Resilient Structures and Sustainable Construction
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INCREASING ROBUSTNESS OF REINFORCED CONCRETE STRUCTURES UNDER COLUMN LOSS SCENARIO

SAID ELKHOLY and BILAL EL-ARISS

Civil and Environmental Engineering Dept, UAE University, Al Ain, UAE

This paper presents a simple technique to enhance robustness of reinforced concrete (RC) plane frames to progressive collapse under column loss scenario. The response of the enhanced/mitigated RC frames is analyzed using fiber force-based finite element analysis and applying displacement-controlled nonlinear static pushdown at the location of failed column. The technique involves addition of external unbounded steel cables to the continuous beam in each floor at anchorage and deviator locations. The cables transfer the loads above the failed column to the anchorages and deviators that are assumed to perform as rigid arms, which in turn redistribute the loads to adjacent columns. The numerical model computes the frame progressive collapse robustness using push-down analysis to simulate a column elimination and estimate the effects of cable catenary action on the frame. Two-dimensional RC frame of six stories and four bays was adopted in the study. The numerical results demonstrate the prospect of increasing robustness of RC frames to progressive collapse using presented technique.

Keywords: Progressive collapse, RC plane frames, External unbounded steel cables, Fiber element approach, Cable deviators, Catenary action.

1 INTRODUCTION

The need to limit the disproportionality of collapse in structures due to abnormal loadings has placed a great emphasis on the consideration of progressive collapse in the design community. Currently, national codes and design standards do not provide explicit guidelines for designing progressive collapse resistant structures. However, there are general provisions for load redistribution capability to satisfy a minimum level of structural integrity. Sudden column loss is the most common direct design method recommended for progressive collapse mitigation by current design codes and guidelines, ASCE/SEI 7-05 (2005) and Unified Facilities Criteria (UFC) (2009). According to these guidelines, also referred to as the alternate load path method, the potential for progressive collapse may be diminished by designing the structure so that it can bridge across the local failure zone resulting from instantaneous removal of a primary vertical support member. A retrofitting scheme by Hadi and Alrudaini (2012) was proposed and examined for viability to resist progressive collapse of a ten-story RC building resulting from a first-floor column failure. The proposed scheme is based on the concept of increasing the building redundancy by mounting steel cables parallel to the columns from top of a hat steel braced frame installed on top of the building and attached to the beam ends. The numerical results demonstrated the possibility of preventing progressive collapse of RC buildings by implementing the proposed scheme. Kim and An (2009) performed static push-down analyses of three and six-story steel structures with and without bracing to investigate the effect of catenary action on the progressive collapse resistance. The numerical result showed that the effect of catenary action increased significantly in braced frames due to the restrained movements of the beam-column joints. Lew *et al.* (2014) performed an experimental investigation of two RC beam-to-column joint specimens exposed to monotonically increasing vertical displacement at the location of a removed column. Experimental results showed that the loads were largely resisted by the catenary action developed in the beam tension steel reinforcement. Reinforced concrete beam-column connections were also examined by Bao *et al.* (2012) using displacement control push down analysis in their finite element models at the location of failed column. The numerical results indicated that catenary action in the beams could be capture. Qian and Li (2012), (2013a), and (2013b) carried out push-down experimental studies on RC beam-column connections and flat-slab structures to simulate progressive collapse behavior in the occurrence of a corner column failure. They concluded that seismic detailing of specimens would contribute to better progressive collapse performance and that analytical studies to capture the tensile membrane action developed in the slabs for progressive collapse should be conducted. Elkoly and El-Ariss (2014) proposed a mitigation scheme and examined its validity to prevent potential progressive collapse in RC continuous beams subsequent to interior column failure. The mitigating scheme comprised of external unbounded unstressed straight fiber reinforced plastic (FRP) cables attached to the beam at anchorage and deviator locations to bridge over the failed column. Elkholy and El-Ariss (2016a) highlighted the effects of different mitigating scheme setups/arrangements on the resistance of such beams to progressive collapse using FRP cables. The mitigating schemes utilized had external FRP cables with different changing profiles and varying deviator locations to examine their effects on the resistance of progressive collapse of beams, and to identify the optimal setup mitigating scheme that efficiently enhanced the beam resistance to progressive collapse due to interior column removal. Elkholy and El-Ariss (2016b) underlined the effects of the mitigating scheme on the resistance of concrete structures to progressive collapse using steel cables. The objective of this paper is to assess the validity and effectiveness of a mitigating scheme on the progressive collapse robustness of RC plane frame buildings subjected to an interior/edge column failure scenario. Extensive nonlinear static pushdown analyses were performed to validate and obtain the optimal frame mitigating scheme configuration. Steel external mitigating cables are considered in this paper because of their ductile nature, cost effectiveness, and convenient availability.

