced, hence referred to as "fuses".

In steel structures, assigning the fuse function to connection parts avoids nesting new devices—with its own connections—to the steel members, using small components already designed taking installation constraints into account. This intent, which merges with the design of connections for supplementary damping and the making of self-centring systems requires complex design strategies and extensive research and testing.

The design of fuses has had its most recent advances in the work of Hu et al. [191], in Wang and Bi's and He at al.'s development of low yield point steel (LYP) connections [105,192], or in Pongiglione et al. novel connection design [193], all with a proven capability to concentrate plastic damage in the replaceable parts while assuring that the remaining structural members behave elastically to a significant intensity of motion.

Moreover, the behaviour of frames with replaceable connection parts has been further studied by Wang et al. [194], and by Lin et al. [195], who proposed a frame-and-fuse system with the ability to withstand significant earthquake actions without noticeable damage and only require the fuse replacement for very rare earthquake motions and by Garoosi et al. who assessed the interaction of Reduced Beam Sections and replaceable connections [196].

Other recent studies of interest can be found in the studies of Feng et al. and Pinkawa et al. on replaceable links, either in its fatigue behaviour [197] or in its optimization [198], respectively, and in Pongiglione et al. research on combining seismic design and design for disassembly (DfD) in industrial building structures [199].

Concerning the design of fuses, the work of Wang et al. [200] may assist practitioners with their proposed approach to assess frames behaviour, as well as Lin et al.'s conclusions on fuses' shape and thickness influence on its performance [195] provides relevant information for design purposes. Moreover, European Commission (RFCS) funded research has had a significant impact on the development of fuse solutions. Starting before the aim of the current review, with the FUSEIS ("Dissipative Devices for Seismic Resistant Steel Frames") project, the programme found continuity in the INNOSEIS ("Valorization of innovative anti-seismic devices") project. The latter has recently depicted the implementation of such devices in the seismic design of four and eight-storey office buildings. As a result, tested solutions and design recommendations are now available for practitioners to employ FUSEIS2 and INERDTM connections [201].

Noteworthy is also the work of Wiebe's group on an integrated brace and gousset design for replacement [202,203]. With this, they were able to address not only the practical difficulties in replacing damaged gousset plates but also to avoid the out-of-plane buckling of braces, known for its economic impact upon façades cladding.

4.12. Bolted Connections in Self-Centring Systems

Residual permanent displacements after seismic events have a significant impact on a building structure repair cost and, in extreme cases, may lead to irreparability. Thus, a recent trend in earthquake engineering lies in developing solutions for ensuring the centring of structural systems once the horizontal action is terminated.

The so-called self-centring systems, in steel structures, are usually integrated in bolted connections, either as bolt-like components or as added post-tensioned high strength strands (PTHSS). Moreover, such a layout is suitable to bear supplementary damping components.

Concerning the PTHSS solution, Abdollahzadeh et al. assessed a layout with strands located between the beam flanges and passing through holes drilled on the column flanges. The connection between beam and column was ensured by angles. After a parametric study, they confirmed the reduction of the residual gap between beams and columns and observed that the connecting angles remained undamaged [204]. Qin et al. added a friction-based energy dissipation device to the PTHSS layout. As a result, they found that the connection bending capacity was mostly determined by the HSS post-tension force and that deformation and energy dissipation, on the other hand, were significantly influenced by the T-stub thickness [205].

Replacing bolts by spring-like components is another possibility for materializing self-centring systems. The employment of SMA ring springs was tested by Wang et al. [206] and Fang et al. [207]. The former were able to observe a very good re-centring capacity and a satisfactory energy dissipation, while the latter results corroborate the adequate behaviour in steel frames but found that the self-centring ability is compromised in composite connections due to reinforcement yielding and concrete cracking.

A different concept was investigated by Gomaa and Osman, who tested a dual system, with EBF for energy dissipation, with a fuse-like function, and MRF to ensure the system re-centring. The system was shown to generally behave as idealized, with most plastic damage to occur in the shear links, yet avoiding interaction with the floor slabs [208]. However, for strong earthquakes, inelastic behaviour was reported in the MRF.

