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Seismic Design Effect on the Progressive Collapse Potential of R.C. Frames

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Abstract
Progressive collapse potential studies are performed on two-dimensional R.C frames with different reinforcement details under the effect of interior column removal. This study aims to determine the effect of seismic detailing on the structural resistance and behavior of RC frames during progressive collapse event. The R.C frames and reinforcement details would be designed according to Egyptian Code ECP-203 [1]. In this study, two prototype frames were executed from concrete structure and were designed. The first frames F1 is designed with non-seismic reinforcement detail. The second frame F2 is designed with seismic rebar detail. Nonlinear software, used for modeling R.C frames, is extreme loading for structures (ELS). The numerical technique is based on the applied element method with suitable stress-strain relations for concrete and steel. It is found that frame with seismic detail improve the frame resistance against progressive collapse event by 40-45% more than the non-seismic detail in all response stages. The load-steel strain curves and the cracking patterns are also compared for the two frames.

Keywords: Reinforced concrete frames; Progressive collapse; Load-deflection curves; Crack patterns; Applied element method; ELS.

1. Introduction

Several structural progressive collapses accidentally took place in the last few decades. For example, in 1968, the collapse of the 22-story Ronan building [2], East London took place due to gas explosion in 18th floor. In 1995, the Murrah Federal Office Building in Oklahoma City was collapsed due to a terrorist bomb explosion at the ground floor [3]. In 2001, the World Trade Center [4], New York, was totally collapsed due to planes impact at the tower upper levels. Recently, design guidelines such as General Services Administration (GSA) [5] and the Unified Facilities Criteria (UFC) [6] addressed progressive collapse due to sudden loss of a main vertical support. Progressive collapse is a critical event that happened due to failure of load carrying element. For this event, it is important to study the different detailing of frames to withstand progressive collapse event. Much attention has been given to the behavior of beams that bridge over removed column areas, which are under amplified gravity loads in beam-column substructures or planar frames (Sadek et al. [7]; Mehrdad et al. [8]; Yi et al. [9]; Hou and Yang [10]; Kim and Choi [11]). It was concluded that a generous reserve capacity of the catenary action in beams that carry the gravity loads in a tension mode is necessary for mitigating progressive collapse.

In this paper, the alternative path method is used to evaluate the resistance of the frames subjected to progressive collapse. It occurs by removing the one or several bearing element and analysis the remaining structures to determine if this initial damage propagates from element to another. Reinforcement details for frames are conducted to investigate the different structural resistance mechanisms during progressive collapse occurrence. Also, the aim of this study is to determine the reinforcement details effect on the response of frames after column removal event.

2. The Applied Element Modelling (AEM) of R.C Frames

AEM is a modeling method adopting the concept of discrete cracking. As shown in Fig. 1(a), the structure in the AEM is modeled as an assembly of elements connected together along their surfaces through a set of normal and shear springs. The two elements shown in Fig. 1(b) are assumed to be connected by normal and shear springs located at the contact points, which are distributed on the element faces [12]. These connecting springs represent the state of stresses, strains and connectivity between elements. They can represent both concrete and steel reinforcing bars. Each single element has six degrees of freedom: three for translations and three for rotations. Relative translational or rotational displacement between two neighboring elements cause stresses in the springs located at their common face as shown in Fig. 2. Two neighboring elements can be totally separated once the springs connecting them rupture [12]. Fully nonlinear path-dependent constitutive models are adopted in the AEM as shown in Fig. 3.

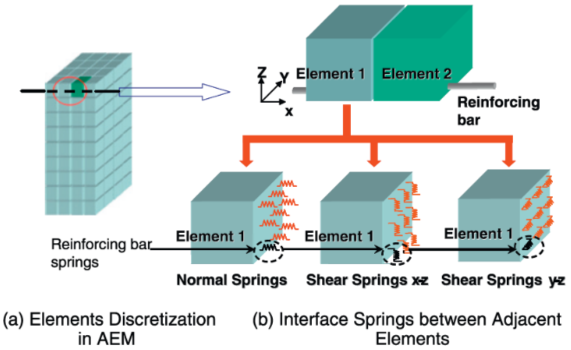


Fig. 1. Modeling of a Structure with AEM.

