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Parameters Affect Flexural Mechanism to Prevent Progressive Collapse of RC Buildings

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Abstract

This study investigates the effect of spans length, reinforcement ratio and continuity of flexural reinforcement on the progressive collapse performance of double span beams over failed columns. The investigations focus on initial flexural resisting mechanisms to prevent progressive collapse. Detailed nonlinear finite element simulation of double span beam-column sub-assemblages subjected to residual gravity loads that initially carried by the failed column is adopted for the investigations. Nonlinear static pushover analysis is conducted in which capacity curves are derived and compared with demanded capacities. The effects of spans length, reinforcement ratio and number of continuous bottom flexural reinforcement on progressive collapse are considered in the investigations. Analysis results show that the strength to resist progressive collapse has decreased by 25.4 % and the ductility increased by 103 % following the increase in span length from 5 m to 7 m. On the other hand, increasing reinforcement ratio of top flexural reinforcement from 0.447 to 1.089 leads to 26.27 % increase in strength accompanied with a decrease in ductility equal to 16.42 %. In addition, extending all bottom bars rather than the minimum specified two bars resulted in 12 % increasing in strength and 40.28 % decreasing in ductility.

Keywords

Progressive collapse, Reinforced concrete, Flexural mechanism, Reinforcement continuity

1. Introduction

Progressive collapse is referred to a series of failures of structural members initiated by local unexpected failure or damage in individual structural member leading to a partial or entire collapse of that structure (ASCE, 2022). In this study, the failure of a supporting column may lead to the failure of the supported beams and floors. The progressive collapse has gained great attention and much research were published within the last two decades to mitigate such collapse (Alogla et al. 2016; Hadi and Alrudaini 2011a; b, 2012; Helmy et al. 2012; Kim et al. 2013; Liu 2010; Panahi and Zahrai 2021; Weng et al. 2017). Also, attention on this type of failure is obvious through the specified analysis methods and design approaches by different governmental and nongovernmental design standards and guidelines (DoD 2009 and Gsa 2003) to confine or prevent progressive collapse of buildings based on the type and importance of the building. Also, several measures were recommended to improve building integrity to confine progressive collapse such as the integrity requirement that recommended by ACI 318-19 (Committee 2019). ACI-318-19 (Committee 2019) requires continuity of at least one third and quarter of the top and bottom longitudinal reinforcement; respectively, along the beam and considering a minimum two continuous bars. These measures are demanded to provide adequate continuity and ductility to the structure to accommodate potential collapse of interior support. Further researches were published in which studied the influence of different design parameters on progressive collapse resistance including flexural reinforcement ratio (Iribarren et al. 2011), load from the above floors on adjacent columns to the failed interior one (Alrudaini and Najem 2016), concrete strength (Deng et al. 2020), reinforcement continuity considering catenary action (Alrudaini 2021; Azim et al. 2020; Li et al. 2014), presence of infill walls (Eren et al. 2019), span length in steel moment resisting frame buildings (Hashemi Rezvani et al. 2015) and slab system (Elkholy et al. 2022). In addition, Azim et al. (Azim et al. 2020) reviewed the main parameters that affect progressive collapse performance including beam dimension, ratio of top and bottom flexural reinforcement and seismic details as well as the effect of slabs and transverse beams. In this paper, the effect of spans length, reinforcement ratio and continuity of bottom flexural reinforcement on progressive collapse resistance of reinforced concrete buildings is studied considering the initial flexural resisting mechanism.

2. Progressive Collapse Assessment

The common resisting mechanisms of developed double span beams above the failed column that resist the residual loads include flexural resisting mechanism and catenary action mechanism. The former mechanism is developed at the initial stage following the removal of the column and the later will developed after exhausting the flexural mechanism. Figure 1 illustrates the resisting mechanisms in double span beams.