4.13. Composite, Hybrid and Complex Connections

State-of-the-art research frequently finds complex solutions in domains that are still afar from current practice or standards prescriptions. That is the case of some rehabilitation approaches, such as Saberi et al.'s [209], that is deemed to transform flexible connections into rigid ones by welding haunches, while improves its hysteretic behaviour, Chang and Yeh's welded and bolted solution [210], and Deng et al.'s simple connection for frames with double-tube sections [211]. Regarding the latter, one shall note that the article analysis was limited to certain extent by the efficiency of translation tools.

Also, Ali et al. proposed a solution with bolts made of steel and recycled rubber for simple connections with promising results in terms of energy dissipation and durability [212].

Another field where current research is vibrant in the endeavour of developing and testing feasible solutions for complex connections lies in the steel-to-concrete joints with enhanced energy dissipation and code-compliant ductility. These solutions, frequently referred to as hybrid when having steel and concrete parts, have had remarkable development in recent years, among which one can highlight Muciaccia's post-installed connection which achieves a behaviour similar to current steel-to-steel connections [213], as well as steel devices to ensure the connection within concrete elements, with the possibility of additional damping [214–216].

As the concrete filed steel tubes research yielded encouraging results for columns under seismic actions, its connection to steel beams arose as a critical field of research. For such an end, several answers using high-strength blind bolts have recently been investigated, including Agheshlui et al.'s full-scale test [217] as well as the experimental studies of Deng et al. and Wang et al. [218,219]. Specific types of concrete-filled sections, as the concrete-filled steel tubular columns (CFST) and the concrete-filled double-skin steel tube (CFDST) have been subjected to investigation regarding its possible connections to steel beams. The former has been investigated by Wang and Wang with good performance [220] and the latter by Wang et al. [221,222].

A further study on concrete slab interaction with composite connections was conducted by Amadio et al. [223]. As a result, it has been found that non-interaction with the bracing system can be ensured through adequate detailing and slab isolation.

4.14. Welded Connections and Welds in Bolted Connections

Welded connections are well known for low ductility and significant damage, including brittle fracture, under earthquake events. That is especially true for site welding, as highlighted by recent research [224]. In fact, the aforementioned study, investigating pre-Northridge joints unveiled a significant impact of site welding onto buildings' expected life [224].

Probably owing to this disseminated knowledge, research in seismic behaviour of welded joints has become a less active topic in recent years. Nevertheless, even bolted connections have significant welded parts, justifying continuous experimental testing as investigated by Liu et al. which recently systematized damage models for weld failure in beam-to-column joints [225].

4.15. Hysteretic Behaviour in Systems with Bolted Connections

Hysteretic behaviour of seismically loaded steel systems is one of the most difficult aspects to capture in structural analysis, either in numerical simulations or experimental testing of complex systems. This is due not only to the lack of guidance and standardization of loading protocols—for which the recent work of Safaei and Erfani may assist [226]—but also to the difficulty in accurately depicting damping in highly non-linear systems.

One critical source of uncertainty in damping modelling in steel structures with bolted connections lies in the so-called stick-slip phenomenon. Its input to the global structural damping may be very significant [227] and it is displacement dependent. Under these circumstances, one can regard as vital the recent Zhang et al.'s proposition of a more straightforward model, mimicking the viscous damping simplicity [227] to a certain degree. Additionally, Rodas et al. conceived a hysteretic model formulation for the behaviour of exposed column–base (ECB) details, accounting for unloading and reloading, pinching, and recentring phenomena [228], and Nath and Bhowmick [229] addressed the hysteretic behaviour of shear panels in moment connections.

