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Review

Survey on the Role of Beam-Column Connections in the Progressive Collapse Resistance of Steel Frame Buildings

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Abstract: The behavior of steel frame buildings under progressive collapse conditions depends on a combination of several parameters, including the interplay between different collapse resistance mechanisms that are mobilized in different structural components. Previous studies have shown that the extent to which these mechanisms may contribute to progressive collapse resistance depends on the ability of the beam-column connections to undergo large inelastic deformations prior to reaching their deformation capacity limits. For this reason, and due to the important role of their flexural strength and tying capacity in the development of essential collapse resistance mechanisms, the response of beam-column connections is one of the most important features of progressive collapse performance. Based on the knowledge gained through previous studies on the mechanics of this problem, the role of these connections are critically reviewed in this paper by examining the results of several experimental studies that have been conducted during the past decade. The factors that may adversely affect progressive collapse resistance–such as the failure modes of certain connection types–are evaluated, and novel approaches to limiting these factors, which are currently under development, are reviewed. The assessment of these parameters leads to useful conclusions of practical significance while highlighting the aspects of these problems that need further study and understanding.

Keywords: beam axial force; bending moment; catenary action; column loss; composite action; compressive arching; rotation capacity; structural robustness

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1. Introduction

Progressive collapse represents a chain-reaction form of structural failure, which usually starts with local damage that progressively spreads to neighboring structural elements and may ultimately result in the collapse of a substantial part of the structural system [1]. Possible triggering events can be accidental actions (e.g., gas explosion, impact, or fire), natural disasters (e.g., earthquakes, hurricanes, etc.), malicious actions, and construction errors. Severe local damage caused by these actions usually results in large deformations in the immediately affected area, which generates large strains and can lead to material nonlinearity and second-order geometry effects. In building structures, the main collapse propagation mechanisms involve the separation of structural members, the collision of failed components, and the instability of members. Resistance to progressive collapse ensures that the extent of failure will not be disproportionate to the triggering event [2]. The ability of a structure to sustain initial damage without suffering disproportionate collapse is referred to as "structural robustness" [3].

The traditional methods that are available in current design codes for assessing the resistance of building structures to progressive collapse have mainly been derived from the work conducted in the UK following the Ronan Point collapse [4]. In their simplest form, these methods are mainly prescriptive since they only impose certain conditions on the basis that their inclusion should simply ensure a potentially better performance. One such approach is the provision of a suitable level of tying resistance in structural components, with the aim of increasing the degrees of continuity, ductility, and load transfer capacity.

Various forms of this tying force method have been addressed in current design codes such as the Eurocodes [5] and the US guidelines for structural robustness [6,7]. However, these approaches do not account for all the key features of the problem, and therefore, they cannot provide a reliable indication of progressive collapse resistance [8–12].

The Eurocode tying force method assumes that beam members and their end connections act as horizontal ties in the event of local damage provided that they possess a minimum tying resistance which can be simply defined as a function of the beam span and the beam design gravity load [5]. A more representative approach that also considers the importance of ductility in these connections, and thus the capability to bridge over a damaged supporting member whilst undergoing large deformations, has been provided in more recent codes [7]. However, apart from the requirements of minimum tying resistance and ductility limits, several other parameters should also be considered, such as dynamic effects, the type of loading, and the interaction with adjacent structural members [13,14].

Since general interest in the more scientific treatment of this problem has heightened significantly during the past two decades, especially after the 9/11 events, the need for a sound understanding of the mechanics of progressive collapse has become more important [15]. Research activities have focused on changing the design basis from prescriptive requirements to performance-based methods that rely on understanding, modeling, and quantitative assessments. Therefore, particular attention has been given to the study of this problem through the "alternate load path" approach, which allows the quantitative assessment of structural redundancy–which is a key component of structural robustness and an essential requirement in resisting progressive collapse–based on the analysis of the mechanics of structural behavior [16–21].

The alternate load path method is usually applied by considering a threat-independent loss of a load-bearing element—e.g., a column member–and by examining the consequences of this action on the remaining structure [22]. It may incorporate essential features of the real structural behavior, such as dynamic effects (provided the load-bearing member is removed instantaneously to simulate the scenarios of blast or impact), loading redistribution, large deformations, material nonlinearity, and second-order geometry effects. Gravity loading that was initially carried by the missing element could effectively be redistributed to the remaining structure through the activation of different collapse resistance mechanisms. Progressive collapse may therefore be arrested once the structure attains a new state of stable equilibrium; otherwise, the increased loading and deformation demands could cause separation at key locations, which may lead to collapse propagation. Most likely, separations may occur at the connections between structural components, and thus, the response of these connections represents a key feature of progressive collapse behavior [23].

The role of beam-column connections in the progressive collapse resistance of steel frame buildings is the subject of this paper, with the aim of reviewing the current state of research and highlighting limitations and issues that require further study. First, the different collapse resistance mechanisms mobilized in steel frame structures under the action of notional column removal are described based on the findings from previous studies. Attention is then focused on the double-span beam mechanism, which represents the lowest level of structural idealization in a progressive collapse analysis, but it is considered particularly suitable for studying the behavior of beam-column connections. While essential links are defined between the key properties of beam-column connections (i.e., rotational stiffness, flexural strength, tensile resistance, and post-limit tensile stiffness) and the different collapse resistance mechanisms mobilized in these beams, the extent to which these collapse resistance mechanisms influence overall performance largely depends on the deformation capacity available in these connections. Certain information about the importance of this parameter can only be obtained through experimental tests; therefore, emphasis is placed on the results of relevant experimental studies. The average deformation capacity limits of conventional steelwork connection arrangements, in conjunction with the effects of slab reinforcement in the presence of composite action, can be evaluated in relation to their expected influence on the overall progressive collapse resistance. The value of innovative approaches under development that are aimed at enhancing the progressive collapse response of beam-column connections is also discussed. Based on the most important conclusions, several aspects of the problem that need further study and understanding are also identified.

2. Collapse Resistance Mechanisms Mobilized by Column Removal

The column removal approach involves the assessment of structural response to the notional removal of a single column. Although a column loss scenario may affect the overall behavior of the structure, simplified idealizations can be considered in the analysis [24], as shown in Figure 1. The basic concept is that performance at a higher level of structural idealization can be assessed based on the detailed modeling of the response at a lower level. This simplification, however, assumes that certain conditions are met. In particular, the interaction with the surrounding structure is simulated by appropriate boundary conditions, where consideration may be given only to the affected bay instead of the full structure. Furthermore, if it is proved that the stability of the remaining columns is not a critical parameter for structural resistance, a reduced model comprising only the floors above the level of the initial damage may be considered. Provided that these floors are identical in terms of structure and loading, the model may be further restricted to a single-floor system. By dividing the floor system into two main constitutive components and by setting aside the contribution of the slab, this model is simplified to a grillage structure. The response of a grillage model following column removal can be established based on the responses of the constitutive beams by employing a simplified assembly approach [24], which allows each beam system to be analyzed individually [10,20,25].

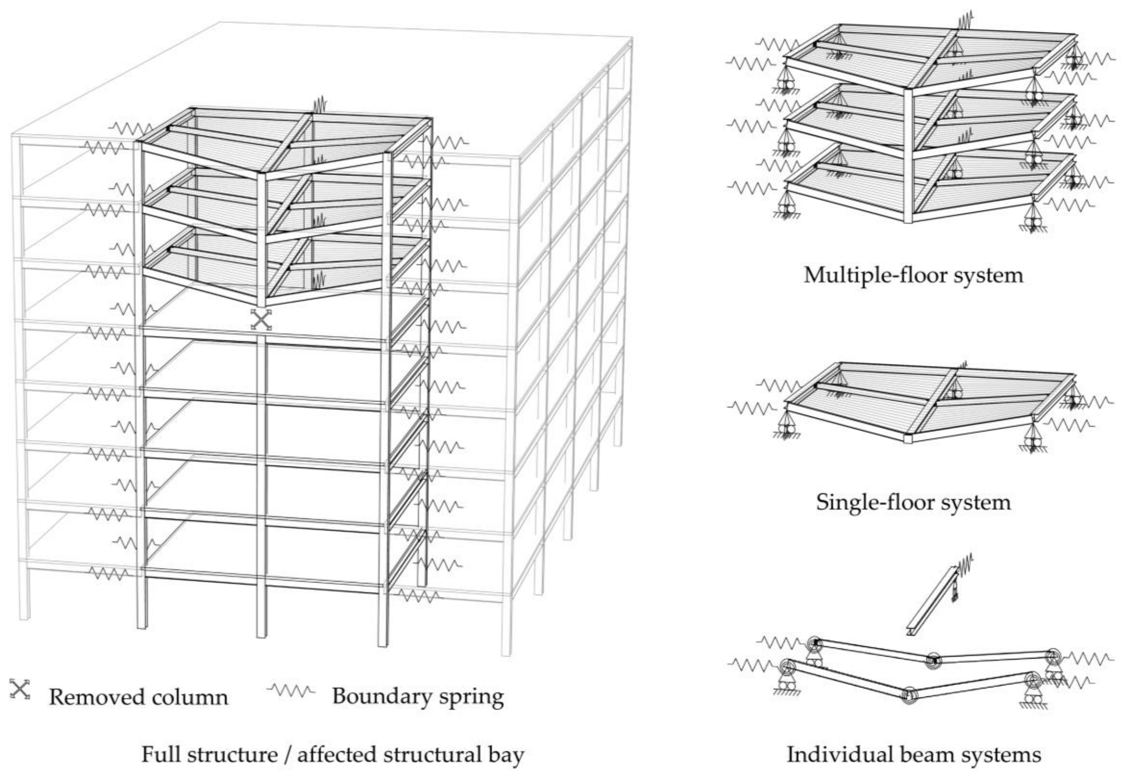


Figure 1. Levels of structural idealization for progressive collapse assessment [25].

Unless the removed column is located at the corner of the structure [26], this latter simplification leads to the concept of a double-span beam mechanism that is created by two adjacent beams after the removal of the intermediate column, which represents a common approach for examining the progressive collapse behavior of beams and beam-column connections, as further discussed in the next section. In the presence of an axial restraint, the progressive collapse resistance of a double-span beam system can be enhanced by the mobilization of different load resistance mechanisms, such as a compressive arching action and/or tensile catenary action. Although these mechanisms may have a substantial contribution to the overall structural performance, the resistance of the entire structure is usually affected by several additional factors, such as the collapse resistance mechanisms mobilized in the slabs when subject to large deflections, the presence of infill walls and/or diagonal bracing in the affected structural bays, and the load-carrying capacity of the remaining columns. The importance of these additional parameters in the progressive collapse resistance, and especially their interaction with the contribution of beams and beam-column connections, should be taken into account.