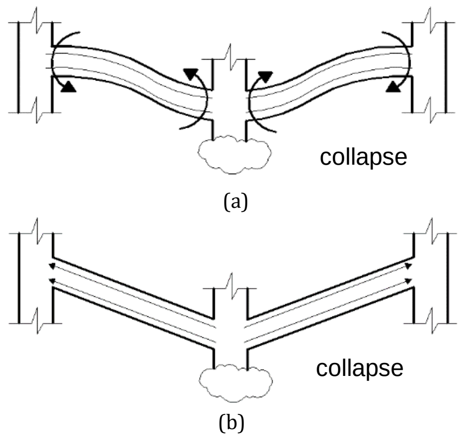


Fig. 1 Alternate load paths above failed interior column; a) flexural mechanism and b) catenary action mechanism.

UFC (DoD 2009) design guidelines specified load combination for progressive collapse analysis and design as;

Load = DIF (1.2 DL + 0.5 LL) (1)

Where; DIF, DL and LL are dynamic increase factor, dead loads and imposed live loads, respectively.

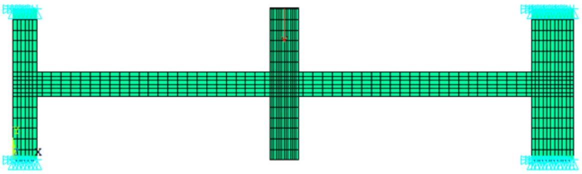
According to UFC (DoD 2009), the dynamic increase factor (DIF) is calculated using the equation;

DIF=1.04+0.45 /(θ_m/θ_y +0.48) (2)

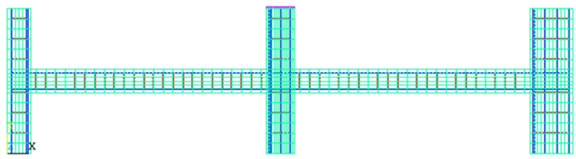
Where, θ_m and θ_y are maximum and yield rotations, respectively.

3. Modeling

A general finite element program ANSYS 11.0 was utilized for modeling. Concrete is simulated using three-dimensional brick elements (SOLID65) that have cracking and crushing capabilities under the effect of tension and compression stresses. Reinforcement steel bars are simulated by three-dimensional line elements (LINK8). The main material parameters include modulus of elasticity of the concrete ($E_c = 4700\sqrt{f'_c}\ (MPa)$ according to ACI 318-19 (Committee 2019)), Poisson's ratio (ν) equals to 0.2. Concrete nonlinearity is simulated using Drucker-Prager elastic plastic model. Drucker-Prager model parameters include cohesion value, angle of internal friction and dilatancy angle were defined as presented in Doran et. al. (Doran et al. 1998). Nonlinear behavior of steel bars was defined using bilinear model. The nonlinear static method allows for both geometric and material nonlinearities. In this method, the vertical load was increased until the maximum ultimate load was reached, or the structure had failed. ANSYS employs the Newton-Raphson approach to solve nonlinear problems. In this approach, the load is subdivided into a series of load steps. At load step, the program will perform several equilibrium iterations to obtain a converged solution. The Newton-Raphson method evaluates the balance load vector, which is the difference between the restoring forces (the loads corresponding to the element stresses) and the applied loads and checks for convergence. If a specified convergence criterion is not satisfied, the out of balance load vector is reevaluated, the stiffness matrix is updated, and a new solution is obtained. Iterative procedure for load vector evaluation and stiffness matrix updates continue until the problem converges for each load step. The analysis stops either at reaching complete specified load or the analysis leads to divergent solution that represent the failure situation. The developed model is verified against experimental test conducted by Choi and Kim (Choi and Kim 2011). Choi and Kim (Choi and Kim 2011) tested double-span beams sub-assemblage model that is part of an eight-story reinforced concrete building. The dimensions of the right column left column and mid column were 285 x 190 mm, 160 x 160 mm and 160 x 125 mm, respectively. Beams dimensions were 125 mm width and 160 mm depth with two 10 mm bars at the top and bottom. The double span length was 3770 mm from outer faces of the exterior columns. The concrete compressive strength $f_c^{'}$, yield strength of main reinforcement bars fy and that for stirrups were equal to 30 MPa, 493 MPa and 363 MPa, respectively. Finite element modeling of the specimen using ANSYS11.0 program is constructed as shown in Fig. 2. Nodes at top and bottom ends of the exterior columns were fixed in which displacements at these nodes were prevented. Vertical downward load was defined to represent the residual loads from the failure of interior column on the top of the middle column of the sub-assemblage. Also, ten load steps were defined to represent gradual application of load in the nonlinear analysis.



