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New Building Scheme to Resist Progressive Collapse

Muhammad N. S. Hadi, M.ASCE;¹ and Thaer M. Saeed Alrudaini²

Abstract: In this paper, a new scheme is proposed for retrofitting RC buildings to resist progressive collapse that may result from a first floor column failure. The proposed scheme is comprised of placing vertical cables connected at the ends of beams and hung on a hat steel braced frame seated on top of the building. In case of a column collapse, the cables transfer the residual loads above the failed column to the hat-braced frame, which, in turn, redistributes these loads to the adjacent columns. A finite-element modeling and a nonlinear dynamic analysis following the alternate path method (APM), as recommended by U.S. General Services Administration guidelines, are used to assess the viability of the proposed scheme. A 10-story RC building designed according to Australian Standard AS 3600 was adopted in the investigations. The investigation results demonstrate the possibility of preventing the progressive collapse of RC buildings by implementing the proposed scheme. DOI: [10.1061/\(ASCE\)AE.1943-5568.0000088](https://doi.org/10.1061/(ASCE)AE.1943-5568.0000088). © 2012 American Society of Civil Engineers.

CE Database subject headings: Progressive collapse; Concrete structures; Buildings; Rehabilitation; Dynamic analysis.

Author keywords: Progressive collapse; Buildings; Retrofitting; Alternate path method; Dynamic analysis.

Introduction

Buildings are structurally designed to support anticipated loads adequately and safely, in addition to fulfilling the needs of clients, which include functional and aesthetic requirements. However, these designs do not normally account for the extreme loading events that may cause progressive collapse. The progressive collapse of a building refers to the phenomenon when a failure of an individual structural element leads to a partial or an entire failure of the building. The extreme loading events that are not considered in the conventional design and can cause progressive collapse include gas explosions, fire, car collision, and bomb explosions (ASCE 2005). Recently, different governmental guidelines [U.S. General Services Administration (USGSA) 2003; Unified Facilities Criteria (UFC) 2009] and some design codes and standards [ASCE 2005; American Concrete Institute (ACI) 2005; National Institute of Standards and Technology (IST) 2007] have addressed the progressive collapse in the design guidelines. Consequently, designing and retrofitting existing buildings to prevent progressive collapse have become imperative challenging issues for structural engineers to satisfy the recent standards that involve structural, architectural, and economic factors.

Different studies on progressive collapse have been published, which include investigations and suggestions for enhancing the resistance of buildings to progressive collapse. Several studies have been undertaken to investigate the relationship between the seismic design and the progressive collapse capacity of buildings (Baldrige and Humay 2003; Bao et al. 2008; Hayes et al. 2005; Tsai and Lin

2008). These studies have shown that buildings designed for high seismic regions performed better and are less vulnerable to gravity-induced progressive collapse than buildings designed for low to moderate seismic risk. However, there are very few explicit design schemes to prevent progressive collapse, especially for RC buildings, which include: using horizontal steel cables benefited from catenary action to prevent progressive collapse (Astaneh-Asl 2003), using steel bracings in RC frames to increase the progressive collapse resistance of the corner panels (Mohamed 2009), and using CFRP to provide sufficient continuity in beams with missing reinforcement continuity to mitigate the potential progressive collapse (Orton et al. 2009).

In this paper, a new scheme is proposed for retrofitting RC buildings to absorb and redistribute the residual gravity loads induced by column loss. The viability of the proposed method has been investigated according to the alternate path method recommended by the USGSA (2003) in which different scenarios for first floor column failures are adopted and used for checking existing buildings and designing new buildings. In this study, a numerical investigation is undertaken for a 10-story RC building to show the building resistance to progressive collapse with and without the proposed retrofitting method. Finite-element software ANSYS 11.0 (ANSYS 2008) is used in the numerical simulation.

