Acta Technica Napocensis: Civil Engineering & Architecture
Volume 59, No. 2, (2016)
Journal homepage: http://constructii.utcluj.ro/ActaCivilEng
ISSN 1221-5848

# Numerical investigation on the progressive collapse behavior of the RC frames dependent on the damage cases

Adrian G. Marchis[*1], Adrian M. Ioani[2]

[1,2] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

(Received 15 July 2015; Accepted 17 June 2016)

## Abstract

*In this paper the ultimate load bearing capacity to progressive collapse of RC frames subjected to different damage cases is investigated. A reinforced concrete planar frame, previously tested during an experimental program performed by Yi et al. (2008), is considered herein for validation. The 3D finite element model is created using the Midas FEA software. A nonlinear static "push-down" analysis is conducted considering three distinct damage cases: the removal of a first-storey column located at the middle, near the middle, respectively at the corner of the frame. The response of the numerical model subjected to middle column failure is similar with the response of the frame during the experimental test. For each damage case, the ultimate load associated to the structural collapse is compared with the load associated to the three hinge failure mechanism (yield load). Thus, the contribution of two supplementary resisting mechanisms (the compressive arch action and the catenary action) to better resist progressive collapse of the RC frames is investigated with respect to three damage cases.*

## Rezumat

*În această lucrare se studiază capacitatea portantă ultimă la colaps progresiv a structurilor în cadre din beton armat supuse diferitelor cazuri de avarie. Pentru validare, se consideră un cadru plan din beton armat testat anterior de Yi și alții (2008) în cadrul unui program experimental. Modelul 3D de element finit este creat în programul de calcul Midas FEA. Se rulează câte o analiză statică neliniară pentru modelul numeric supus la trei cazuri de avarie structurală, astfel: îndepărtarea unui stâlp de la primul nivel amplasat la mijlocul deschiderii, în apropiere de mijlocul deschiderii, respectiv la colțul cadrului. Răspunsul modelului numeric supus cedării stâlpului amplasat la mijlocul deschiderii este similar cu răspunsul structurii reale din cadrul programului experimental. Încărcarea ultimă asociată colapsului structural este comparată cu încărcarea asociată formării mecanismului de cedare prin trei articulații, pentru fiecare caz de avarie. Astfel, se evidențiază contribuția celor două mecanisme suplimentare de rezistență (efectul de arc comprimat și efectul de lănțișor) asupra capacității portante ultime la colaps progresiv a structurilor în cadre din beton armat în funcție de cazul de avarie considerat.*

[*] Corresponding author: Tel./ Fax.:0264 401 345
E-mail address: Adrian.Marchis@mecon.utcluj.ro

**Keywords:** progressive collapse, RC frame, nonlinear static analysis, compressive arch action, catenary action

# 1. Introduction

Progressive collapse is defined as a situation where a local failure of primary structural components leads to the collapse of adjoining members which, in turn, leads to additional collapse, the total damage being disproportionate with the original cause [1]. The engineering community had been engage in preventing progressive collapse of building after the structural failure of the Ronan Point Apartment Building from London, England (1968). The interest in this field has been intensified after the collapse of the Murrah Federal Building (Oklahoma, U.S.A, 1995) and after the total failure of the World Trade Center (New York, U.S.A, 2001). Therefore, in order to minimize the human losses, it is essential to design the buildings (especially those classified as of major importance) to resist progressive collapse when subjected to abnormal loads (terrorist attacks, gas explosion impact by vehicle, etc). In this category are included all the loads not considered in the initial phase of the structural design.

However, from an economical point of view, since abnormal loads are extremely rare events that can occur during the lifetime of a building, it is more appropriate to minimize the risk for progressive collapse in buildings than to design them to resist for all possible threats: gas explosion, terrorist attack, impact by vehicle, etc. In this context, two major guidelines [2, 3] for progressive collapse analysis of the new and existing buildings, released by the U.S General Service Administration (GSA) and the U.S Department of Defense (DoD) are available. The Alternative Path Method has been selected by both agencies as the basic approach for providing resistance to progressive collapse. A structure should be capable of developing alternative load paths over a vertical support suddenly removed as a result of abnormal loading. This means that a structure should be designed with an adequate level of continuity, ductility and redundancy, characteristics which are found in the seismic design codes, too: Eurocode 8 [4], ASCE 41-06 [5] and P100/1-2013 [6].