(a)



(b)

Fig. 2 Finite element verification model, a) concrete elements and b) reinforcement elements.

Fig. 3 shows load displacement curve obtained from the developed model and that presented by Choi and Kim (Choi and Kim 2011). The results obtained from the developed model are limited to the ultimate value in which the solution unconverged beyond this point in which forced controlled method was adopted for the analysis. After this point the capacity degraded without any increase in the load. The load deflection relation before and at the ultimate point represents the initial flexural resisting mechanism which is the focus of this study. This study focuses on the behavior at which the progressive collapse is prevented by flexural resisting mechanism. Comparison results show very good agreement between experimental test and numerical predictions.

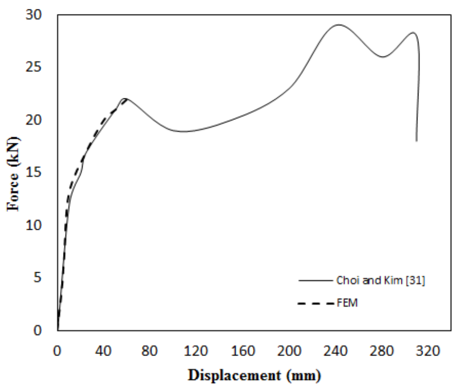


Fig. 3 Load-deflection curve at the point above the removed column.

4. Building Model

In this study, 2-storey reinforced concrete ordinary moment resisting frame structure is considered for building models and designed according to ACI 318-19 (Committee 2019) to carry live load of 2 kN/m2 and dead load 2.3 kN/m2 on floors in addition to self-weight of slabs. One-way slab system was selected in which slabs were designed with a constant thickness equal to 170 mm. Material properties include compressive strength of concrete equal to 30 MPa and yield strength of reinforcement bars equal to 414 MPa. The plan and elevation of the designed moment frame building are illustrated in Fig. 4.

4.1 Effect of span length

In conventional design, using larger spans leads to greater flexural reinforcement area and/or larger sections size to resist the increases in bending moments due to larger spans. The combined effect of using larger spans and greater reinforcement area on progressive collapse is investigated in this study within the initial flexural resisting mechanism. This study focuses on the effect of varying span length on progressive collapse considering constant applied load. Constant applied loads on different span lengths are found in buildings with similar applied loads and constant lateral spans as shown in Fig. 4.

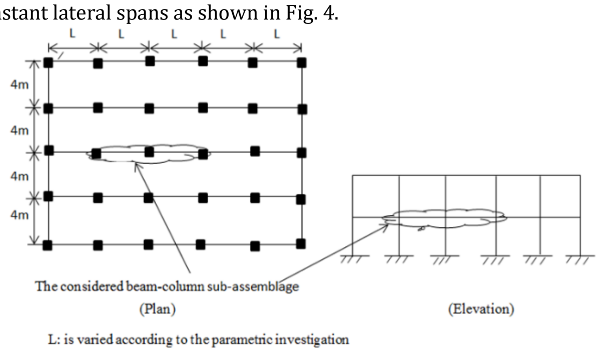


Fig. 4 Plan and elevation of the designed moment frame building.

Three span lengths are considered in the analysis including 5 m, 6 m and 7 m. In this section, only the effect of reinforcement area on progressive collapse was investigated. Increasing span length for the same design loads results in increasing required reinforcements for the adopted spans. Further increasing in span length may result in greater sections as well as increasing reinforcement ratio. To avoid the interaction effect of increasing beams cross sectional area with the increase in the amount of reinforcement, longer spans have been excluded in which only lengths of 5 m, 6 m and 7 m are considered. Dimensions and reinforcement details of the selected beam column sub assemblages are illustrated in Fig. 5 and Table 1.

The reinforcements in Table 1 are selected according to the flexural design. The applied load on floor is constant considering same building function. Also, similar transferred distributed load on beams due to equal lateral span (4m) as shown in Figure 1. However, the variation in flexural reinforcement resulted from different span length of the beams that affect the resulting bending moments in beams.

