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Numerical study of the effect of reinforcement detailing on progressive collapse behaviour of reinforced concrete structure

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Abstract:*Building structures subjected to explosion may result in the loss of column. In such case the ability of the structure to resist collapse depends on the structural system especially the reinforcement detailing at the beam column joint of the damaged column. This study investigated the effect of different detailing schemes on the progressive collapse behaviour of reinforced concrete building. Three different detailing schemes were considered namely Model A, B and C. In Model A, the bottom reinforcements were continuous through the joint. In Model B, the bottom reinforcements were lapped within the joint at the same section while in Model C, the bottom reinforcement were lapped within the joint but in a staggered manner. The result showed that Model C had a higher load resistance capacity compared to Model B although their flexural behaviour were the same. Using the pseudo static curve, the dynamic load resisting capacity for Models A, B and C at failure were 127KN, 95KN and 116KN respectively. This shows that lapping reinforcement in a Staggering way as allowed by Eurocode 2 can help resist progressive collapse much better than lapping the reinforcement at the same section.*

Key Word*:Progressive collapse, Column loss, Reinforcement detailing, Pseudo static curve.*

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I. Introduction

Building structures are designed to resist loads based on ultimate flexural capacity. This does not take into consideration any abnormal load. When a structure is subjected to abnormal loads, the vertical structural members such as column may be damaged. In such scenario, the load transfer path of the building is altered. If the directly affected part is not strong enough to resist the effect, failure will initiate andmay eventually lead to the failure of the entire building or a disproportionate part of it. According to American Society of Civil Engineers[1], Progressive Collapse is the failure of the entire structure or disproportionate part of it, caused by a local member failure which spread from member to member.

Interest in progressive collapse began after the collapse of the 22-storey Ronan Point structure by gas explosion in 1968. Following this collapse, the UK incorporated measures aimed at preventing progressive collapse in the National Code. The interest was re-awakened after the terrorist attack on the World Trade Centre building in 2001. Accordingly, US codes[2,3] have specified two methods for assessing and designing against progressive collapse. These are; indirect and direct methods. The indirect method consists of the tie force while the direct method consists of the Alternate Load Path Method (ALP) and the Specific Load Factor Method (SLFM).

In the design of RC buildings, Eurocode 2 allows for different detailing schemes at the beam-column joint. Several experimental test have been carried out in the last decade to understand the structural behaviour of Reinforced Concrete (RC) beam-column connection under a middle column removal scenario using Alternate Load Path (ALP) method. The researches showed that in the presence of axial restraint, flexural action, compressive action and catenary action can develop[4,5,6,7]. However, study on the behaviour of different reinforcement detailing schemes at the beam column-jointon the progressive collapse resistance capacity of RC building is yet to be investigated. This study therefore focused on the effect of different reinforcement detailing schemes on the progressive collapse of RC structure.

II. Material And Methods

For this study, design of reinforced concrete building was considered. The structure is a 4-storey office building designed according to Eurocode 2. As shown in Figure 1,the building has a plan dimension of 16m by 16m and a slab thickness of 150mm. Dead load due to slab was determined as $3.6KN/m^2$ based on concrete density of $24KN/m^3$. For an office building, the imposed load according to Eurocode 2 is $5KN/m^2$. A typical periphery of an office building is usually cladded, thus a load of 10KN/m was assumed for additional load due to cladding. From the tributary area supported by the beam along gridline A/1-5 in Figure 1, the factored load on the periphery was calculated as 30KN/m.

Figures 2shows the bending moment diagram from elastic analysis of frame in grid line A/1-5 with and without the middle column. It can be seen that the moment at the column removal point changes direction from hogging to sagging moment. Such a change is never considered for buildings under normal loading conditions. Studies by Yu and Tan[7]showed that failure of RC sub-assembly was caused by the fracture of reinforcement at the middle beam column interface. This implies that different detailing schemes which is satisfactory under the conventional loading with intact column may have different behaviour at large displacement regime under column removal scenario.