Floor slabs could contribute to structural resistance and progressive collapse through membrane action. In the presence of lateral restraint, compressive membrane action may increase the punching shear capacity at small deflections, but this usually has only a little effect on collapse resistance [27]. On the other hand, tensile membrane action could considerably enhance the resistance to progressive collapse at large deflections [28–31]. This performance could be supported by the presence of a lateral restraint and/or by compressive ring action, which can enable self-equilibration [32–34]. However, some studies have shown that, although tensile membrane action can make a significant contribution to resistance against progressive collapse, it cannot form the principal collapse-resisting mechanism in the absence of other load redistribution paths [28,35]. Therefore, a comparable contribution from the framework of the supporting beams and their end connections through the effective activation of flexural, arching, and/or tensile catenary actions is required [10,20,36–39].

Recent studies have examined the progressive collapse behavior of steel frame structures in the presence of masonry infill walls on structural bays above the removed column. A numerical study reported in [40] revealed that infill walls might considerably contribute to collapse resistance, but they could also increase the stiffness of the structure and thus lead to different failure modes. In a different numerical study [41], it was shown that the presence of openings on the walls might affect performance, while the extent to which infill walls could enhance structural resistance depends on the structural properties of beam-column connections. The experimental study conducted by Qian et al. [42] focused exclusively on the possible effects of infill walls and on the progressive collapse resistance against corner column removal. It was found that the progressive collapse resistance of corner areas of steel frame structures–which are usually prone to progressive collapse due to the absence of sufficient load redistribution paths–was extremely enhanced by the development of effective compressive struts in the walls, which could contribute to the transfer of gravity loading from the exposed area to the surrounding structure.

The presence of steel bracing members in structural bays above the missing column could also influence progressive collapse resistance, as demonstrated by recent experimental and numerical studies. This, however, depends on the type of bracing system since X-bracing, for instance, may ensure increased resistance while V-bracing could have a detrimental effect on performance [43]. Concentric diagonal bracing can affect performance in very different ways depending on whether the bracing members are subject to tension or compression following column loss [44]. These conflicting conclusions indicate that the response of bracing systems should not be considered a reliable collapse resistance mechanism; however, instead, special attention should be paid to their possible negative influence. The only exception is probably the cases of corner areas, which are extremely prone to progressive collapse due to the absence of effective alternate load paths, the response of which could benefit from any type of bracing system [45].

So far, research studies have mainly focused on the behavior of specific structural components such as beams, connections, and slabs. One main reason is that these components have a major contribution to the overall structural resistance to progressive collapse. Another reason is that the problem, whether approached by experimental, numerical, or analytical methods, is considerably simplified when focusing on a reduced model of structural idealization. Few experimental studies have considered the behavior of two-story [46,47] or three-story [48,49] plane frames following simulated column loss. These studies were mainly aimed at identifying possible differences between the contribution of different stories to the total resistance. However, what is probably more important in a multi-story frame structure is the stability of the surrounding columns. While it is necessary to make sure that beams and slabs can safely transfer their loads to the remaining columns, the latter should also be able to carry these loads. Very few studies have focused exclusively on this problem [50–55], but their findings indicate that the bucking of the remaining columns is a possible mode of failure in a column removal scenario, which may lead to extensive collapse propagation.

3. Basic Features of the Double-Span Beam Mechanism

3.1. Mechanics of Structural Behavior

As noted previously, the present study focuses on the role of beam-column connections in the progressive collapse response of steel frame buildings. The double-span mechanism created by two adjacent beams following the removal of the intermediate column, as illustrated in Figure 2 [56], represents a simple approach for examining the progressive collapse behavior of beams and beam-column connections. This is because the double-span beam mechanism can incorporate the basic characteristics of this behavior through the detailed modeling of beams and beam-column connections. In the presence of a concrete slab, the beams can be modeled as composite elements based on the effective breadth concept. Detailed representations of beam-column connections could be considered, where the rebar contribution in the presence of composite action can be taken into account.

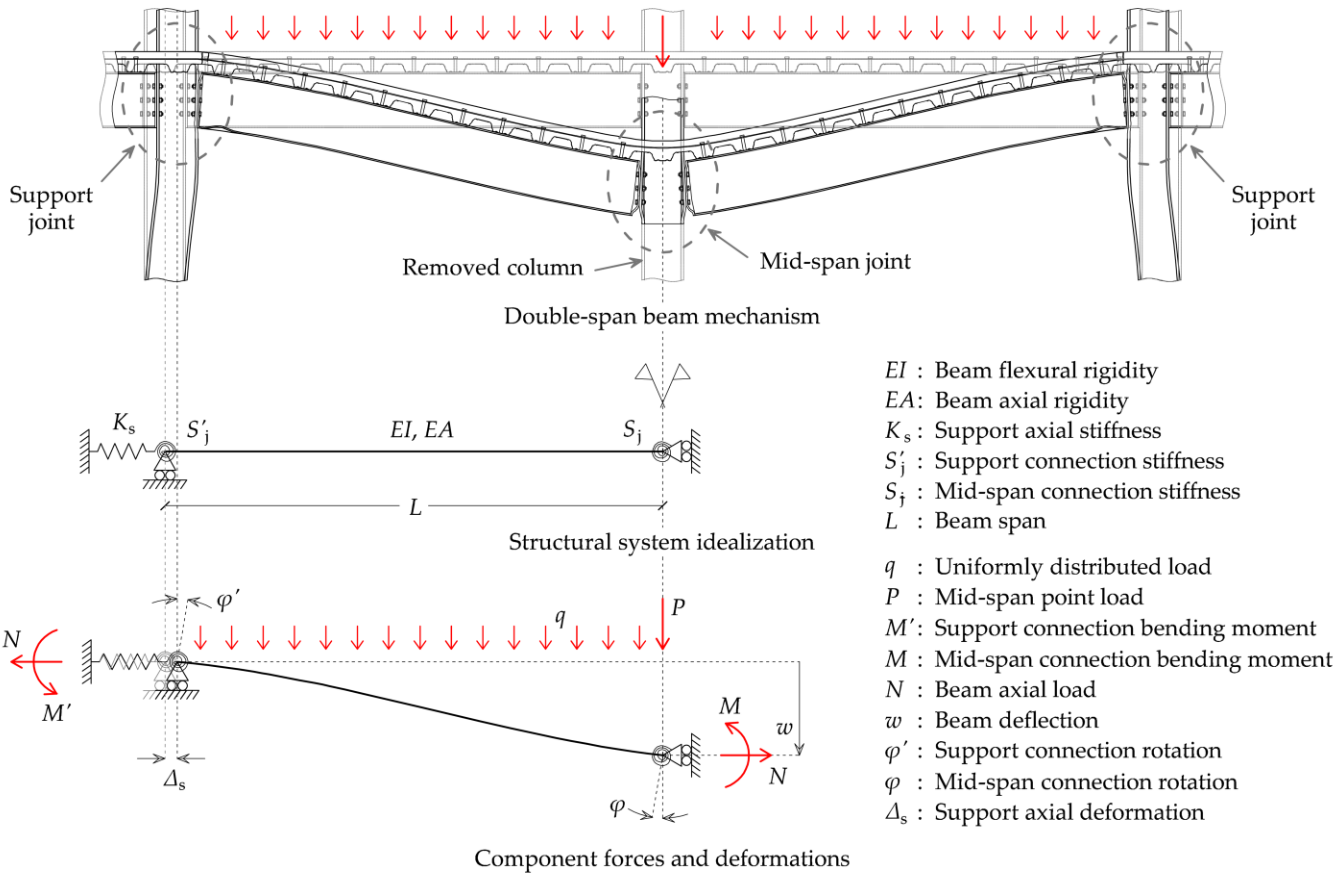


Figure 2. Representation of the double-span beam mechanism [56].

The axial restraint provided by the surrounding structure should be simulated by appropriate boundary conditions. The degree of axial restraint may depend on several parameters [49], such as the position of the removed column, the number of floors above the removed column, the lateral stiffness of the frame structure, and the type of lateral force-resisting system. However, unless the support axial stiffness is substantially small compared to the axial stiffness of the beams, it barely influences the performance, especially at large deflections (i.e., during the tensile catenary stage) where the stiffness of the beam-column connections is considerably reduced [57].

Assuming a symmetric structure (e.g., equal spans for the two adjacent beams), this response could be analyzed based on the structural model shown in Figure 2, where various structural parameters that influenced the performance are identified. A suitable representation of the connection behavior is required, which explicitly accounts for the connection moment-rotation response in the presence of an axial force [58–61]. The behavior of the structure is described by a set of component forces (i.e., beam axial force and connection bending moments) and deformations (i.e., beam deflection, connection rotations, and support axial deformation), as shown in Figure 2.

A representation of the typical behavior of a double-span beam system is given in Figure 3 [62]. In practice, it is not always possible to reach every stage of the response since the deformation capacity can be exhausted at any point along the load–deflection curve [10,23,25,61–63]. The comparison with the response of an equivalent axially unrestrained system demonstrates the significant effects of axial restraint. The axial force generated in axially restrained beams may mobilize the compressive arching and tensile catenary actions at small and large deflections, respectively. In both cases, the load-carrying capacity could considerably increase. As shown in Figure 3, the maximum compressive arching capacity is reached at a beam deflection less than the beam depth (D), while tensile catenary action is mobilized when the beam deflection became greater than twice the beam depth ($2D$).