For concrete in compression, an elasto-plastic and fracture model is adopted [13] as shown in Fig. 3(a). When concrete is subjected to tension, a linear stress strain relationship is adopted until cracking of the concrete springs, where the stresses then drop to zero. The residual stresses are then redistributed in the next loading step by applying the redistributed force values in the reverse direction. For concrete springs, the relationship between shear stress and shear strain is assumed to remain linear till the cracking of concrete. Then, the shear stresses drop down as shown in Fig. 3(b). The level of drop of shear stresses depends on the aggregate interlock and friction at the crack surface. For reinforcement springs, the model presented by Ristic et al. [14] is used as shown in Fig. 3(c). Applied element method is a good way to simulate column removal. Also, it is a good way to describe the post failure event. Four different experimental frames were verified using applied element method by using ELS software as in [15]. Generally, the numerical results are in a good agreement with experimental results.

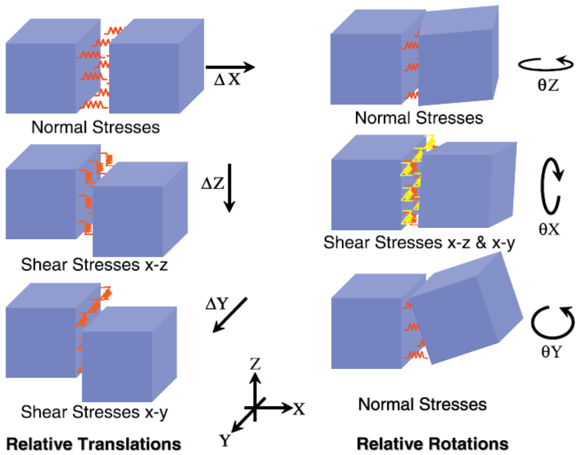


Fig. 2. Stresses in Springs due to Elements Relative Displacement.

