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Progressive collapse resistance of precast beam–column sub-assemblages with engineered cementitious composites

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ABSTRACT

This paper addresses the experimental investigation on the behaviour of precast concrete beam–column sub-assemblages with engineered cementitious composites (ECC) in structural topping and beam–column joints under middle column removal scenarios. Sub-assemblages comprising of a double-span beam and a middle beam–column joint were laterally connected to a rigid frame on one side and a reaction wall on the other side through load cells. Quasi-static vertical loading was imposed onto the middle column. Compressive arch action (CAA) and catenary action developed sequentially in the bridging beam with increasing middle joint displacement. Unlike conventional concrete, structural topping made of ECC exhibited multi-cracking behaviour with limited crack width and compatible deformations with embedded reinforcement at the initial stage of CAA. However, beyond the tensile strain capacity of ECC, major cracks formed near the face of the end column stub, causing severe strain localisation of the top longitudinal reinforcement in the beam. Besides, at large deformations, high tensile strain capacity of ECC actually hindered the development of flexural cracks in the beam, in particular, at the curtailment point of the top longitudinal reinforcement near the end column stub, thereby substantially reducing the deformation capacity of sub-assemblages. Eventually, fracture of beam longitudinal reinforcement at the face of end column stub led to failure of beam–column sub-assemblages. Through linear variable differential transducers mounted in the plastic hinge region, rotation capacity of plastic hinge could be calculated at the catenary action stage. Finally, the effectiveness of ECC on mitigating progressive collapse was examined to gain a better insight into its potential structural applications.

1. Introduction

Under middle column removal scenarios, the ability of reinforced concrete (RC) structures to develop alternate load paths is dominated by the ductility and continuity of bridging beams over the damaged zone [1,2]. Deformation capacity of the beam primarily depends on the reinforcing bars in plastic hinge regions at the beam ends [3]. Seismic detailing in the beam, which requires more stirrups in the plastic hinge region and longer embedment length of longitudinal reinforcement, has shown little influence on the behaviour of beam–column sub-assemblages subjected to column removal [4]. However, enhanced structural performance may be achieved through seismic design in which seismic loads are considered in the design and more longitudinal bars and shear links are provided in the beams [5]. Besides, development of catenary action in RC beam-slab sub-structures significantly enhance the load-carrying capacity under quasi-static column removal scenarios [6]. An assessment framework was also proposed for building structures subjected to sudden column removal scenarios, in which the pseudo-static response can be determined by the quasi-static load–displacement history [7].

Although extensive experimental and analytical studies have been conducted on the progressive collapse resistance of RC sub-assemblages and frames [8–12], much less attention is paid to precast concrete structures. Continuity of longitudinal reinforcement in these beam–column joints is achieved through 90° bend anchorages, lap-splices, mechanical couplers [13,14] or even welded joints. Mechanical couplers are expensive and usually pose construction difficulty in congested areas such as the joints. It has also been shown that precast concrete frames with welded joints developed poor performance under middle column removal scenarios [15]. By using either lap-splice or 90° bend of bottom reinforcement in the joint, precast concrete beam–column sub-assemblages exhibited similar behaviour to monolithic RC sub-assemblages [16]. Besides, precast concrete structures also allow innovative materials such as engineered cementitious composites (ECC) to be placed in critical regions, such as structural topping and the

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joint region, so as to enhance structural performance under various loading conditions.

The choice of ECC is spurred by Japan Society of Civil Engineers [17], of which the tensile strength and tensile strain capacity can be considered in the design. As a unique high-performance fibre-reinforced cementitious composite with extreme tensile ductility of several hundred times that of concrete, ECC has been intensively investigated in terms of material design [18–20] and structural performance under different loading conditions [21–24]. One potential advantage of applying ECC lies in its compatible deformations with steel reinforcing bars [25], which could significantly reduce the required embedment length or lap length between steel bars to develop the full yield strength. This enhancement in bond strength can relieve reinforcement congestion in the joint regions. Nevertheless, potential applications of ECC to mitigate progressive collapse have not been explored thus far. The behaviour of the joint regions made of ECC remains a concern due to significant deformation demand on bridging beams to develop compressive arch action (CAA) and subsequent catenary action at large deformation stage. Hence, an experimental programme is undertaken to shed light on the behaviour of precast beam–column sub-assemblages incorporating ECC in the critical joint regions.

This paper presents an experimental study on the behaviour of six beam–column sub-assemblages subjected to column removal, in which ECC was placed in the structural topping and the beam–column joint. The resistance and failure modes of sub-assemblages were investigated in the experimental programme. Besides, total deformation of sub-assemblages was decomposed to compare the deformation capacities of sub-assemblages made of conventional concrete and ECC.

2. Experimental programme on beam–column sub-assemblages

2.1. Specimen design

A prototype precast concrete structure was designed in accordance with Eurocode 2 [26]. In the building, column spacing in two orthogonal directions was 6 m. Dimensions of transverse and longitudinal beams were 300 mm by 600 mm. The column section was 500 mm square. In the experimental programme, the prototype structure was scaled down to one-half model. Thus, cross sections of the beams and columns became 150 mm by 300 mm and 250 mm square, respectively.