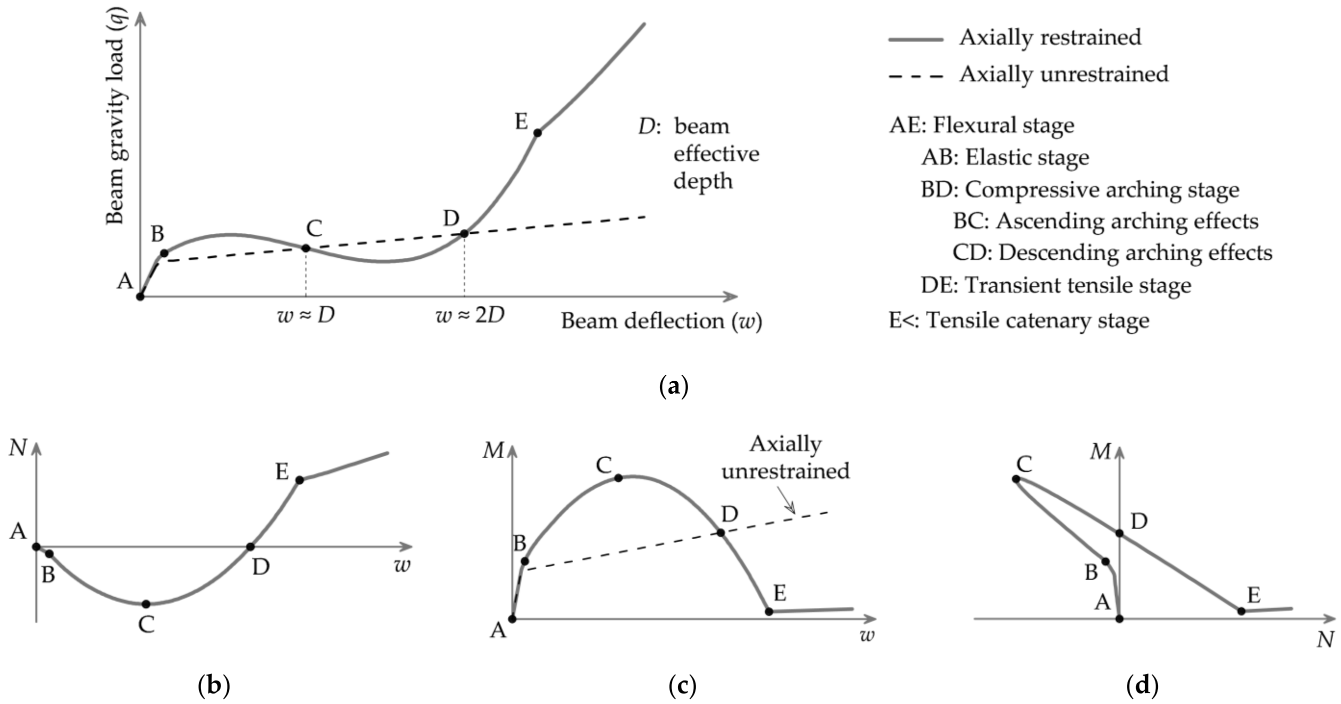


Figure 3. Structural response following column removal involving elastic compressive arching action [62]: (**a**) Beam load-deflection; (**b**) Beam axial force; (**c**) Connection bending moment; (**d**) Connection moment–axial force interaction.

The above conclusions only concern the case of elastic compressive arching behavior, which means that all structural components that are subject to axial compression (e.g., beam compression flanges) exhibit linear elastic behavior [64]. When the resistance of any of the components participating in the development of the compressive arching action is exhausted, the system response changes as described in the examples in Figure 4 [57]. Previous studies [57,64] have shown that compressive arching action may still occur, provided that the beam-column connections are classified as partial-strength; that is, the connection tension zone resistance is less than the connection compression zone resistance. In this case, the beam deflection corresponding to the maximum compressive arching capacity decreases, while tensile catenary action is activated earlier, at a deflection value between the beam depth (*D*) and at twice the beam depth (2*D*). The response of a system with full-strength connections–which practically corresponds to the case when the connection tension zone's resistance is no less than the connection compression zone resistance–can only be governed by flexural action at small deflections, followed by the tensile catenary action at large deflections in the presence of an axial restraint. In this case, tensile catenary action is activated when the beam deflection equals the beam depth (*D*). On the other hand, beam systems with nominally pinned connections might resist gravity loading only through tensile catenary action. This is essentially equivalent to the absence of connection compression resistance, which means that connections are not able to resist bending moments [64].

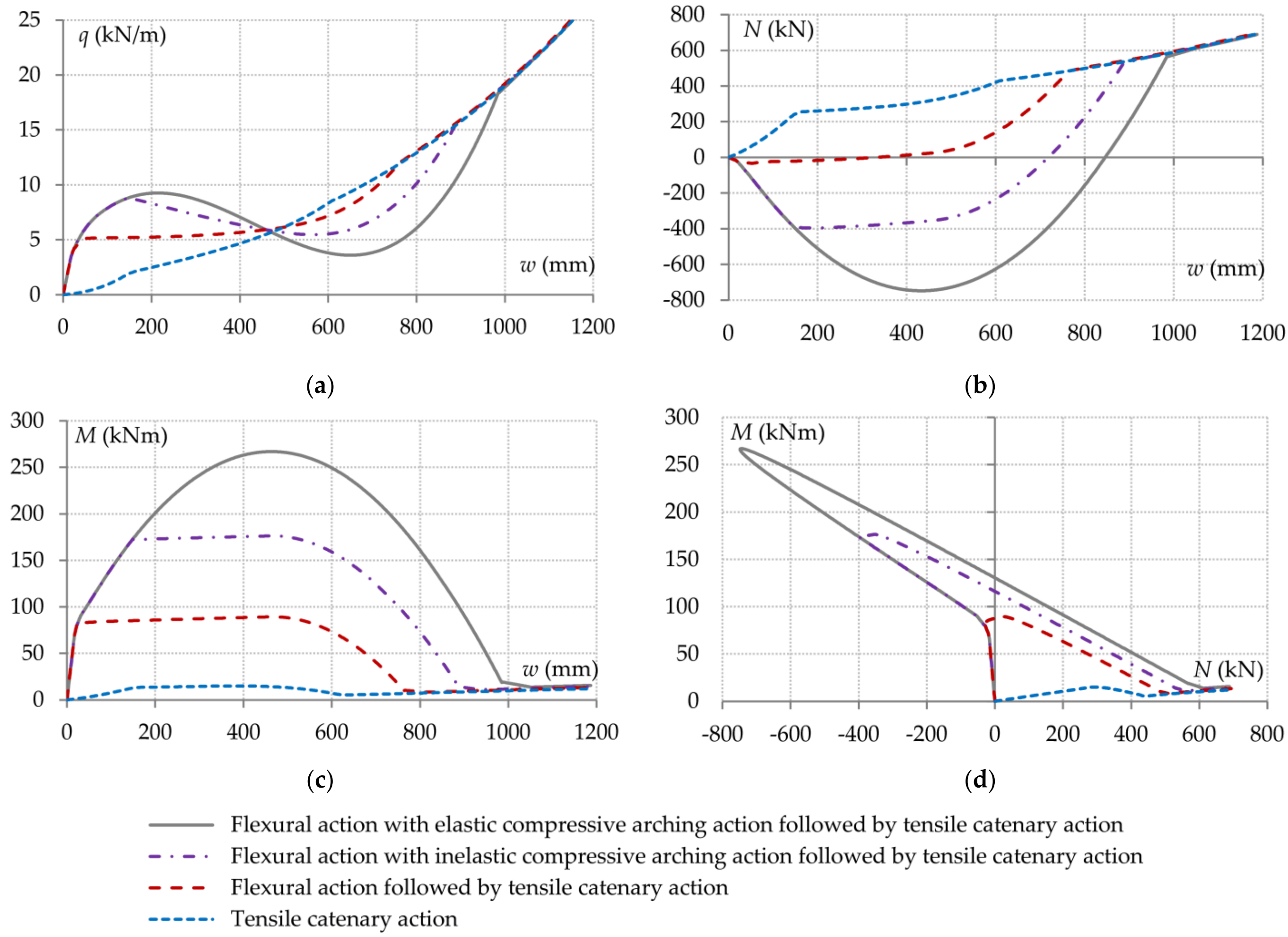


Figure 4. Comparison between responses governed by different collapse resistance mechanisms [57]: (**a**) Load–deflection curves; (**b**) Beam axial forces vs. beam deflections; (**c**) Connection bending moments vs. beam deflections; (**d**) Connection bending moments vs. beam axial forces.

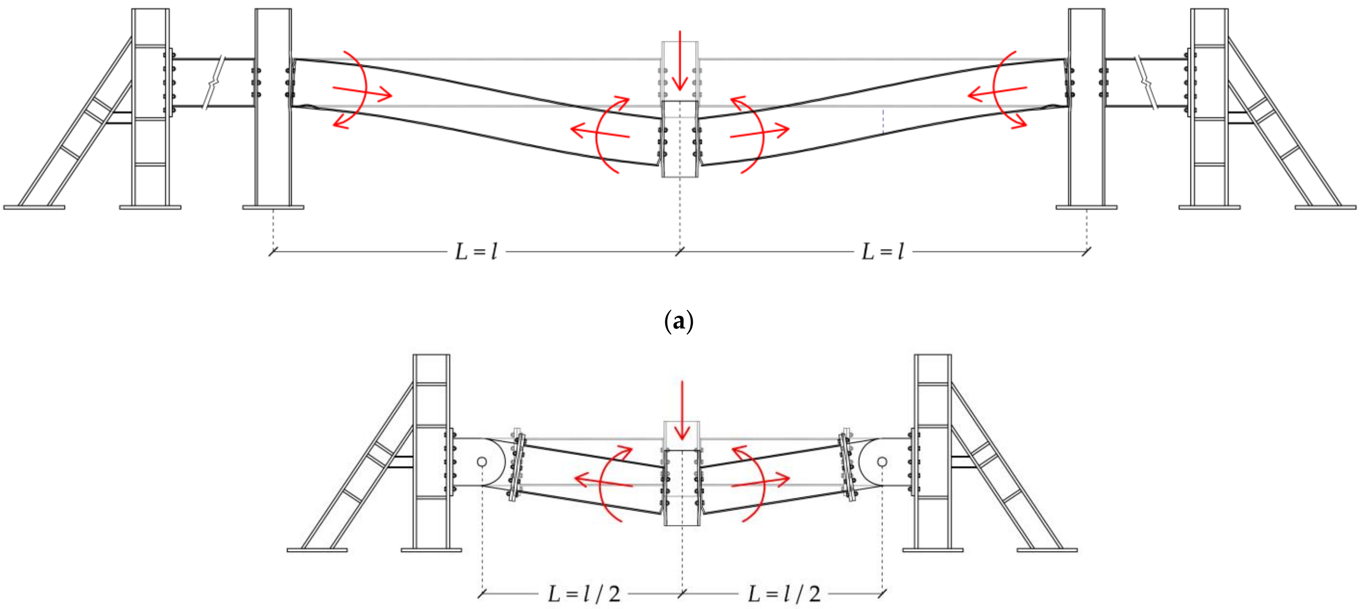
The previous studies also demonstrated how different collapse resistance mechanisms are affected by various structural parameters [64,65]:

- The flexural action effect increases as the beam span decreases and/or the bending moment capacities of the beam-column connections increase.
- The compressive arching action effect in creases as the beam span-to-depth ratio decreases, and/or the compression zone resistances of connections increase, and/or the degree of axial restraint increases.
- The tensile catenary action effect increases as the beam span decreases, the tensile resistance of the connections increase, the post-limit tensile stiffness of the connections increase, and/or the degree of axial restraint increases.

