Computational Model for Nonlinear Behavior of R/C Beam-Column Assemblies and Full-Scale Experimental Validation under a Column Removal Scenario

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ABSTRACT

A number of computational tools have been developed in recent years to simulate the disproportionate (progressive) collapse behavior of multi-story reinforced concrete buildings. Under disproportionate collapse scenarios, beam-column connections of moment-resisting reinforced concrete frames undergo large rotations in addition to sustaining large axial tension forces as a result of the development of catenary action in the beams. Because of this large tension force development, it is important to characterize the interaction of bending moment, shear and axial force accurately in modeling the beam-column connections up to failure. However, it has been difficult to validate the computational models that have been developed for the disproportionate collapse behavior of multi-story concrete buildings due mainly to the lack of experimental data. This paper presents two types of computational models, detailed and reduced finite element models, of reinforced concrete frame structures. To validate the numerical models, two full-scale tests of beam-column assemblies were carried out under a column removal scenario. One of the assemblies represents part of a ten-story reinforced concrete frame building designed for the high seismic region (Seismic Design Category D), and the other represents part of a similar frame building designed for the moderate seismic region (Seismic Design Category C). The analyses showed a good agreement between the experimental results and the computational predictions. Both detailed and reduced models were capable of capturing the primary response characteristics.

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1. INTRODUCTION

After the event of the terrorist attack on the World Trade Center towers in 2001, the engineering community, including codes and standards development organizations and regulatory agencies, has paid greater attention to the performance of buildings when subjected to local damage sustained from abnormal events. In the U.S., the American Society of Civil Engineers Standard 7 (ASCE 2010), and the documents of the U.S. General Services Administration (GSA 2003) and the Department of Defense (DoD 2009) provide guidance to prevent partial or total collapse of a structure resulting from progression of local damage. To avoid such a disproportionate collapse, the guidelines require either (1) providing sufficient strength to structural members that are critical to global stability or (2) providing alternate load paths so that local damage is absorbed and major collapse is averted to maintain structural integrity. This latter approach is treated through the general continuity requirements in the ASCE Standard 7 (2010). In this approach, the structural integrity is assessed by analysis to ascertain whether alternate load paths exist around failed structural members. For concrete structures, the American Concrete Institute Building Code (ACI 318-08) requires minimum continuity reinforcement to provide overall structural integrity. At present, these guidelines and code requirements are mainly based on engineering experience with limited support of experimental data. The National Institute of Standards and Technology (NIST) is currently carrying out a comprehensive analytical and experimental research program to study the behavior of structures that when exposed to abnormal loads, might experience disproportionate (or progressive) collapse.

This paper presents two types of computational models, one a high-fidelity finite element model and the other a macro model with reduced complexity, as well as full-scale testing of two reinforced concrete (RC) beam-column assemblies representing portions of ten-story buildings; one designed for Seismic Design Category C (SDC C) and the other designed for Seismic Design Category D (SDC D).

2. COMPUTATIONAL MODELS

Two types of computational models were developed for reinforced concrete frame structures. The detailed finite element model is a high-fidelity model while the reduced finite element model is a component-based model. The high-fidelity model captures local behaviors in more detail than the reduced model. Due to its greater accuracy, the detailed finite element model was used to verify the reduced model when test data for validation were not available (Bao 2008). However, the high cost of model development and the computational expense limit its application to large-scale structural systems. The reduced model is computationally efficient. With careful calibrations, the reduced model can predict structural responses both efficiently and accurately.

In this study, the above-mentioned computational models were implemented in LS-DYNA (Hallquist 2007) , a general-purpose finite element software package using explicit time integration.

Analyses account for both geometric and material nonlinearities, including element failure and erosion.

2.1. Detailed Model

In the detailed finite element model, concrete is modeled using solid elements and reinforcing bars are modeled using beam elements. Bond slip between reinforcing bar and surrounding concrete is modeled using a CONTACT_1D card which defines one-dimensional slide lines for reinforcing bar in concrete. An elastic-plastic material model (material 124 in LS-DYNA) is used to describe the reinforcing bar properties. In this model, unique stress versus plastic strain curves can be defined for compression and tension separately. Once the failure strain is reached, the element is eroded, simulating fracture of the reinforcing bar. A continuous surface cap model (material 159 in LS-DYNA) is used as the material model for concrete. This model captures important features of concrete such as confinement effects and tensile or compressive softening behavior. A regulation technique is also implemented in this model to reduce the mesh sensitivity caused by energy localization.