To develop alternate load paths, one middle supporting column was assumed to be "forcibly removed" without any damage to the beam–column joint [1]. This is a threat-independent approach and it embodies a number of assumptions. Chiefly amongst them is the assumption that only one column is removed at one time of an analysis, which implies that the approach can only be used for a small blast charge. In this regard, the double-span bridging beam with a middle joint above the removed column was extracted from the damaged region and tested under quasi-static push-down loading condition to investigate the resistance and ductility of the joint. Besides, two enlarged concrete stubs were designed at the beam ends to provide horizontal and vertical restraints for the beam–column sub-assemblages, as shown in Fig. 1. It is noteworthy that the hatched zones represent the precast concrete beam units.

To investigate the integrity of precast concrete structures subjected to progressive collapse, precast beam units with cast-in-situ concrete topping were selected in the experimental programme based on local and international construction practices [14,27]. Beam units with bottom longitudinal reinforcement were prefabricated in the casting yard, and then assembled with top reinforcement prior to placement of cast-in-situ concrete topping and beam–column joint. This type of construction technology enables precast concrete structures to perform as well as monolithic RC structures, but at the same time seeks to achieve higher productivity through reinforcement detailing [28]. In the experimental programme, continuous longitudinal reinforcement was placed in structural topping and passed through the middle beam–column joint of sub-assemblages. Two types of bottom reinforcement detailing were used in the joint of sub-assemblages, as shown in Fig. 1. The first joint detailing consisted of 90° bend of beam bottom reinforcement protruding from the beam end and anchored in the joint, as shown in Fig. 1(a). The second detailing was characterised by lap-spliced bottom reinforcement in the joint, as shown in Fig. 1(b). For the second detailing, precast beam units with a trough at each end were cast first, and bottom reinforcement was placed in the middle joint to provide continuity. Based on a concrete cylinder strength of 30 MPa, the anchorage length of bottom steel reinforcing bars was calculated as 470 mm (36 times the rebar diameter). It has been reported that precast concrete beam–column sub-assemblages exhibited horizontal cracking across the interface between precast beam unit and cast-in-situ concrete topping under column removal scenarios [16]. Thus, to prevent horizontal cracking and to ensure adequate composite action between the precast beam unit and the cast-in-situ structural topping, sufficient stirrups at equal spacing were placed along the beam length. Besides, horizontal interface between precast concrete beam units and ECC was intentionally roughened to 3 mm deep, so as to comply with requirements from Eurocode 2 [26]. In the middle column and end column stub, continuous longitudinal reinforcement was provided through the joint and welded to steel plates at two ends, as shown in Figs. 1(a and b).

Control specimen MJ-B-1.19/0.59 made from conventional concrete was designed against gravity loads in accordance with Eurocode 2 [26], with a combination of $1.35w_d + 1.5w_l$ (w_d is dead load and w_l is live load). Besides, another five sub-assemblages with different reinforcement detailing and longitudinal reinforcement ratios in the beam were fabricated, in which ECC was used to replace conventional concrete in the structural topping of the double-span beam and the beam–column joint. Table 1 lists the reinforcement details of beam–column sub-assemblages. In the notations, the alphabets "MJ" and "EMJ" represent precast beam–column sub-assemblages of middle joints incorporating conventional concrete and ECC, respectively, and "B" and "L" stand for 90° bend and lap-splice of bottom bars in the middle joint. The first and second numerals denote the respective percentages of top and bottom reinforcement at the middle joint. Beam–column sub-assemblage MJ-B-1.19/0.59 with conventional concrete beam and structural topping served as the control specimen, in which 90° bend of beam bottom reinforcement was used in the joint. In EMJ-B-1.19/0.59, concrete topping and beam–column joint were replaced by ECC, whereas the other parameters remained the same as MJ-B-1.19/0.59, so as to study the effect of ECC on structural resistance. In comparison with EMJ-B-1.19/0.59, lap-spliced beam bottom reinforcement was applied in the middle joint of EMJ-L-1.19/0.59 (Fig. 1(b)) to study the effect of reinforcement detailing on the middle joint behaviour. To quantify the effect of beam top reinforcement ratio on progressive collapse resistance, the top reinforcement ratio of EMJ-B-0.88/0.59 and EMJ-L-0.88/0.59 was reduced from 1.19% to 0.88% (see 'A-A' section), but the bottom reinforcement ratio was kept the same at 0.59%. Lastly, compared with EMJ-L-0.88/0.59, only the bottom reinforcement ratio was increased from 0.59% to 0.88% in sub-assemblage EMJ-L-0.88/0.88 to investigate the influence of bottom reinforcement ratio.

2.2. Test set-up and instrumentation

Fig. 2(a) shows the test set-up for beam–column sub-assemblages. Restraints connected to each end column stub consisted