In all cases, the size of the column and reinforcement are kept constant considering the largest values that satisfies all span lengths of the beams. The reason behind this is to focus on the performance of the bridging beams above the failed column. Fig. 6 and 7 illustrate the finite element simulation of the models. Due to symmetry, only half of the models are simulated by the finite element to reduce computational storage and analysis time. Figure 7 illustrates nonlinear capacity curves above the center of the removed column in terms of load factor against vertical deflection in which the resulted loads have normalized to the demanded load capacity. On the other hand, the values of the ultimate normalized load factor as well as the determined ductility factor μ (the ratio of maximum deflection to yield deflection) are given in Table 2. Load factor is calculated by dividing applied load at the end of each load step by the specified total load by the design guidelines (equation 1) to resist progressive collapse in which equal to;

$$\text{Load factor} = \frac{\textit{Applied load}}{\text{DIF (1.2 DL + 0.5 LL)}} \tag{3}$$

The ductility factor is given by

$$\text{Ductility factor } \mu = \frac{\Delta_{ultimate}}{\Delta_{yield}} \tag{4}$$

Where, $\Delta_{ultimate}$ and Δ_{yield} are the ultimate deflection and yield deflection underneath the applied load at the middle column; respectively.

The results show an obvious decrease in ultimate load factor with the increases in span length. The model with double 5 m spans length exhibits the greatest load factor equal to 1.18, while the double 7 m model has the smallest load factor that equal to 0.88. In contrast, it is shown that the deflection increases with the increases in span length in which the double 7 m model performed the largest deflection (342 mm) compared to the smallest deflection (108 mm) corresponding to the double 5 m model. The increases in ultimate deflection with increases in span length lead to increases in ductility.

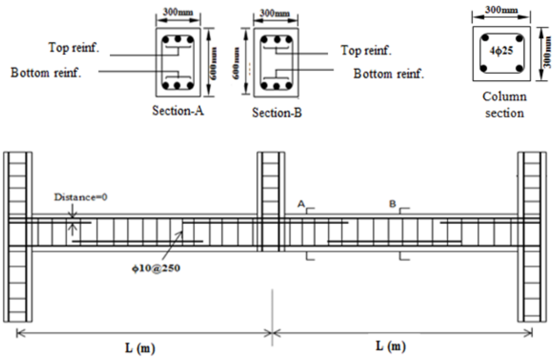


Fig. 5 Dimensions and reinforcement details of the considered double span beams.

Table 1. Reinforcement details of the considered beams with different span lengths.

Model	Span length (m)	Flexural reinforcement			
		At support		Mid span	
		top	bottom	top	bottom
L5	5	4ϕ16	2ϕ16	2ϕ16	3ϕ16
L6	6	4ϕ20	2ϕ16	2ϕ20	4ϕ16
L7	7	3ϕ25	2ϕ20	2ϕ25	3ϕ20

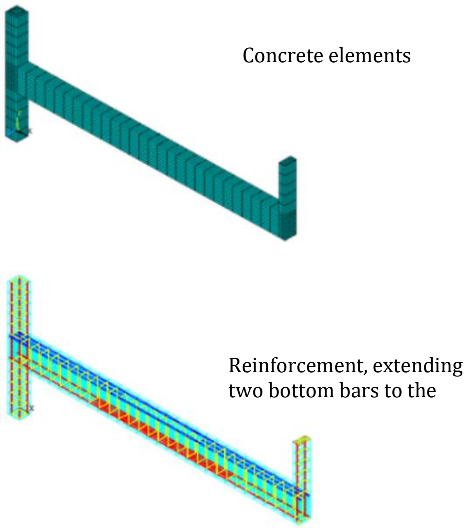


Fig. 6 Finite element modeling of symmetric sub assemblage models.

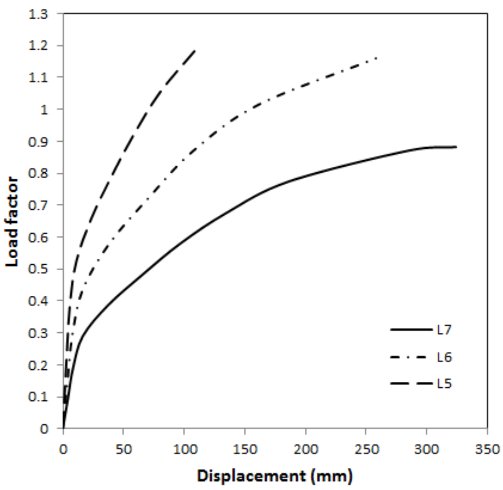


Fig. 7 Variation of capacity curves with span length.