2 RC FRAME ANALYZED

2.1 Control RC Frame

The control frame analyzed in this study is a six-story, four-bay 2D frame with 3 m story height and 4 m bay length. T-beams were used in the frame model where the effective flange width on each side of the web was set equal to one fourth of the span length (ACI 318-08). Reinforcing steel bars used have an average yielding strength of 360 MPa. Concrete material used has a mean compressive strength of 25 MPa, a mean tensile strength of 2.2 MPa, and a modulus of elasticity of 24,870 MPa. Material and geometric nonlinearities are accounted for. Dimensions and reinforcements of RC frame members are listed in Table 1.

2.2 Improved/Mitigated RC Frame

The technique proposed in this study for improving (or mitigating) frame structure progressive collapse robustness is based on the concept of increasing the structure redundancy to bridge over the prospective interior failed column. To attain this goal, external unbounded steel cables are hung longitudinally to the web of the continuous beam of each floor of the control frame structure. The cable installation is done by externally attaching the cable to the beam web at two anchorage zones, located at edge ends of the continuous beam, and at scattered deviator locations along the beam length to ensure transferring the loads above the failed column to the external cables, which in turn redistribute these loads to the adjacent beams and columns. The purpose of the external cables is to offer a mixture of strength and ductility to the frame.

Table 1. Dimensions and reinforcements of six-story, four-bay 2D RC frame.

		Floor Number	
		1st, 2nd, 3rd	**4th, 5th, 6th**
Beams	Cross Section (m2)	0.25 x 0.50	0.25 x 0.50
	Reinforcement (Top and bottom)	4 ϕ 16	4 ϕ 16
	Reinforcement Ratio (%)	0.64 %	0.64 %
Edge Columns	Cross Section (m2)	0.25 x 0.80	0.25 x 0.70
	Reinforcement	10 ϕ 16	10 ϕ 16
	Reinforcement Ratio (%)	1.00 %	1.15 %
	Stirrups	5 ϕ 10 /m	5 ϕ 10 /m
Inner Columns	Cross Section (m2)	0.60 x 0.60	0.50 x 0.50
	Reinforcement	20 ϕ 16	16 ϕ 16
	Reinforcement Ratio (%)	1.11 %	1.29 %
	Stirrups	5 ϕ 10 /m	5 ϕ 10 /m

3 ANALYTICAL MODELING OF THE RC FRAME

In the analytical model, each beam of the frame is modeled as an assemblage of straight beam elements simulating the beam member, an assemblage of cable elements simulating the external steel cables, and an assemblage of rigid arms connecting the external cable nodes (anchorage zones and deviator points) with the beam nodes as shown in Figure 1. The failed column could be any interior column of any floor. However, the damaged column considered in this study is the middle column in the ground floor of the frame, Figure 1. As for the external steel cables, the cable area is varying from 10% up to 60% of the beam internal reinforcing tension steel area. In this study, both control and improved/mitigated RC frame structures are modeled by fiber section based elements and analyses are performed by applying displacement-controlled nonlinear static pushdown (also known as vertical pushover) at the location of failed column to predict the progressive collapse robustness of the damaged control and improved/mitigated structures. The displacement-controlled pushdown analysis for progressive collapse is carried out by gradually increasing the vertical displacement of the bridging beam at the location of the failed column to simulate the removal of the middle column.

3.1 SeismoStruct Distributed Plasticity Model

Fiber element-based software, SeismoStruct (2011), was used in this study to model both the control and mitigated frames. SeismoStruct employs the displacement-based element formulation and the corresponding displacement-based elements adopt inelastic cubic displacement shape functions and present distributed plasticity. The cross-section of the members is modeled by a number of individual fibers (typically 100-150) separately representing concrete and steel; the discretization of a typical reinforced concrete cross-section is depicted in Figure 2.

