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Effect of beam web bolt arrangement on catenary behaviour of moment connections

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ABSTRACT

In a column-removal scenario for a building structure, the catenary action will play an essential role for the frame in resisting a progressive collapse. This paper investigates the catenary behaviour of welded unreinforced flange-bolted web connections (i.e. WUF-B connection) in plane frames by means of full-scale testing and numerical simulation. Two different layouts of bolts at the beam web were considered, with four bolts arranged in one row or two rows in the two specimens, respectively. The results demonstrate that both specimens of the WUF-B moment connection were able to develop an effective catenary action via the bolted web following the primary flexural phase. The failure modes of the bolted web vary with different bolt arrangements under the catenary action. When all (four) bolts were arranged in one row, the lowest bolt bearing area on the web tends to be compressed to fracture before bolt tear-out failure occurs near the weld access hole. When the bolts were arranged in two rows, however, the shear tab cracked at the section across the bolt holes. The former failure mode is deemed to be more robust than the latter under a column removal scenario.

1. Introduction

As a general guide to preventing disproportionate or progressive collapse in the event of a critical local failure, a structure should be designed to possess an adequate degree of continuity and ductility, in addition to strength [1–3]. As far as a frame structure is concerned, the structural system should be able to bridge over the failed load-carrying member, particularly a failed column. Moment-resisting beam-column connections, which hold the critical path of the gravity load in a framed structure, are generally beneficial in terms of the structural redundancy [4]. After the removal of a column, a “double-span” scenario arises, and the soundness of the affected moment connections will play a central role in withstanding and redistributing the gravity loads from the upper storeys over the emerged double-span [5–11]. In this process, the connection(s) and the adjoining members will typically experience an intensified flexural action stage, followed by a catenary action phase as the deflection in the double span becomes large.

It has been demonstrated [11–17] that the catenary action mechanism has the potential to considerably supplement and eventually replace the flexural action in carrying the vertical load. However, it can be understood that the realisation of an effective catenary action depends upon two basic conditions: a) a sufficiently large axial tension can develop in the beams, and b) such axial tension can maintain whilst large deformation (and hence large slope) advances, which would effectively enable the transfer of the vertical load via the axial tension of the beams to adjacent columns. In this respect, the ability of the connections in withholding a necessary degree of integrity into the large deformation regime becomes critically important.

According to the preceding experimental investigation of beam-to-tubular column moment connections under the column removal scenario [17], different connecting methods at the web may provide a similar flexural capacity but they could end up with considerably different catenary action capacity after flexural failure occurred. For a welded-web connection, the flexural action and catenary action mechanisms tend to deteriorate simultaneously because of continuous crack propagation after the bottom flange of the beam section fractured. In contrast, a bolted web connection enables the catenary action to develop more effectively, thanks to the interaction of the beam web with the bolts and shear tabs even after fracture occurs in the bottom flange. In another experimental study on the bolted web connections under a column removal scenario conducted by Sadek [12], the loading capacity was observed to reduce following the fracture of the bottom flange near the weld-access hole. Unfortunately, the test terminated shortly after the bottom flange fractured, so the performance of the moment connection in the catenary phase could not be examined.

This paper investigates the catenary behaviour of the typical H-beam and square-column moment connections with a bolted web connection, with a particular focus on the influence of different bolt layouts on the structural resistance in the large deformation regime. Two full-scale

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beam-to-column assemblies with welded unreinforced flange-bolted web connection (i.e. WUF-B connection) were designed in detail in accordance with a prototype steel building frame, and they were constructed and tested under a push-down action applied at the unsupported centre column location. The experimental results are presented and discussed comprehensively. In conjunction with the experiments, numerical simulations with a detailed finite element model incorporating material fracture are conducted to verify the load transfer and failure mechanisms of the WUF-B connections, especially in the catenary response phase.

2. Experimental programme

2.1. Test specimens

The test specimens were designed to represent the beam-to-column connection region in a column removal scenario. For this purpose, a beam-joint-beam (B-J-B) assembly is considered appropriate [17]. Such an assembly, as depicted in Fig. 1, is extracted from the directly affected spans of the frame when a middle column is removed, assuming that the inflection point is located around the mid-span of the original beam members in such a scenario. This configuration allows the full details at the connection to be reproduced, whilst the column removal scenario can be simulated by a push-down action via a centre column, as will be shown in Section 2.2.