Table 2. Normalized ultimate loads and ductility factors variation with span length.

Model	Load factor	Yield deflection (mm)	Maximum deflection (mm)	Ductility factor μ
L5	1.18	8.6	109.0	12.67
L6	1.16	12.0	258.0	21.50
L7	0.88	12.6	324.2	25.73

Table 2 shows that models with span lengths equal to 5 m, 6 m and 7 m have ductility factors equal to 12.67, 21.5 and 25.73; respectively. It is obvious that increasing span length from 5 m to 7 m leads to 103 % increases in ductility. The results have demonstrated a significant decrease in ultimate strength with increasing span length despite of larger flexural reinforcement due to developing larger double spans. The increase in developed double span overcome the increase in flexural strength of the original spans. This indicates that beams with smaller spans exhibit more redundancy than beams with larger spans. In contrast, increases in span length resulted in significantly larger ductility that assist in developing larger catenary action which is the following mechanism in resisting progressive collapse.

4.2 Effect of steel area

The effect of steel area on load redistribution to resist progressive collapse is investigated. Three models of double span beam column sub-assemblage have constructed with span length 5 m and have flexural reinforcement given in Table 3. Figure 8 illustrates the capacity curves in terms of load factor to deflection relations of double beam models corresponding to three different amount of flexural reinforcement. Values of load factors and ductility factor are presented in Table 4. The comparison results show increases in capacity and decreases in ductility following the increases in reinforcement area. It is found that the models A1, A2 and A3 with steel ratios of top reinforcement equal to 0.447, 0.700 and 1.089 at the faces of the columns accompanied with increase in bottom reinforcement at the middle spans equal to 0.335, 0.447 and 0.700 resulted in normalized ultimate load factor equal to 1.18, 1.26 and 1.49; respectively. The obtained ductility factors are 12.67, 11.75 and 10.59 corresponding to models A1, A2 and A3. The increase in reinforcement ratio from 0.447 to 1.089 leads to increase in progressive collapse resistance equals to 26.27 % but with a decrease in ductility equal to 16.42 %.

Table 3. Reinforcement details of the considered symmetric beams.

Model	Span length (m)	Flexural reinforcement			
		At support		Mid span	
		top	Bottom	top	Bottom
A1	5	4ϕ16	2ϕ16	2ϕ16	3ϕ16
A2	5	4ϕ20	2ϕ16	2ϕ20	4ϕ16
A3	5	4ϕ25	2ϕ20	2ϕ25	4ϕ20

Stirrups ϕ10 @ 250

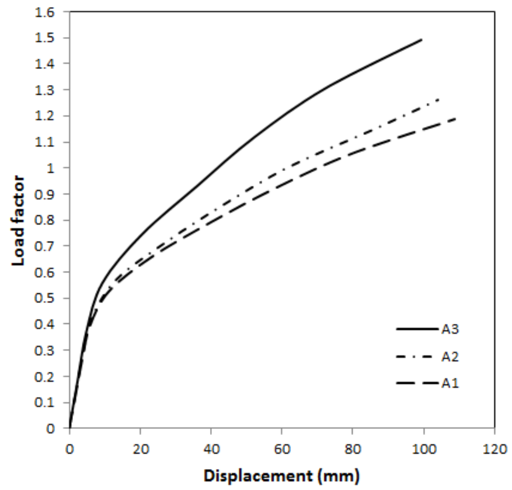


Fig. 8 Variation of capacity curves with reinforcement area.

Table 4. Normalized ultimate loads and ductility factors variation with flexural reinforcement area.

