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Experimental and numerical investigation on progressive collapse resistance of reinforced concrete beam column sub-assemblages

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ABSTRACT

Based on alternate load path approach, an experimental program was conducted for investigating progressive collapse resistance of reinforced concrete (RC) beam-column sub-assemblages under a middle column removal scenario. Two one-half scaled sub-assemblages were designed with seismic and non-seismic detailing to check the effect of detailing on structural behavior. During the tests, with increasing deformation of the specimens, different structural mechanisms developed subsequently, i.e. flexural action, compressive arch action (CAA) and catenary action. Compared with conventional yielding strength (i.e. capacity of flexural action without considering the existence of beam axial forces), both CAA and catenary action can significantly enhance the structural resistance. The understandings towards these two mechanisms were illustrated at structural, sectional and fiber levels. To simulate the structural responses of the specimens with severe geometric and material nonlinearity, a component-based joint model was proposed and incorporated into macromodel-based finite element analysis in which beams were modeled with fiber elements. The joint model consisted of a series of springs to characterize bond-slip behavior under large tension. Numerical results agreed well with test results. Then the numerical model was used to conduct some parametric studies on the boundary conditions of test specimens, including the axial and the rotational restraints.

1. Introduction

Since the Ronan Point collapse event, the considerations to improve the integrity and resilience of structures to resist progressive collapse have been incorporated into building codes [1,2]. The design methodology in these codes can be broadly classified into indirect and direct methods. The indirect method specifies a minimum level of connectivity among various structural components, and no structural analysis is required. Currently, it is widely used in practice [3]. However, the actual effect of the indirect method is rarely verified by test results, especially for reinforced concrete (RC) structures.

The alternate load path (ALP) method, one of direct methods, is used to evaluate progressive collapse resistance by removing one or several major structural bearing elements and analyzing the remaining structure to determine if this initiating damage propagates from elements to elements. For an RC frame structure, possible ALPs include Virendeel (or flexural) and Catenary action. However, GSA 2003 [2] and British Document A3 [4] do not discuss whether the ALP method involves a catenary mechanism [5]. So far, almost all analytical tools cannot account for catenary action. As a result, these analyses may have the presumption that the ultimate capacity of structure elements is limited by the yield strength of reinforcement.

Catenary action in beams involves large deformations and utilizes tensile force to balance the amplified gravity loads due to doubling of span (associated with the loss of a middle column) and dynamic effect (associated with the sudden loss of supporting force). Currently, only a few published papers referred to catenary action of RC structures subjected to static loads [6–9]. However, in accordance with UFC 4-023-03 [1], static resistance of structures against progressive collapse can be converted to dynamic resistance with a dynamic increase factor, which is the function of the rotation ductility of RC members.

This paper shows an experimental and numerical study on progressive collapse resistance of two one-half scaled RC beam-column sub-assemblages designed in accordance with ACI 318-05 [10], with seismic and non-seismic detailing, respectively. The effect of seismic detailing, mainly the arrangement of stirrups and lap splice of longitudinal reinforcing bars, on catenary action is investigated. Test results demonstrate that the ultimate failure was more intensely developed near the middle joint and the beam ends, suggesting that connection behavior, such as bond-slip, is very crucial to progressive collapse resistance. Due to highly geometric and material nonlinearity in catenary action, a component-based joint model was proposed to facilitate the frame analysis taking account of catenary action. The joint model

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comprising a group of equivalent nonlinear springs was incorporated into macromodel-based finite element analyses in which beams and columns were modeled with fiber elements. The joint model was validated by experimental results, and finally a parametric study of axial and rotational restraints at the beam ends on structural resistance was conducted.

2. Experimental program

Two one-half scaled specimens were designed and tested under a middle column removal scenario. The specimens consisted of a two-bay beam, a middle joint and two enlarged end columns stubs. The specimens were supported by a statically determinate test rig, and a point load was applied at the top of the middle joint with displacement control until the specimens completely failed. During the tests, catenary action of RC beams was mobilized at large displacements.

2.1. Design of specimens

For a public building, perimeter frames are most vulnerable due to ease of accessibility. Therefore, the prototype of test specimens is assumed to be located at the middle of a multi-bay perimeter frame, as shown in Fig. 1. The perimeter frame is located in a 5-story commercial building, of which the ground story is 4.0 m high and a typical story is 3.3 m high. The spans at two orthogonal directions are 6 m. The live load is 4.8 kN/m^2, and the total dead load including self-weight is 7.1 kN/m^2. A uniform dead load of 2.0 kN/m is used for non-structural exterior components applied on the perimeter frames. As a result, the unfactored dead load and live load applied on the perimeter frames are 23.3 kN/m and 14.4 kN/m, respectively. For seismic consideration, it is assumed that the building is designed as a special moment-resisting frame with a base shear coefficient of 0.034 in two directions.