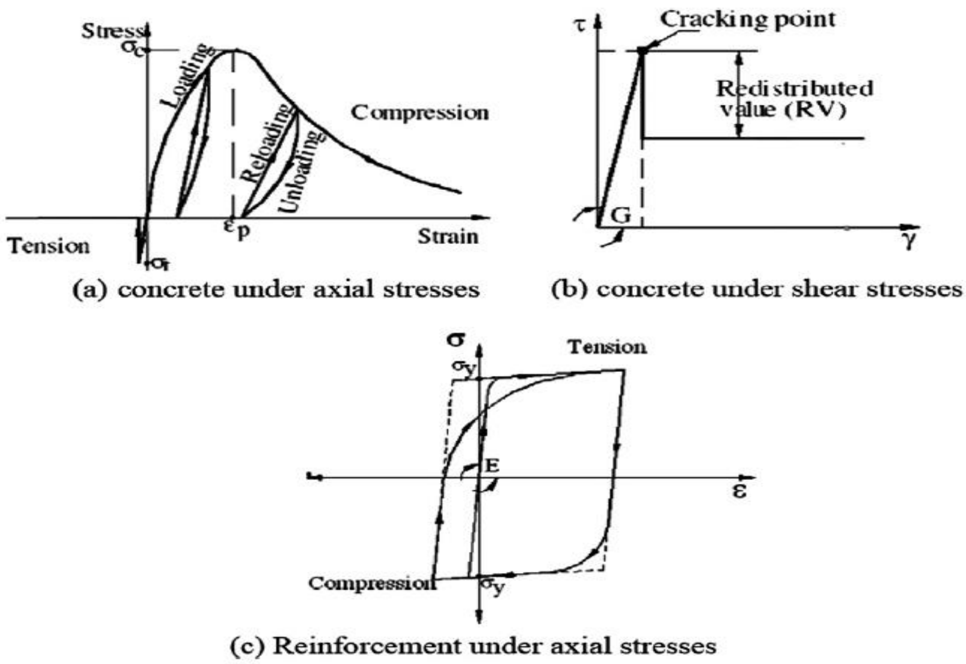


Fig. 3. Constitutive Models for Concrete and Reinforcing Bars

3. Design of Prototype Frames

The prototype frames are full scaled and assumed to be located at the middle of a multi-bay perimeter two-story frame as shown in Fig. 4. Two frames were designed according to ECP 203 [1]. Frame F1 was designed under the vertical load only with non-seismic detail. Frame F2 was designed under seismic and gravity loads with seismic detail. The geometry properties of protype frames are given in Table 1. For frame F1, as shown in Fig. 4, the top reinforcement ratio at the middle column and at the beam ends was 0.62% (4T18) that extended an anchorage distance of beam depth (d) after the end point of negative bending moment. The extended distance form column center line would be 1.80 m. The negative steel bars would be lapped with secondary steel of (4T12). The lap distance between two bars group would be 500 mm. This lap is carried out to investigate effect of lap on the progressive collapse event. The bottom reinforcement ratio was 0.37% (3T16). The bottom reinforcement was extended from column to another.

Fig. 5 represents the seismic steel details of frame F2 and the location of the removed middle column. The top reinforcement at the corner column and middle columns was (5T18) that represents a ratio of 0.77%. The top reinforcement extended a distance of 3.70 m from the centerline of each columns as shown in the figure. No secondary steel is added to the top reinforcement at mid span to comply the seismic details specifications. The bottom reinforcement ratio for both specimens was 0.49% (4T16). The bottom reinforcement extended form the column to column to achieve bottom moment capacity equal to at least half of top moment capacity at each support. The top reinforcement was a constant value at all sections of frame to achieve a reinforcement ratio not less than 0.25% at any section.