While the effects of flexural action and the compressive arching action are solely dependent on the corresponding parameters listed above because they govern performance at small deflections, the effects of tensile catenary action also depend on the deformation capacity available in the connections. For a better understanding of the extent to which this parameter affects the development of tensile catenary action, the results of relevant experimental studies should be considered.

3.2. Common Configurations of Test Specimens

In relevant experimental studies, one of the two main test configurations shown in Figure 5 is usually adopted. In both cases, the test is conducted under static loading by applying an incremental point load at the mid-span under displacement control until failure. The first configuration shown in Figure 5a represents a complete double-span beam system that was created after column loss, which comprises the midspan region and two support regions, including details of the corresponding beam-column connections. The second configuration shown in Figure 5b represents a simplification of the double-span mechanism. On the presumption that the point of inflection is located at the mid-span of each beam section only a half of the section is considered, and the supports are modeled as hinged. This configuration is probably more appropriate for studying the behavior of the midspan joint rather than the response of the complete double-span beam system.



(**b**)

Figure 5. Common configurations of test specimens (where *l* is the span of the beam) [65]: (**a**) Detailed; (**b**) Simplified.

Previous experimental studies that have employed a simplified setup approach have provided essential information about the behavior of different connection types, especially in relation to their potential contribution to the load-carrying capacity and the deformation capacity of the structure. Nevertheless, the focus of these studies has been the prediction of the load-deflection response. It should be noted, however, that the results obtained from a simple setup configuration should be carefully evaluated because these could lead to inaccurate conclusions regarding the significance of each collapse-resisting mechanism [65]. In particular, according to the information given in Section 3.1:

- The span length influences each collapse resistance mechanism differently.
- The exclusion of support connections results in a decrease in the flexural action effects.
- The use of hinged supports results in a decrease in the effective depth (D) used for the prediction of the span-do-depth ratio, which influences the compressive arching action effects.
- The replacement of the support connections by axially stiff pinned supports increases the axial stiffness of the system and, therefore, enhances the catenary action effects.

Therefore, the load–deflection curves obtained using the simplified configuration in Figure 5b may underestimate the potential effects of flexural and compressive arching actions while overestimating the effects of tensile catenary action.

4. Summary and Review of Experimental Studies

4.1. Characteristics of Test Specimens

A list of several experimental studies that examined the behavior of steel and composite double-span beam structures is given in Table 1. These studies are described in terms of their structure type (bare steel or steel–concrete composite beams), the test setup configuration (detailed or simplified, as defined in the previous section), the beam span length (L), the beam depth (D), and the type of steelwork connections employed. Their main results, as defined in terms of the main parameters that governed the different collapse resistance mechanisms described in the previous section, are collected in Table 2. In addition to the span-to-depth ratio (L/D), which has a direct influence on the compressive arching action, two other ratios were defined:

- The ratio between the ultimate load-carrying capacity at a level of deflection greater than the beam depth (P_{u}) and the maximum value of the load-carrying capacity up to a deflection level equal to the beam depth ($P_{w \leq D}$).
- The ratio between the ultimate deflection (w_{u}) and the beam depth (D).

A value of the former ratio ($P_{\mathrm{u}}/P_{w \leq D}$) greater than 1.0 indicates that the progressive collapse resistance had been enhanced by tensile catenary action since catenary action effects might develop beyond $w = D$ unless tensile catenary action is the only collapse resistance mechanism (i.e., in the absence of flexural action and/or compressive arching action). A value of the latter ratio (w_{u}/D) less than 1.0 indicates that the structure exhibited a limited deformation capacity (thus, it has failed during the flexural/arching stage), while a value greater than 2.0 indicates that the structure exhibited a high deformation capacity (thus, it failed after the development of substantial tensile catenary action forces).

In accordance with the values of these ratios, and based on further information obtained from each study, the collapse resistance mechanisms (i.e., flexural action, compressive arching action, and tensile catenary action) that were activated in each specimen are also specified in Table 2. The last column of the table defines the failure mode, which is the mode of failure of the most critical connection component that essentially triggers the overall failure of the specimen. The load–deflection curves of selected specimens from those listed in Tables 1 and 2 are shown in Figure 6, on which the various key parameters described above are identified.

Table 1. Summary of test specimens employed in experimental studies.

Specimen No.	References	Structure Type	Test Setup Configuration	Beam Span (mm) *	Beam Depth (mm) **	Steelwork Connection Type ***
1	Demonceau and Jaspart, 2010 [66]	Composite	Detailed	4000	260	Flush endplate
2a	Yang and Tan, 2013 [67]	Bare steel	Simplified	2434	303.4	Web cleat
2b	Yang and Tan, 2013 [67]	Bare steel	Simplified	2434	303.4	Top-seat angles
2c	Yang and Tan, 2013 [67]	Bare steel	Simplified	2434	303.4	TSWA
2d	Yang and Tan, 2013 [67]	Bare steel	Simplified	2434	303.4	Fin plate
2e	Yang and Tan, 2013 [67]	Bare steel	Simplified	2434	303.4	Flush endplate
2f	Yang and Tan, 2013 [67]	Bare steel	Simplified	2434	303.4	Extended endplate
3a	Lew et al., 2013 [68]	Bare steel	Detailed	6096	539.5	WUF-B
3b	Lew et al., 2013 [68]	Bare steel	Detailed	6096	615.95	RBS
4	Guo et al., 2013 [69]	Composite	Detailed	2000	300	Welded
5	Guo et al., 2015 [70]	Composite	Detailed	2000	300	Flush endplate
6a	Li et al., 2015 [71]	Bare steel	Simplified	2250	300	WUF-B (1)
6b	Li et al., 2015 [71]	Bare steel	Simplified	2250	300	WUF-B (2)
7a	Wang et al., 2016 [72]	Bare steel	Simplified	2250	300	Welded
7b	Wang et al., 2016 [72]	Bare steel	Simplified	2250	300	WUF-B
8a	Yang et al., 2016 [73]	Composite	Detailed	3000	282	Web cleat
8b	Yang et al., 2016 [73]	Composite	Detailed	3000	282	Flush endplate
9a	Dinu et al., 2017 [74]	Bare steel	Detailed	3000	220	WCF-B
9b	Dinu et al., 2017 [74]	Bare steel	Detailed	3000	220	Haunch endplate
9c	Dinu et al., 2017 [74]	Bare steel	Detailed	3000	220	RBS
9d	Dinu et al., 2017 [74]	Bare steel	Detailed	3000	220	Extended endplate
10a	Zhong et al., 2017 [75]	Bare steel	Detailed	1500	150	WUF-B
10b	Zhong et al., 2017 [75]	Bare steel	Detailed	1500	150	TSWA
10c	Zhong et al., 2017 [75]	Bare steel	Detailed	1500	150	Web cleat
11	Li et al., 2017 [76]	Bare steel	Simplified	1200	300	WUF-B
12a	Xu et al., 2018 [77]	Bare steel	Simplified	1800	244	Flush endplate
12b	Xu et al., 2018 [77]	Bare steel	Simplified	1800	244	Extended endplate
12c	Xu et al., 2018 [77]	Bare steel	Simplified	1800	244	Stiffened angle
13a	Gao et al., 2019 [78]	Bare steel	Simplified	1800	244	Stiffened angle (1)
13b	Gao et al., 2019 [78]	Bare steel	Simplified	1800	244	Stiffened angle (2)
13c	Gao et al., 2019 [78]	Bare steel	Simplified	1800	244	Stiffened angle (3)
14a	Alrubaidi et al., 2020 [79]	Bare steel	Detailed	2000	194	Fin plate
14b	Alrubaidi et al., 2020 [79]	Bare steel	Detailed	2000	194	WUF-B
14c	Alrubaidi et al., 2020 [79]	Bare steel	Detailed	2000	194	Extended endplate
15a	Meng et al., 2020 [80]	Composite	Detailed	1500	205	WUF-B
15b	Meng et al., 2020 [80]	Composite	Detailed	1500	205	WUF-B/RWS
16a	Qiao et al., 2020 [81]	Bare steel	Simplified	1520	200	Welded
16b	Qiao et al., 2020 [81]	Bare steel	Simplified	1520	200	RBS
16c	Qiao et al., 2020 [81]	Bare steel	Simplified	1520	200	RBS-RWS
17a	Lin et al., 2021 [82]	Bare steel	Simplified	1510	200	Welded
17b	Lin et al., 2021 [82]	Bare steel	Simplified	1510	200	RWS (1)
17c	Lin et al., 2021 [82]	Bare steel	Simplified	1510	200	RWS (2)
18a	Kukla and Kozlowski, 2021 [83]	Bare steel	Simplified	945	300	Flush endplate
18b	Kukla and Kozlowski, 2021 [83]	Bare steel	Simplified	945	300	Extended endplate

* The beam span is defined for each test setup configuration as shown in Figure 5. ** The beam depth includes the height of the slab in the presence of composite action. *** TSWA: top/seat and web angle; WUF-B: welded unreinforced flange-bolted web; WCF-B: welded cover plate flange-bolted web; RBS: reduced beam section; RWS: reduced web section.

Table 2. Summary of results obtained from experimental studies.