2.2. Reduced Model

Beams and columns in the reduced model are represented by beam elements with cross-section integration. In this beam formulation, the cross section is divided into fiber sections which are represented by integration points. Reinforcing bars in concrete are modeled by assigning the steel property to the corresponding integration points. A constitutive model that incorporates confinement effect is specified for the integration points which represent the core concrete in order to represent the enhancement of concrete strength and ductility of concrete due to steel hoops or spirals. The beam-to-column connection model is depicted in Fig. 1. The joint region is modeled by 4 rigid elements with rotational springs located at the corners. Shear behavior of the joint region is implemented through the moment-rotation relationship of the rotational springs. The beam-joint interface is modeled by a Hughes-Liu beam element which includes finite transverse shear strains. Bond-slip effects are considered by modifying the constitutive relationship of reinforcing bars as follows (Lew et al. 2011):

$$\varepsilon = f(L \ \sigma_s, s \ , L_0) = f \ \sigma_s, L_0 \tag{1}$$

$$L(\sigma_s, s) = \Delta l \ (\varepsilon) + s = \Delta l(\sigma_s) + f'(\sigma_s) \tag{2}$$

where L_0 is the original length, L is the deformed length including slip, σ_s is steel bar stress, and s is slip between reinforcing bars and surrounding concrete. The slip s can be expressed as a function of the steel stress σ_s; therefore it can be removed from the expression.

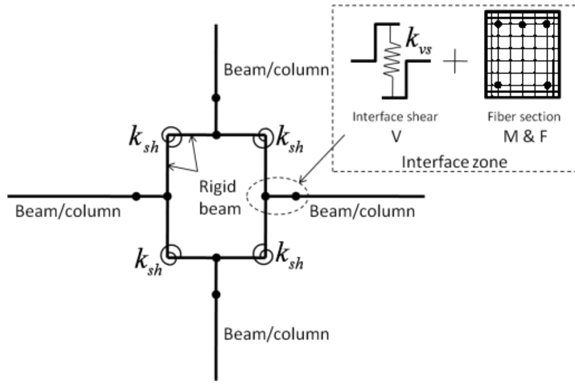

Fig. 1. Reduced beam-to-column connection model

3. EXPERIMENTAL VALIDATION

3.1. Test Specimens

Test specimens, which consist of two end columns, two beams and one center stub column, are part of two ten-story prototype buildings which were designed for Seismic Design Category C (SDC C) and SDC D. For both the buildings, exterior moment frames were designed to resist lateral loads. The prototype buildings have plan dimensions of 30.5 m × 45.7 m with five bays in the longitudinal and transverse directions. First story height is 4.6 m and the height of the other stories is 3.7 m. The building designed for SDC C used intermediate moment frames (IMFs) as defined by ACI 318 (ACI 2008) and similarly, the building designed for SDC D used special moment frames (SMFs). Fig. 2 shows member sizes and reinforcement details for the IMF and SMF specimens. Two end columns are taken from the mid height of the second story to the mid height of the third story. The center stub column has extension of 0.305 m above the beam and 0.61 m (2 ft) below the beam.

The compressive strength of concrete for the IMF and SMF specimens was 32.4 MPa and 35.9 MPa, respectively. Grade 60 A706 reinforcing bars were used for both specimens. The mechanical properties of the reinforcing bars are listed in Table 1.