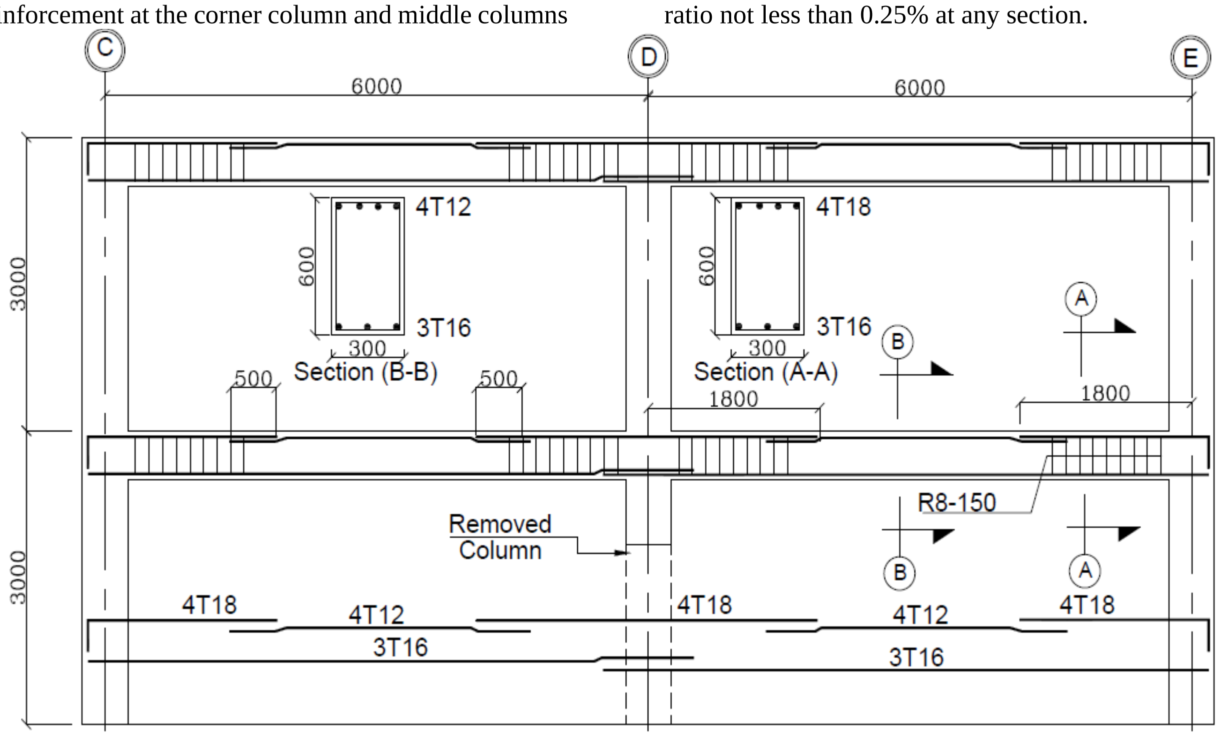


Fig. 4. Reinforcement Details for Non-Seismic Frame F1-ECP 203 [1]
(All Dimensions are in mm)

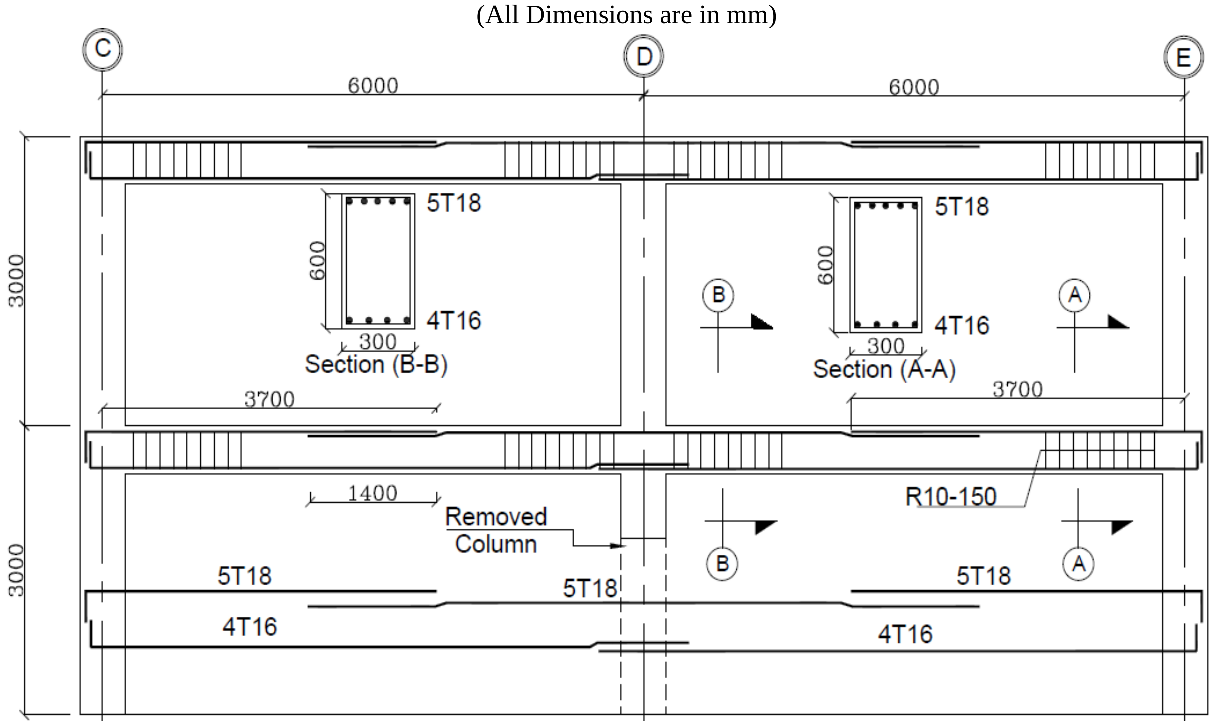


Fig. 5. Reinforcement Details for Seismic Frame F2- ECP 203 [1]
(All Dimensions are in mm)