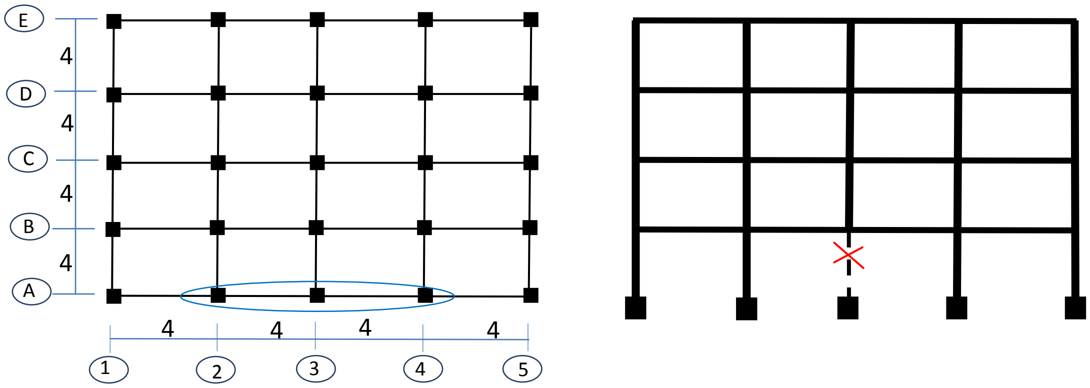


Figure 1: (a) Plan view of the building (b) Elevation of the building (all unit are in mm)

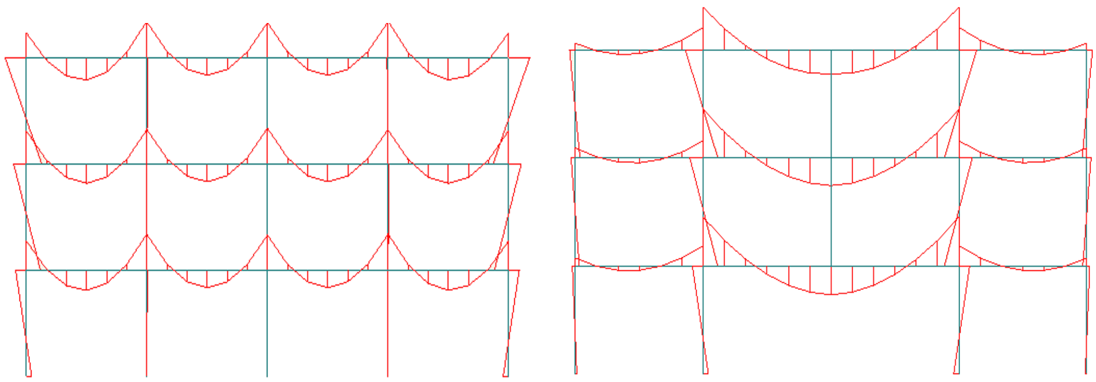
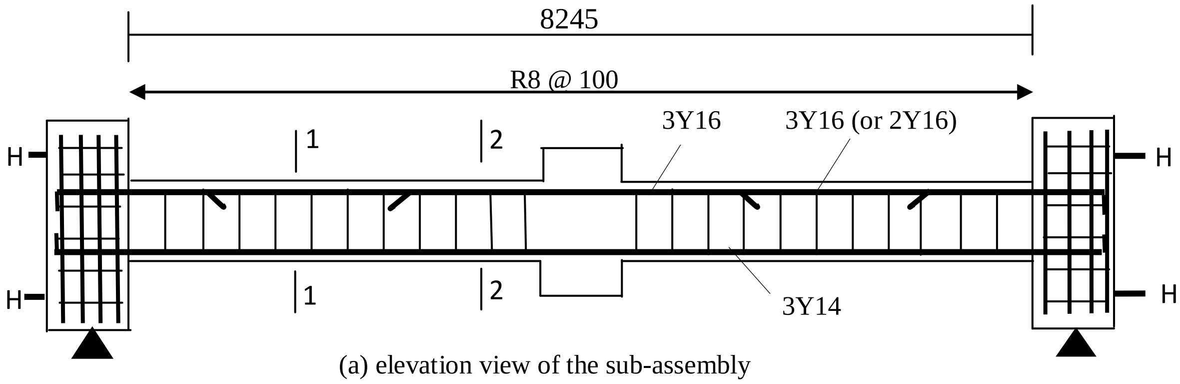


Figure 2: elastic bending moment distribution (a) with middle column (b) without middle column

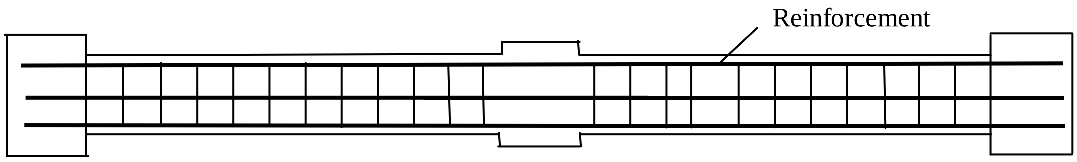
In this paper, the behaviour of the sub-assembly directly under the removed column is investigated. To this end, the columns adjacent to the loss column are replaced with enlarged column stub. Three different detailing schemes identified as Model A, B and C are considered. Model A has continuous reinforcement at the top and bottom section. For models B and C, one of the top reinforcements is curtailed at a distance of 1300mm from the column position. According to Eurocode 2, reinforcement bars may be lapped at the same section or staggered at different section. These two different lapping techniques are used for lapping bottom reinforcement in model B and C respectively. Table 1 shows the detail of the reinforcement and the lap splice arrangement of bottom bars in models B and C are shown in Figure 3.