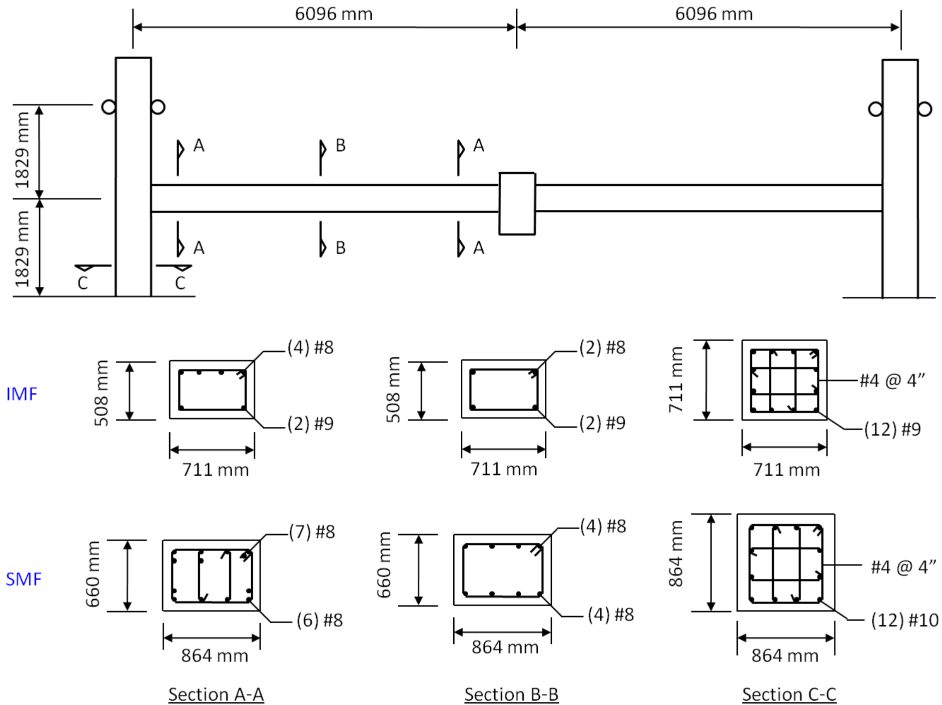

Fig. 2. Dimensions and reinforcement details of test specimens

Table 1 Properties of Reinforcement

Bar Size	f_y (MPa)	f_u (MPa)	Rupture Strain
8	476	648	21 %
9 (Beam)	462	641	18 %
9 (Col)	483	690	17 %
10	503	731	16 %
4 (IMF)	537*	710	14 %
4 (SMF)	555*	676	15 %

*0.2% offset yield strength

3.2. Test Setup

A schematic view of the test setup for both the IMF and the SMF specimen is shown in Fig. 3. The longitudinal bars in the columns with 610 mm-long 90˚ hooks were fully embedded in the footings. Each footing was clamped down to the test floor using ten 32 mm post-tensioning bars. The total clamping force on each footing was about 6670 kN. Horizontal support at the top of each column was provided by rollers. These rollers reacted against 51 mm thick steel plates attached to the interior and exterior faces of the column. The horizontal movement of the steel plates at the exterior face of the column was restrained by four 32 mm post-tensioning bars which were anchored to a reinforced concrete block. The block was fixed to the reaction wall by post-tensioning bars.

In order to avoid premature lap splice failure, the top and bottom longitudinal reinforcing bars in the beams were spliced with threaded couplers located at midspan. All beam longitudinal bars were anchored at the exterior beam-column joints by means of threaded mechanical anchorage devices. The concrete adjacent to the devices was confined with spirals made with Grade 40 plain wire.

As shown in Fig. 3, the vertical load was applied to the top of the center stub column by means of four post-tensioning rods that were pulled down by four hydraulic rams, each having a capacity of 534 kN and a stroke of 100 mm. Under this loading scheme, the out-of-plane movement of the stub column was restrained. The load was applied under displacement control at a rate of 25 mm/min. Instrumentation used in the concrete assembly tests included displacement transducers, inclinometers, vertical displacement encoders, Optotrak (a surface position measuring device) targets, whittemore gages, and strain gages cemented to reinforcing bars. The uncertainty in the measured data from the load cells, deflection and strain gages, and inclinometers was within ±1 %.