Table (1): The Geometry Properties of the Prototypes Frames

Frame	***Beam Dimensions b x t***	***Position of rebar curtailment (mm)***	***Longitudinal Reinforcement***				***Top rebar Continuity***
			Section A-A		***Section B-B***		
		Lo	***Top***	***Bottom***	***Top***	***Bottom***	
F1	300 x 600	1800	0.62% (4T18)	0.37% (3T16)	0.27% (4T12)	0.37% (3T16)	Non- Continuous
F2	300 x 600	N/A	0.77% (5T18)	0.49% (4T16)	0.77% (5T18)	0.49% (4T16)	Continuous

4. Analysis Results of R.C Frames

Comparison between the results of (R.C) Frames named F1 and F2 that are designed with different reinforcement details is presented to study the performance under progressive collapse.

4.1 Predicted Load-Deformation Curve

For the two Fames (F1 and F2), the predicted load and deflection curves are shown in Fig. 6. The simulation of middle column removal is divided into four zones as following:

1) (Zone OA)

Zone (OA) can be considered as the elastic stage with cracking of frame beams observed at this zone. This stage represent the linear relation between the load and displacement. The load transfer mechanism through this zone is due to the shear force developed in beams due to double curvature bending. It is represented by the flexural action mechanism (Vierrendeel action). The displacements of F1 and F2 at end of this zone are 48 and 68 mm, respectively. The resisting loads of frames at end of this stage are (301 and 443 kN) as shown in Table (2). Progressive collapse detail of frame F2 improves the elastic zone by 42% in the level of displacement and 47% in the level of resisting load capacity more than non-seismic detail of frame F1.

2) (Zone AB)

Second zone in part (AB) can be considered as the elasto-plastic stage. At this stage the frame reach to ultimate state and the strain in steel bars exceed the yield limit. This stage represents the continuity of the flexural action mechanism through load transfer mechanism. At end of this zone, the predicted displacement of frames as shown in Fig. (6) are 138 and 156 mm for frames F1 and F2, respectively. The resisting loads for the same frames are (403 and 566 kN) as shown in Table (2). Consequently, using seismic details for F2 enhances the displacement by 13 % and load resisting by 40 % more than the non-seismic detail in frame F1.

3) (Zone BC)

Third stage in zone (BC) can be considered as the plastic hinges formation stage. The concrete starts crashing at this interval and the beams loss their flexural capacity. The crushing of concrete do not cause the lose of full flexural of beams but it causes decreasing of resistance force capacity. The load transfer mechanism at this stage represent the flexural action. This stage represents the last Vierrendeel action stage. The displacements of F1 and F2 at end of this zone are 378 and 415 mm, respectively. The resisting loads of frames at end of this stage are (246 and 263 kN) as presented in Table (2). Frame F2 detail improves the displacement by 10% and the resisting load capacity by 7% more than non-seismic detail of frame F1.

4) (Zone CD)

In the last zone of frame response, an increased in the resistance capacity is observed as shown in Fig. 6. The increase of resisting force due to formation of another mechanism called Catenary action [9]. This mechanism depends of the tension forces formed in the beams above the removed columns. This action depends on essentially two factors: the axial tensile force capacity of the beams bridging over the removed column, and the capacity of the rest of the structure to resist the tensile forces formed at the beam ends. The catenary action is activated at point C (the end of flexural action mechanism) until the point D (failure of frame). At end of this zone, the predicted displacement of frames as shown in Fig. 6 are 744 and 1008 mm for frames F1 and F2, respectively. As shown in Table (2), the resisting loads for previous frames are (426 and 636 kN). The improvement in the progressive collapse detail in F2 referring to F1 is 36% in level of displacement and the enhancement in resisting load level is 49%.