Specimen No.	References	L/D *	$P_u/P_{w \le D}$ **	w_u/D ***	Mechanisms **** FA	CAA	TCA	Failure Mode
1	Demonceau and Jaspart, 2010 [66]	15.38	1.92	2.38	✓		✓	Rebar rupture
2a	Yang and Tan, 2013 [67]	8.02	1.60	1.22			✓	Web angle fracture
2b	Yang and Tan, 2013 [67]	8.02	1.00	2.01	✓		✓	Flange angle fracture
2c	Yang and Tan, 2013 [67]	8.02	1.36	1.21	✓		✓	Flange angle fracture
2d	Yang and Tan, 2013 [67]	8.02	-	0.86			✓	Bolt shear failure
2e	Yang and Tan, 2013 [67]	8.02	1.28	1.15	✓		✓	Bolt tensile fracture
2f	Yang and Tan, 2013 [67]	8.02	0.54	1.48	✓		✓	Weld fracture
3a	Lew et al., 2013 [68]	11.30	0.62	1.33	✓	✓	✓	Bolt shear failure
3b	Lew et al., 2013 [68]	9.90	1.17	1.40	✓		✓	Beam flange fracture
4	Guo et al., 2013 [69]	6.67	1.16	1.47	✓		✓	Weld fracture
5	Guo et al., 2015 [70]	6.67	-	0.95	✓	✓		Bolt tensile fracture
6a	Li et al., 2015 [71]	7.50	1.09	1.13	✓			Beam flange fracture
6b	Li et al., 2015 [71]	7.50	1.00	1.30	✓			Beam flange fracture
7a	Wang et al., 2016 [72]	7.50	-	0.67	✓			Beam flange fracture
7b	Wang et al., 2016 [72]	7.50	0.93	1.33	✓			Beam flange fracture
8a	Yang et al., 2016 [73]	10.64	2.89	2.27	✓	✓	✓	Web angle fracture
8b	Yang et al., 2016 [73]	10.64	2.32	2.04	✓	✓	✓	Bolt tensile fracture
9a	Dinu et al., 2017 [74]	13.64	2.31	2.36	✓		✓	Flange plate fracture
9b	Dinu et al., 2017 [74]	13.64	1.85	2.00	✓		✓	Bolt tensile fracture
9c	Dinu et al., 2017 [74]	13.64	2.00	2.18	✓		✓	Beam flange fracture
9d	Dinu et al., 2017 [74]	13.64	0.36	1.66	✓			Bolt tensile fracture
10a	Zhong et al., 2017 [75]	10.00	1.09	1.13	✓			Beam flange fracture
10b	Zhong et al., 2017 [75]	10.00	2.65	2.37	✓		✓	Flange angle fracture
10c	Zhong et al., 2017 [75]	10.00	5.69	2.83			✓	Web angle fracture
11	Li et al., 2017 [76]	4.00	0.82	1.03	✓			Beam flange fracture
12a	Xu et al., 2018 [77]	7.38	2.00	1.27	✓		✓	Bolt thread stripping
12b	Xu et al., 2018 [77]	7.38	1.02	1.27	✓		✓	Bolt thread stripping
12c	Xu et al., 2018 [77]	7.38	1.11	1.17	✓		✓	Bolt pull-out
13a	Gao et al., 2019 [78]	7.38	-	0.74	✓	✓		Bolt pull-out
13b	Gao et al., 2019 [78]	7.38	1.11	1.19	✓		✓	Bolt pull-out
13c	Gao et al., 2019 [78]	7.38	-	0.94	✓	✓		Bolt pull-out
14a	Alrubaidi et al., 2020 [79]	10.31	2.52	1.80			✓	Fin plate fracture
14b	Alrubaidi et al., 2020 [79]	10.31	1.05	1.16	✓			Beam root fracture
14c	Alrubaidi et al., 2020 [79]	10.31	0.71	2.35	✓		✓	Bolt thread stripping
15a	Meng et al., 2020 [80]	7.32	1.16	1.34	✓		✓	Beam flange fracture
15b	Meng et al., 2020 [80]	7.32	1.67	1.78	✓		✓	Beam flange fracture
16a	Qiao et al., 2020 [81]	7.60	-	0.90	✓		✓	Weld fracture
16b	Qiao et al., 2020 [81]	7.60	1.23	1.15	✓		✓	Beam flange fracture
16c	Qiao et al., 2020 [81]	7.60	1.26	1.18	✓		✓	Beam flange fracture
17a	Lin et al., 2021 [82]	7.55	-	0.93	✓		✓	Weld fracture
17b	Lin et al., 2021 [82]	7.55	1.00	1.55	✓		✓	Beam flange fracture
17c	Lin et al., 2021 [82]	7.55	1.20	1.13	✓		✓	Weld fracture
18a	Kukla and Kozlowski, 2021 [83]	3.15	-	0.33	✓			Bolt tensile fracture
18b	Kukla and Kozlowski, 2021 [83]	3.15	-	0.42	✓			Bolt tensile fracture

* Beam span-to-span ratio based on the span and depth values given in Table 1. ** Ratio between the ultimate load-carrying capacity and the maximum value of the load-carrying capacity up to a deflection level equal to the beam depth. *** Ratio between the ultimate deflection and the beam depth. **** FA: flexural action; CAA: compressive arching action; TCA: tensile catenary action.

4.2. Characteristics of Pseudo-Static Approximation

All the tests listed in Tables 1 and 2 were conducted under static loading by applying an incremental vertical force at the mid-span under displacement control. Therefore, the nonlinear static load-deflection responses of the specimens were obtained. However, since events in progressive collapse usually take place during a short timescale, it is quite important to identify how the behavior changes when the load is applied instantaneously, which corresponds to a sudden column removal scenario (e.g., an initial failure caused by blast or impact). Only a few experimental studies have examined this problem through instantaneous column removal [84–87]. These studies have confirmed that a sudden column loss induces larger beam deflections under a certain level of applied gravity loading, and therefore, the connections are subject to increased deformations. However, the main objective of these studies was to measure the maximum deflection of the double-span beam system, which required the structure to obtain a new state of stable equilibrium without collapsing due to failure of the connections. Therefore, some information about the deformation capacity and the potential failure modes of these connections can only be obtained by examining the deformation modes and the extent of their deformation at the new equilibrium state.

Since the dynamic analysis of this problem is particularly demanding in both experimental and numerical applications, alternative solutions have been proposed. Izzuddin et al. [24] derived a simplified method to predict a dynamic response based on static analysis principles. It has been proved that the maximum dynamic displacement of a single-degree-of-freedom system (such as the case of a double-span beam system) under the action of a certain level of loading develops when the work conducted by the load becomes equal to the energy absorbed by the structure. Therefore, from a static load–deflection curve, which essentially represents the relationship between the statically incremental displacement and the corresponding load-carrying capacity, a relationship between the dynamic displacement and load-carrying capacity is obtained. Since the resulting load–deflection curve is based on static analysis principles, it is referred to as a "pseudo-static response". This approach can be used to assess the maximum dynamic deflection of the double-span beam system under a certain level of gravity loading. Alternatively, provided that the deflection limit is known (i.e., the deflection that corresponds to connection failure), the load-carrying capacity of the system under the action of sudden column removal can be defined as the maximum load value on the pseudo–static curve up to this deflection limit.

By examining the different pseudo-static predictions derived from the experimental load–deflection curves shown in Figure 6, some important conclusions are drawn:

- The poor behavior of a double-span system with nominally pinned connections which is solely governed by tensile catenary action, further degrades when considering dynamic effects, as shown in Figure 6a,b.
- The increase in the load-carrying capacity due to tensile catenary action that developes after the flexural/arching stage becomes much less significant when the static response is converted to pseudo-static, as shown in Figure 6c,g,h.
- When the load-carrying capacity is gradually decreased due to the successive failure of various connection components, without leading to the total collapse of the double-span system, the rate of decrease in the pseudo-static capacity is smaller, as shown in Figure 6d.
- A system that demonstrates a stiff initial response and substantial flexural action tends to exhibit enhanced pseudo-static resistance, as shown in Figure 6e,f.
- The effects of compressive arching action are less pronounced in a pseudo-static load–deflection curve, as shown in Figure 6g.

These conclusions are very useful to better assess the true significance of various collapse resistance mechanisms in the progressive collapse response of steel frame structures following sudden column removal.

4.3. Review of Experimental Results

One of the earliest experimental studies was performed at the University of Liège as part of the collaborative research project on structural robustness described in [66]. This study adopted the concept of column removal to examine the progressive collapse resistance of a composite frame building that was designed according to the provisions of the Eurocode. A substructure was isolated from the frame and tested against simulated column removal by adopting the detailed test setup configuration described previously.

This structure comprised HEA160 steel columns and IPE140 steel beams acting compositely with a 500 × 120 mm solid concrete slab. Flush endplate connections with two bolt rows and an endplate thickness equal to 8 mm were employed. In addition, 3Ø8 rebars were arranged on either side of the columns. The response was first governed by flexural action without evident compressive arching effects, but the structure was able to undergo large inelastic deformations and exhibited substantial tensile catenary action prior to failure due to rebar rupture at a beam deflection of approximately 2.4 times the beam depth.

The simplified test specimen was first adopted by Yang and Tan [67] in a study that examined the progressive collapse behavior of bare steel beams with various steelwork connection types. Beam systems with simple shear connections, such as double web cleat and fin plate, were only able to sustain gravity loading through tensile catenary action (Figure 6a,b). The behavior of beams with more substantial arrangements (e.g., top and seat angle, top/seat and web angle) was somewhat enhanced by flexural action, but their resistance was limited by the premature fracture of the seat angles at the mid-span joints. The effects of flexural action were better illustrated by the behavior of beam systems with flush and extended endplate connections. The former was able to demonstrate significant tensile catenary action (Figure 6c) prior to the tensile fracture of the bolts, while the latter did not benefit from tensile catenary action due to the premature fracture of the welds, which caused the successive failure of other connection components and, thus, led to a progressive reduction in the load-carrying capacity (Figure 6d).

By employing the detailed setup configuration, Lew et al. [68] tested two bare steel double-span beam specimens with welded unreinforced flange-bolted web (WUF-B) and reduced beam section (RBS) connections, respectively. The response of the former was initially described by the effects of compressive arching action, which, however, had a minor influence on the overall performance. The resistance of both specimens was greatly enhanced by the development of tensile catenary action at large deflections. The resistance of the WUF-B connection was first reduced by the shear failure of the beam web bolts prior to the complete failure of the specimen due to fracture of the beam flange. The RBS specimen failed at a comparatively later stage due to the fracture of the beam flange at a reduced area of the section.

The progressive collapse behavior of a double–span composite beam with two different steelwork connections was examined by Guo et al. [69,70]. In both cases, the total depth of the composite section was 300 mm, which comprised the steel section with a height of 200 mm and the slab with a height of 100 mm. The slab had a width of 800 mm and included a series of closely spaced longitudinal reinforcement bars Ø12. Welded [69] and flush endplate [70] beam-column connections were employed in these two specimens, respectively. The first specimen exhibited a limited compressive arching action; however, it exhibited substantial tensile catenary action (Figure 6e) since failure occurred at a deflection of approximately 1.5 times the beam depth due to the fracture of the welds. The effects of compressive arching action were rather more pronounced in the response of the second specimen; however, the tensile catenary action effects were limited (Figure 6f) by the premature tensile fracture of the bolts at the mid-span joint. The premature failure of the mid-span connections was mainly due to the presence of the concrete slab in the compression zone. Since mid-span connections were subject to sagging bending moments, the center of rotation was in the concrete slab and, therefore, was further away from the lower bolt rows. For this reason, these bolt rows underwent increased deformations for lower connection rotation values.

