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Effects of span-to-depth ratios on moment connection damage evolution under catenary action

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ABSTRACT

This paper proposes an improved method for determining the gravity resistance of a moment resisting beam-column assembly following an interior column loss. The proposed method accounts for the connection's damage evolution and for the catenary mechanism developed by the assembly as it deflects downward. Through a full-scale laboratory test and finite element simulations, the complete responses of moment resisting beam-column assemblies including the connection's damage evolution are investigated under different beam span-to-depth ratios. The welded unreinforced flange-bolted web (WUF-BW) connection method is used for its robustness in developing the catenary action. It is found that, under the same span-to-depth ratio, beam-column assemblies exhibit similar normalized load-rotation relationships, even with different beam depths. The assembly with a larger span-to-depth ratio is able to develop the gravity resistance earlier, and provides a higher ultimate resistance by developing a more effective catenary mechanism. On the other hand, the assembly with a smaller span-to-depth ratio exhibits a more ductile response. A simplified curve model of the gravity resistance development of a moment beam-column assembly with damage evolution has been proposed for a convenient assessment of the progressive collapse resistance following a central column loss.

1. Introduction

There have been several guidelines [1–3] for the progressive collapse design and analysis of building structures under extreme or abnormal load, all of which employ basically the same principles and analysis methods. According to UFC 4-023-03 "Design of building to resist progressive collapse" [2], a progressive collapse design may use different methods depending on the occupancy category of the building, including the Tie Force (TF) method for the entire structure, the Alternate Path (AP) method and the Enhanced Local Resistance (ELR) method for some specific structure regions.

The Alternate Path method [4], as both the design and the analysis methods, is the most popular for the study of progressive collapse prevention [1–3]. A structure must be able to bridge over vertical load-carrying elements notionally removed from itself by satisfying the requirements of the Alternate Path method, otherwise it must be re-designed or retrofitted to increase the structural bridging capacity [2,3]. In this method, any further failure of structural components (connections, beams and columns) following the notional column removal is prevented by ensuring the components meet certain criteria for various building materials including reinforced concrete, structural steel, masonry and wood [2,3].

It has been found [5–10] that the structural bridging capacity depends on the performance of the connections. There have been a number of experimental tests and numerical simulations focusing on the behaviour of various connections [11–21] following an interior column loss. The moment connections were found to work firstly by flexural action and later by catenary action [6,14,15,18–20]. It was found [15,18–21] that a steel moment connection usually acquires a meaningful contribution to the gravity resistance from the catenary mechanism at chord rotations greater than 0.03 rad.

When the nonlinear static analysis procedure is employed, nominally rigid moment connections must deform within the prescribed deformation limits so as to meet the acceptance criteria [2]. The acceptance criteria for moment connections are given in terms of the plastic rotation, whose values for a primary component correspond to its plastic deformation limit prior to capacity degradation [2,3]. Moment connections are permitted to deform within a small range of plastic rotations, below 0.025 rad for the typical "improved welded unreinforced flange-bolted web" (WUF-BW) connection [2,3], which does not allow any significant catenary action to be developed [15,18–22]. However, the capacity degradation does not usually occur until a much larger rotation, typically greater than 0.06 rad [15,18–21].

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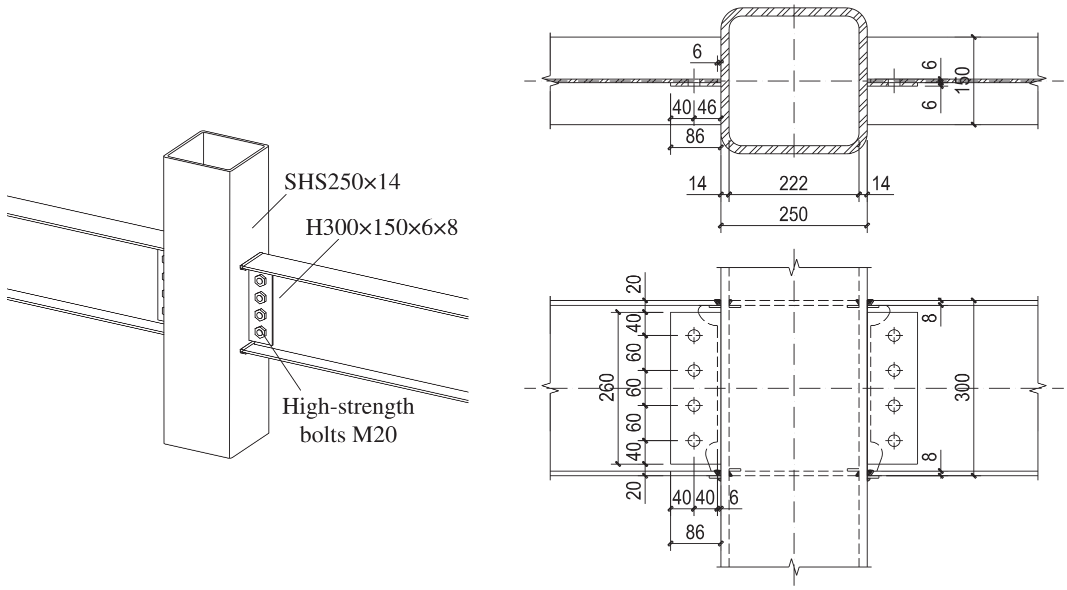