Recent numerical studies [7, 8, 9, 10] have indicated the beneficial influence of the seismic design to better resist progressive collapse when subjected to column removal. These results were validated by experimental studies [11, 12, 13, 14] carried out on RC beam column sub-assemblages; it was shown that the specimen with seismic detailing could develop supplementary resisting mechanisms – the compressive arch action and the catenary action - before failure, which increases the ultimate load bearing capacity to progressive collapse of the tested specimens. Yi et al. (2008) [15] had shown that a four-bays three-stories RC planar frame is capable to resist for a peak load of $F_u$=105kN, which is 35% higher than the yield load ($F_y$) associated to the development of the three hinge failure mechanism. Also, Sadek et al (2011) [12] had shown that a seismically designed beam-column subassemblage is capable of resisting for a peak load $F_u$=1092kN which is 50% higher than $F_y$. In this context, the ultimate load bearing capacity to progressive collapse of RC planar frames dependent on the damage cases is investigated herein. The RC planar frame experimentally tested by Yi et al. (2008) [15] is modeled using the Midas FEA software [16]. A nonlinear static “push-down” analysis is performed for the numerical model considering three distinct damage cases: the removal of a first-storey column located at the middle, near the middle, respectively at the corner of the frame. For each case, the peak load attained before failure is compared with the yield load when the three hinge failure mechanism is activated. Thus, the contribution of the supplementary resisting mechanisms – the compressive arch action and the catenary action – to better resist progressive collapse is discussed.

A secondary objective of this study is to test the ability of the Midas FEA (available for the researchers from the Faculty of Civil Engineering from Cluj-Napoca), in order to capture the response of the RC frames in the large displacement range during the progressive collapse analysis. The Extreme loading for Structures software was successfully tested in a previous paper [17].

## 2. Numerical model

### *2.1 Details of the experiment (Yi et al., 2008)*

A four-bay and three-storey one-third scale model representing a segment of a larger planar RC frame was experimentally tested by Yi et al. (2008) [15]. The frame consists of four 2667mm bays and three stories. The storey height was 1100mm except for the first one which has 1567mm height. Dimensions of the structural components as well as the longitudinal and transverse reinforcement are provided in Table 1.

Table 1. Design details of the planar frame (Yi et al., 2008) [15].

| Element | Dimensions w•h [mm] | Bottom rebars | Top rebars | Shear rebars |
|---|---|---|---|---|
| Beam | 100x200 | 2Φ12mm | 2Φ12mm | Φ6/150mm |
| Column | 200x200 | 4Φ12mm | | |

The material parameters provided by the experiment are given in the following. The concrete compressive strength f'c measured on 15x15x15cm cube was 25MPa. The yield and ultimate strength for steel was $f_y$=416MPa respectively $f_{yu}$=526MPa. The ultimate strain for steel $\varepsilon_{su}$ of 23% was measured with strain gauges having 120mm length. For strain gauge length of 60mm $\varepsilon_{su}$ was measured as 27.5%. The gradual failure of the first-storey middle column $C_3$ was performed in a displacement controlled manner, as follows. First, a vertical load F=109kN (refer to Fig. 1) was applied on the top of the middle column by a servo-hydraulic actuator in order to simulate the gravity loads of the upper stories not considered in the experiment. Then, while maintaining the load at the maximum value, the progressive failure of the first-storey middle column was simulated by lowering the mechanical jacks located on the bottom of the middle column $C_3$.