The building is designed to minimize the discrepancy of longitudinal reinforcement of frames based on seismic and non-seismic design, respectively. Typically, seismic design will enlarge the sections of structural members and increase the longitudinal reinforcement ratio. Therefore, for given dead loads and live loads, buildings with seismic design have higher resistance to mitigate progressive collapse since the actions leading to progressive collapse are dead loads and reduced live loads. In this paper, the interest is in whether seismic detailing rather than seismic design will increase structural resistance against progressive collapse.

In Fig. 1b,[1] the directly-affected part (enveloped by red dash lines) is just above the removed column and the adjacent frames (enveloped by green dash lines) are indirectly-affected due to load redistribution. Assuming the two adjacent columns do not fail and the adjacent frames have sufficient lateral stiffness, the shaded part in Fig. 1b becomes the most critical element in the whole frame due to the combined effects of doubling of span and amplified gravity loads.

Based on foregoing loads, a prototype frame was designed in accordance with ACI 318-05 [10], with seismic and non-seismic detailing, respectively. The specimens were then scaled down to one-half of the prototype frame [11]. Both the geometric dimensions of the prototypes and specimens are listed in Table 1. However, to simplify the boundary conditions and focus on structural mechanisms of the beams and the middle joint, the extensions of the columns and beams at both ends of the two-bay beam (in Fig. 1) are replaced by two enlarged column stubs (in Fig. 2). The column stubs are 400 mm wide by 450 mm deep. This design could provide sufficient anchorage for the longitudinal reinforcement in the affected beams. Since the specimens were scaled down, concrete chippings with the maximum aggregate size less than 10 mm were used. Due to symmetry, the detailing of one-half of both specimens S1 and S2 is shown in Fig. 2.

The top reinforcement ratio at the middle joint and the beam ends is 0.90% (1T13+2T10) and 0.73% (3T10) for S1 and S2, respectively. The bottom reinforcement ratio for both specimens is 0.49% (2T10). Note that "T" represents high-yield strength reinforcement with nominal yield strength of 460 MPa. One of the top rebars is curtailed at specified positions for both specimens, as shown in Fig. 2. The bottom reinforcement is lap spliced to investigate whether the lap splice will affect the development of catenary action. According to ACI 318-05 [10], the tension splice of specimen S1 is Class B splice (i.e. equal to 1.3 times the development length l_d of reinforcement) and that of specimen S2 is Class A splice (i.e. l_d). All stirrups are designed as two-legged R6 hoops with a 135° hook. "R" represents low-yield strength reinforcement with nominal yield strength of 250 MPa. The stirrups are distributed at 100 mm on centers except within the plastic hinge locations (two times the beam depth from the joint interface) and lap spliced regions. The stirrups inside the middle joint of S1 consisted of three-legged R6 with a center-to-center spacing of 40 mm.

2.2. Material properties

The material tests included tensile tests for reinforcement, compressive cylinder and split-cylinder tests for concrete. Material properties are listed in Table 2. Note that fracture strain herein is the strain at the ultimate tensile strength, and reinforcement stress is evaluated according to nominal bar areas. The stress–strain relationships of T10 and T13 bars are shown in Fig. 3a.

During the compressive cylinder tests, three linear differential variable transducers (LVDT) were installed along the circumference of the cylinders with 100 mm gage length so that the whole compressive stress–strain curve of concrete could be obtained. The compressive stress–strain relationship of concrete in Fig. 3b indicates that Maekawa's concrete model [12] can fit the test results with acceptable accuracy.

2.3. Test set-up and procedure

Fig. 4 shows the test set-up. To simulate the axial restraints of indirectly-affected frames to a directly-affected frame, the ends of specimens were, respectively connected to a steel frame and a reaction wall through two horizontal pin connections. In vertical direction, each end of the specimens was supported by a pin connection seated on three steel rollers, as shown in Fig. 5a. The steel rollers were used to eliminate the effect of horizontal forces on vertical support reaction forces. Therefore, the measurements of vertical and horizontal reaction forces were independent of each other. Compression load cells were placed at the bottom of the vertical supports, as indicated in Fig. 5a. Two tension/compression load cells were installed at horizontal restraints towards the A-Frame side, as shown in Fig. 5b. Strain gages were mounted at horizontal restraints towards the reaction wall, as shown in Fig. 5c. The load was applied at the top of the middle joint through a hydraulic actuator with displacement control until the specimens completely failed. The actuator was installed onto a steel portal frame. The applied force was measured by a built-in load cell of the actuator. Since all the reaction forces and the applied load were measured, the test system was statically determinate. Because the specimens were quite slender, two lateral restraints were installed to prevent out-of-plane failure. Steel plates with a steel roller were mounted onto the specimen to minimize friction when they came

[1] For interpretation of color in Fig. 1, the reader is referred to the web version of this article.