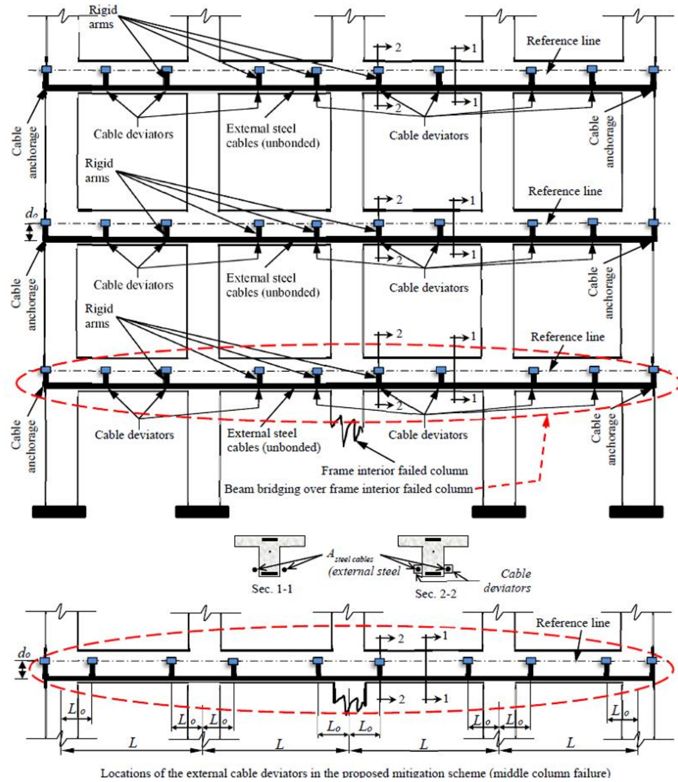


Figure 1. Multi-story improved/mitigated RC frame building.

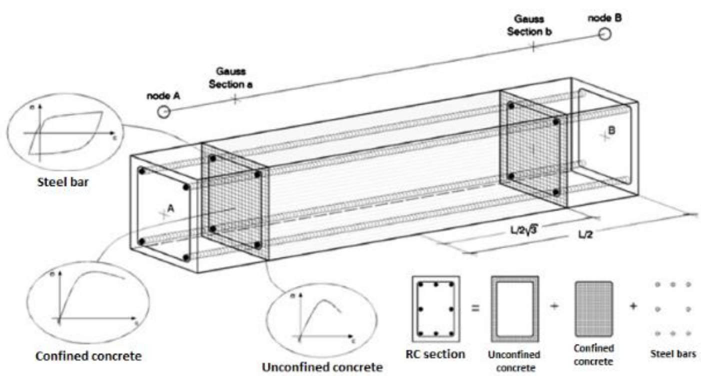
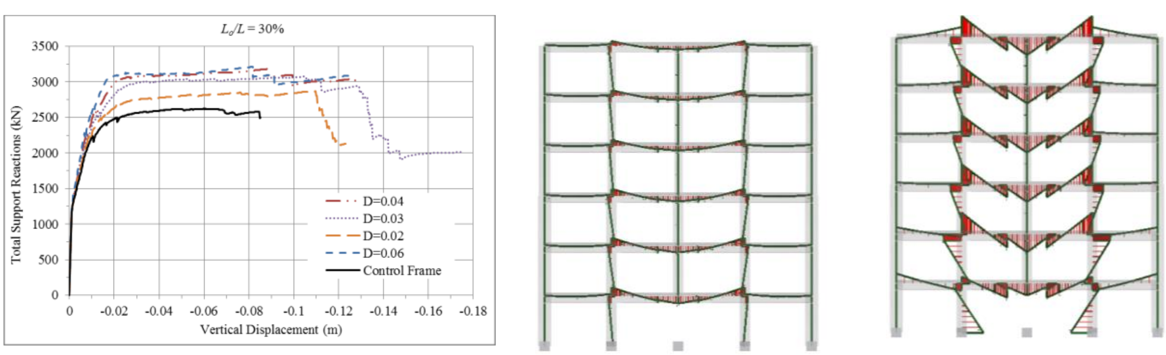


Figure 2. Multi-story improved/mitigated RC frame building.