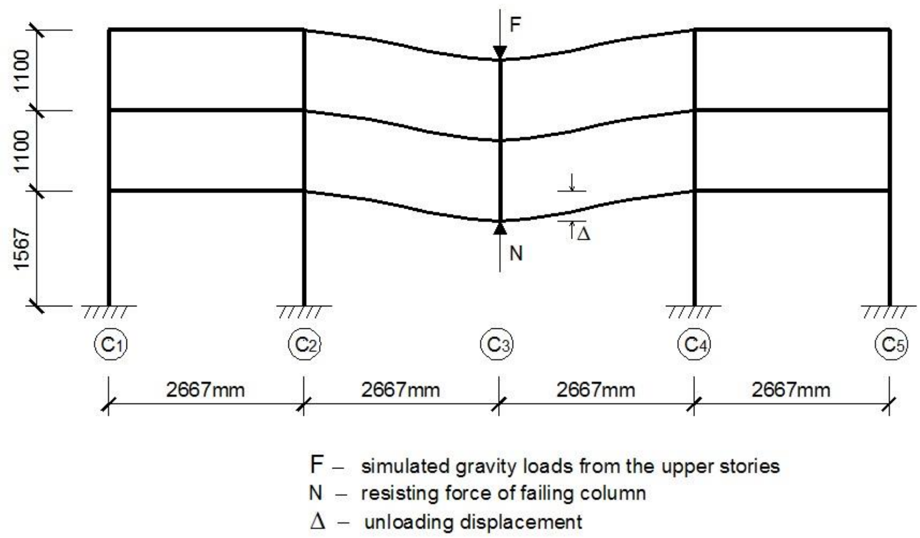
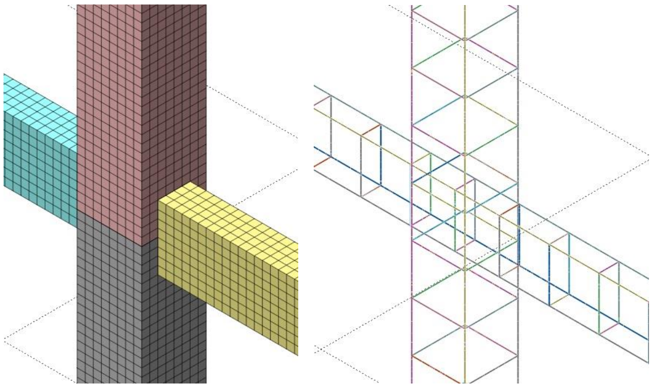



Figure 1. Test setup for the RC planar frame (Yi et al, 2008) [15].

### 2.2 Validation of the FEM model

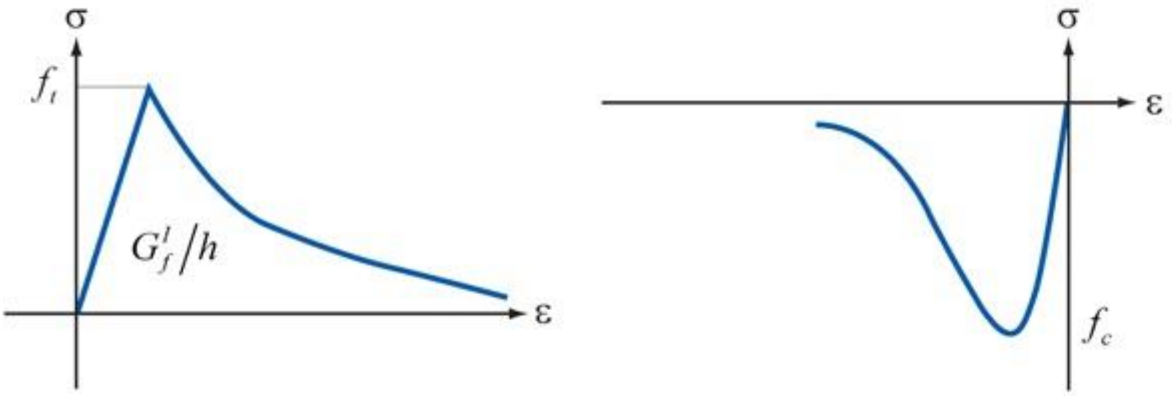
A computer program Midas FEA [16] was used to develop the 3-D finite element model for the RC planar frame under investigation. Solid elements were considered for the FEM model; the concrete and reinforcement bars are modeled separately, as illustrated in Fig. 2. A mesh size of 2.5cm was considered for each structural component.



(a) (b)

Figure 2. Modeling of the frame with Midas FEA: (a) concrete, (b) reinforcement.

Nonlinear behavior is adopted for the constitutive models (refer to Fig. 3). A *Total Strain Crack* model was considered for the behavior of concrete which assumes that the generated cracks are scattered over a wide surface [16]. The Hordijk model was adopted for the behavior of concrete in tension as illustrated in Fig. 3(a). The tensile strength was considered as $f_t$ =2.2MPa. The softening function is governed by the ratio $G_f/h$, where $G_f$ is the fracture energy and is related to the compressive strength ($f_{ck}$) and the maximum aggregate size ($D_{max}$).




(a) (b)

Figure 3. Constitutive models for concrete: (a) in tension – Hordijk model, (b) in compression – Thorenfeldt model [16].

