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RESEARCH ARTICLE

Investigation on a mitigation scheme to resist the progressive collapse of reinforced concrete buildings

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ABSTRACT This study presents the investigation of the approach which was presented by Thaer M. Saeed Alrudaini to provide the alternate load path to redistribute residual loads and preventing from the potential progressive collapse of RC buildings. It was proposed to transfer the residual loads upwards above the failed column of RC buildings by vertical cables hanged at the top to a hat steel braced frame seated on top of the building which in turn redistributes the residual loads to the adjacent columns. In this study a ten-storey regular structural building has been considered to investigate progressive collapse potential. Structural design is based on ACI 318-08 concrete building code for special RC frames and the nonlinear dynamic analysis is carried out using SAP2000 software, following UFC4-023-03 document. Nine independent failure scenarios are adopted in the investigation, including six external removal cases in different floors and three removal cases in the first floor. A new detail is proposed by using barrel and wedge to improve residual forces transfer to the cables after removal of the columns. Simulation results show that progressive collapse of building that resulted from potential failure of columns located in floors can be efficiently resisted by using this method.

KEYWORDS prevent progressive collapse, alternate load path, reinforced concrete buildings, nonlinear dynamic, retrofitting, cable, steel hat braced frame, barrel and wedge

1 Introduction

According to the American Society of Civil Engineers (ASCE 7-05) [1], progressive collapse is defined as "spread of an initial local failure from an element to another, eventually resulting to collapse of an entire structure or a disproportionately large part of it". Among a number of building codes, standards, and design guidelines for progressive collapse, General Services Administration (GSA) [2] and Department of Defense (DoD) [3,4] address progressive collapse mitigation explicitly. They provide quantifiable and enforceable procedures to resist progressive collapse [5]. A few specific design procedures are available to prevent the progressive collapse of buildings especially for reinforced concrete buildings. Among these studies; Asteneh-Asl [6] proposed using steel cables horizontally placed parallel to the steel beams in composite steel and concrete floors of the building to reduce potential progressive collapse by catenary mechanism. He proposed embedding the cables in the floor slab for the new buildings and installing cables under the slabs for retrofitting existing buildings. Orton et al. [7] proposed using carbon fiber reinforced polymers (CFRP) in retrofitting reinforced concrete beams with deficient continuity to bridge over the potential failed column and reduce the risk of progressive collapse. This study presents the investigation of the approach which was presented by Alrudaini [8] to increase the progressive collapse resistance of reinforced concrete buildings in case of accidental column failure. The proposed method follows the alternate path method recommended by Unified Facilities Criteria (UFC 4-023-03) document [3,4]. In the alternate path (AP) method, the design allows local failure to occur, but seeks to prevent major collapse by providing alternate load paths. Failure in a structural member dramatically changes load path by transferring loads to the members adjacent to the failed member [5]. The overall objective of this research is to assess the potential for progressive collapse of RC building with and without mitigation scheme through computa-

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tional modeling and analysis. To achieve this goal, a computer program SAP2000 [9] is used to model and analyze the buildings, following the Unified Facilities Criteria (UFC 4-023-03) document [3,4]. A numerical investigation by utilizing nonlinear dynamic analysis is adopted to investigating viability of proposed method. Following sections define proposed scheme, building model that is adopted in the investigations and the analysis method. Based on UFC document [3,4], the structure must be checked for separately removal of columns, in the first story above grade, story directly below roof, story at mid-height and story above the location of a column splice or change in column size, in these positions: external columns near the middle of small side, large side, and at the corner of building. For each internal column removal, the AP analysis is only performed for the column on the ground floor or parking area floor and not for all stories in the structure. In each removal case, the AP analysis must be conducted to ensure appropriate load redistribution occurs, required capacities in adjacent members exist and acceptance criteria are satisfied. Nine independent failure scenarios are adopted in the investigation, including six external removal cases in different floors and three removal cases in the first floor.

2 Geometric model, material properties and loads

In this study, a ten-storey regular structural building has been considered to investigate progressive collapse potential. This building has four bays in both directions. The assumed building plan is shown in Fig. 1. Structural design is based on ACI 318-08 [10] concrete building code for special RC frames. Material properties and the design Loads of the structure are shown in Tables 1 and 2, respectively. The self-weights of the structure can be automatically generated by SAP2000 program [9] based on element volume and material density.

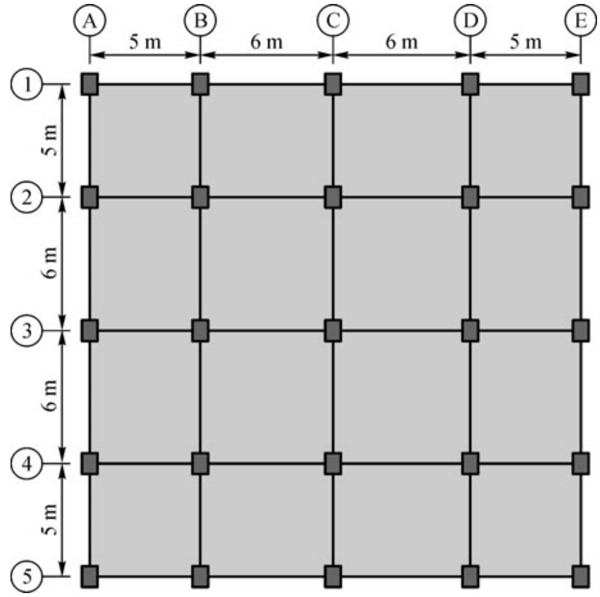


Fig. 1 The assumed building plan

The seismic response coefficient (C_s) is assumed to be 0.046 based on (ASCE 7-05) [1]. The reinforcement of beam sections are applied by considering the design