In a different study, Li et al. [71] employed the simplified test setup configuration to explore the behavior of welded unreinforced flange-bolted web (WUF-B) connections between I-beams and square hollow sections under simulated column removal. Two different connection geometries were considered, with the same number of bolts arranged in one and two rows, respectively. Both specimens exhibited similar responses, which were mainly governed by flexural action, but their resistances were limited by the fracture of the beam bottom flange at the midspan region. An extension of this study was presented elsewhere [72], where another alternative WUF-B arrangement and an equivalent welded connection were examined. Similar experimental results in terms of the overall performance and load resistance mechanisms as those reported in [66] were obtained. The fracture of the beam bottom flange in the region of the mid-span joint was the initial failure mechanism for both connection types.

Yang et al. [73] compared the role of different steelwork connections in the progressive collapse response of composite frames. Detailed test setup configurations were employed, in which the composite beams had a total depth of 282 mm, comprising UB203 × 133 × 30 steel sections and a composite slab with or without longitudinal reinforcement bars. The steelwork connections were either double web cleat or flush endplate. Although these connections are classified differently in terms of stiffness and strength, the responses of both specimens in the presence of longitudinal reinforcement were similar since they were both enhanced by compressive arching and, especially, by tensile catenary effects (Figure 6g,h). This demonstrates the significant contribution of slab reinforcement to the tensile catenary behavior. The failure modes were the same as those observed in [67], that is, web angle fracture and bolt tensile fracture, respectively.

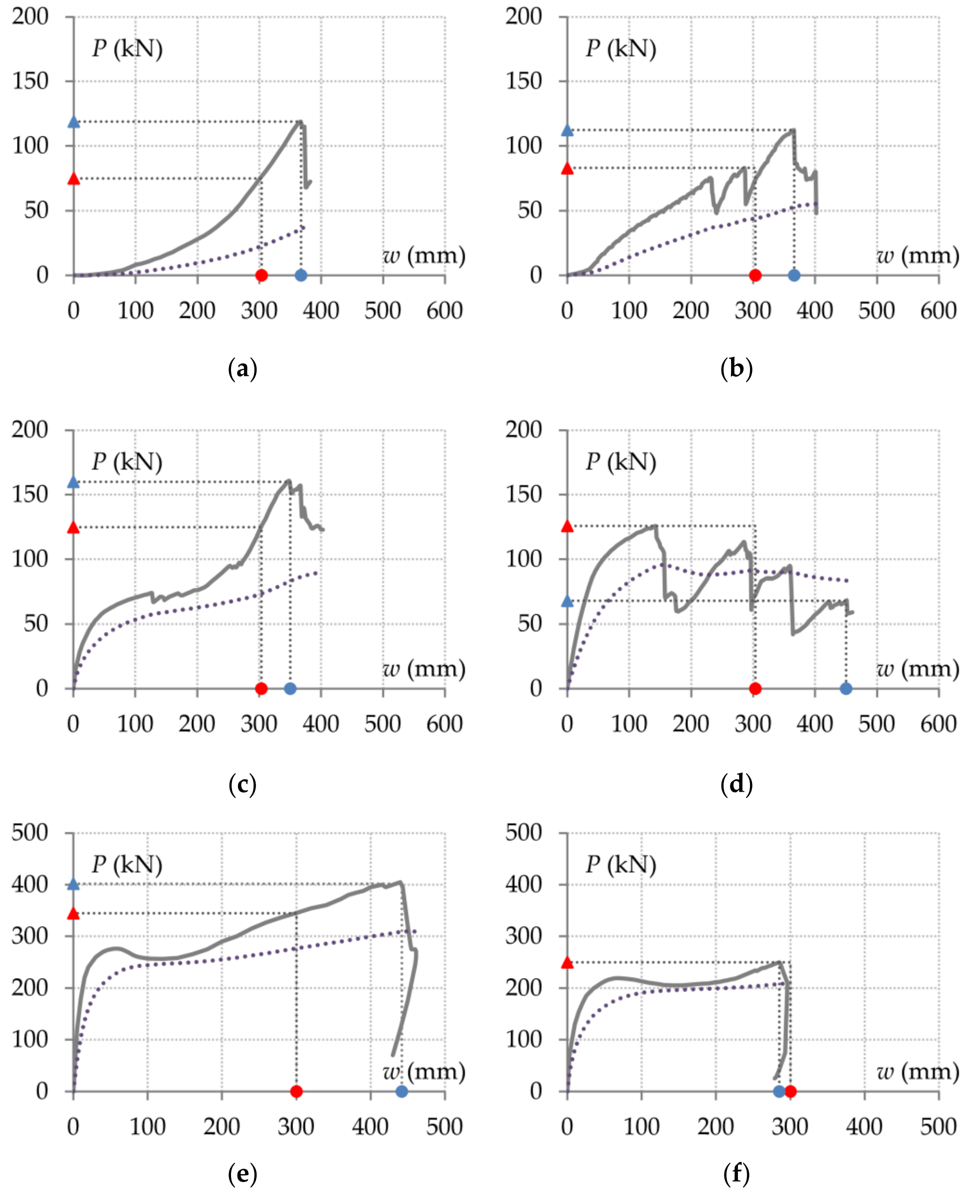


Figure 6. *Cont.*

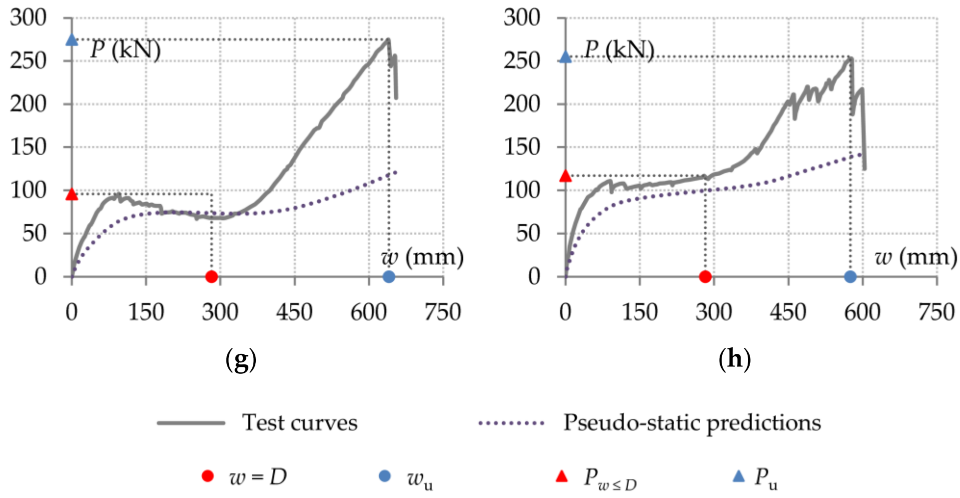


Figure 6. Experimental load–deflection curves and pseudo-static predictions: (**a**) Bare steel specimen 2a [67]; (**b**) Bare steel specimen 2c [67]; (**c**) Bare steel specimen 2e [67]; (**d**) Bare steel specimen 2f [67]; (**e**) Composite specimen 4 [69]; (**f**) Composite specimen 5 [70]; (**g**) Composite specimen 8a [73]; (**h**) Composite specimen 8b [73].

The objective of the study conducted by Dinu et al. [74] was to investigate the contribution of different types of semi-rigid and rigid connections to the progressive collapse response of steel frame structures. Based on a detailed representation of the double-span beam mechanism, four specimens with different connections–i.e., welded cover plate flange-bolted web, haunch endplate, reduced beam section, and extended endplate–were tested. Apart from the latter, the other three specimens were able to sustain large deformations, allowing for the development of tensile catenary action prior to failure. The flange cover plate fracture, bolt tensile failure, beam flange fracture, and premature bolt tensile fracture were the observed failure modes for the four specimens, respectively.

Another study performed by Zhong et al. [75] employed the detailed test setup configuration to study the performance of different types of bare steel connections. One of these connection types was a welded unreinforced flange-bolted web (WUF-B), which demonstrated similar behavior to that described by previous studies [68,71]. Apart from the WUF-B connection, this study also examined the behavior of beam systems with double-angle cleat and TSWA (top/seat and web angles) connections. Due to their substantial deformation capacity, both connections enabled the development of tensile catenary action prior to their failure. The failure modes of these connections were similar to those observed in relevant previous studies [67].

Based on the simplified test setup configuration, the behavior of bare steel systems was further studied by Li et al. [76]. The test specimen comprised two I-beam sections that were connected with a square hollow column through WUF-B connection arrangements. Similar to relevant previous studies, the load-carrying capacity was mainly limited by the premature fracture of the beam bottom flanges in the region of the mid-span joint, which occurred at a relatively small beam deflection. Although this considerably reduced the load-carrying capacity, the specimen was able to undergo quite large inelastic deformations to attain the tensile catenary stage, but these catenary action forces were not sufficient for the structure to recover its initial load-carrying capacity.

Xu et al. [77] and Gao et al. [78] studied the progressive collapse resistance of steel frame systems comprising I-beam sections and concrete-filled steel tubular (CFST) columns based on the simplified test setup configuration and by considering alternative solutions for beam-column connections. Instead of the conventional blind bolts that are commonly used in these frame systems, long bolts passing through the column section were adopted. The former study examined the behavior of flush endplate, extended endplate, and stiffened angle connections, while the latter study focused on different arrangements of stiffened angle connections. The failure of all specimens was triggered by either the thread stripping or pull-out of the most heavily loaded bolts. The responses of specimens with the flush endplate and the extended endplate connections were enhanced by tensile catenary action effects, while the specimens with stiffened angle connections failed at comparatively earlier stages and, thus, prior to the significant activation of tensile forces in the beams.

Another experimental program was conducted by Alrubaidi et al. [79] to study the behavior of bare steel frames with different connection types based on the detailed test setup configuration. Three connection types were considered, including fin plate, WUF-B, and extended endplate. Similar to previous studies, the response of the specimen with fin plate connections was mainly described by tensile catenary action. Although these connections exhibited an increased deformation capacity, allowing for the beam deflection to reach a value of approximately 1.8 times the beam depth, the tying resistance of these connections was not sufficiently high to ensure a significant load-carrying capacity, especially compared to the other two specimens. Owing to the increased flexural strength of the WUF-B and extended endplate connections, the ultimate resistance of these specimens was 3–4 times greater than the load-carrying capacity of the specimen with fin plate connections. However, the capacity of the WUF-B specimen was limited by the premature fracture of the beam bottom flange prior to the activation of tensile catenary action. Although the behavior of the third specimen was enhanced by tensile catenary action at large deflections, this occurred after the thread stripping of some bolts, and thus, it had a minor practical significance.