For the numerical model under investigation $G_f$=0.069kN/m, considering $f_{ck}$=25MPa and $D_{max}$=16mm; h represents the total crack band width which is taken as the mesh size of the finite element (h=0.025m). For concrete behavior in compression, a Thorenfeldt model was adopted (refer to Fig.3b), where $f_{ck}$ =25MPa, a value provided by the experiment. Since the Young's modulus for concrete is not given by the experimental test, it is calculated using the relation provided by ACI

318-11 [18]:

$$E_c = 4700\sqrt{f_{ck}} \approx 24GPa \tag{1}$$

where the concrete compressive strength $f_{ck}$=25MPa at the time of testing [15]. In order to model the behavior of the reinforcing steel both in tension and compression, a Von Mises model was considered. As illustrated in Fig. 4, a bilinear stress-strain relationship (continuous line) was adopted for the longitudinal reinforcement. The yield strength for steel $f_y$=416MPa was considered. In the experiment, the ultimate strength for steel was $f_{yu}$=526MPa which corresponds to an ultimate strain of $\varepsilon_{su}$=23%.After calibration, a lower value for $f_{yu}$=498MPa and $\varepsilon_{su}$=17% was considered in the numerical model, in order to capture the failure of the numerical model (fracture of the longitudinal bars) at the same step of loading as in the experiment. The Young's modulus for steel ($E_s$) was considered as 200GPa. For the transverse reinforcement, the yield strength for steel $f_y$=370MPa was adopted as in the experiment.

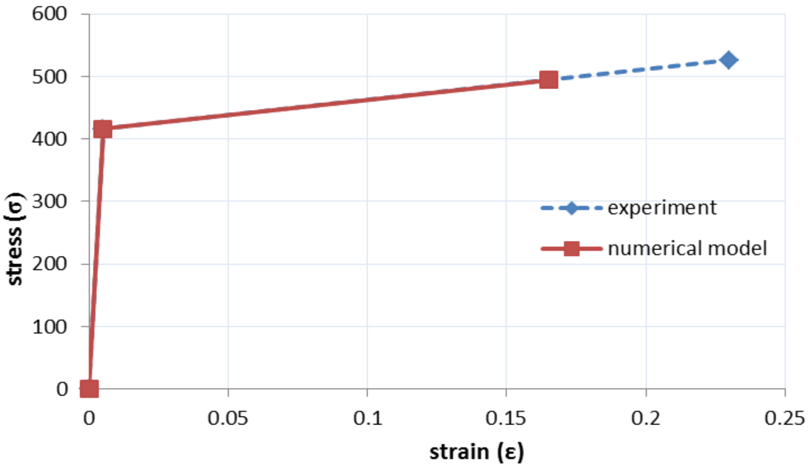



Figure 4. Constitutive model for the longitudinal reinforcement.

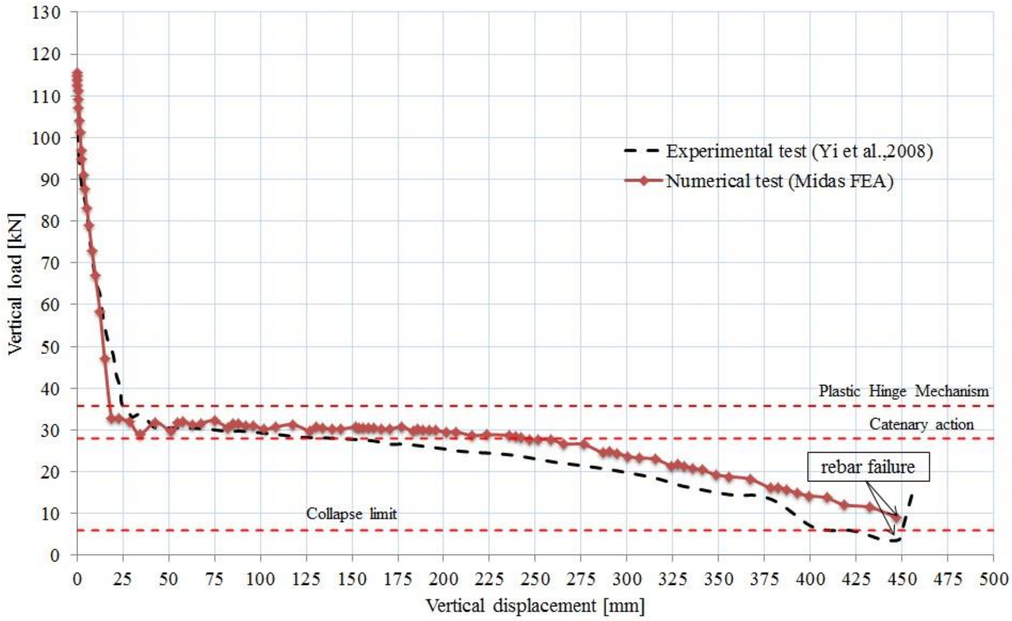



Figure 5. Middle column load versus unloading displacement of the column- removed point: the experimental test (Yi et al., 2008 [15]) *vs.* the numerical test.