Table 1 Material properties

Young's modulus of concrete E_c	Young's modulus of steel E_S	Poisson's ratio	characteristic compressive strength of concrete f_{ck}	design yield strength for longitudinal reinforcement F_Y	design yield strength for transverse reinforcement F_{YS}
2×10^4 MPa	2.34×10^5 MPa	0.20	25 MPa	400 MPa	300 MPa

Table 2 Design loads of the building

load	dead load/(kN·m^{-2})	live load/(kN·m^{-2})	wall load on all beams/(kN·m^{-2})	perimeter wall weight/(kN·m^{-1})
on ground and first floor	5.7	5	–	5.0
on floors 2–9	5.6	2	1.5	6.1
on the roof	5.7	2	–	1.7

Table 3 Beam types of assumed building

storey	storey 1–4	storey 5	storey 6–8	storey 9,10
beam dimension/cm	70 × 45	60 × 45	50 × 45	40 × 45

Table 4 Column types of assumed building

storey 1	storey 2–4	storey 5	storey 6,7	storey 8	storey 9	storey 10
Col1	Col2	Col3	Col4	Col5	Col6	Col7

Table 5 Reinforcement of column section

section	dimension/cm	reinforcement/mm
Col1	70 × 70	32d22
Col2	70 × 70	28d22
Col3	60 × 60	20d22
Col4	50 × 50	20d22
Col5	50 × 50	16d22
Col6	40 × 40	12d22
Col7	40 × 40	12d18

requirements in all storeys, Table 3 shows Beam types of assumed building. Column types, dimensions and reinforcement in all storeys are shown in Tables 4 and 5. The Damping ratio of 0.5% is considered in this model.

The steel sections for the hat braced frame are designed following the UBC97-ASD [11]. Cables and bracing frame properties are shown in Tables 6. The designed Cables are in accordance with ASTM-A416 standard [12]. Figure 2 shows the view of the hat braced frame utilized in the mitigation scheme and the designed steel sections are given in Table 7.

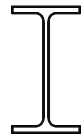
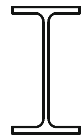
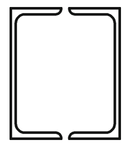
3 The proposed mitigation scheme

The proposed method to prevent progressive collapse is summarized by connecting the ends of the beam with vertical Cables parallel to the columns and hanged at the top to a braced frame placed on the top of the building which is seated on the top of the columns [8]. In the proposed scheme at each floor, the cable is locked above and below the floor. Steel plates are fabricated and welded to form a seating base to hang beam ends by the cables as illustrated in Fig. 3 [8].

Typical barrel and wedges are shown schematically in Fig. 4. The wedge may be formed into two or three parts as shown. The inner taper angle of the barrel and the outer taper angle of the wedge are approximately equal. The wedge made from hardened steel and has sharp teeth formed at the inner surface that makes contact with the cable. The initial tension developed in the cable after removal of the column is also depends on the barrel/wedge and the wedge/cable interface condition. Forces acting on hanging seat, barrel, wedge and the cable are shown in Fig. 5 [13].

In Fig. 5 [13]:
R = force between the barrel and the hanging seat,
T = tension in the cable,

Table 7 Bracing steel frame sections

element	section	shape
column	IPE240	
beam	IPE220	
bracing	UPA300	

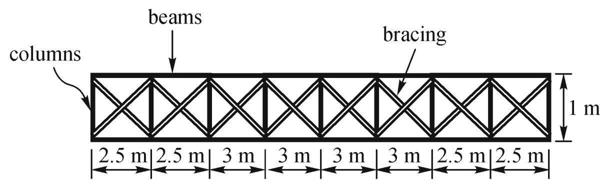


Fig. 2 The view of the hat braced frame utilized in the mitigation scheme

W = normal force acting across the barrel/wedge interface,
C = normal force acting at the wedge/cable interface,
S_w = shear force at the barrel/wedge interface,
S_c = shear force at the wedge/cable interface,
α = Wedge taper angle.
Equilibrium of forces for the barrel requires:

$$R = S_w \cos\alpha + W \sin\alpha, \tag{1}$$

$$C = W \cos\alpha - S_w \sin\alpha. \tag{2}$$

4 Structural modeling

Using SAP2000 program [9], beams, columns and the members of the hat braced frame are modeled as a 3-D frame. All live and dead loads are distributed to the adjacent beams according to the tributary area. In this research, the slab is considered as one-way spanning slab. Cables are modeled using tendon elements with only tension capability and connected to column elements at nodes.