(a) Components. (b) Dimensions (in mm).

Fig. 1. Details of the WUF-BW connection.

In traditional seismic structural designs, the occurrence of fracture signifies the ultimate limit state of a moment connection due to the loss of its flexural capacity. However, in an interior column removal scenario, catenary action can still be developed by the tensioning of the connected beam members under large deflection following fracture, provided the connections are designed appropriately [18–22]. Two types of moment connection failure modes, being the beam-end interrupted failure mode and the column-wall failure mode, have been identified [18–20] as being able to allow the assembly to obtain a higher gravity resistance (from the catenary mechanism) in the post-fracture stage than its previous peak resistance (under the flexural mechanism). It is therefore rational to explore new design criteria that take advantage of the catenary mechanism that develops following an interior column loss.

Among the various levels of sub-structure idealisation in the simplified framework proposed by Izzuddin et al. [23] for multi-storey buildings, the double-span beam-column assembly within the bays above the lost column is the lowest level of sub-structure whose response is used for composing the higher level sub-structures. The beam's span-to-depth ratio has been found to significantly affect the response of the double-span beam-column assembly following the column removal [24–27]. However, these investigations did not account for the damage evolution of the beam-to-column connections.

In this paper, the complete responses of the moment resisting beam-column assemblies under the column removal scenario are investigated. The welded unreinforced flange-bolted web (WUF-BW) is used to connect the beams and the column as such a connection facilitates the development of the catenary mechanism following an initial fracture. The development of the assembly's gravity resistance in the post-fracture stage and the effects of the span-to-depth ratio are studied in detail.

A full-scale laboratory test is conducted where a pushdown action at the central column is applied in order to simulate the column removal scenario. The test results are used to verify the refined finite element model incorporating material fracture, which is employed in subsequent parametric analyses of the effects of the beam span-to-depth ratio on the gravity resistance of the beam-column assemblies. Based on the parametric analyses results, an improved development model will be proposed for the structural gravity resistance taking into account the damage evolution of the connection region.

2. Full-scale laboratory test

2.1. Test specimen

Due to its robustness during the beam-end interrupted failure and column-wall failure under a central column removal scenario [18,19], the welded unreinforced flange-bolted web (WUF-BW) connection was used for the test specimen whose details are given in Fig. 1. The double-span assembly consisted of two I-section beams (H300 × 150 × 6 × 8) and a square hollow section column (SHS250 × 14) with two inner diaphragms (thickness $t = 8$ mm) at locations corresponding to the top and the bottom flanges of the beam, as illustrated in Fig. 1 (b). The beam webs were bolted to the shear tab welded to the column via four M20 Grade 10.9 frictional type high-strength bolts arranged in one vertical row. More manufacturing details could be referred to in literature [19]. The measured material properties of the specimen are summarized in Table 1.

The Beam-Joint-Beam (B-J-B) assembly [18] was employed for the specimen, as illustrated in Fig. 2 (a). A relatively small span of the beam $l_0 = 2400$ mm was used, giving a gross span-to-depth ratio of $l_0/H = 8$, in order to obtain the complete response of the beam-to-column connection including the damage evolution since there was a limited vertical displacement range (approximately 400 mm).

Table 1
Material properties of test specimen.

Components	Yield strength f_y (MPa)	Tensile strength f_u (MPa)
Plate of SHS250 × 14	410	655
Corner of SHS 250 × 14	415	750
Beam flange ($t_f = 8$ mm)	400	670
Beam web ($t_w = 6$ mm)	405	640

2.2. Test setup and instrumentation

The test specimen, mounted on a purpose-built test rig as illustrated in Fig. 2 (b), was loaded vertically at the unsupported central column by the actuator at a stroke rate less than 7 mm/min. The central column