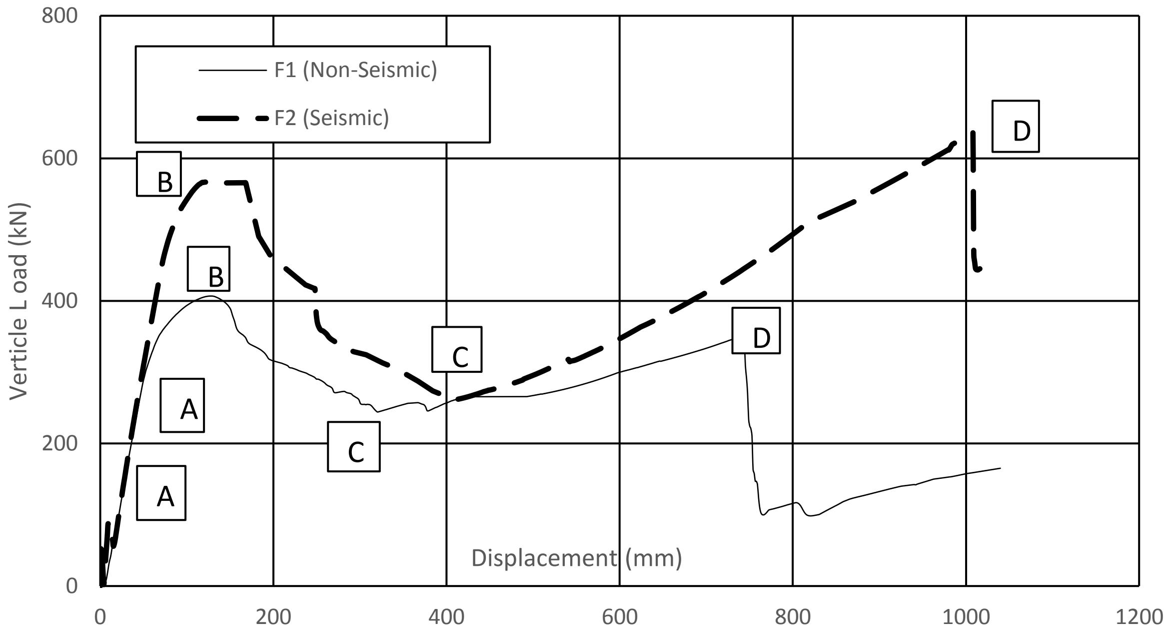


Fig. 6 Predicted Load-Deformation Curve for F1 and F2 Frames Cases

Table (2): Results at Critical load Transfer Stages

Frame	End of elastic stage			End of elasto-plastic stage		
	P (KN)	Δ (mm)	ε_s	P (KN)	Δ (mm)	ε_s
F1	301	48	-0.00044	403	138	-0.00115
F2	443	68	-0.00074	566	156	-.0013

Table (2): Results at Critical Load Transfer Stages (Cont.)

Frame	End of plastic hinge stage			End of catenary stage		
	P (KN)	Δ (mm)	ε_s	P (KN)	Δ (mm)	ε_s
F1	246	378	-0.00112	426	744	+0.00049
F2	263	415	-0.00129	636	1008	+0.00075

4.2 Predicted Steel Strain Curve

Fig. 7 represents the predicted steel strain of top steel bars in frame F1. At displacement 48 mm, steel strain is measured 0.000443 in compression. The strain value is decreasing gradually till displacement level of 340 mm, the measured steel bars strain is 0.00112 in compression. After this point the steel strain starts to increase gradually and the sign of steel strain changed from compression to tension due to activation of catenary action mechanism. After displacement level of 626 mm, the steel strain is measured a positive value after subjecting to axial tension force. At failure point, the steel strain is measured 0.000373 in tension value.