Table 6 Cables and bracing frame properties

Young's modulus of the cables	diameter of cables	Young's modulus of steel	yield strength of steel	ultimate strength of steel
1.86×10^5 MPa	76.24 mm	2×10^5 MPa	240 MPa	400 MPa

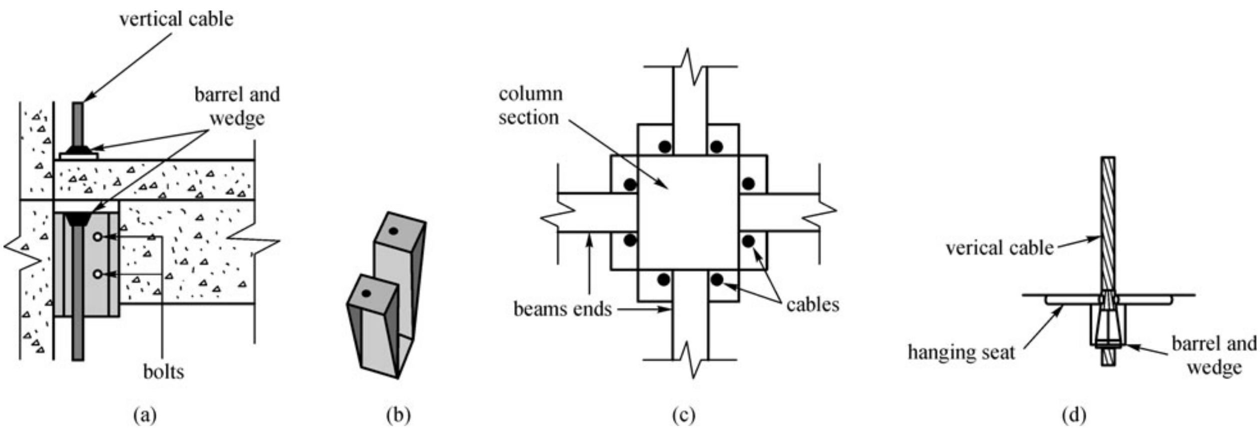


Fig. 3 Steel plates are fabricated and welded to form a seating base to hang beam ends by the cables

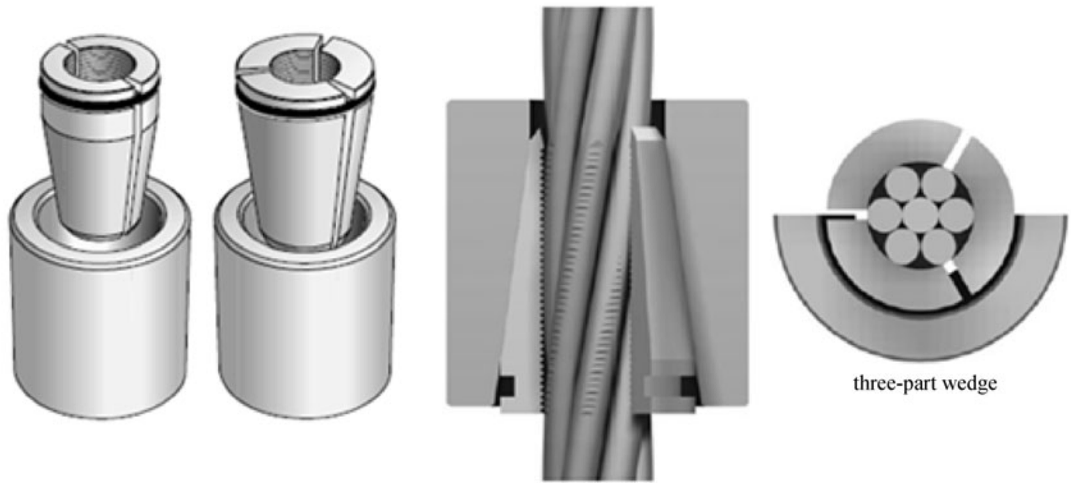


Fig. 4 Typical barrel and wedges

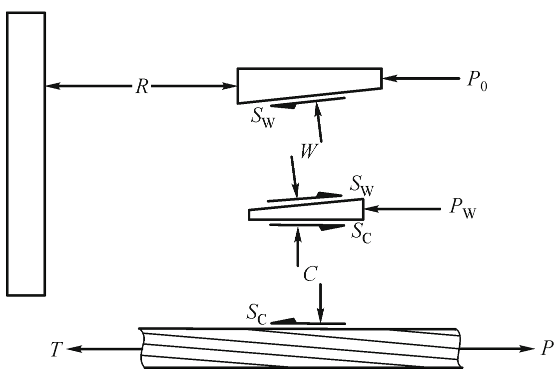


Fig. 5 Forces acting on hanging seat, barrel, wedge and the cable

5 Analysis method

SAP2000 [9] computer program is used to create a model of building and to examine redistribution of loads after removing columns. Figure 6 illustrates a typical plan view of this building that shows the location of failed columns, and Fig. 7 shows the elevation view of the building before and after setting the cables and the hat braced frame. Nine independent failure scenarios are adopted in the investigation, including six external removal cases in different floors and three removal cases in the first floor. The nonlinear dynamic procedure is described in the following sub-sections.

