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Experimental testing and numerical modelling of steel moment-frame connections under column loss

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ABSTRACT

The beam-to-column connections of moment-resisting steel frames should exhibit capacities that allow them to transfer the forces that develop under normally expected loading conditions. However, when a column is lost owing to accidental loading, these conditions change, and the forces are redistributed to the adjacent beams and columns. In such cases, the connections must be capable of resisting the combined axial and flexural loads and allow for the redistribution of the loads, so that progressive collapse development is prevented. In this study, we investigated the performances of four types of beam-to-column connections, namely, the welded cover plate flange connection (CWP), the haunch end plate bolted connection (EPH), the reduced beam section welded connection (RBS), and the unstiffened extended end plate bolted connection (EP), against progressive collapse. Two span frames were constructed and tested under a central column removal scenario until failure. The results from the experimental tests were used to validate finite element models. The CWP, EPH, and RBS specimens showed good ductility, with the catenary action making a significant contribution to the ultimate load resistance. Further, the ultimate rotations of the beams were greater than the deformation limit given in the latest Unified Facilities Criteria guidelines for design of buildings to resist progressive collapse. Specimen EP showed the lowest ductility and ultimate load resistance, with the bolts in the rows under tension fracturing before the catenary action could develop. Further, the failure mode for specimen EP indicated that bolt strengthening is necessary for improving its progressive collapse resistance.

1. Introduction

Steel frame structures are used widely in multistory buildings as they offer a number of advantages (e.g., high resistance, structural and architectural flexibility, and high sustainability and durability). Even though building structures are not always designed for situations involving accidents, they must be able to resist any action without being damaged to an extent disproportionate to the original cause [1]. This disproportionality refers to situations wherein the failure of one member causes a major collapse, with the magnitude being disproportionate to the initial event; this is also known as progressive collapse [2]. Potential damage to structures may be limited or even prevented through several means, which can range from the elimination or reduction of hazards to the selection of a structural form that is not as sensitive to hazards and can survive the accidental removal of an individual member or of a limited part of the structure. For the latter option to be effective, it is necessary to ensure continuity between the member, thus ensuring the redistribution of loads through alternate load paths. The alternate path (AP) method provides a formal check on the ability of the structural system to resist the removal of specific members, such as columns. The AP method, with its emphasis on continuity and ductility, is similar to those employed in current seismic design practice [2]. Seismic design procedures [3,4] can be adopted as references for designs for preventing progressive collapse. This assumption is supported by the FEMA 277 report [5], which concluded that, if the Murah Building (see Oklahoma City Bombing, 1995) had been designed to resist seismic action, its progressive collapse would have been precluded. The adoption of seismic provisions is also encouraged by the latest DoD code [6], which recommends that all primary steel frame beam-to-column moment connections be one of the special moment frame (high ductility moment resisting frame) connections. The primary reason for using such a connection is to secure the connection characteristics that provide a minimum level of rotational capacity.

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Although the seismic design philosophy may be considered as a model for controlling the collapse of structures subjected to extreme events, there are specific issues that should be considered to forestall localized failures, particularly of columns [7]. One such issue refers to the values of the deformation capacity for beams and connections. Thus, in typical tests to evaluate the seismic performance of connections and members, cyclic loads with increasing magnitude are applied, without axial loading, and the resulting curves are used to develop "backbone" curves. In progressive collapse, the connection and member experiences one half cycle of loading, often in conjunction with a significant axial load, due to large deformations and catenary response [6]. More relevant data on the performance of connections in the event of column loss are therefore needed.

The complex behavior that is exhibited by the components of a steel frame structure prior to failure makes it difficult to predict their ultimate capacity using analytical models. Therefore, experimental testing and numerical simulations are necessary for addressing the phenomenon of progressive collapse of steel moment frames [8]. Demonceau and Jaspart [9] experimentally tested a two-dimensional (2D) composite frame (composite beams, steel columns, and partial strength composite joints) simulating a column loss. The test results indicated that the configuration exhibited ductile behavior, with the catenary action that developed in the beams making an important contribution. Xu and Ellingwood [10] investigated the performance of steel frames with partially restrained connections fabricated from bolted T-stubs following damage to the load-bearing columns. They reported that the frames with strong T-stub connections could resist collapse in damage scenarios involving the notional removal of one column, while the robustness of the frames with weak T-stub connections was questionable. In a similar study, Gong [11] conducted tests on 31 bolted double-angle connections under a purely tensile load, with the aim of developing a method for assessing the robustness of the design of bolted-angle connections. The results showed that, in order to ensure ductile behavior that would produce a catenary action, capacity design philosophy should be followed when designing the connections for robustness. Thus, in the case of bolted-angle connections, tensile bolt fracture before angle rupture should be avoided.