Proposed Scheme

The proposed retrofitting scheme is based on the concept of increasing the redundancy of the building, to bridge over the potential failed columns. To achieve this goal, a hat-braced steel frame is placed on the top of the building, and vertical steel cables are placed parallel to the columns to provide an alternate path over the potential failed column. These vertical steel cables are connected at the ends of the beams and hung at the top of the hat-braced steel frame that is seated on the top of the building. The retrofitting scheme includes installing vertical cables after constructing the building structure. Fig. 1(a) illustrates the elevation view of a typical building after installing the cables and the hat-braced steel frame. Steel plates are fabricated and welded to form a seating base for hanging the beam ends by the cables, as illustrated in Figs. 1(b and c). In effect, in the case of a column failure, the loads that are transferred through the

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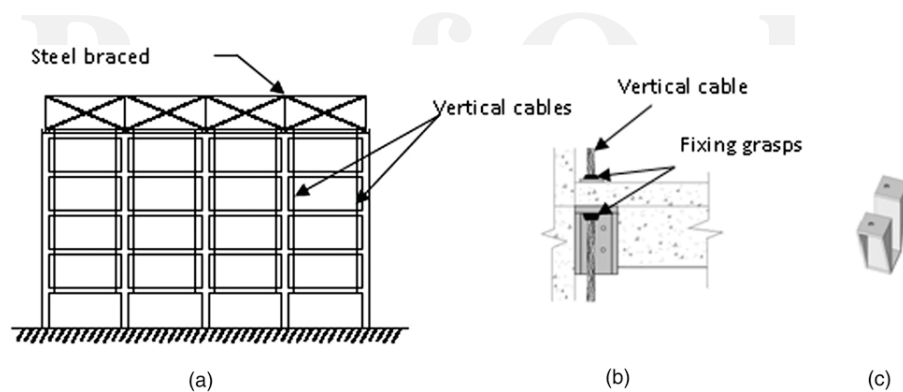


Fig. 1. Proposed retrofitting scheme: (a) building view after applying retrofitting scheme; (b) cable connection at end of beam; (c) hanging seat of the beam

column down to the footings are transferred up to the roof through cables. A hat-braced frame that is seated on the roof of the building redistributes the loads to the adjacent columns. The proposed scheme utilizes steel cables to compensate for the tension deficiency of RC columns in transferring the floor loads upward to the hat-braced steel frame.

In the proposed scheme, cables are designed to carry gravity loads considering the tributary area of the associated columns. The static load combination equal to $2(D.L. + 0.25L.L.)$, as specified by the USGSA (2003), is adopted for the preliminary design of the cables. The hat-braced frame is designed to carry the same load used in designing the cables, considering double span or cantilever span developed over the failed column depending on the location of the potential failed columns.

Structural Modeling

The finite-element program *ANSYS 11.0* (ANSYS 2008) is used to model the structural members of the building. Beams and columns are modeled using three-dimensional (3D) frame elements (BEAM4). All live loads and dead loads carried by the slabs, including the slab weight, are distributed onto the supporting beam elements according to the tributary area. The assumption of distributing slab weight onto the supporting beams is used to simplify the modeling. The assumption of ignoring the slab effect neglects the contribution of its stiffness, which leads to conservative results. The inelastic behavior of beams is modeled by placing nonlinear rotational springs (COMBIN39) at the ends of beams, to account for the potential plastic hinges. The characteristics of these springs are determined using section analysis (Park and Paulay 1975). Fig. 2 shows the moment rotation relation of the nonlinear rotational spring used in this study, which includes both positive and negative yield and ultimate moment capacities and their associated rotations. The moment of inertia of beams used in this study was half the uncracked moment of inertia, and for columns the moment of inertia was 0.7 the uncracked moment of inertia [Federal Emergency Management Agency (FEMA) 2000]. The members of the top hat-braced frame are modeled using 3D frame elements (BEAM4) and the bracings are modeled using axial elements capable of carrying axial compression and tension forces (COMBIN8), whereas the cables are modeled using axial elements with tension-only capability (COMBIN10).