5.1 Plastic hinge properties

Taking nonlinear behavior into account is performed by assigning plastic hinges to the most probable locations in the model. In beams, moment hinges are assigned to relative lengths of 0.00, 0.50 and 1.00; also, in columns, axial–moment interaction hinges are assigned to both ends. Moment-rotation curves are defined according to Appendix C of the UFC document [4]. Figure 8 presents a graphical representation of the hinge definition for the beams. No plastic deformation occurs until point *B*, where the hinge yields. Point *C* represents ultimate capacity of hinge and point *D* corresponds to residual strength of it.

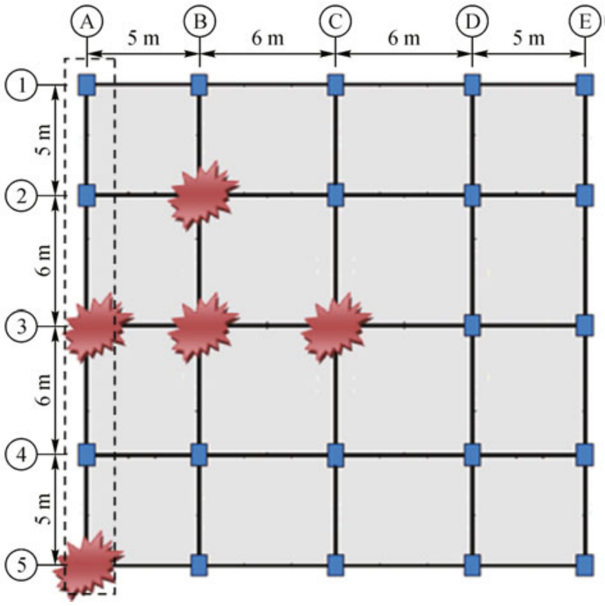


Fig. 6 A typical plan view of this building that shows the location of failed columns

Point E represents the ultimate displacement capacity of hinge after reaching to total failure [3,4]. Table 8 demonstrates rotations of points A to E in beams. These properties are adapted from the reinforced concrete member rotation requirements of the UFC [4]. It should be noted that reinforced concrete member allowable nonlinear capacity is based on absolute rotation, independent of member section properties. For nonlinear analysis, automatic hinge properties and user-defined hinge properties can be assigned to frame elements. When automatic or user-defined hinge properties are assigned to a frame element, the program automatically creates a generated hinge property for every hinge. These hinges are based on maximum moment values, which were calculated by using phi factors and over-strength factors per UFC [3,4]. Any shear or torsion values that would cause a hinge to form would result in an immediate failure [3,4]. Hinge properties of columns are selected from Table 9-7 of ATC-40 [14].

5.2 Load combinations

For nonlinear dynamic analysis, UFC document [4] recommended following gravity and lateral loads combination to calculate the deformation-controlled and force-controlled actions.

$$G = (0.9 \text{ or } 1.2)D + (0.5\,L \text{ or } 0.2\,S), \tag{3}$$

where D = dead load; L = live load; S = snow load [3,4].

The lateral load is applied orthogonal to each exterior face, one at a time. In this research four separate analyses must be performed, one for each principal direction of building, in combination with gravity loads. UFC [4] recommended the use of 0.2 wind load as the lateral load and the following lateral loads are recommended in the UFC document [3]:

$$L_{\mathrm{LAT}} = 0.002\Sigma P, \tag{4}$$

where L_{LAT} = lateral load, $0.002\sum P$ = notional lateral load applied at each floor; $\sum P$ = sum of the gravity loads [3].

Nonlinear dynamic procedure requires several analysis cases for each column removal. Analysis cases are created

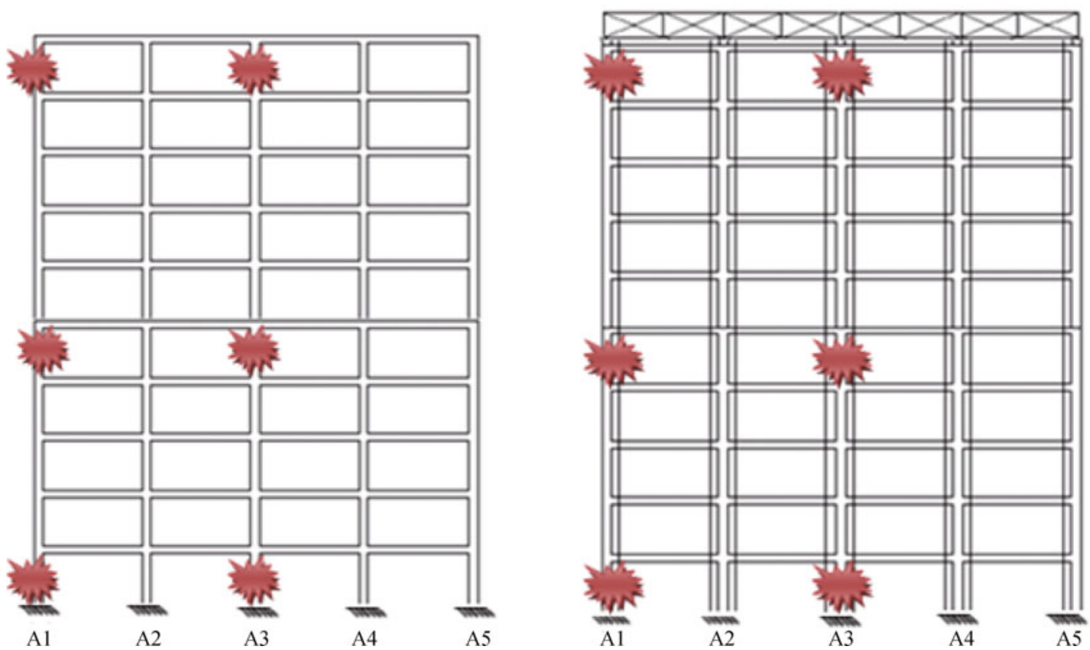


Fig. 7 The elevation view of the building before and after setting the cables and the hat braced frame