Among the significant recent research in innovative solutions which bear a noteworthy work on hysteresis, as well, one can highlight Xue et al.'s cast steel replaceable connector [230], shown in Figure 17, Huang et al.'s H-shaped moment connection [231] and Ushio et al.'s joints in crane masts [232]. It should be highlighted that the connector displayed in Figure 17 is a unique piece cast. The remaining visible components are the connected column and beam.

Figure 17. Replaceable cast steel connectors [230] reproduced under the Creative Commons Attribution 4.0 International License, http://creativecommons.org/licenses/by/4.0/).

4.16. Material Properties and Fatigue Issues in Seismically Loaded Bolted Connections

Fatigue, brittle fracture, and post-earthquake fire-related failures are three examples of earthquake-related failure mechanisms which gather little attention, especially from the codes and designers' point-of-view. Yet, those have been shown to pose significant threats to structural safety.

Low-cycle fatigue (LCF) and ultra low-cycle fatigue (ULCF) are critical as earthquake loading lies within the range of high-stress and a low number of cycles actions deemed to cause severe damage to steel connections and welded parts. Accordingly, Wang et al. provided important insight on the LCF response of semi-rigid joints to HSS columns under seismic loading [233], while Sousa and Nussbaumer leveraged the knowledge of ULCF in welded steel connections [234].

Fractures due to microstructural issues in steel alloys, welds, and thermally affected zones have been studied by Stillmaker et al. with a focus on column splice fracture in seismically loaded MRFs [235] and by Liao et al. concerning microstructural modelling of steel alloys' fracture under strong earthquake motions [236].

Investigation on post-earthquake fire (PEF) has found significant developments in recent years, not only with the aforementioned Song et al. work [160,161], but also with Petrina's efforts to dissect the issue [237,238] with an emphasis in its likelier failure modes, to propose a deterioration coefficient [239] and to establish testing setups [240]. Moreover, Johnson and Sarif concluded that pre-earthquake fire, if succeeded by light or inexistent repair, will drastically reduce the cyclic response of the affected members, at least in welded unreinforced flange web (WUF-W) and RBS welded joints [241].

4.17. Bolted Connections in Cold-formed Steel Sections

Cold-formed steel sections are especially prone to local buckling, given its thin parts. This is a critical deterrent to its use in earthquake-resistant structural systems, thus facing significant hindrances from seismic codes. However, sensible developments are being pursued to solve this issue. The American Seismic Evaluation and Retrofit of Existing Buildings standard, ASCE 41, undertook an experimental data-based revision which led to stipulating modelling parameters and safety criteria for cold-formed members [242], even if the objective of establishing a PBSD practical procedure for structural systems with steel cold-formed sections remains to be achieved. Likewise, the Australian and New Zealander AS/NZS 4600 standard for "Cold-Formed Steel Structures" has been updated concerning connections design and seismic requirements [243].

Moreover, the research on the seismic behaviour of these members' connections has not been totally absent from recent publications. That is the case of transverse shear stiffness of frames, where bolted joints local deformation has been regarded as a critical source for a significant mismatch between experimental data and FE models [244]. To overcome this issue with a critical impact on seismic design, Talebian et al. developed modelling techniques, attaining good results compared with published experimental data [244]. Another interesting development can be found in recent Fiorino et al.'s experimental investigations on connections for cold-formed sections [245].

5. Discussion on Recent Advances and Research Needed

Assessing the data investigation on published scientific literature pertaining the shared domain of bolted steel connections and seismic design, one can observe that the number of publications was limited until the last 25 years. During the latter period, the research output has been continuously growing.

Over 99% of the published papers are either research articles or conference papers, leaving less than 1% for review articles. As such, one can hypothesize that this is still a field of study lacking consolidation and meta-analysis.

Arguably, this may explain why most research endeavours are at connections or frame levels and usually follow research groups' established and very specific previous research lines.

System-level analyses, especially with holistic assessments of the influence of employing novel solutions, and investigations deemed to improve seismic regulations and general practice are still scarce, but greatly needed. Such analyses would be useful not only for practice but also for identifying novel solutions and technological improvements that will have the chance to be implemented.