The predicted steel strain of top steel bars of frame F2 is shown in Fig. 7. At elastic stage, the displacement measures 68 mm and steel strain is measured 0.000737 in compression. The strain value is decreasing gradually by increasing displacement to reach 0.00129 in compression at displacement level of 409 mm. After maximum compression strain value, the steel strain starts to increase gradually and the sign of steel strain changed from compression to tension due to activation of catenary action mechanism. After displacement level of 837 mm, the steel strain is measured a positive value after subjecting to tension force. At failure point, the steel strain is measured 0.000702 in tension value.

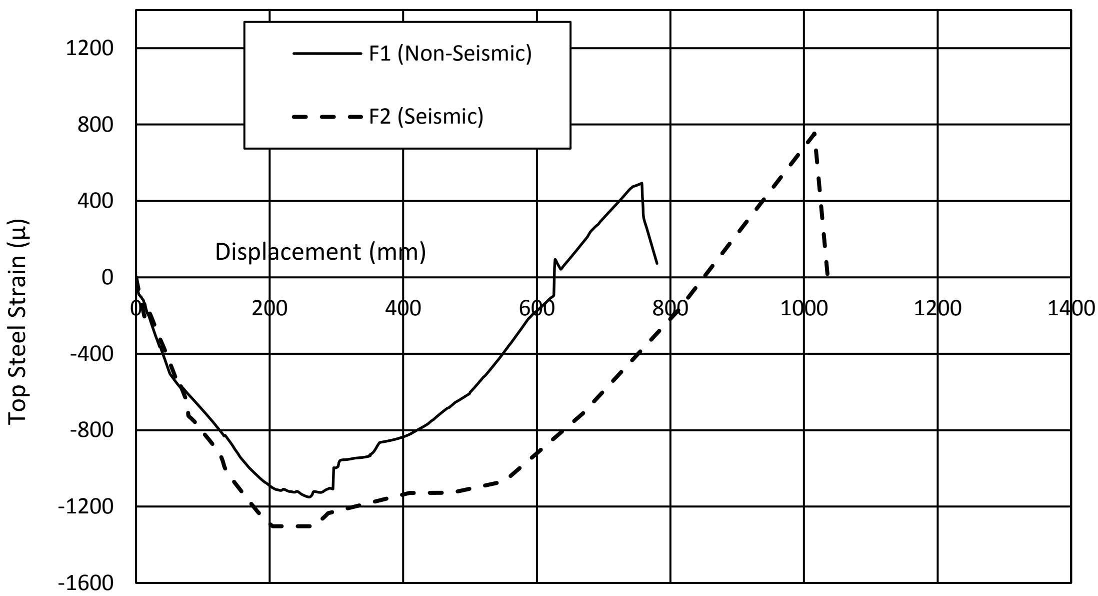
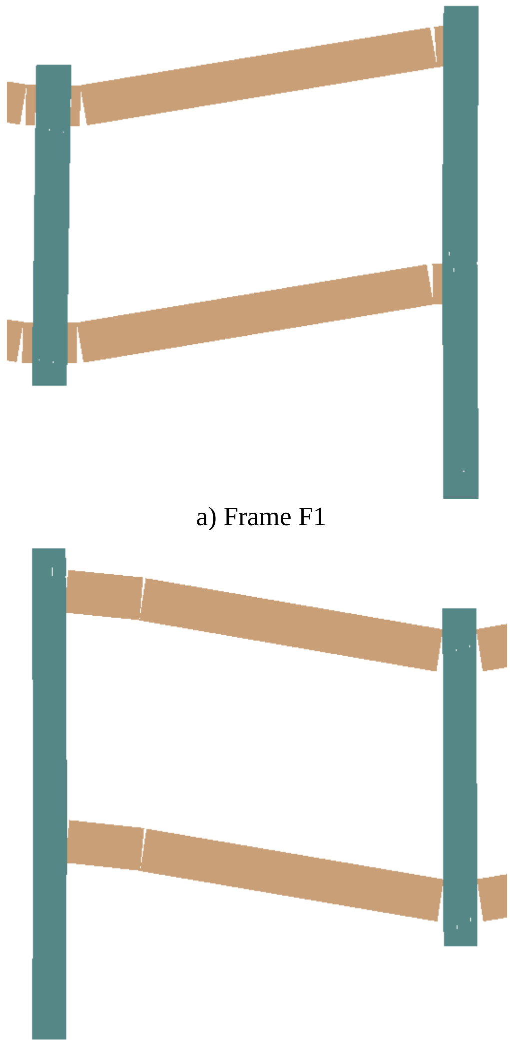