In this study, the hat-braced frame is linked to the columns using contact elements (CONT178) that only allow the transfer of compression forces from the hat-braced frame to the columns. However, transfers of forces above the failed column are transferred by the cables to the hat-braced frame. Rigid beam elements (BEAM4) are used to connect the cable nodes with the hat-braced frame nodes, to

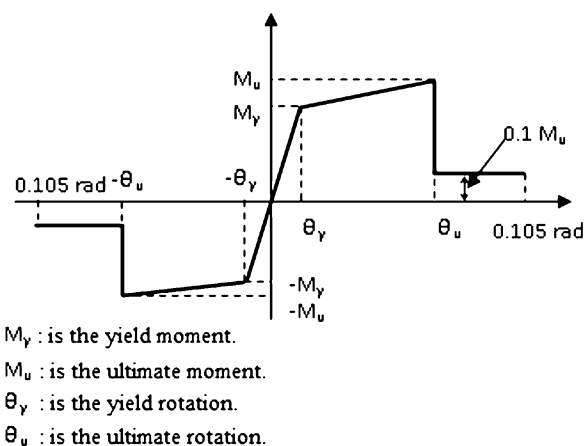


Fig. 2. Rotational spring properties used in modeling plastic hinge in simplified finite-element procedure

allow the transfer of loads from the cable to the hat-braced frame. The connection points of the hat-braced frame to the column and the braced frame to the cables are illustrated in Fig. 3.

The RC columns usually have different compression and tension stiffness. FEMA (2000) recommended modeling RC columns with a compression stiffness equal to the axial stiffness of the gross concrete area and a tension stiffness equal to the axial stiffness of the longitudinal reinforcing bars of the column.

Conventionally, most columns are subjected to compression forces in which the compression stiffness is utilized in the modeling. In this study, columns above the failed column are modeled with different tension and compression stiffness, to capture the expected reversal of forces. This was conducted by modeling the columns above the potential failed column by combining two elements: the first element is a 3D elastic frame element (BEAM4) with only a defined section moment of inertia to account for the bending capability, whereas the second element is a nonlinear axial line element (COMBIN39) with defined compression and tension stiffness, as shown in Fig. 4. Table 1 summarizes the four different types of elements used in modeling the structural members and their properties. As shown in Table 1, all elements used have the capability of geometric nonlinearity. BEAM4 has linear elastic material. COMBIN39 in the beams represents the plastic hinges with defined moment rotation properties; however, COMBIN39 in the columns above the failed column is used to define the nonlinear axial behavior of RC columns with defined force deflection properties. COMBIN10 defined only tension capability that is used to model

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cables. Finally, COMBINE8 has inelastic material to define bracings in the hat-braced frame.

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Assessment Method

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The investigations utilized a nonlinear dynamic analysis method recommended in the USGSA (2003) guidelines, in conjunction with the alternative path method (APM). The load combination (dead load + 0.25 live load), omitting the magnification factor “2,” as recommended by USGSA (2003) guidelines, was used in the dynamic analysis. In addition, the USGSA (2003) has specified different failure

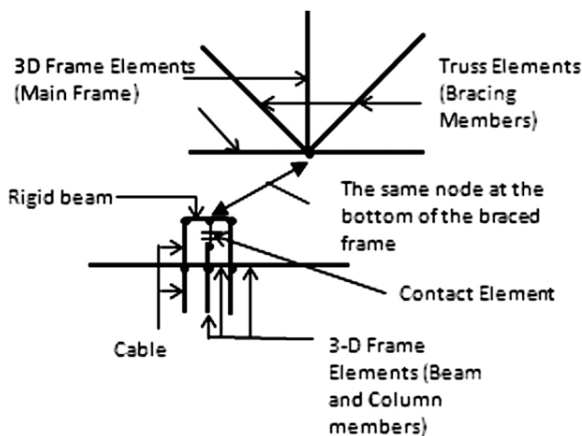


Fig. 3. Finite-element representation of the interaction point between the braced frame and the top of the building

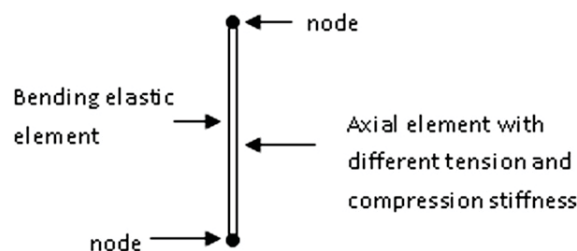


Fig. 4. Finite-element model of columns along the potential failed column

Table 1. Adopted Elements in the Modeling and Their Properties

Structural members	Elements	Material	Large deflections
Beams			
Members	BEAM4	Elastic	Available
Plastic hinges	COMBIN39	Defined moment rotation	
Columns	BEAM4	Elastic	Available
Columns above the failed column	BEAM4	Elastic	Available
	COMBINE39	Defined load deflection	
Cables	COMBIN10	Tension only capability	Available
Hat-braced frame			
Main members	BEAM4	Elastic	Available
Bracings	COMBIN8	Inelastic	