Table 1: Reinforcement detail for different models considered

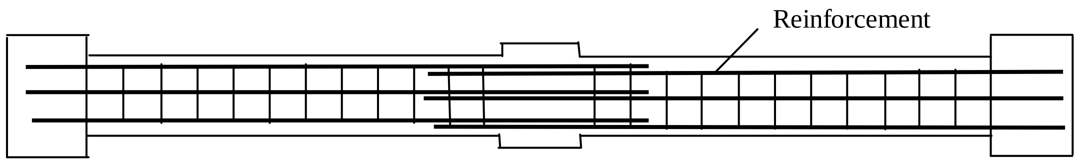
Model Identification	Longitudinal reinforcement				Reinforcement arrangement type
	Section 1-1		Section 2-2		
	Top	Bottom	Top	Bottom	
A	3Y16 (1.1%)	3Y14 (0.84%)	3Y16 (1.1%)	3Y14 (0.84%)	Continuous
B	2Y16 (0.73%)	3Y14 (0.84%)	3Y16 (1.1%)	3Y14 (0.84%)	lap spliced
C	2Y16 (0.73%)	3Y14 (0.84%)	3Y16 (1.1%)	3Y14 (0.84%)	lap spliced



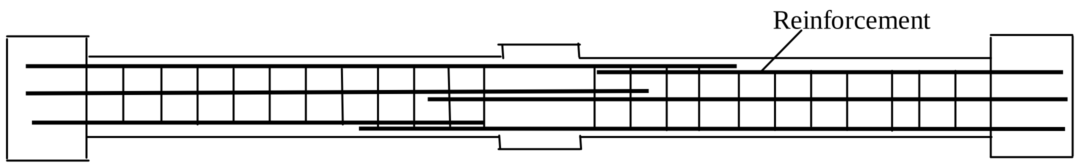
(a) elevation view of the sub-assembly



(b) Plan view showing the continuous bottom reinforcement in model A



(c) Plan view showing the lapping of bottom reinforcement in model B (at same section)



(c) Plan view showing lapping of bottom reinforcement in model C (staggered lapping)

Figure 3: Reinforcement detail of the sub assembly for models A, B and C

Material Models

Concrete was modelled with continuous surface cap model (MAT 159 in LS DYNA) with reduced integration solid elements. Key features of this model include isotropic constitutive equation, a yield surface which is formulated in terms of three stress invariants and a hardening cap which can contract and expands. The properties were generated automatically by inputting the concrete compressive strength of 35MPa and aggregate size of 10mm. Reinforcing bar was modelled with piecewise linear plastic model (MAT_024 in LSDYNA). A tri-linear stress strain curve was used for the reinforcement. The yield and ultimate strength of the rebar was 500 and 600MPa respectively with rupture strain assumed to be 11%.

Finite Element Models

Three different sub-assembly models were developed to study the effect of different detailing on progressive collapse. Mesh size of solid element ranges from 17.5 to 20mm giving a total of 133932 elements. For beam element the mesh size was between 20 and 40mm giving a total of 5302, 5282 and 5270 elements for models A, B and C respectively. The nodes between solid elements representing concrete and beam element representing reinforcement were mesh together assuming a perfect bond. Bottom nodes of column were constrained in the vertical direction. Axial restraint was simulated using beam element with elastic property. The end of these beam elements were constrained in the horizontal direction. To avoid local damage to the middle column, a steel plate was placed on top of the middle column. The steel plate is made of solid element with elastic property. Load was applied to the middle column by pushing all nodes of the steel plate according to the prescribed velocity-time curve.

The developed FE model was validated with experimental test conducted by Yu and Tan[7]. The subassembly consisted of 2 beams with 3 column stubs. The span, width and depth of the beam was 5750, 150 and 250mm respectively. Top reinforcement was 3Y13 with one of the top reinforcement curtailed at a distance of 1000mm from the middle and end columns. Bottom reinforcement was 2Y13 and continuous throughout the beam. Column stubs at the end were connected to a reaction wall on one side and A-frame on the other side. The middle column was pushdown in a displacement control manner using the hydraulic actuator. Mesh size of solid element representing concrete was between 17.5 and 20mm. and that of reinforcing bar was 20mm. In the experiment, movement of the end column stubs were measured. This movement was represented in the FE model by defining a non-linear spring with the load-deformation curves reported in the experiment.