However, some exceptions are noteworthy. Among those, one can highlight Landolfo et al.'s [6] assessment of Eurocode 8 future developments in need, as well as recent research on bolted connections in concentrically X braced frames, with a strong focus on Eurocode 8 criteria, as provided by Campiche and Costanzo [113] and various contributions by Silva et al. [115–117].

Considering the aforementioned research, the conclusion that connections simulation in X concentric braces under seismic actions requires more complex modelling techniques and standards prescriptions revision seems embracing. This conclusion can also be supported by the findings yielded by recent research in the domain of replaceable connections by Wiebe's group [202,203].

Further proposals for Eurocode 8 rules enhancement were brought forward by Silva et al., [119], Costanzo et al. [120], and Bosco et al. [121].

Hence, the forthcoming issue of the European code for seismic design is expected to address several issues that have been stressed out by designers and addressed in the aforementioned research articles.

Beyond addressing current shortcomings in material choice and overstrength [6], the establishment of prequalified connection types is expected, which could leverage the use of partial-strength and semi-rigid connections, approaching European to American practice [43,101–104]. However, prequalified connections cannot solve alone many issues of concern, especially when other disputed code prescriptions remain [52,99,129].

Recent research on connections in moment resisting frames is expected to have an immediate impact on seismic design of bolted connections. That may be the case of the new connection concept idealized by Qian and Astaneh-Asl [144], with excellent ductile behaviour and adequacy to meet practical and code-prescribed demands, or of the specifications resulting from the European EQUALJOINT project [132]. In a different time horizon, the approach by Radmehr and Homami [136] may influence connections analysis, with explicit accounting of several significant sources of uncertainty. In fact, reliability-based approaches are expected to become increasingly prevalent in seismic analysis and design of structures and components [32–34].

On the other hand, promising developments in connections, such as replaceable, self-centring and slotted-holes connections or solutions with supplementary damping, most of those encapsulated within the partial strength and semi-rigid connections domain, have their applicability limited by current code constraints and may not be utilized unless prescribed as pre-qualified solutions.

Within the aforementioned group, the role of slipping connections, based on slotted-holes plates, on providing efficient and economical solutions for increasing damping at the system level by exploiting frictional mechanisms, has been consistently confirmed by different researchers.

Unlike what could have been expected, considering its ductility and aptness for forming replaceable solutions, research on bolted connections in eccentrically braced frames has not been profuse. Moreover, solutions such as slit dampers and shape memory alloys with superelastic properties for increasing damping with minimal residual deformation at the end of the earthquake shaking, or post-tensioned high strength strands for replaceable and self-centring connections may require further investigations before its immense potential can be put to practice, even if the latter has been one of the most prominent research topics in the connections' domain.

Considering the analysed recent research, partial-strength and semi-rigid connections stand out as the domain which encapsulates most of the potentially lightweight, efficient, and economical solutions, already in a mature state where conflicting conclusions in research are rare. Hence, creating regulatory conditions for their use may be the prospective step for technical authorities. On the other hand, connections in braced frames, in dual systems, to hollow sections, with supplementary damping or with self-centring and replaceability abilities seem to be still in need of further developments in the short term, providing promising research lines.

As novel industrial approaches accompanied by advances in material science are expected to drastically change the construction industry in the near future, with enhancements as the additive manufacturing, steel connections manufacture will probably evolve towards lightweight, low-carbon, and case-specific solutions. As a result, research in connection solutions for earthquake actions can become even more relevant if issues such as topology optimization, additive manufacturing materials, and techniques, as well as its design codes framework, are properly addressed.