Model	Load factor	Yield deflection (mm)	Maximum deflection (mm)	Ductility factor μ
A1	1.18	8.6	109.0	12.67
A2	1.26	8.84	103.9	11.75
A3	1.49	9.38	99.3	10.59

4.3 Effect of continuity of bottom reinforcement

The effect of continuity of bottom reinforcement on the response of double span beams to the removal of interior column is investigated. Three models of double span beam column sub-assemblage have constructed with span length 7 m and have flexural reinforcement consist of 3 bars of ϕ 25mm at support (top) and 5 bars of ϕ 16 mm at middle (bottom) as shown in Fig. 9 and Table 5. The effect of extending two bars, three bars and five bars from middle bottom reinforcement to the support on the progressive collapse response is investigated. ACI 318 (2019) code requires extending two bars from bottom reinforcement of the middle span to the support for integrity purposes. In this section, the effect of extending two bars, three bars and all the bottom five bars of the bottom reinforcement to the column is investigated in order to obtain clear comparison regarded to the number of extended bars to the support. Half of the model is simulated by the finite elements due to symmetry to reduce computational time. Fig. 10 shows the finite element models of the three cases of reinforcement extensions in beams assemblages. Figure 11 compares the capacity curves in terms of normalized load deflection relations of double beam models corresponding to three different amount of extended bars to the supports. Also, values of ultimate load factor and ductility factors are presented in Table 6. The comparison results show increases in capacity due to increases in number of extended bars to the support.

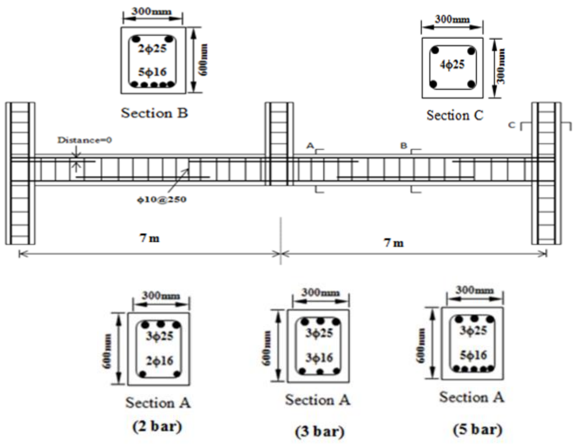


Fig. 9 Dimensions and reinforcement details of the considered double span beams.

Table 5. Reinforcement details of the considered symmetric beams.

Model	Span length (m)	Flexural reinforcement			
		At support		Mid span	
		top	bottom	top	bottom
Ae1	7	3ϕ25	2ϕ16	2ϕ25	5ϕ16
Ae2	7	3ϕ25	3ϕ16	2ϕ25	5ϕ16
Ae3	7	3ϕ25	5ϕ16	2ϕ25	5ϕ16

Stirrups ϕ10 @ 250

It is found that the model with 5 extended bars, 3 extended bars and 2 extended bars exhibit load factor capacities equal to 0.92, 0.86 and 0.82, respectively. Extending bars to the middle support slightly improves the capacity by increasing the flexural strength at the faces of the removed column where the moment reversed from the negative to the positive. The increases in capacity by extending 3 bars and 5 bars over that of 2 bars are 5.56 % and 12.9 %, respectively. However, results show large decrease in ductility of 15.68 % and 40.28 % corresponding to extending 3 bars and 5 bars; respectively, less than the ductility obtained by extending 2 bars. From the analysis results, it is evident that beams perform little more redundancy but less ductility by increases in ultimate capacity with increasing number of extended bars to the supports.