4 FRAME PROGRESSIVE COLLAPSE ANALYSIS AND RESULTS

The control frame is a frame without the proposed improving/mitigation scheme. The analysis of the control frame is demonstrated through its capacity curve which is a relationship between the vertically controlled displacement of the bridging beam at the damaged column location and the frame total vertical support reaction, as shown in Figure 3(a). The controlled displacement was carefully increased, gradually, to capture the instant at which each performance criterion (stage) is reached within the frame structure. The mitigated frame of Figure 1 is actually the control frame with the externally installed improving/mitigating scheme. The control and mitigated frame performance criterion (stage) sequence of occurrence are obtained directly from SeismoStruct "Analysis Log" and are depicted in Figure 3(a). Nonlinear static vertical (down) pushover analysis is performed by applying a pseudo static displacement in the bridging beam at the collapsed column location and by using the displacement control approach for the incrimination of the loading factor. The limit failure of the mitigated frame considered in this work is the steel cable rupture limit, which may or may not correspond to the ultimate strength limit of the mitigated frame. Parameters considered in this study, where the external steel cable profile is straight, include total cross-sectional area of the external cables and cable deviator locations with respect to the bean span (L_o and L as in Figure 1). The total area of the external steel cables is referred to in the subsequent discussions and charts as "equivalent diameter" (D). Equivalent diameter (D) is meant to indicate a diameter of a cable whose area is equivalent to the total cable steel cross-sectional area. The mitigated frame was analyzed with a mitigating cable configuration of L_o/L = 30% and D = 0.03 m and two corresponding bending moment diagrams are revealed in Figure 3(b) and Figure 3(c). A bending moment diagram at early stage before plastic hinge formations is demonstrated in Figure 3(b), and a bending moment diagram at a vertical displacement of 0.125m in the bridging beam at the failed column location is established in Figure 3(c) with plastic hinge formations (before failure). As the plastic hinges develop in the beams, the bending moments are redistributed as depicted by the sudden changes in the moments at the locations of rigid arms/deviators, Figure 3(c), indicating that the mitigating cable system has produced counter acting moments at the deviator locations. The counter acting moments produced by the cables enhance the structure performance by redistributing the maximum moments in the beams. The ultimate magnitudes of the counter acting moments at the deviators control the efficiency of the mitigating cable system and; therefore, govern the performance of the mitigated frame structure.



(a) Capacity curves of control and improved/mitigated frames; Lo/L = 30%.

(b) Bending moment diagram at early stage.

(c) Bending moment diagram at vertical displacement of 0.125 m in bridging beam at location of failed column.

Figure 3. Capacity curves and bending moment diagrams of the improved/mitigated frame.

5 CONCLUSIONS

In this study, a mitigating scheme is proposed to improve the progressive collapse robustness of RC frame buildings subjected to interior/edge column failure. The scheme is simple and comprises of placing external unbounded unstressed steel cables attached to the frame continuous beam in each floor at anchorage and deviator locations. The cables transfer the loads above the failed column to the deviators and anchorages that are assumed to perform as rigid arms, which in turn redistribute the loads to adjacent columns. Control and mitigated frame buildings were modeled by a fiber force-based finite element software, SeismoStruct, to perform structural analyses by applying displacement-controlled nonlinear static pushdown at the location of the failed column to predict the progressive collapse robustness of the damaged control and mitigated frames. The numerical results indicate that he most efficient mitigating scheme is that with a ratio of L_o/L =30%, an equivalent cable diameter of D=0.03m (representing area of external steel cables equals to 30% of the beam tension steel reinforcement). The results show that the maximum ductility of the mitigated frame is about 61% more than that of the control frame and the frame ultimate and failure load capacities increased by 24% and 30%, respectively. It should be mentioned that the results of the analysis in this paper were based on ignoring compression membrane response of beams and slabs, assuming that the beam-column joints were rigid enough to initiate catenary action in beams, and ignoring the friction in external cables. It would be recommended that further investigations considering the above simplifications should studied as well as experimental investigations be conducted to demonstrate the practicality and efficiency of the proposed scheme in real structures.

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