scenarios for the first floor column as representative scenarios that include independent failure of the corner column, the exterior edge column, and the interior column. The dynamic analysis is conducted considering the sudden effect of the column failure. The sudden removal of the column is achieved by applying the gravity loads gradually to the full capacity and keeping the imposed load stable for a certain time. Then, the support below the potential failed column is removed suddenly, and the time history response of the building is tracked for a sufficient time. A 5% Rayleigh mass proportional damping is assumed, considering the first mode of vibration corresponding to the failed column (Chopra 2001).

Case Study

To investigate the viability of the proposed method, a 10-story RC building designed to carry gravity loads according to Australian design standard AS3600 (AS 2009) is considered. This building consists of four longitudinal bays by four transverse bays of 6.5 m center-to-center span length in both directions. Fig. 5 shows a typical plan view of this building. The height of the first story is 5.0 m, and the height of the other stories is 3.0 m. The floor slabs are two-way slab systems. The columns are 0.6 × 0.6 m square cross sections. The thickness of the floor slabs is 0.18 m, and the total depth and width of all beams are 0.6 and 0.30 m, respectively. Material properties of the structure are: yield strength of reinforcement bars, $f_{sy} = 500$ MPa; compressive strength of concrete, $f'_c = 32$ MPa; modulus of elasticity of steel, $E_s = 200$ GPa, and of concrete, $E_c = 30.1$ GPa. The designed imposed live load on the slabs is 3 kPa. In addition to the self weight of the structural elements, an assumed wall and partitions dead load of 1.5 kPa and an additional dead load of 1.15 kPa—counting for floor finishing, ceilings, and mechanical utilities—are considered in the design of the building. The interior beams are designed to have 3N20 (N class represents deformed bars with 500 MPa nominal tensile strength) top reinforcement at the supports and 4N16 bottom reinforcement in the middle of the beam, whereas the exterior beams are designed to have 4N16 top reinforcement at the supports and 4N16 bottom reinforcement in the middle of the beam. Two bars of both top and bottom reinforcements are extended to the other sections. According to the section analysis, the yield and ultimate moments and their associated rotations, which represent potential plastic hinges at the ends of both exterior and interior beams, are given in Table 2. Although, beams are subjected to negative bending moments under the gravity loads near supports,

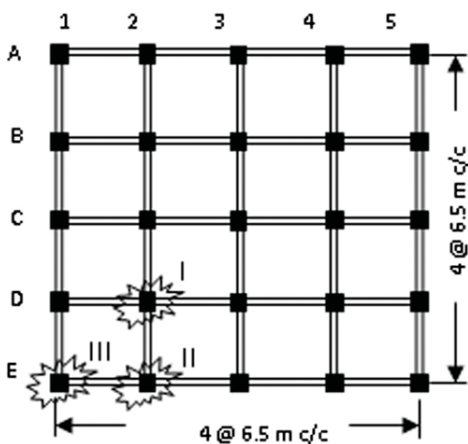


Fig. 5. Plan view of the example building showing the location of failed column

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positive bending moments are considered to account for the possible moment reversals above the failed support.

In this study, the braced frame was designed by using one structural section (ST 310 UC 118) (AS/NZS 3679.1; AS/NZS 1996) for all vertical, horizontal frame members, and inclined bracing. Fig. 6 illustrates the configuration and a three-dimensional view of the adopted steel hat-braced frame. The yield strength and modulus of elasticity of steel sections are taken as $f_y = 340$ MPa and $E_s = 200$ GPa, respectively. The cables used in this study at each side of the ends of beam members were galvanized strands (AS2841; AS 2005). The designed cables in this study have a diameter of 36 mm, a minimum breaking force of 1150 kN, a nominal cross-sectional area of 789 mm², and a modulus of elasticity of 166 GPa.