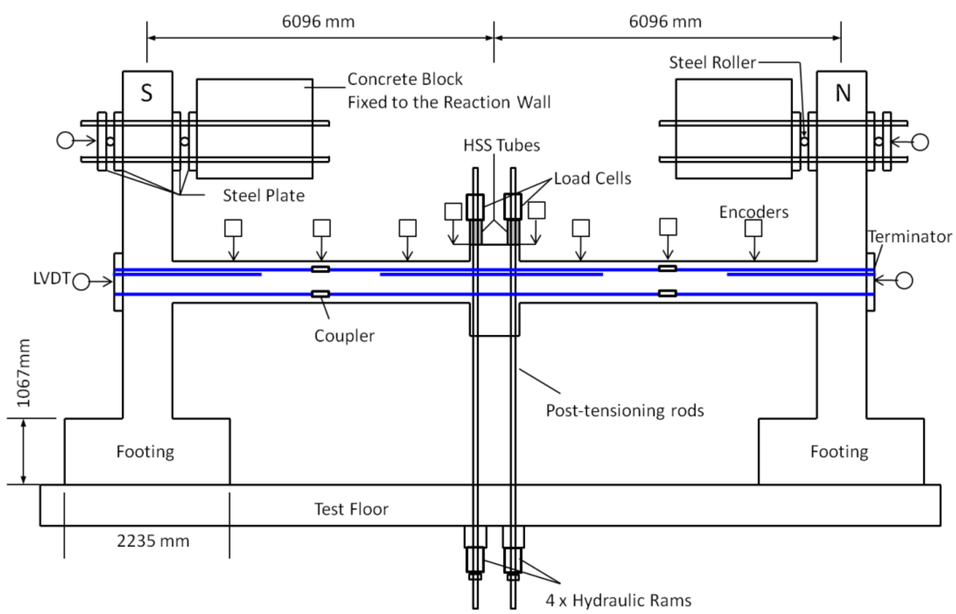

Fig. 3. Test configuration

3.3. Results and Comparison

The vertical displacements of the center stub column versus the corresponding load levels for the IMF specimen are plotted in Fig. 4. Similar plots for the SMF specimen are shown in Fig. 5. As the plots indicate, the specimens were slightly unloaded when the hydraulic rams were stopped every 64 mm to 76 mm to take instrument measurements and photographs. As seen in Fig. 4 and Fig. 5, no significant loss of stiffness was observed at early loading stages until yielding commenced in the longitudinal reinforcing bars of the beam due to bending moment. After yielding, the load continued to increase until it reached a peak of 296 kN at a center stub column deflection of about

127 mm for the IMF specimen, and 903 kN at a center stub column deflection of 112 mm for the SMF specimen. After reaching a peak load, the load started to drop with increasing deflection for both specimens. The end columns at the beam mid-height deflected outward initially as the load at the center stub column increased. After the load began to drop, the column started to move in the opposite direction and eventually changed to inward movement. This indicates a transformation of the beam axial force from compression to tension as shown by the model predictions in Fig. 6. As the deflection increased, splitting cracks and spalling of concrete developed in the compressive regions at both ends of the beams. The load leveled off at 196 kN at a center stub column deflection of 406 mm for the IMF specimen and at 640 kN at a center stub column deflection of 500 mm for the SMF specimen. Subsequently, the load started to increase again with increasing deflection. When the stub column deflection was about one-depth of the beam, the strain of the top reinforcing bars at the mid span of the beams changed from compression to tension, indicating the development of catenary action. Full-depth cracks were also observed within the center region of the beam span which is another evidence for development of tensile axial force in the beams. Catenary action enabled the assemblies to reach a maximum load of 547 kN at a center deflection of 1092 mm for the IMF specimen and 1232 kN at a center deflection of 1219 mm for the SMF specimen. The failure of the assemblies was brought about by fracture of the bottom- most longitudinal reinforcing bars, which occurred at a major crack in the beam near the center stub column.

Figure 4 and Fig. 5 show that both the detailed and the reduced models predict reasonably close to the experimental value. The computational models captured the initial peak load, and subsequent dropping of load due to the formation of plastic hinges at the beam ends, and the development of catenary action. Good agreement between these results provides a validation for the developed finite element models.