III. Result and Discussion

Figure 4 shows the applied load versus vertical displacement and the horizontal force versus vertical displacement of the middle column. It can be seen that the FE model predict failure mechanism similar to that of the experiment. The sudden reduction in the experimental test was due to the fracture of reinforcement near the middle joint interface which was followed by fracture of reinforcement near the end column stubs. Similar failure modes were observed in the FE simulation although the fracture of bottom reinforcement at middle column occurred at a much higher displacement in the FE model.

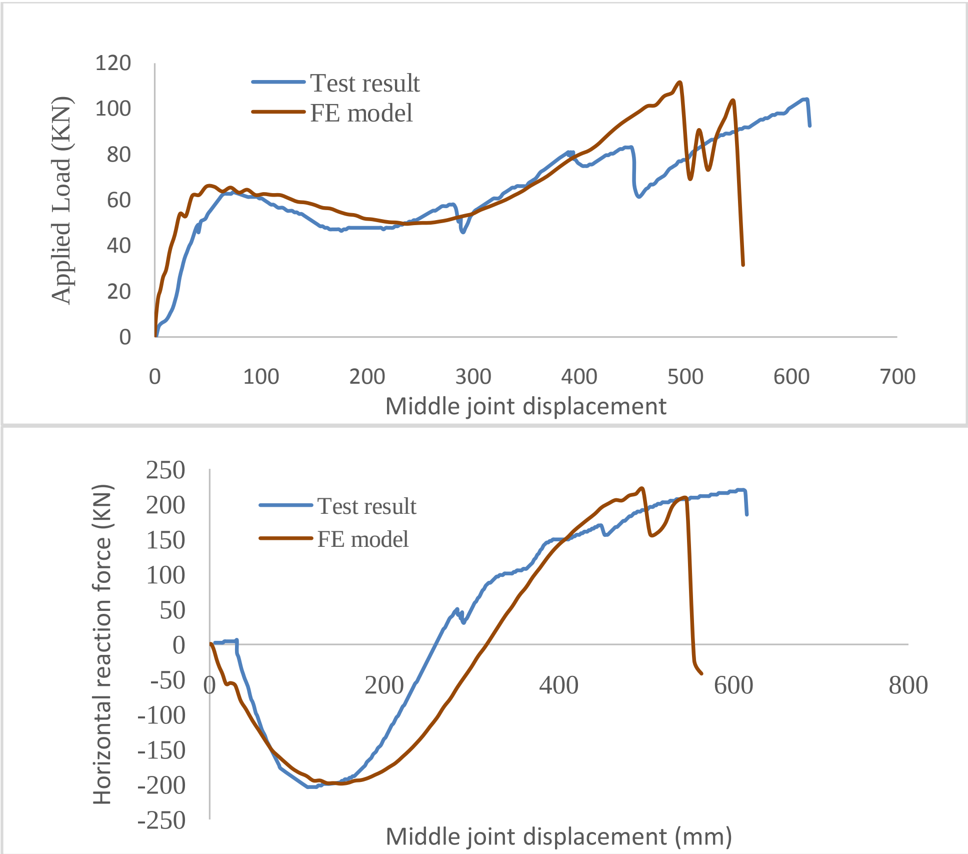


Figure 4: Comparison between experimental and FE result (a) Applied load versus middle column displacement (b) Axial force versus middle column displacement

Figure 5 shows the applied load-middle column displacement and axial force middle column displacement for the three models considered in the study. It can be seen from these figures that the response up to the flexural stage are similar with development of catenary action in all the models. As expected, model A with continuous reinforcement sustained loads until fracture of bottom reinforcements at the middle beam column interface and further loading caused fracture of top reinforcement at the middle beam-column interface leading to complete failure of the substructure. In model B, damage concentrated at the end of lap splice, although catenary action developed, fracture of bottom reinforcement around the lap splice positon occurred at a much lower displacement of the middle column. Subsequent fracture of the top reinforcement at the beam middle column resulted in complete loss of load carrying capacity. Model C had similar failure mechanism as models A and B and failed by fracture of bottom reinforcement near the middle column. Compared to model B, the fracture of reinforcement in Model C occurred at a much higher displacement of middle column. This may be due to the fact in the event of column loss, the beam column interface near the removed column is the most critical area. Hence, lapping of reinforcement in this critical section lead to early failure of the sub-assembly. Figure 6 present the axial force-middle column displacement for section near the middle column and mid-way between the middle and the end column.