The building is analyzed by considering cases with and without the application of the proposed scheme. The finite-element simulation

of the building model with and without the application of the proposed scheme is shown in Fig. 7. Three independent first-floor column failure scenarios are considered in this study (Fig. 5). These failure scenarios (denoted by I, II, and III) correspond to removal of the interior first floor column (Column D2), the edge first floor column (Column E2), and the corner first floor column (Column E1), respectively.

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Analysis and Results

Sudden removal of the first floor column causes the downward displacement at the points in different stories above the removed column, for the building without retrofitting. Fig. 8 shows the deformed shape of the building models following the removal of the

Table 2. Yield and Ultimate Moments and Their Associated Rotations That Represent the Potential Plastic Hinges at the Ends of Beams

Beam type	Negative moments				Positive moments			
	M_y (kNm)	θ_y (rad)	M_u (kNm)	θ_u (rad)	M_y (kNm)	θ_y (rad)	M_u (kNm)	θ_u (rad)
Interior beams	233.560	0.006	242.019	0.022	103.317	0.0053	116.043	0.046
Exterior beams	202.939	0.0058	210.596	0.024	103.731	0.0053	115.364	0.046

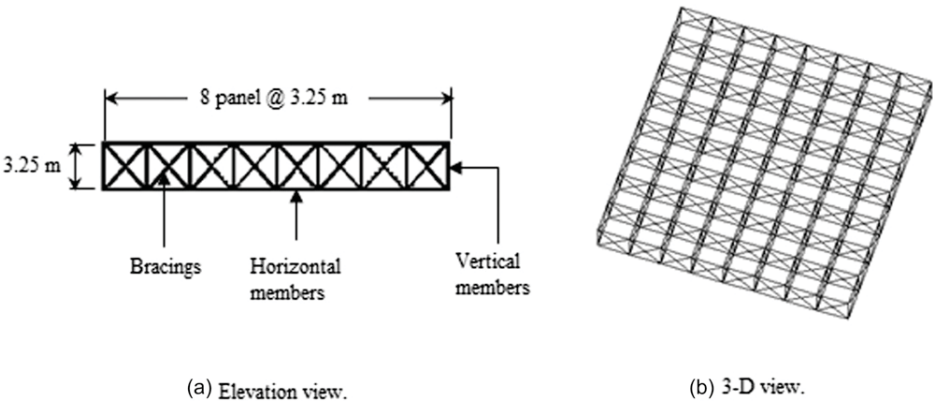


Fig. 6. Configuration of the adopted steel hat-braced frame

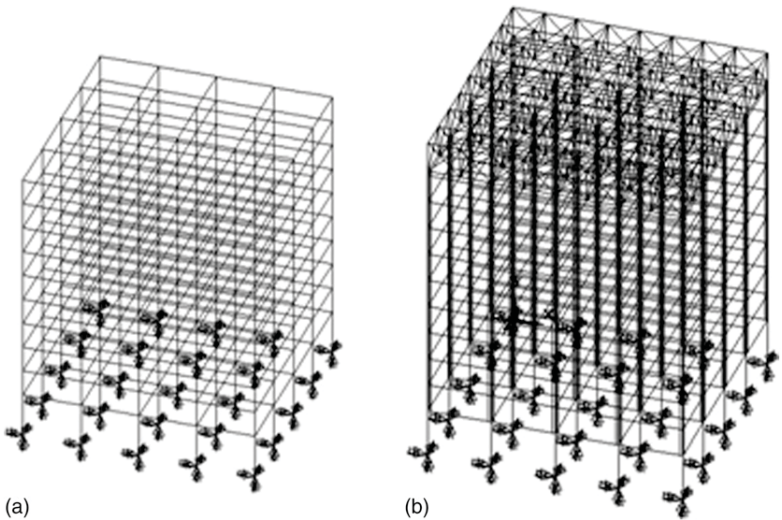


Fig. 7. Finite-element modeling of the building model (a) with and (b) without the retrofitting scheme