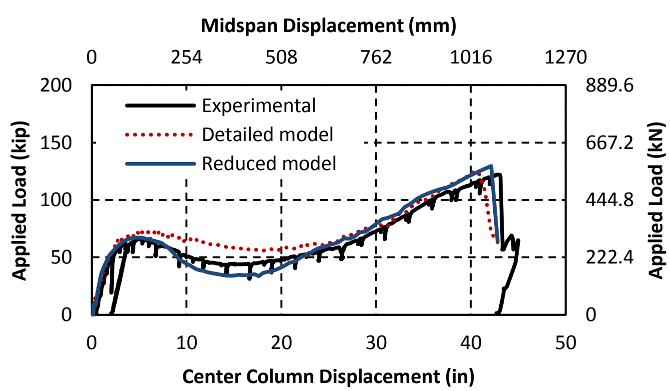

Fig. 4. Vertical load versus vertical displacement of center stub column for the IMF specimen

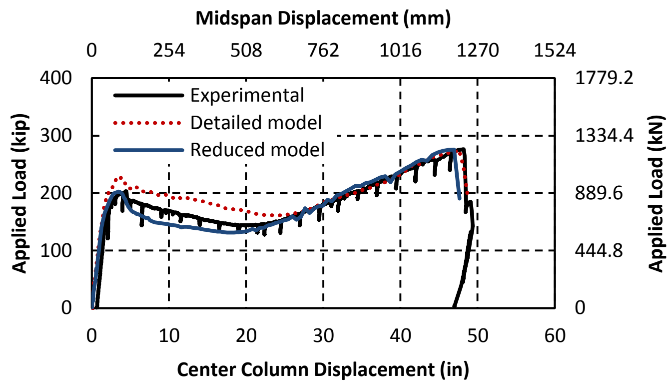

Fig. 5. Vertical load versus vertical displacement of center stub column for the SMF specimen

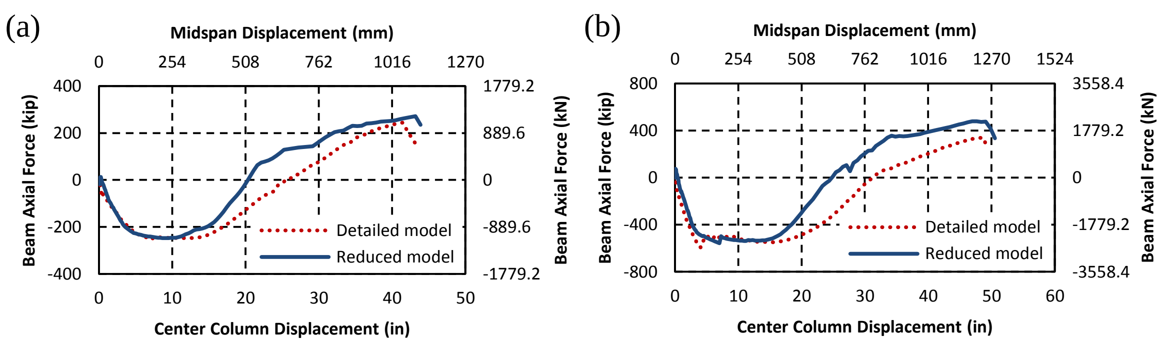

Fig. 6. Beam axial forces versus vertical displacement of center stub column for (a) IMF specimen and (b) SMF specimen

SUMMARY AND DISCUSSION

This paper presented two types of computational models for reinforced concrete frame structures: one a high-fidelity detailed model; and the other a component-based reduced model. The detailed model requires fewer assumptions and simplifications. It involves a large number of elements and is capable of representing the behavior and failure modes of the structures in great detail. It can be used for verification of the reduced model when experimental data are not available. The reduced model, which involves much fewer elements than the detailed model, can be executed much more efficiently.

Two full-scale beam-column assemblies, one designed for SDC C and the other designed for SDC D, were tested in order to validate the computational models. Good agreement between the experimental results and the computational predictions was confirmed. The analyses also showed that the numerical models captured three stages of force transfer under a column removal scenario: 1) compressive arching action due to axial restraint by boundary columns, 2) plastic hinge development with concrete crushing and reinforcing bar yielding, 3) catenary action due to tensile forces mobilized in the beams. The validated reduced model can be used to predict the behavior and failure load of structures, and would be valuable in the disproportional collapse analysis of complete structural systems.

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