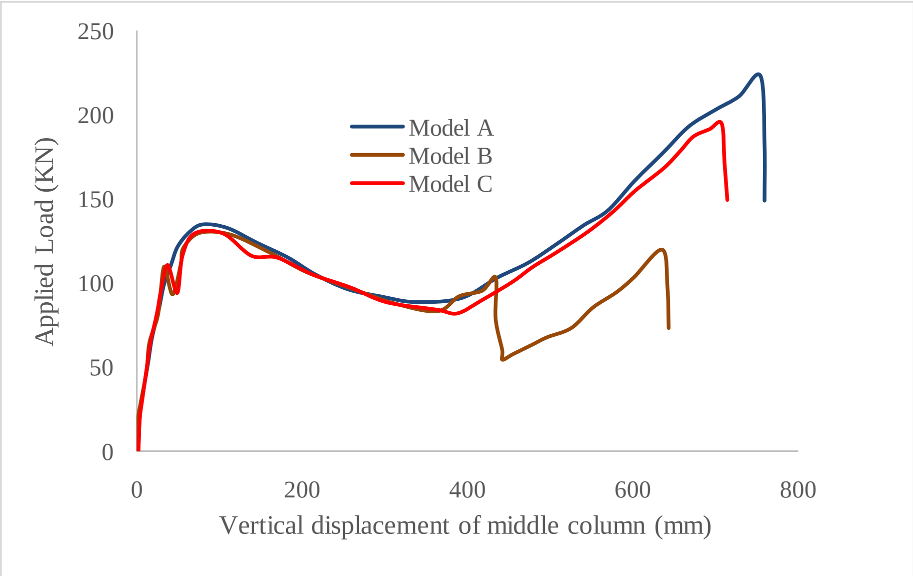
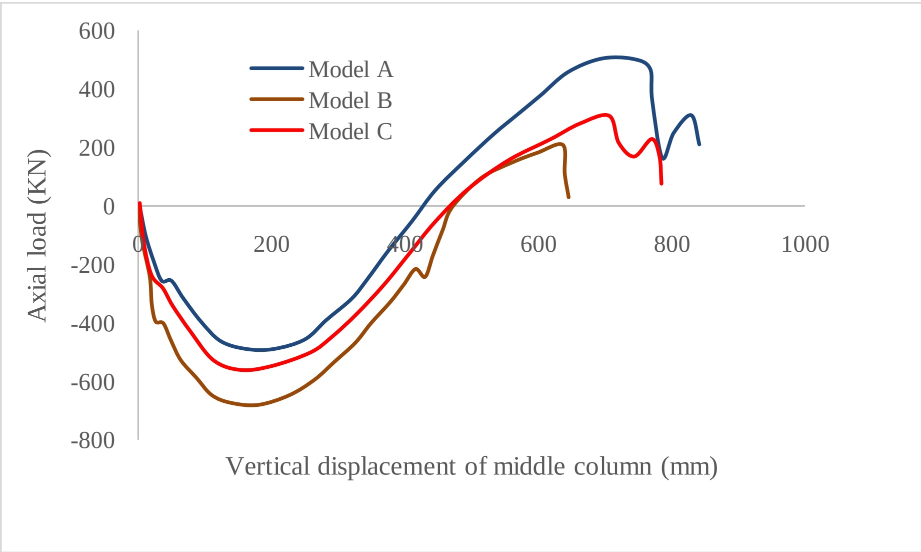


Figure 5: Applied Load versus middle column displacement



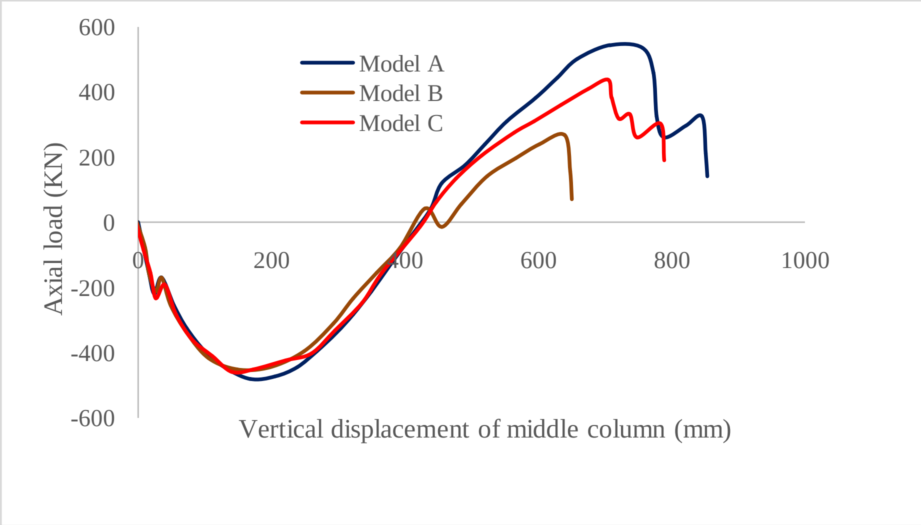


Figure 6: Axial force versus middle column displacement (a) section near middle column (b) section mid-way between middle column and end column