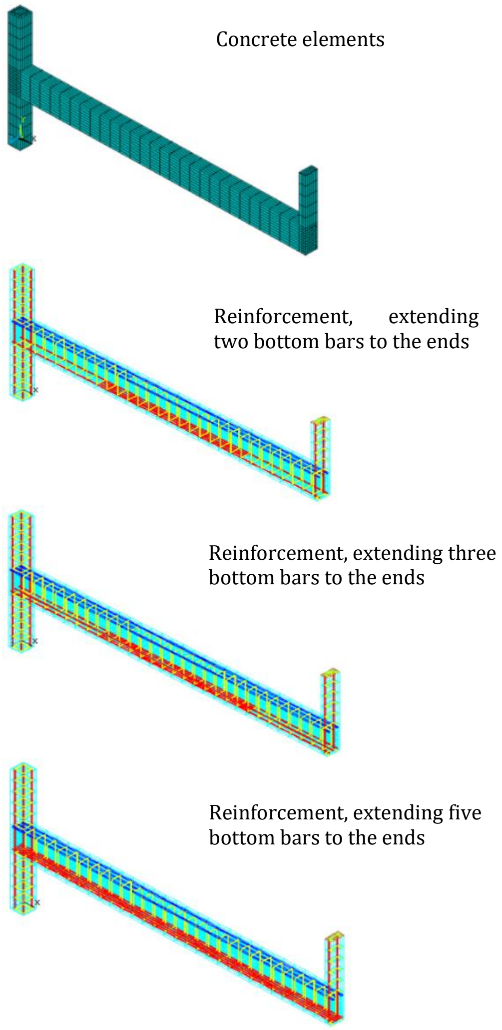


Fig. 10 Finite element modeling of the four cases of reinforcement extensions in beams assemblages.

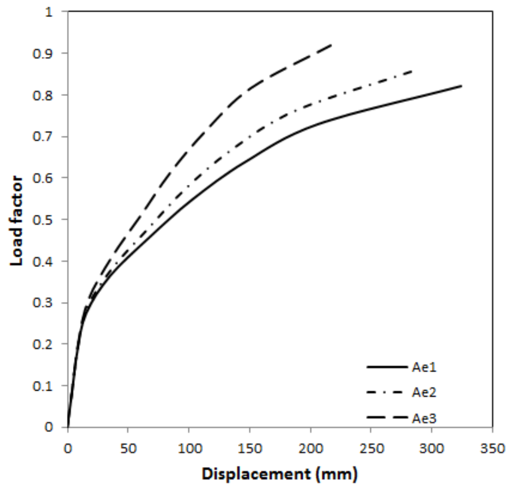


Fig. 11 Variation of capacity curves with number of extended bottom bars.

Table 6. Normalized ultimate loads and ductility factors variation with number of extended bottom bars.

Model	Load factor	Yield deflection (mm)	Maximum deflection (mm)	Ductility factor μ
Ae1	0.82	15.4	324.2	21.05
Ae2	0.85	16.1	285.8	17.75
Ae3	0.92	17.2	216.2	12.57

5. Conclusions

Two main successive mechanisms are developed following failure of interior support to continuous beams that may resist progressive collapse.

These two main resisting mechanisms include flexural mechanism and catenary action mechanism. Design parameters have different degrees of influence on resisting mechanisms. One of the main design parameters is the span length in which increasing span length resulted in increases in bending moments accompanied by increases in required flexural reinforcement. This study investigates the effect of increasing span length along with increasing flexural reinforcement ratio on the progressive collapse performance of double span beams over the failed column considering flexural resisting mechanism. On the other hand, the effect of flexural reinforcement ratio and continuity on progressive collapse performance considering flexural mechanism is investigated. The progressive collapse investigations are conducted using nonlinear static pushover analysis in which capacity curves are derived and compared with demanded capacities. A three-dimensional nonlinear finite element method is adopted in this study to model the double span beam-column sub-assemblage specimens subjected to the residual gravity loads above the failed column. The considered models were simulated using a finite element program ANSYS 11.0 and verified against previous experimental tests considering only ultimate moment resisting mechanism. Analysis results show that increasing span length from 5 m to 7 m resulted in decreasing the strength to resist progressive collapse by 25.4 %. However, increasing span length from 5 m to 7 m resulted in increasing the ductility by 103 %. On the other hand, increasing reinforcement ratio of top flexural reinforcement ratio from 0.447 to 1.089 leads to the increase in strength equal to 26.27 % accompanied with decreasing ductility by 16.42 %. In addition, extending all bottom bars rather than the minimum specified two bars resulted in increasing strength and decreasing in ductility by about 12.9 % and 40.28 %, respectively. The predicted performance and conclusion are related to flexural resisting mechanism. Further investigations considering the effect of the considered parameters on the catenary action are recommended for future study. Also, the effect of column reinforcement as well as column size is recommended for future investigations considering the locations of bridging beams along the height of the building.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work related in this paper.

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