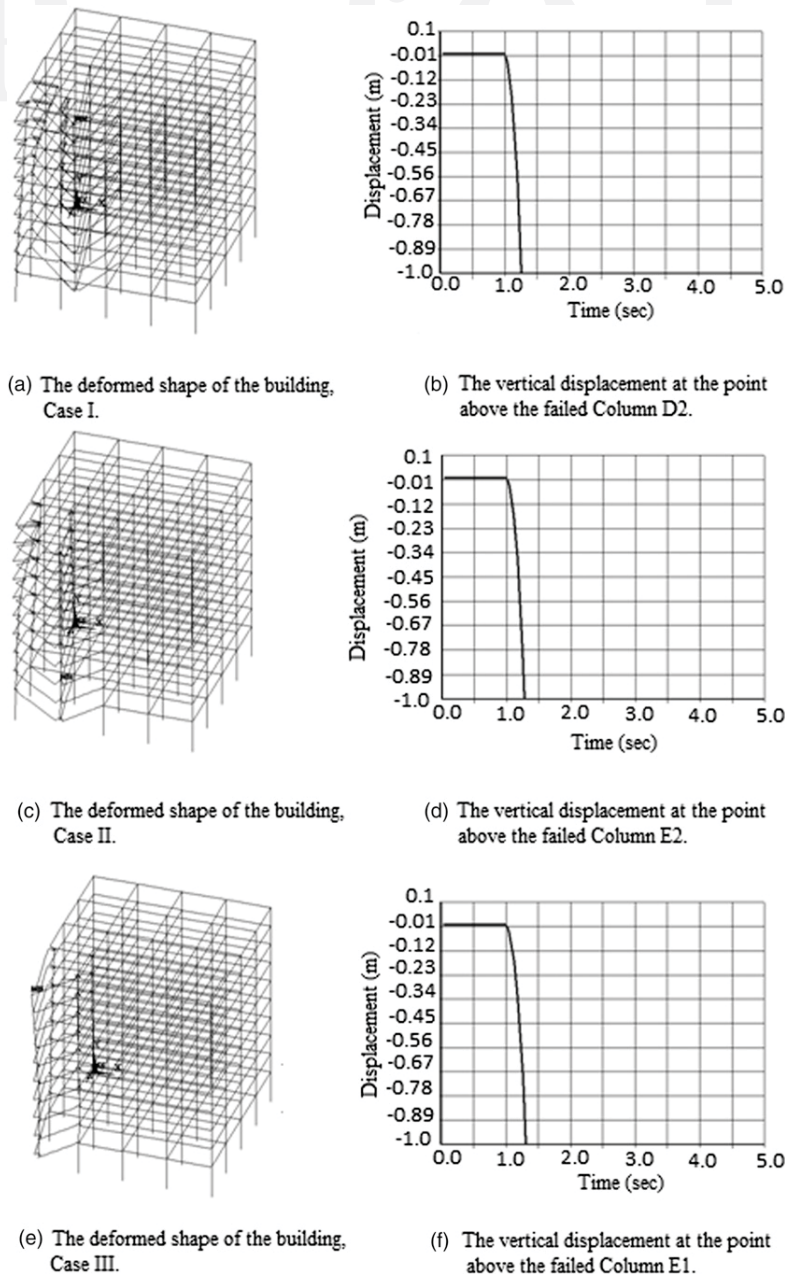
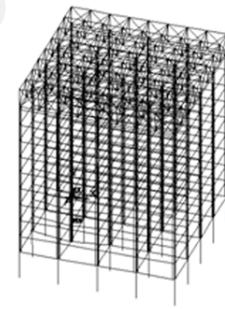


Fig. 8. Response of the 10-story building to different scenarios of first floor column failure; building without retrofitting scheme

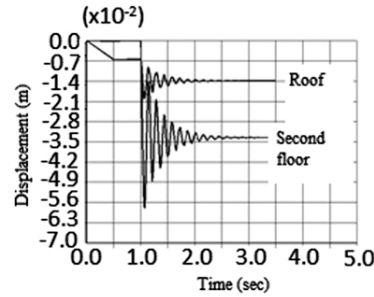
first floor Columns D2, E1, and E2, which correspond to the independent failure Cases I, II, and III, respectively (see Fig. 5). In addition, Fig. 8 shows the downward displacement time history of the points in the second floor above the failed columns. It is shown that these downward displacements at the points in the second floor above the failed column have increased dramatically. In this case, vertical displacements above the failed columns are stopped at 1 m, which is associated with rotations of 0.169 rad at the ends of the beams in which they far exceeded the ultimate rotations at the ends of beams (see Table 2). Also, the determined rotation 0.169 rad, which is associated with 1 m deflection above the failed column, exceeded the rotation limit 0.105 rad that is specified by USGSA (2003) guidelines. Furthermore, the results show that the excessive rotations in the plastic hinges at the ends of the bridging beams developed in all floors above the failed column. Accordingly, it is demonstrated

that the bays above the failed columns experience progressive collapse following the failure of these columns.