For section near the middle column, axial compression force increased due to the lapping of reinforcement, this increase in axial compression is only within the lapping zone hence have no effect on the global response of the subassembly as seen in the global load displacement response. Curtailment of top reinforcement at some point away from the middle column has no pronounced effect on the overall response. The damage contours of the models at three (3) different middle column displacement are shown in Figures 7-9.

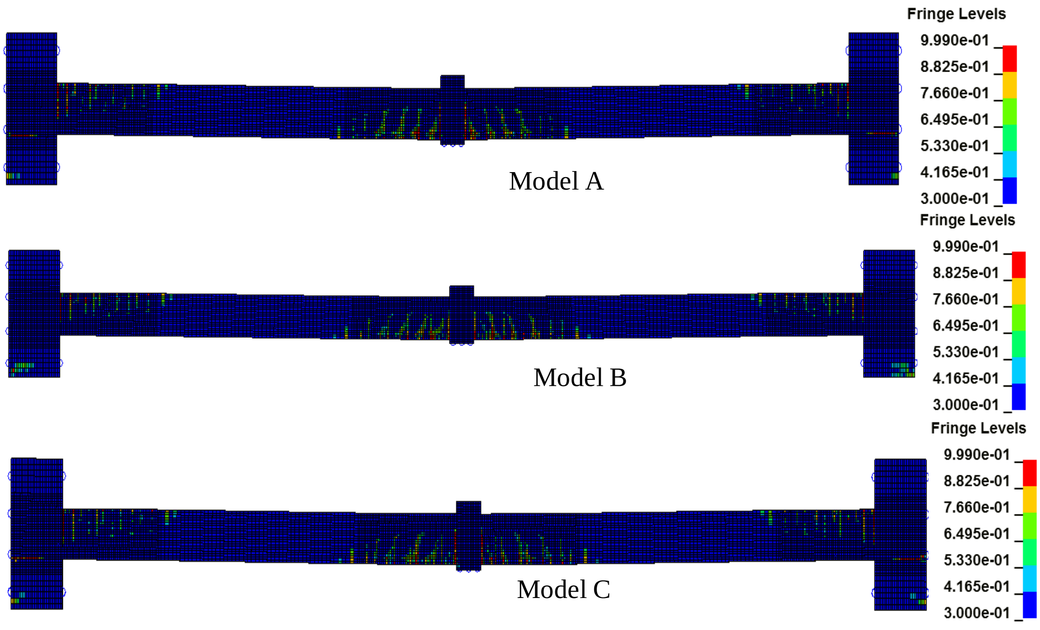


Figure 7: Concrete damage index contours at a displacement of 65mm (contours represent damage. The damage is shown from 0.3 to 1 where 1 means complete)

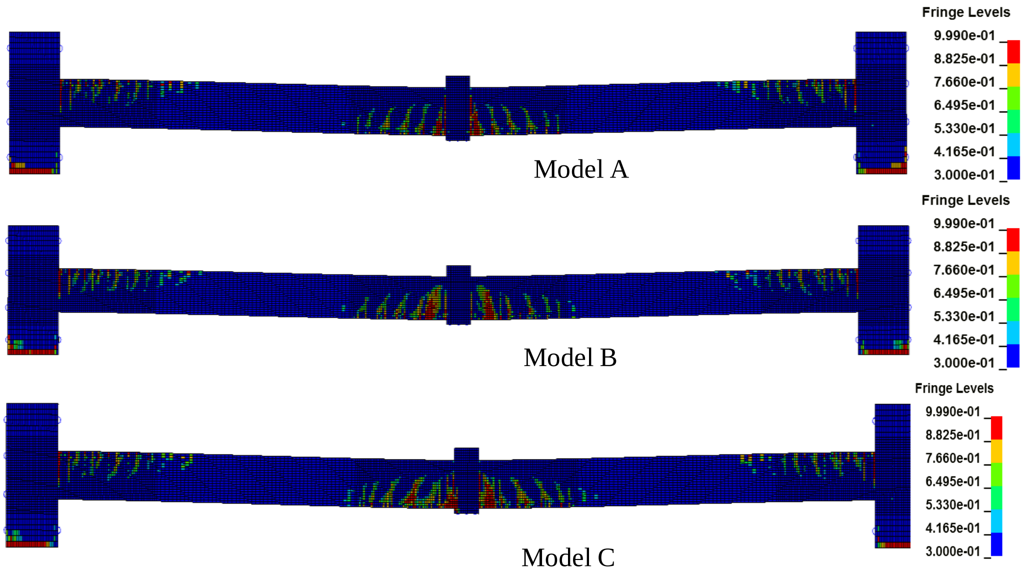


Figure 8: Concrete damage index contours at a displacement of 100mm (contours represent damage. The damage is shown from 0.3 to 1 where 1 means complete damage)