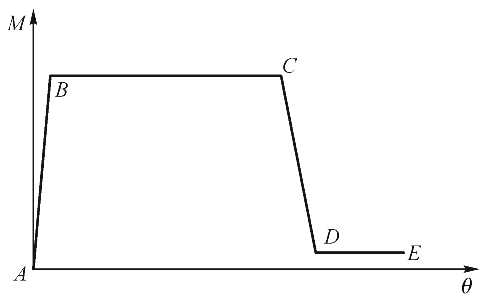


Fig. 8 A graphical representation of the hinge definition for the beams

for each lateral load direction in order to determine present forces at equilibrium in each column to be removed.

5.3 Element removal technique

In performing nonlinear dynamic analysis, nonlinear static cases with initial conditions are used in order to consider deformations due to gravity loads. Removal technique includes four steps [15]. First, the structure is analyzed under load combinations 1 and 2. Second, after reaching to equilibrium for framed structures, the column is removed and, instead, the effects of its top end forces are exerted to the structure along with present forces. This step is used as the initial condition of the third step. It is important to notice that the deformed shape and internal forces of the structural elements are the same in first two steps. Third, forces equal and opposite to the direction of forces that were applied in step two are exerted to the structure [15]. In the case of nonlinear dynamic analysis, the reaction forces are suddenly exerted. Duration for removal must be less than one-tenth of the period associated with structural response mode for vertical motion of bays above the removed column [3,4]. Analysis shall continue until maximum displacement is reached or one cycle of vertical motion occurs at column section removal location [3,4].

Progressive collapse is an inherently dynamic event. Dynamic effects may come from many sources during the collapse. After a structural member is failed, the structure transfers the load of that member and comes to rest in a new equilibrium position. During this dynamic load redistribution, internal dynamic forces affected by inertia and damping are produced and vibrations of building elements are involved. The analysis is run using the nonlinear time-history option in SAP2000 [9]. This method requires the users to define several dynamic and nonlinear parameters including time step, damping ratio, and plastic hinges. The steps required in performing the analysis are given below:

1) Build a 3-D model in the SAP2000 computer program [9].

2) Use the hinge properties for RC buildings provided in UFC4-023-03 document [4].

3) Define a damping ratio. Damping coefficient is assumed to be 5%.

4) Apply dynamic load combinations.

5) For each column removal, the column member is deleted in structural model and the internal forces determined from the equilibrium model are applied to the structure as a load case to the joint or joints at each column end. These static nonlinear analysis cases (1 for each combination of column removal and lateral load direction) are used as the starting conditions for the column removals.

6) Set “Load Case Type” to be “Time History”, “Analysis Type” to be “Nonlinear”, and “Time History Type” to be “Direct Integration”. Select Nonlinear parameters button and choose P-delta option. Define initial conditions and an appropriate time step in the time history function definition. The output time step size of 0.01 is selected in this research. Remove the column by ramping down the column forces under a duration for removal of less than one tenth of the period associated with the structural response mode for the element removal.

7) Perform nonlinear time history analysis.

8) Verify and evaluate analysis results such as the maximum ductility and plastic hinge rotation values. SAP2000 [9] design procedures may be used to evaluate whether columns are deformation or force controlled. Design checks also aid in definitions of column hinges by determining axial load demand and capacity. Create a design combination for each analysis case.

5.4 Acceptance criteria in AP method

Acceptance criteria include resistance requirements and deformation limits. To check resistance criteria, moments, axial and shear forces, which are considered as required, strengths must be lower than design strength of the element [15]. Determination of design strength is performed according to common procedures of ACI 318 [10], but a factor of 1.25 is exerted to the concrete compressive strength (f_c) and to yield strength of reinforcement steel. Also, in order to control deformation limits, calculated deflections and rotations in elements are compared to allowable limits of the UFC document [4].