For the building retrofitted with the proposed scheme, the results show that the points above the failed column abruptly reach the peak downward vertical displacements because of the sudden loss of the column. Finally, the responses rest at steady state downward displacements. The resulting peak displacements in the second floor above the failed columns are 0.058, 0.057, and 0.066 m, following the failure of the first floor interior Column D2, edge Column E2, and corner Column E1, respectively. Fig. 9 shows the deformed shape of the building models following the removal of the first floor columns D2, E1, and E2, corresponding to the independent failure Cases I, II, and III, respectively (see Fig. 5). In addition, Fig. 9 shows the downward displacement time history of the points in the second floor and the roof above the considered removed columns.



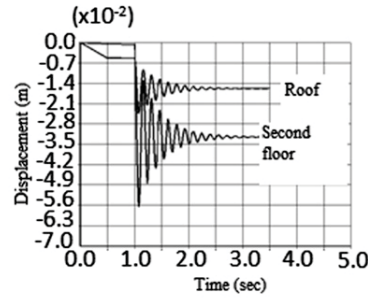
(a) The deformed shape of the building, Case I.



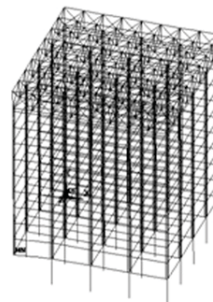
(b) The vertical displacement at points above the failed Column D2.



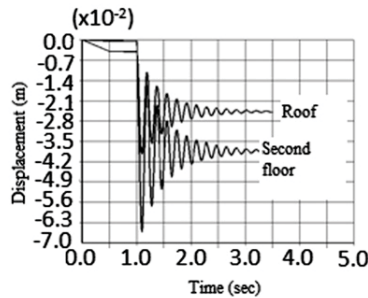
(c) The deformed shape of the building, Case II.



(d) The vertical displacement at points above the failed column E2.



(e) The deformed shape of the building, Case III.



(f) The vertical displacement at points above the failed column E1.

Fig. 9. Response of the 10-story building to different scenarios of first floor column failure; building with retrofitting scheme

For the building model retrofitted by the proposed scheme, the peak of maximum displacements are far less than the ultimate displacements (0.1298 m) associated with the ultimate rotations of the critical section of the bridging beams above the removed columns. It is obvious from the results that the plastic hinges are formed in the bridging beams above the failed column; however, the rotations in these plastic hinges are far less than the ultimate rotations at these sections where the plastic hinges were formed. Therefore, the building model successfully absorbs the loss of the first-floor column and the building will not suffer progressive collapse following any of the adopted failure scenarios.

Fig. 9 shows that the displacements in the second floor are significantly larger than those in the roof above the failed column, which indicates larger displacements in the lower stories compared with the higher stories. The larger displacements in the lower stories compared with the higher stories stems from the elongation of the columns and cables because of the developed tension forces in these members that result from reversing the loads through the alternate load path.

Load Redistribution in Building with the Proposed Scheme

The results depicted in Fig. 10 show the release of the compression forces in the failed columns D2, E2, and E1, which correspond to Cases I, II, and III, respectively. The release of compression forces in the potential failed columns is accompanied by sudden, instantaneous development of tension forces in the cables above the failed columns. Fig. 10 shows the axial force developed in the cable just over the roof and beneath the hat-braced frame. The results show that the tensile force at the top of cables just beneath the hat-braced frame reached the peak values of 655.93, 650.14, and 702.45 kN following the independent removal of columns D2, E2, and E1, respectively; then, the tension forces are damped out to the stable values of 383.76, 387.39, and 393.125 kN. In all cases, it is obvious that the developed forces in the cables are far less than their capacities (1,150 kN), and the cables are responding in the elastic range.