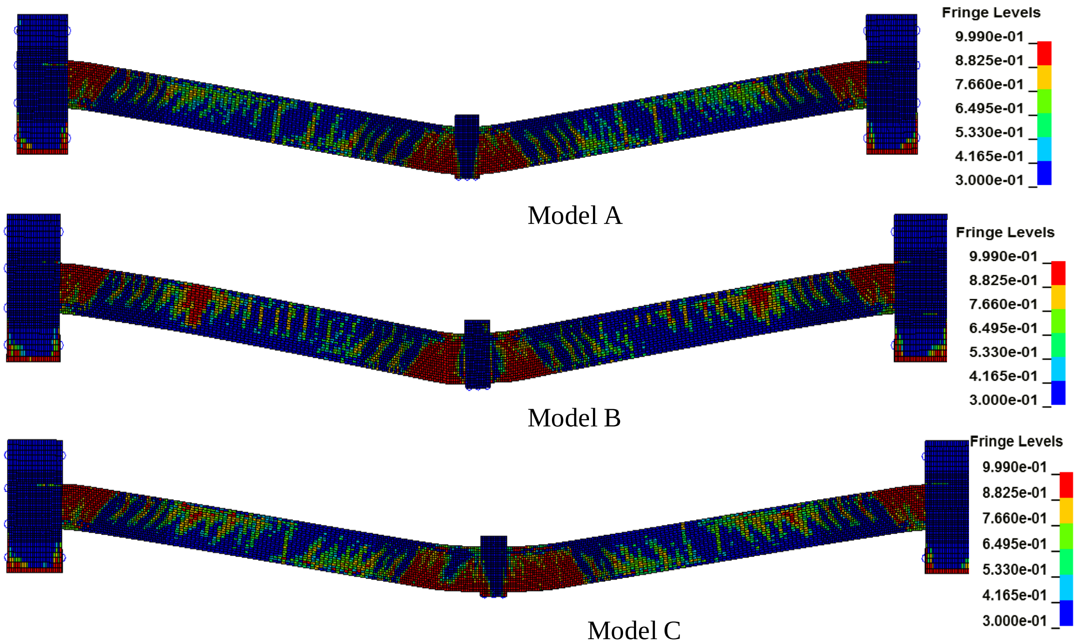


Figure 9: Concrete damage index contours at a displacement of 500mm (contours represent damage. The damage is shown from 0.3 to 1 where 1 means complete damage)

At a middle column displacement of 65mm, the damage contour in the three models are similar. As the displacement increased, damage concentrated near the middle column interface. At a displacement of 500mm, damage zone in Model C was more than damage in Model B. This is because the lapping zone in Model C is much larger than lapping zone in Model B. This could also be responsible for the higher loads sustained by Model C compared to Model B. In Model A, damage only concentrated near the middle and end column but in Models B and C damage also occurred at the position of curtailment of the top reinforcement.

Assessment of Progressive Collapse Resistance

The resistance functions (load-displacement curves) for different models in Figure 5was determined under static conditions without any significant effects due to strain rate. In reality, progressive collapse is a dynamic event, and accurate assessment of the amplitude of the response needs to take into consideration the dynamic effect arising from the sudden loss of column.

In this study, the energy balance method proposed by Izzuddin[9]was used to convert the static load displacement curve into the dynamic curve. The method involves (1) determination of nonlinear static resistance function of the structure without the loss column, (2) determination of maximum dynamic response under sudden column removal using simplified dynamic assessment i.e. converting the nonlinear static resistance function to pseudo static curve. The pseudo static curve is determined on the assumption that part of the building bridging over a loss column has a dominant deformation mode and respond like a single degree of freedom systems. The accuracy of this method depends on the accuracy of the nonlinear static resistance function.

An illustration for conversion of nonlinear static curve to pseudo static curve for a simple bilinear nonlinear static resistance function is illustrated below and the result is shown in Figure 10.