Meng et al. [80] examined the effects of web openings on the progressive collapse resistance of composite frames. Two specimens with detailed setup configurations that included welded unreinforced flange-bolted web (WUF-B) connections were tested. The steel beam cross-section was the same for both specimens. The total beam depth was 205 mm, including a concrete slab with a height of 55 mm. While in the first specimen, the beam webs were solid, in the second specimen, the beam webs had circular openings at closely spaced intervals throughout the span. The experimental results showed that the web openings had a beneficial influence on progressive collapse resistance. Both specimens failed due to the fracture of the beam tension flange in the mid-span joint region. Meanwhile, however, the mid-span connections of the second specimen had already undergone greater deformations compared to the first specimen due to the presence of web openings in the vicinity of the mid-span region. For this reason, the response of the specimen with web openings was enhanced to a significantly greater extent by the effects of tensile catenary action.

The significance of web openings in the vicinity of the beam-column connections was further investigated by the experimental studies conducted by Qiao et al. [81] and Lin et al. [82]. In these studies, there were no web openings along the entire length of the beam but only one opening next to each connection. For this reason, the connection type was defined as the reduced web section (RWS). The studies employed simplified test setup configurations, and they were restricted to bare steel specimens. The beam-column connections were welded. Qiao et al. [81] compared the cases of a solid beam section, a reduced flange section (RBS), and a section with both reduced flange and reduced web (RBS-RWS). Lin et al. [82] compared a solid beam section with two reduced web sections, RWS1 and RWS2. In the RWS2 specimen, the web opening had a smaller diameter, and it was located at a greater distance from the edge of the beam. The failure occurred due to the fracture of either the welds or the beam flange. It was found, however, that performance can be improved by a reduction in either the beam flange or the beam web. The latter could become more effective depending on the dimensions and the location of the web opening.

The experimental study conducted by Kukla and Kozlowzki [83] confirmed the significance of the beam span-to-depth ratio on the progressive collapse resistance of steel frame structures. This study employed the simplified test setup configuration, but the span-to-depth ratio on either side of the removed column was only 3.2. Six specimens with flush endplate and extended endplate connections and various endplate thicknesses were examined. Table 2 only presents the results for an endplate thickness equal to 20 mm. Regardless of the connection type and the endplate thickness, the responses of all specimens were very similar. They were mainly governed by flexural action without evident compressive arching effects, but the specimens failed prior to the development of tensile catenary forces due to the premature fracture of the bolts. This was because the beam-column connections were subject to large rotations at relatively small beam deflections.

4.4. Summary and Conclusions

The results of 44 test specimens from 18 different experimental studies were collected and evaluated with respect to the information provided in Section 3. Only 7 out of 44 specimens comprised composite beams of steel and concrete, while the remaining 37 specimens comprised bare steel beam sections. In addition, 19 specimens employed the detailed test setup configuration, whereas the simplified configuration was adopted for the remaining 25 specimens. The average span-to-depth ratio of the 44 specimens was 8.6, where only one of them had a span-depth ratio greater than 15, and three specimens had a ratio value less than 6.5. For the detailed specimens, the average span-to-depth ratio was 10.6, while for the simplified specimens, the corresponding value was only 7.1. The high span-depth ratios of the detailed specimens justify the limited contribution of compressive arching action, while the smaller value for the simplified specimens has minor significance since this simplified configuration does not favor the development of compressive arching action.

Except for the four specimens that involved nominally pinned beam-column connections (i.e., double web cleat or fin plate), the responses of the remaining 40 specimens that employed moment connections were governed by flexural action. These results confirm that in the absence of flexural action, the progressive collapse resistance of a double-span beam system is rather low since tensile catenary action alone cannot ensure a substantial load-carrying capacity, regardless of the deformation capacity of the connections. The beneficial effects of flexural action, however, are of essential practical importance only when the structure has an increased deformation capacity. From the 40 specimens that exhibited a flexural action response, 8 specimens with different connection types failed at a beam deflection value less than the beam depth. This means that these structural systems failed prior to the development of tensile catenary action. Among these specimens, however, only one of them employed a detailed setup configuration, but this specimen had a comparatively low span-to-depth ratio.

The remaining 32 specimens with moment connections that exhibited flexural action were able to undergo large deflections on the order of the beam depth or greater. Nine of them, in particular, were able to undergo even larger deflections on the order of twice the beam depth or greater. The load-carrying capacity of only six of these specimens decreased within the deflection range between the beam depth and twice the beam depth due to the partial failure of these connections (i.e., failure of some connection components that did not lead directly to a total failure of the connection). The load-carrying capacity of the remaining 26 specimens was increased by the effects of tensile catenary action. The average ultimate deflection of these 32 specimens was 1.56 times the beam depth. The load-carrying capacity of the 26 specimens increased by 50% on average due to tensile catenary action.

From the seven composite specimens, six failed at a beam deflection greater than the beam depth. From the 33 bare steel specimens that exhibited flexural action, 26 failed at a beam deflection greater than the beam depth. These results demonstrate that both composite and bare steel beams with moment connections may exhibit a substantial deformation capacity. From the 26 bare steel specimens that exhibited flexural action and failed at a deflection greater than the beam depth, 10 were tested using the detailed setup configuration. Overall, from the 17 specimens (bare steel or composite) that exhibited flexural action and employed the detailed setup configuration, only one of them failed at a deflection that was less than the beam depth. This important conclusion indicates that many of the conventional steelwork connections employed in practice may possess a substantial deformation capacity to allow for beam deflections to exceed the beam depth–or, in some cases, twice the beam depth–even in the presence of composite action.

It could be concluded that the development of flexural action in a double-span beam mechanism is particularly important for progressive collapse resistance. Based on the information provided in Section 3, it is, therefore, established that beam-column connections should have sufficient flexural stiffness and strength. They should therefore be classified as rigid or semi-rigid moment connections. The greater the initial stiffness and/or the bending moment capacity, the higher the progressive collapse resistance of the structure. Along with these properties, beam-column connections must also possess a substantial deformation capacity, which could allow beam deflections on the order of the beam depth or higher in the event of column removal. The beam depth should be considered as the limit value, while twice the beam depth should be the target value for beam deflection. Depending on the beam span-to-depth ratio, the connection rotation requirements can be defined accordingly. The experimental studies have shown that the rotation capacity of conventional steelwork connections in bare steel or composite beams may satisfy these requirements; however, this is not always guaranteed. Therefore, certain connection types and arrangements that have a limited deformation capacity should not be adopted.

5. Novel Approaches to Improve Performance

Although the experimental studies described in the previous section have shown that beam-column connections in steel frame structures may respond adequately to progressive collapse demands, they have also revealed several factors that may adversely affect their structural resistance. These factors mainly concern the limited deformation capacity of certain types of connections due to the premature failure of specific components, such as the fracture of welds, the shear or tensile failure of bolts, the fracture of angle cleats, the fracture of beam flanges (especially in reduced beam sections), etc. Based on these findings, recent research studies have focused on possible methods of limiting these effects, thus enhancing their overall structural performance.

Reduced beam sections (RBS) offer important advantages in terms of their increased rotation capacity and increased energy absorption capacity under seismic actions. Under the action of column removal, however, structural resistance heavily depends on the ability of a reduced section to carry the combined forces that develop, especially the combined action of bending moment and axial tension. Therefore, the failure of frame systems comprising reduced beam sections is usually triggered by the fracture of beam tension flanges due to the high tensile stresses developed in these components [68,74,81]. In order to overcome this limitation, Meng et al. [88] proposed a novel strengthening approach, which involved the addition of V-shaped reinforcing plates on the inner faces of the beam flanges, bridging over the reduced areas. The aim was for these plates to act as backup components and prevent rapid failure propagation in the case of a flange fracture. This strengthening technique has only been studied through numerical modeling, but quite promising results have been obtained. In particular, it was found that the load-carrying capacity and ultimate deflection could increase up to 180.9% and 85.8%, respectively.

According to previous experimental studies, the progressive collapse resistance of steel or composite beams with fully welded beam-column connections is usually limited by a premature weld fracture or brittle cracking at the beam root [69,72,81,82]. The study presented in [89] aimed to resolve this problem by proposing a different design solution for welded connections. The objective was both to limit the concentration of stresses at the critical area of the welds and to enable the formation of plastic zones outside this area. The solution involved the addition of suitably configured energy dissipation cover plates on the outer surfaces of the beam flanges, which would indirectly connect the beam flanges with the column. The mechanical behavior of the cover plates was studied in isolation through tensile tests. However, the progressive collapse behavior of the proposed connection configuration was only studied through numerical modeling. This study has shown that energy dissipation cover plates could increase the progressive collapse resistance and deformation capacity by 78–120% and 140–182%, respectively.

Previous experimental studies have also shown that extended endplate connections might suffer premature failure when exposed to the effects of column removal due to weld fracture [67], the tensile fracture of bolts [74,83], or bolt thread stripping [77,79]. Avoiding these failure modes could increase the connection deformation capacity and, thus, enhance structural resistance to progressive collapse. Toward this goal, Meng et al. [90] proposed a strengthening technique for extended endplate connections, which involved the addition of bending stiffened plates between the outer surfaces of the beam flanges and the connected column. The objective was for these bending plates to replace the loss of welds or bolts in extreme loading conditions. A numerical study was conducted to examine the prospect of this solution, and it was found that the resistance of a double-span beam sub-structure to progressive collapse could increase up to 248%, while the deformation capacity could increase up to 151%.

The progressive collapse resistance of the top-and-seat angle with double web-angle (TSWA) connections is usually controlled by the ability of the top/seat angles to respond to increased loading demands while undergoing very large deformations. Previous experimental studies have shown that failure is usually governed by the fracture of these angles [67,75]. The study presented in [91] focused on the idea of replacing the traditional right web angles with bending angles, as well as adding bending angles to the inner surfaces of the beam flanges. On the presumption that bending angles would have a greater deformation capacity, the overall deformation capacity of the connection could increase. The study presented in [91] was only restricted to tensile tests of T-stub models comprising bending angles. These results indicated that the proposed solution offers greater strength and deformation capacity, and it might, therefore, become effective in enhancing the progressive collapse behavior of steel frame structures with TSWA connections.

Since shear connections such as fin plates and double cleats are normally designed as nominally pinned and are only required to resist shear forces, their bending moment capacity under normal loading conditions is of minor significance. This is the reason why tensile catenary action is the only mechanism that can be mobilized in a double-span beam system with shear connections under the action of gravity loading, according to the conclusions of relevant experimental studies [67,75,79]. With the aim of improving the performance of these structures, Alrubaidi et al. [92] investigated possible strengthening techniques. The objective was to achieve a level of performance that was equivalent to that of corresponding structures with moment connections. The results of a previous experimental study conducted by the same research group [79] were used as a guide to determine the strengthening requirements. Two strengthening approaches were proposed, including welded double-side plates and pretensioned high-strength hot-rolled steel rods within the connection region, respectively. This study, which involved both experimental testing and numerical analyses, showed that the two strengthening schemes could effectively enhance the performance of shear beam-column connections, leading to substantial increases in progressive collapse resistance. A possible disadvantage of these approaches, however, was that they seemed to be rather complex and potentially costly solutions.