The WUF-B connections between the H-beam and the square hollow section (SHS) column are investigated in this paper. The geometrical characteristics of the assemblies are given in Table 1. The main difference between the two test specimens under consideration, namely SI-WB and SI-WB-2, lies in the arrangement of the web bolts. The span of the beam is $l_0 = 4500$ mm (giving a gross span-to-depth ratio of $l_0/H = 15$), and the height of the centre column is 1100 mm, as will be illustrated later in the test set-up. The design of the beam-to-column assemblies was made following a strong column–weak beam seismic design philosophy and specific requirements in Chinese codes [18,19].

Fig. 2 illustrates the details of the connections. In each specimen, two H-section beams were connected to the SHS column via the WUF-B connection, and within the joint region two inner-diaphragms were installed inside the column at the locations corresponding to the top and the bottom flanges of the beam. It is worth noting that such a connection configuration with internal diaphragms in a square tubular column is commonly used in steel construction to maintain the column continuity and at the same time to ensure sufficient beam-to-column joint flexural stiffness [20,21].

The flanges of the beam, as well as the inner-diaphragms inside the column, were jointed to the column wall using complete joint penetration (CJP) groove welds, and weld access holes of the beam were cut from the beam web in accordance with a standard recommendation [22]. The beam was bolted on the web to a shear tab which was pre-welded to the column, via four M20 Grade-10.9 frictional type high-strength bolts. Four bolts were arranged with two different layouts in the two specimens; Specimen SI-WB had all four bolts arranged in a single row along the depth of the web (see Fig. 2(a)), whereas Specimen SI-WB-2 had the four bolts arranged in two rows around the mid-height region of the web (see Fig. 2(b)). The pre-tightening force and torque applied on the bolts were 155 kN and 440 N-m, respectively, according to standard requirements [23]. All the contact surfaces were pre-treated with sand blasting. The measured material properties of the SHS column and the H-section beams are summarised in Table 2.

2.2. Test setup

A purpose-built test setup was employed for the series of tests, as schematically illustrated in Fig. 3. The test specimen was supported by a horizontally self-balanced support frame, whilst the vertical load was supported by a vertical reaction frame mounted on the strong floor.

The test specimens were loaded vertically at the top of the centre column to simulate the effect following the removal of the middle column below. To avoid complication in the loading condition, the centre column was guided at the bottom end using a sliding support so that only vertical movement was possible. This configuration effectively simulates a symmetrical condition which is considered representative in a building collapse scenario (see Fig. 1 of the paper) and it also allows a simpler setup for the application of the pushdown load from the top of the column. The sliding support at the column bottom end consists of an interior connector and a rigid exterior guiding box, and the interior connector is made of multiple ball-joints arranged around the connector. As the connector is ball-jointed, the friction incurred during the tests was negligible and this was confirmed by checking the applied load with the internal forces obtained from analysing the measured strains, as described in Section 4.4. The specimens were pin-supported at the two horizontal ends with latch-type rollers to realise free rotation at the

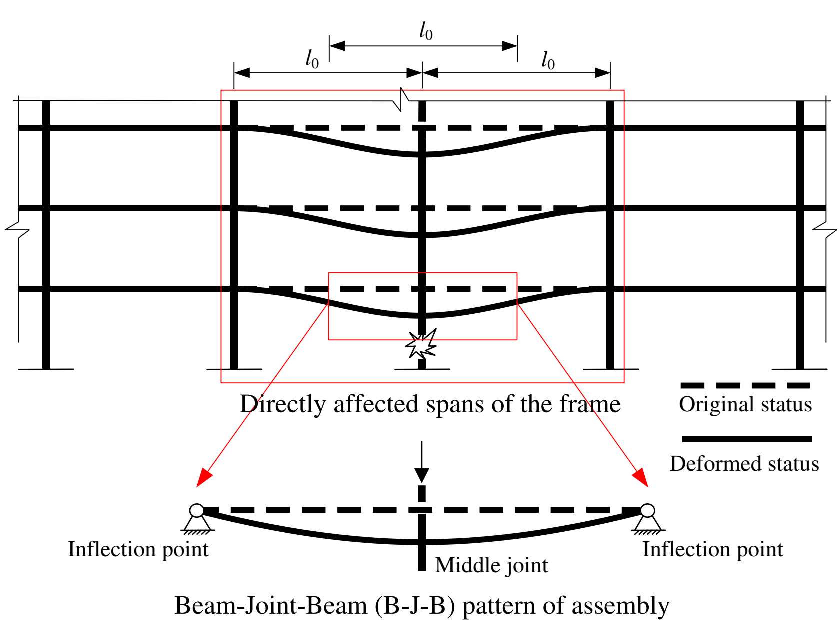

Fig. 1. A beam-joint-beam assembly extracted from a framed structure.