Table 8 Moment-rotation curve of plastic hinges

A		*B*		*C*		*D*		*E*	
M_{rel}*	θ**	M_{rel}	θ	M_{rel}	θ	M_{rel}	θ	M_{rel}	θ
0.01	0.0000	1.00	0.0000	1.00	0.0523	0.01	0.0525	0.01	0.0550

* Hinge moment (defined relative to yield moment)
** Hinge rotation (defined as absolute rotation)

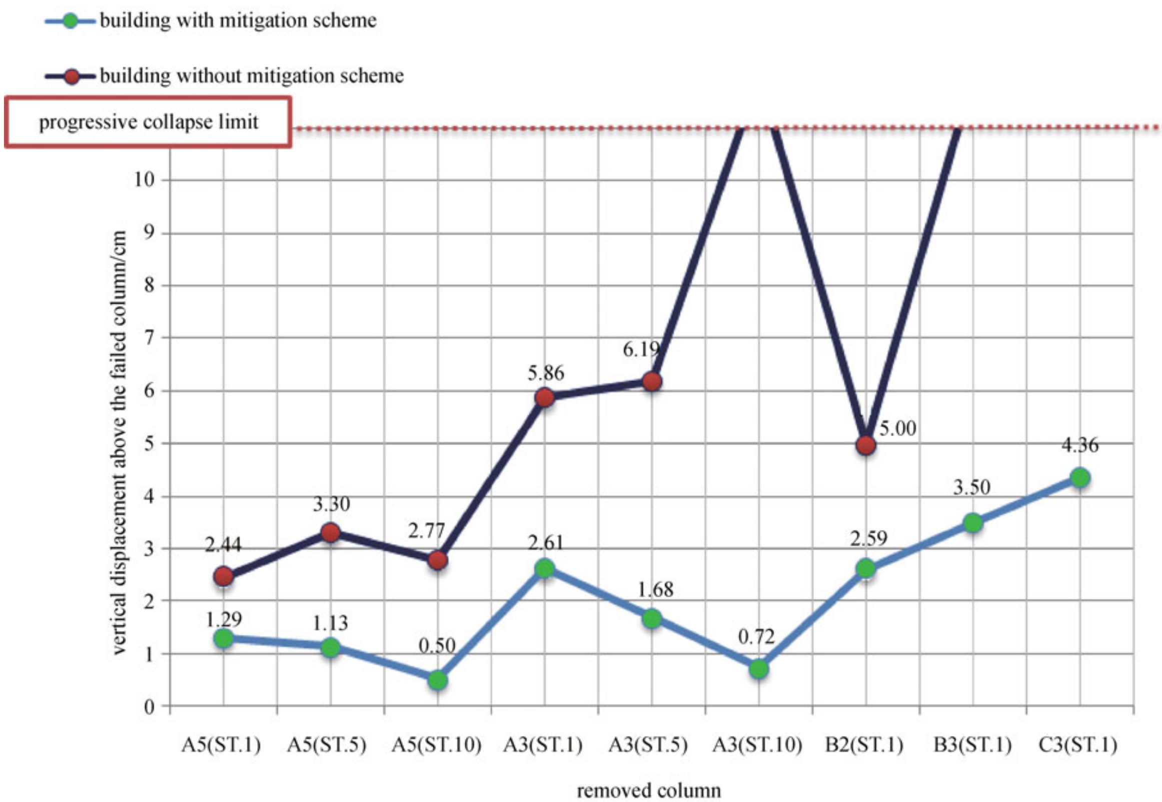


Fig. 9 The maximum downward displacement at the point above the removed column

6 Results and discussions

Figure 9 shows the maximum downward displacement at the point above the removed column in dynamic analysis for the building with and without the mitigation scheme. For the building without mitigation scheme, the analysis results show that the vertical deflection at the point above the failed column exceeded the maximum displacement in the removal of the first storey columns B3, C3 and the tenth storey column A3, that correspond to the ultimate rotation of these beams. Figures 10 and 11 depict the downward displacement after removing the C3 column in

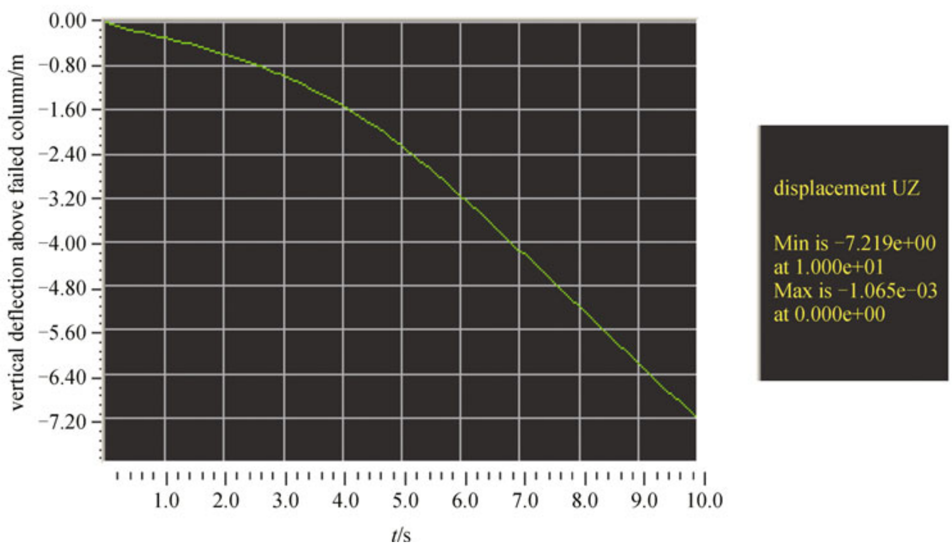


Fig. 10 The downward displacement after removing the C3 column in the first storey for the building without the mitigation scheme