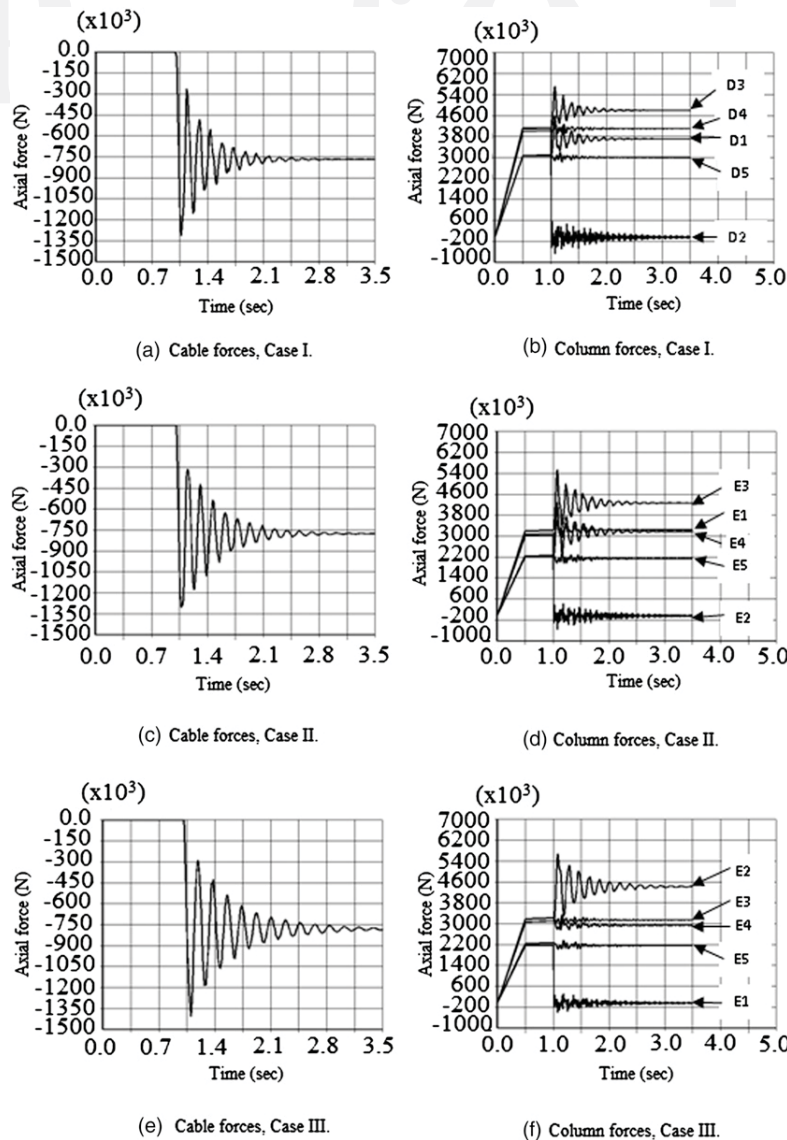


Fig. 10. Tension forces formed in the cable above the failed column at a point just beneath the braced frame; the force represents the total tension force formed in the two cables on either side of the beam (see Fig. 3) and column forces in the adjacent columns

Fig. 10 illustrates the maximum axial forces that developed in the adjacent columns to the failed columns. The results also show that the developed forces in the columns are far less than their capacities in all cases of column failure. Therefore, it is obvious that progressive collapse can be eliminated by adopting the scheme proposed in this paper.

Conclusions

In this study, a new scheme is proposed to prevent the potential progressive collapse of RC buildings resulting from column failures. The investigation results show that the conventionally designed RC building without a retrofitting scheme experiences progressive collapse resulting from the different column failure scenarios. However, the results also show that the building example set up with the proposed scheme successfully absorbs the different column-failure scenarios without spreading the failure. It can be concluded from the numerical results that the proposed scheme of using the

vertical cables and hat-braced frame is efficient in resisting the potential progressive collapse of the sample building used in this study in the event of a first floor column failure. However, before applying the proposed scheme in actual structures, experimental investigations are recommended for future studies to demonstrate the applicability of the proposed scheme in the actual structures.

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