The following four studies, which focused on the performance of steel frames with moment-resisting connections in the case of column removal scenarios, revealed that different conclusions can be drawn regarding the capacity of the connections in allowing the development of a significant catenary action.

In the first study [12], two full-scale steel beam-column assemblies (the first one had a welded unreinforced flange and bolted web connections while the second one had reduced beam–section connections) were tested under a central-column-loss scenario. The test results showed that the rotational capacities of both connections under monotonic column displacement were approximately twice as large as those based on seismic test data. In the second study [13], Yang and Tan experimentally tested the performances of common types of bolted steel beam–column joints under a central-column-removal scenario. The results of this study also demonstrated that the rotational capacities of the beam–column joints were much higher than the recommended values. In the third study [14], three tests were performed on double-span frames with circular hollow sections subjected to column removal. The types of connections used were the welded flange-weld web connection with internal diaphragms, the welded flange-bolted web connection with internal diaphragms, and the welded flange-bolted web connection with short through diaphragms. Based on the results of the study, it was concluded that, during a sudden-column-loss scenario, progressive collapse can be triggered upon the initial fracturing of the bottom beam flange; this was true for all the specimens tested. In the fourth study, performed within the framework of the CODEC research project [15], Dinu et al. [16] experimentally investigated the ability of a three-dimensional (3D) steel frame structure realized using extended end-plate bolted beam-to-column connections to support the loss of a central column. The test specimen, which consisted of two bays and two spans of 3.0 m each, was constructed and tested under a monotonic load, which was applied at the top of the central column until complete failure. The beam-to-column connections were designed to exhibit an overstrength compared to the beams. The results showed that the end-plate-bolted connections possessed sufficient strength to resist the catenary forces that developed in the beams, with the rotation capacity being approximately 0.20 radians.

The connections in moment-resisting frames (which are designed as primary lateral load-resisting systems) can be either full-strength or partial-strength connections. A similar classification system applies for the stiffness. That is to say, the connection can be fully rigid or partially rigid (semi-rigid). The use of full-strength (and fully rigid) joints is the main option for DCM (medium ductility) and DCH (high ductility); however, partial-strength (and semi-rigid) joints are also allowed (see [3]). The second option is a more economical one while ensuring a safe and reliable response (for "normal" loading conditions, including seismic action). As a result, it is of interest to determine whether partial-strength and semirigid connections can provide the capacity (strength, ductility) required for arresting the progressive collapse that follows the loss of a column.

In the present study, also performed under the aegis of the CODEC project, we investigated the effects of various beam-to-column connections on the ability of steel frames to resist the loss of a column. The specimens tested, which had identical geometries and were fabricated using sections and materials similar to those used for the 3D steel frame tested and reported in [16] but were realized as 2D assemblies, were subjected to a monotonic load, which was applied on the top of the removed central column until complete failure. Four connection types were designed and fabricated to meet the seismic design requirements for special moment-resisting frame connections, namely, the welded cover plate flange connection (CWP), the haunch end plate bolted connection (EPH), the reduced beam section welded connection (RBS), and the unstiffened extended end plate bolted connection (EP).

One of the bolted connections (i.e., the unstiffened end plate bolted connection EP) was similar to the one used in the 3D steel frame tested in [16] but was weaker than the beam (partial strength and semirigid). Finally, based on the results of the experimental tests, numerical models were validated using the finite element software ABAQUS [17].

2. Specimens and test setup

A typical multistory steel frame structure (Fig. 1a) was used as the reference for extracting the experimental specimens. The design of the structure and the element sections used were identical to those of a three-bay, four-span, and six-story structure described elsewhere [18]. The bays and spans measured 8.0 m each. The structure was designed using the Eurocodes while keeping the effects of gravity loads (permanent and variable actions) and lateral loads (wind and seismic actions) in mind. The dead and live loads were both 4.0 kN/m^2 and the reference wind pressure was 0.5 kN/m^2. The building site was characterized by a design ground acceleration, a_g, of 0.08g and a control period, T_C, of 0.7 s. It should be noted that the seismic intensity and response spectrum used during the design were those given in the