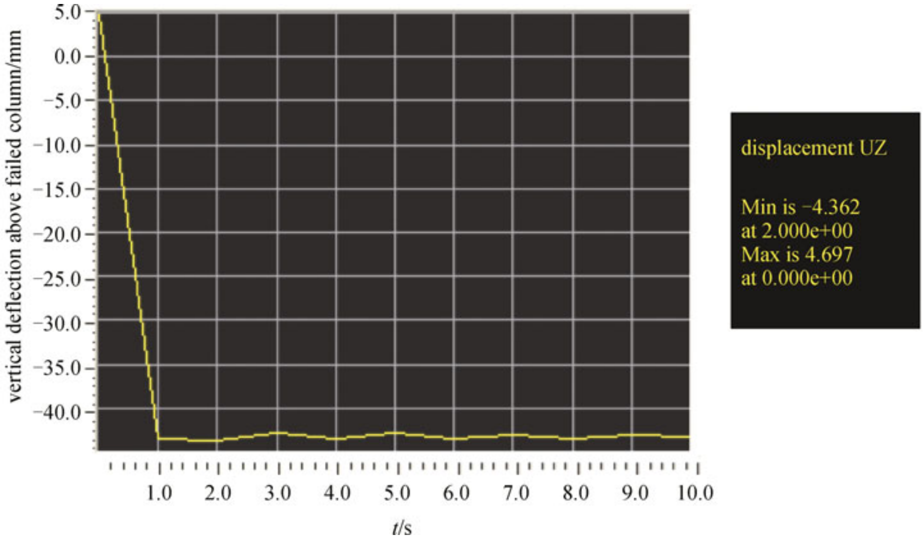


Fig. 11 The downward displacement after removing the C3 column in the first storey for the building with the mitigation scheme

the first storey for the building without and with the mitigation scheme, respectively.

Figure 12 depicts the plastic hinge formation of the model structure obtained by dynamic analyses in case of the elimination of C3 column in the first storey for the building with and without mitigation scheme.

The simulation results of the building show that the presented method of using vertical cables and hat steel braced frame is efficient in absorbing the loss of the individual columns and the building will not suffer progressive collapse following any of adopted failure scenarios in this paper. In the building with mitigation scheme the shear forces in the beams and the axial forces in the columns are below their capacities.

In Fig. 13, the maximum developed forces in the cables at a point at the top of the building and just beneath the hat braced frame is shown. Analysis results show that these cables still behave in the elastic range.

In this section the effect of the cable's stiffness on the performance of the proposed mitigation scheme is