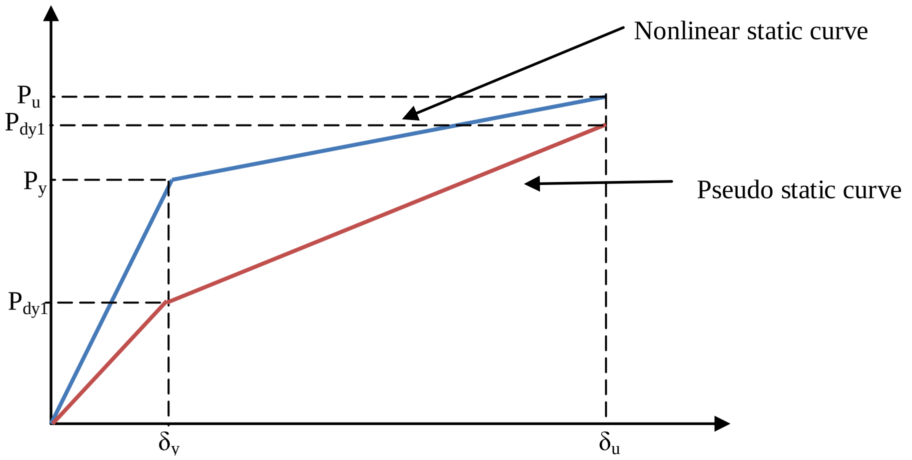


Figure 10: Illustration of the conversion of nonlinear static curve to pseudo static curve

From energy principle;
Internal work done = external work done
At displacement δ_y;

$$P_{dy1}\delta_y = 0.5P_y\delta_y \Rightarrow P_{dy1} = 0.5P_y \quad (1)$$

At displacement δ_u;

$$P_{dy2}\delta_u = 0.5P_y\delta_y + P_y(\delta_u - \delta_y) + 0.5(\delta_u - \delta_y)(P_u - P_y) \quad (2)$$

$$\Rightarrow P_{dy2} = \frac{0.5P_y\delta_y + P_y(\delta_u - \delta_y) + 0.5(\delta_u - \delta_y)(P_u - P_y)}{\delta_u} \quad (3)$$

Using the principle above, the static load displacement curve in Figure 5 is converted to pseudo static curve as shown in Figure 11.It could be further seen that Models A and C can resist dynamic load up to a capacity of 127KN and 116KN respectively. This is much higher than Model B which can only resist maximum load of 95KN at failure.

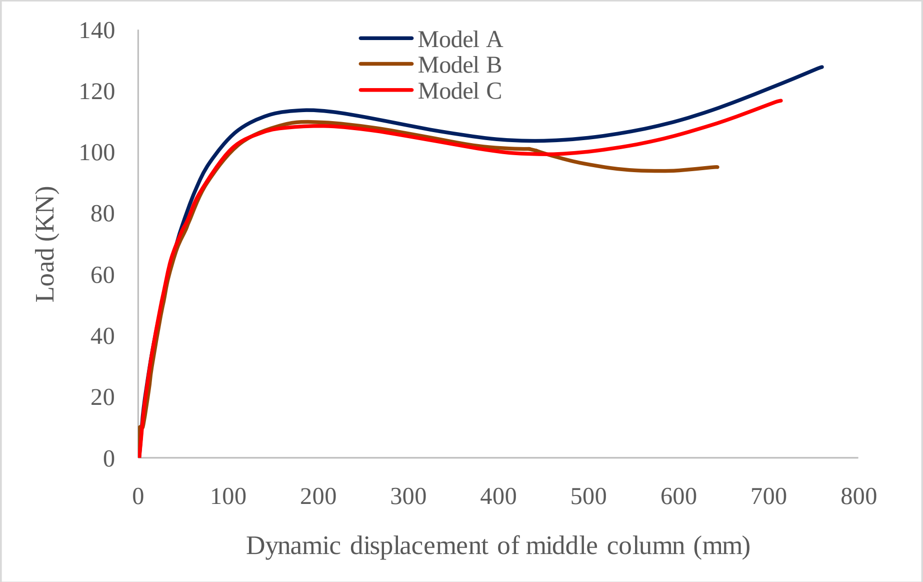


Figure 11: Pseudo static curves for different RC sub-assemblages

IV. Conclusion

This paper focused on investigating the effect of reinforcement detailing on progressive collapse resistance. The following conclusions can be deduced;

1. Different detailing schemes result in different resistance at the catenary action stage.
2. Lapping of reinforcement in a staggered way favours development of catenary action much better than lapping at the same section.
3. The static load-displacement curve can be converted to pseudo static curve which can be used for assessing progressive collapse resistant of a structure.

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