Fig. 7 Top Steel Strains Curve of First Floor Beams for F1 and F2 Frames Cases

4.3 Deformed Shape and Crack Pattern

Frame F1 is initially responded in an elastic trace and at displacement of 48 mm, cracks start to appear. The location of cracks were at the start and end of the beams as shown in Fig. 8(a). Due to the non-continuity of top reinforcement bars cracks formed at the locations where the bars of negative moment reinforcement cut-off, which ended at a distance of 1800 mm from the face of outer columns. These cracks formed in the all beams. As load increased, the cracks spread and widened and plastic hinges start to form at the same locations of cracks starting. Next, the concrete crushed and fallen off where the beams were suspending by steel bars. Finally at ultimate failure load, the longitudinal bars in one area are completely fractured. In addition to these cracks, plastic hinges formed also near to the center column.

Fig. 8(b) shows the deformed shape and crack pattern for Frame F2. The frame behavior was elastic in the early stage of loading. As the frame responded in an elastic manner till displacement of 70 mm. After that, cracks start to form due to the increasing of the loading. The cracks position was at the start and end of the beams above removal column. Cracks did not form at any other position due to the continuity of steel bars. As load increased, the cracks spread and widened and plastic hinges start to form at the same locations of cracks starting. Next, the crushing of concrete happened and concrete started the fallen off process. At this stage beams resist by tension force in bars as mentioned before. Finally at ultimate failure load, the longitudinal bars in one area are completely fractured. In addition to these cracks, plastic hinges formed also near to the center column.



a) Frame F1

b) Frame F2

Fig. 8 Crack Pattern for F1 and F2 Frames

5. Conclusions

From the results obtained from the numerical results for reinforcement parameters, the following conclusions can be drawn:

1. Generally, after column removal event, the reinforced concrete frame is subjected to partial failure and passed through different stages till the total failure. The first one is the elastic stage, the frame transfer the load of the removed by shear forces due to flexural beam capacity. Then the frame transfers to another stage after rebars steel yielding, called elasto-plastic stage. The third stage called plastic hinge formation stage. At this stage frame loss the flexural capacity and cracks are spread in the compression section zone. The last stage is the catenary stage, at this stage the removed column transfer is due to tension force formed in beams.

2. Comparing between seismic details and non-seismic details of R.C frames, the seismic details provide a good performance more than non-seismic details in all resistance stages. The seismic detail increases the range of elastic stage by 42%. The elasto-plastic and plastic hinge formation stage ranges is increased by almost 12%. The catenary stage is increased by 36%. It concluded that, the seismic detail of R.C frame affects highly on the flexural and catenary stages.

3. At different response levels, seismic detail of R.C frames has high capacity more than non- seismic details in all stages of load transfer. For the elastic stage, the capacity of seismic details is increased by 47%. The elasto-plastic capacity and catenary capacity are increased by almost 40%. The capacity at plastic hinge formation stage is increased by 7%. Generally, seismic detail has a high effect on the elastic, elasto-plastic and catenary capacities.

4. The crack pattern position formed in R.C due to progressive collapse is depended on the continuity of steel rebars. For seismic details cracks formed at the start and end of the beams bridging over the fallen column due to continuity of rebars. In non-seismic detail, the cracks formed at the position of top rebars curtailment, start and end of beams.

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