6. Conclusions

This systematic review, targeting the intersection of bolted steel connections and seismic design with an eye on practice and the other on research, was able to identify the roots of scientific literature on the matter approximately 25 years ago. This period includes over 400 articles, 68% to 75% of which are research articles in journals, depending on the searched keywords, 22% to 29% are papers in conferences, and 1% or less are review articles. After a brief outlook on seismic design philosophy and regulations, assisted by recent research and reviews on these subjects, it has been concluded that the forthcoming leap in Eurocode 8, from the third-and-a-half generation to the fourth one, whereby already American FEMA 445, FEMA 695, and FEMA 750 codes exist, is due in the near future with expected developments in pre-qualified joints, material choice, and overstrength, sectional ductility. However, shifting to a full performance-based philosophy can multiply design complexity multiple folds.

Current European rules were assessed and systematized into a step-by-step guide for designing bolted steel connections exposed to earthquake actions, assisting practitioners and researchers with a tool that has not been published yet to the best of our knowledge. From such investigation and aided by a literature review, European standard current research gaps were identified.

Current significant research was examined and nested under 17 fields. Within each, a review was performed. It has been found out that bolted connections in concentrically braced frames, despite being a long-lasting research line, are still very active and in need of development. By contrast, the research in standard solutions for bolted connections for moment-resisting frames is now less prevalent, having most innovations inserted into the semi-rigid and partial-strength groups.

However, slipping connections for increased damping, connections to hollow and concrete-filled sections, self-centring, and replaceable solutions allocate a significant research interest and are deemed to produce sensible developments, mostly for moment-resisting frames. The downturn is likely to be the difficulty in leveraging its use under current and forthcoming seismic design regulations.

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Nomenclature

AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
BSL	Japanese Building Standard Law
CBF	Concentrically Braced Frames
CEN	European Committee for Standardisation
CFDST	Concrete Filled Double-skin Steel Tube
CFST	Concrete Filled Steel Tubular columns
DBD	Displacement-Based Design
DCH	High Ductility Class
DCM	Medium Ductility Class
DfD	Design for Disassembly
EBF	Eccentrically Braced Frames
EC	Elliptical Clearance Method

ECB	Exposed Column–Base
EFTA	European Free Trade Association
EP	End-Plate
EU	European Union
FCP	Flange Cover Plates
FEA	Finite Element Analyses
HPDF	High-Performance Dissipating Frame
HSS	High Strength Steel
HSS	High Strength Strands
HSS	Hollow Structural Sections
IBC	International Building Code
IDA	Incremental Dynamic Analyses
IFBD	Improved Force-Based Design
IMK	Ibarra–Medina–Krawinkler model
IV-CBF	Concentrically Braced Frames with Inverted-V Braces
KBF	K Braced Frames
LCF	Low-cycle Fatigue
LYP	Low Yield Point steel
MDOF	Multiple Degree of Freedom
MRF	Moment Resisting Frames
N-CBF	Concentrically Braced Frames with N Braces
NBC	National Building Code
PBCSC	Prefabricated Beam-Column Steel Connections
PBCSJ	Prefabricated Beam-Column Steel Joints
PBSD	Performance-Based Seismic Design
PEF	Post-Earthquake Fire
PR	Partially Restrained connections
PSJ	Prefabricated Steel Joint
PT	Post-Tensioned
PTFE	Polytetrafluoroethylene
PTHSS	Post-Tensioned High Strength Strands
RBS	Reduced Beam Section
RFCS	Research Fund for Coal and Steel
RSBC	Rotational Slotted Bolted Connections
RSM	Response Surface Methodology
SBC	Slotted Bolted Connections
SBC	Standard Building Code
SLC	Standard Linear Clearance Method
SMA	Shape Memory Alloy
SRSM	Simplified Response Surface Method
SSBC	Shear Slotted Bolted Connections
SSL-SSBC	Short Shear Link with Shear Slotted Bolted Connection
SZBF	Suspended Zipper Braced Frames
UBC	Uniform Building Code
ULCF	Ultra-Low Cycle Fatigue
V-CBF	Concentrically Braced Frames with V Braces
WUF-W	Welded Unreinforced Flange Web
X-CBF	Concentrically Braced Frames with X Diagonal Braces
Z-CBF	Concentrically Braced Frames with Z Braces
ZBF	Zipper Braced Frames

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