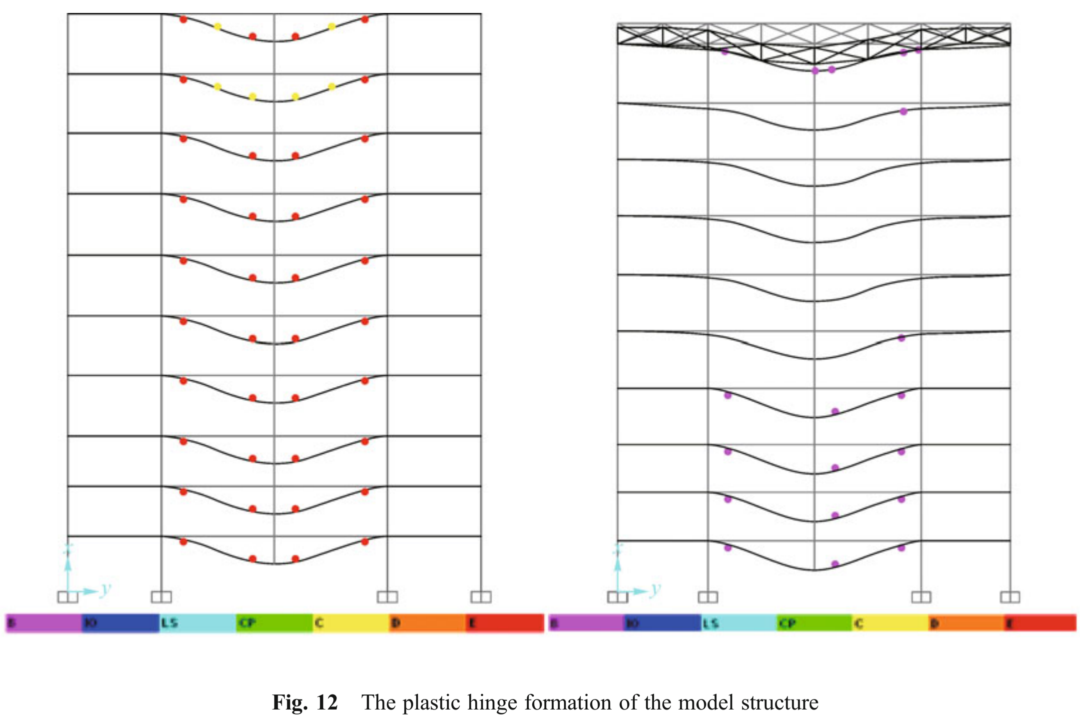


Fig. 12 The plastic hinge formation of the model structure

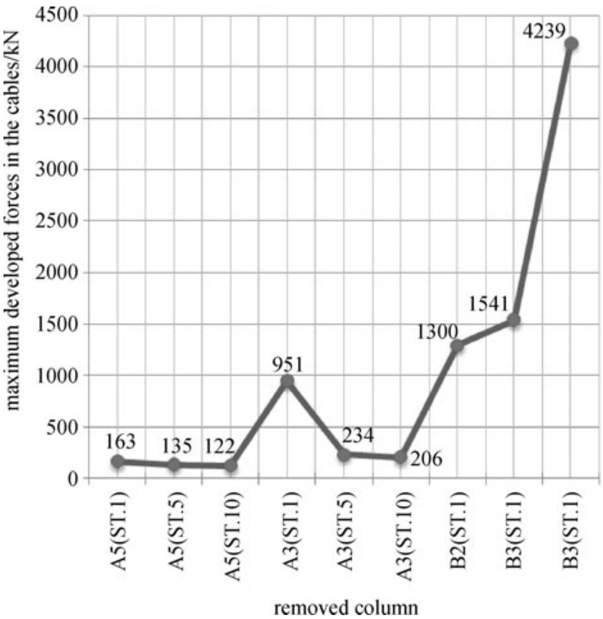


Fig. 13 The maximum developed forces in the cables at a point at the top of the building and just beneath the hat braced frame

investigated. The variation of cable's stiffness is accomplished by the varying the cross sectional of the cables as well as the elastic modulus of the cables. Figures 14 and 15 show the effect of the cables' diameter and elastic modulus on vertical displacement above the failed columns and developed loads in cables, respectively. The elastic modulus of the cables are $E_1 = 2 \times 105$ MPa and $E_2 = 1.86 \times 105$ MPa. These cables have diameter equal to 5, 12 and 16 cm, respectively.

It is shown that the vertical deflections are decreased with the increase of the cables' cross-sectional area and modulus of elasticity. In addition the parametric results show that the developed forces in the cables are increased with the increase of the cables' stiffness.

7 Conclusions

According to performed analyses we surveyed in this paper, we can have the following conclusions:

1) In this study, a mitigation scheme consists of

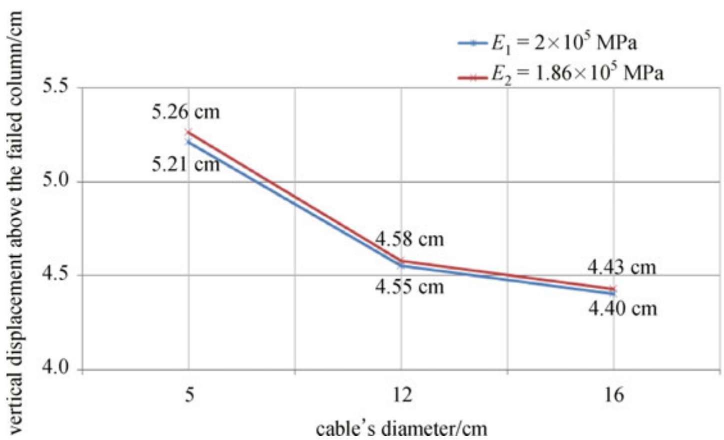


Fig. 14 The effect of the cables' diameter and elastic modulus on vertical displacement above the failed columns

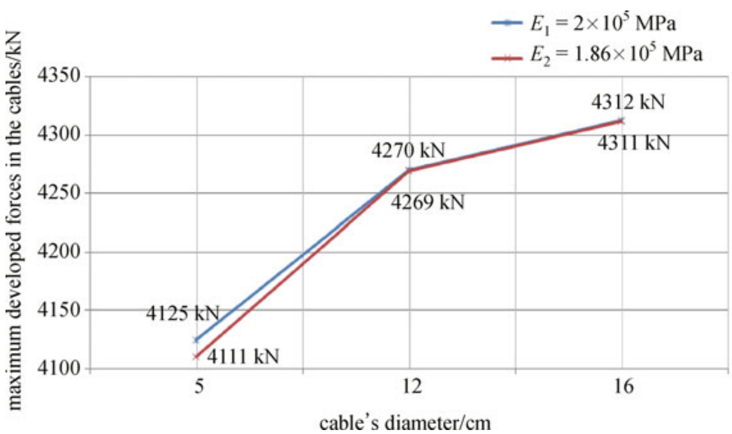


Fig. 15 The effect of the cables' diameter and elastic modulus on developed loads in cables

installing steel cables along the columns and hanging these cables at the top from a hat braced frame seated on the top of the building to resist the progressive collapse of reinforce concrete buildings that resulted from the potential failure of a column is investigated.

2) The investigation results show the effectiveness of the proposed method for the building example in this study. The proposed method of using cables to prevent progressive collapse of building is efficient in eliminating progressive collapse in the event of first, fifth and tenth storeys column loss.

3) It is founded that the vertical deflections are decreased with the increase of the cables' cross-sectional area and modulus of elasticity. Developed forces in the cables are increased with the increase of the cables' stiffness.

4) A new detail is proposed by using barrel and wedge to improve residual forces transfer to the cables after removal of the columns. The developed forces in the cables after removal of the columns are also depend on the barrel/wedge and the wedge/cable interface condition.

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