In order to increase the tying resistance and rotation capacity of nominally pinned connections, Ghorbanzadeh et al. [93] and Bregoli et al. [94] proposed and investigated a different strengthening approach. This involved the reinforcement of the joint region by duplex stainless-steel pins that were passed through the beam web, which aimed at carrying tensile catenary action forces at large deformations through bending without affecting the shear resistance of connections under normal loading conditions. Experimental results have shown that this technique could enhance, to a considerable extent, the tying resistance and rotation capacity of fin plate connections. This could lead to a substantial increase in the load-carrying capacity of a double-span beam mechanism, which could become more than eight times higher compared to the capacity of an equivalent un-strengthened arrangement. Despite this significant improvement, however, tensile catenary action remains the dominant collapse resistance mechanism.

According to several experimental studies, the failure of flush and extended endplate connections when exposed to progressive collapse conditions is usually triggered by the tensile fracture of bolts [67,70,73,74,83]. This represents an undesirable brittle mode of failure which usually results in a significant reduction in the connection strength and overloading of other connection components, thus rapidly leading to a complete failure of the connection. A relatively simple solution to this problem was proposed by Shaheen et al. [95]. This involved the addition of a steel sleeve between the steel endplate and the washer, with the primary purpose of enhancing the deformation capacity of the bolt. The sleeve must be properly designed by specifying the appropriate length, thickness, and wall curvature to ensure the best possible performance. Numerical studies have exploited this approach to evaluate its effects on the progressive collapse performance of flush endplate [96] and extended endplate [97] connections. The results indicated that significant increases in the connection rotation capacity could be achieved on the order of approximately 50–150%. However, there is still a lack of experimental validation.

As described in the previous section, recent studies have shown that the deformation capacity of beam-column connections under the action of column removal can be considerably enhanced by the presence of openings on the webs of connected beams in the vicinity of the connection regions [80–82]. This essentially represents another effective approach for increasing the progressive collapse resistance of steel frame structures since a higher deformation capacity allows for the development of tensile catenary forces at large deflections prior to reaching the ultimate deformation limit. Experimental results are already available for both cases of circular [81,82] and rectangular [98] openings, with the findings and conclusions of all studies appearing quite promising. However, while the shape and dimensions of the web opening as well as its distance from the beam edge, may influence the structural response in certain ways [82], this could also be controlled and limited by other factors, such as the beam shear capacity and seismic performance, which need to be examined in future studies.

The common objective of the above studies was the increase in the deformation capacity of the connections. Most of the proposed methods have been proven to be highly effective, but only a few of them have been validated experimentally. While this limitation is likely to be temporary, as it is expected that these studies will continue at an experimental level, some questions arise regarding the applicability of these methods to real structures. The cost of connections in steel structures usually represents a large proportion of the total construction cost. This cost increases further as the connection configuration is upgraded by adding more components such as bolts, welds, stiffeners, etc. Therefore, despite the scientific significance of the advanced strengthening approaches described above, their practical importance may be limited due to their increased cost. Another important parameter that should be carefully taken into account is the possible influence of these strengthening techniques on conventional design requirements. That is, progressive collapse-resistant connections should still be consistent with gravity loading and seismic design requirements.

Regarding the last point raised above, it should be noted that in recent years special importance has been given to the study of novel design techniques which could enhance both the progressive collapse resistance and seismic performance of structures [99–102]. Lu et al. [103] proposed a novel design solution for steel–concrete composite frame systems involving prestressed steel strands and energy-dissipating components in beam-column connections, which has been proven to be effective in enhancing both the progressive collapse resistance and seismic behavior. In a subsequent study, Tian et al. [104] further improved this particular solution to minimize the possibility of local instability in the compression zone of the beam-column connections due to a compressive arching action. The main objective was to restrict the extent of damage to the energy-dissipating components of the connections, which enhanced the repairability of the structure.

6. Conclusions

Considerable progress has been made in recent years to develop an understanding of the role of beam-column connections on the overall behavior of steel frame buildings in progressive collapse. A common approach for assessing structural performance, which is based on the alternate load path concept, considers the consequences of a threat-independent column loss on the surrounding structure. Beams and beam-column connections, slabs, infill walls, and bracing members may act as alternate load paths to enable the redistribution of loading to neighboring intact columns, which should be able to respond to these increased loading demands. In this process, the role of the beam-column connections is extremely important because they should be able to transfer a varying combination of bending, axial and shear forces from the supported beams to the supporting columns whilst undergoing very large inelastic deformations.

The axially restrained double-span beam mechanism is the most common approach when examining the response of beam-column connections to column loss. A double-span beam system could resist gravity loading through flexural, compressive arching, and tensile catenary actions. An important feature of this problem is that events in progressive collapse usually take place over a very short timescale, and therefore, dynamic effects should also be taken into account. In this case, the performance of a double-span beam structure could be described more appropriately by instantaneous column removal or by a sudden application of gravity loading. Alternatively, the results of the static analysis could be utilized for the prediction of a dynamic response by converting the static load–deflection curve into a pseudo-static response based on an energy-balance approach proposed in the literature. In a pseudo-static representation, however, the effects of compressive arching and tensile catenary actions on collapse resistance can become considerably less significant.

It was found that the most important properties of the connection response are the initial stiffness, the flexural strength, and the deformation capacity. As the initial flexural stiffness and the flexural strength of the connections increase, the static capacity and pseudo-static capacity increase. As the deformation capacity of the connections increases, the static capacity may increase provided that the response is governed by tensile catenary action or could decrease when the fracture of some connection components has already occurred. Both effects, however, become much less significant when considering the pseudo-static response. In either case, the deformation capacity of the connections could enable the structure to undergo large deflections, which may enable the mobilization of additional collapse resistance mechanisms, such as membrane tensile action in the slabs.

The deformation capacity of connections in progressive collapse conditions could mainly be determined through experimental tests. At the experimental research level, the double-span beam mechanism was used in two main alternative ways. Its detailed representation involves the complete sub-structure comprising a missing column between two adjacent beams, which are supported by two other columns on their other ends, while its simplified representation considers only half of the beam spans and hinged supports on either end, simulating the points of inflection. Although this simplified representation cannot describe accurately the actual behavior of a double-span beam system, it could provide essential information about the performance of beam-column connections when subject to large deformations under a combined bending moment and axial force.

Although the general impression is that certain connection types (e.g., flush endplate, welded, RBS, etc.) may perform better compared to others (e.g., extended endplate, WUF-B, TSWA, etc.), this is not always the case according to the results of experimental studies. What is more important is that the interplay between the key connection properties, regardless of the connection type, ensures the best possible behavior. Therefore, rigid or semi-rigid moment connections with a proven capability of sustaining large deformations are likely to make a positive contribution to progressive collapse resistance. While any required level of flexural stiffness and strength could potentially be achieved by the common assemblies of steelwork connections employed in design practice, the required levels of ductility that ensure beam deflections in the event of column loss on the order of approximately twice the beam depth are not always guaranteed.

This is the reason why current studies mainly focus on deriving novel approaches for enhancing the deformation capacity of common connection types by employing additional components that aim at minimizing the consequences of the premature failure of certain connection components (e.g., bolts, welds, angles, etc.). The prospects of the solutions proposed in these studies are quite promising, but most of these solutions have not been validated through experimental testing yet, while their expected increased costs may become a limiting factor for widespread application in practice. In addition, these novel strengthening techniques should also be consistent with more conventional design requirements, as beam-column connections are primarily designed against gravity and seismic loading conditions. Therefore, despite the scientific significance of these approaches, their practical importance may remain limited or may only be restricted to special-purpose designs.

For these reasons and given that conventional connection arrangements may possess a substantial deformation capacity, as confirmed by several experimental studies, more emphasis should be placed on simpler solutions. Beam-column connection assemblies that are widely used in design practices and have been shown to have an acceptable performance under conventional gravity or seismic actions could also perform adequately under progressive collapse conditions. This requires that the interplay between key connection properties–i.e., initial stiffness, flexural strength, and rotation capacity–could ensure optimal performance under these different design requirements. Based on the possibility of intervening in the configuration of connections to define the different characteristics of the various constitutive components, such optimal solutions may be possible. Therefore, special attention should be paid to the detailed study of these possible solutions before turning to more advanced solutions.

In addition to the significance of the connection deformation capacity, another aspect of the problem that requires further study and understanding is the interplay between the different collapse resistance mechanisms. The main resistance mechanisms that are mobilized individually in a double-span beam system or in a reinforced concrete slab have been studied extensively, and they are now well understood. Different mechanisms that are mobilized in the presence of masonry infill walls or steel bracing systems have also received particular attention. However, the simultaneous activation of these mechanisms and the interplay between various parameters that may have different effects on some of these mechanisms may considerably change the problem. Therefore, the connection strength and deformation capacity demands may be significantly different when considered in the context of a wider structural system. Although this appears to be a quite complex problem, it should be properly studied and understood.

Similarly, little attention has been paid so far to the behavior of the surrounding columns, which are expected to carry an increased level of gravity loading transferred from beam-column connections through varying combinations of shear, axial, and bending forces. This is a particularly important aspect of the problem since the failure of surrounding columns can trigger different modes of progressive collapse compared to the failure of beam-column connections. The response of these components largely depends on the magnitude of the forces transferred to them. For example, when substantial tensile forces are transferred from supported beams through beam-column connections, these columns are expected to undergo lateral displacements, which generate second-order effects and may, therefore, affect their structural stability, leading to undesirable failure modes such as flexural buckling. Therefore, enhancing the tensile capacity or the beam-column connectionswith the aim of increasing the tensile catenary action forces could ensure an enhanced resistance of the floor system but may also lead to a different collapse propagation mode. Although this involves a somewhat complicated form of structural behavior, it is an aspect of the problem that still requires substantial study and understanding.

From the above discussion, it is made clear that the role of the beam-column connections in the progressive collapse resistance of steel frame structures has many implications, as the response of beam-column connections may affect the overall performance in different ways, while it may be affected by various parameters. On a research level, significant progress has been made during recent years in understanding the basic features of the problem and estimating potential design requirements. However, to draw comprehensive conclusions about the actual design requirements, research studies should focus more on broader aspects of this problem by effectively exploring the complex interplay between the connection behavior and the overall structural performance.

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