The numerical test was performed in a displacement controlled manner by applying a target displacement $\Delta$=50cm at the bottom of the middle column ($C_3$). 400 steps were set up in the nonlinear static analysis. Fig. 5 displays the load-displacement curve obtain for the numerical model (continuous line) in comparison with the one provided by Yi et al. (2008) [15] in the experiment (dashed line). The response of the FEM model subjected to column failure is in very good agreement with the response of the tested structure during the experiment. When the vertical displacement of the middle column measured $\Delta$=450mm, the longitudinal rebars from the critical beams of the first storey ($C_3$-$C_4$) ruptured, indicating the collapse of the planar frame. In the numerical test, the analysis stops when the ultimate strain for steel $\varepsilon_{su}$=0.17 is attained in the longitudinal rebars from the beams of the third storey, indicating the failure of the FEM model.

## 3. Progressive collapse resistance with respect to different damage cases

The aim of this study is to investigate the ultimate load bearing capacity to progressive collapse of RC planar frames with respect to different damage cases. After the mechanism of three hinge type is formed, two supplementary resisting mechanisms may develop which improves the ultimate load bearing capacity to progressive collapse of the RC frame when subjected to column removal.
The numerical model considered herein, was validated in the previous section with the experimental failure test performed by Yi et al. (2008) [15]. In addition to the damage case considered in the experiment (column loss $C_3$), two supplementary damage cases were accounted for: the removal of the first-storey column $C_4$ (near the middle) and $C_5$ (the corner column).

### *3.1 Column loss $C_3$*

In order to establish the ultimate load bearing capacity to progressive collapse, the numerical model was subjected to three distinct damage cases: the removal of the first-storey columns $C_3$, $C_4$ and $C_5$, each in turn. A nonlinear static "push-down" analysis was performed by considering a target displacement $\Delta$=50cm of the column-removed point. The results of the FEA model subjected to the three damage cases are provided in Fig. 6. The response of the numerical model subjected to the removal of the first-storey column $C_3$ is very similar (Fig. 5) with the experimental test performed by Yi et al. (2008) [15]. The elastic-plastic state between point A and B is characterized by the concrete cracks in tension from the critical beams ($C_2$-$C_3$ and $C_3$-$C_4$) associated to the middle column $C_3$.

When the vertical load reaches point B from Fig. 6, a failure mechanism of three hinge type is formed. The limit value for the load $P_{limit}$ is determined from the plastic analysis. This means that plastic hinges are introduced at both beams ends from the structural bays associated to the removed column $C_3$ where the cross sections enter the plastic stage; the moment resistance reaches the limit value $M_{pl}$. When the plastic hinges appear at all beams ends associated to the column $C_3$, a failure mechanism of three hinge type is formed as illustrated in Fig. 7. The limit value for the load $P_{limit}$ is given by the following equation, a value determined by Yi et al. (2008) [15] as well.

$$P_{limit} = 12 \text{ x } M_{pl} / L = 73.26\text{kN} \qquad (2)$$

Where $M_{pl}$ is the plastic moment of the cross section ($M_{pl}$=15.06kNm) and L is the clear bay dimension (L=2.467m).

The section B-C is the plastic state characterized by large deformation with small increasing of the load. Two resisting mechanism are developed before the collapse of the frame. First, the compressive arch action, also called in the literature as the Vierendel action is identified in Fig.6 until point B'. The second resisting mechanism – the catenary action – is activated when the undamaged columns $C_2$ and $C_4$ start to move toward to the removed column $C_3$. Fig. 8 shows the vertical displacement of the column removed point ($C_3$) versus the horizontal displacement of the undamaged column $C_2$ for each storey. The horizontal displacement of the column $C_2$ is changing to an opposite direction when the vertical displacement reaches $\Delta$=150mm ($3/4 \cdot h_{beam}$) indicating the incipient phase of the catenary action. These results are similar with the response of the tested structure during the experimental program performed by Yi et al. (2008) [15].

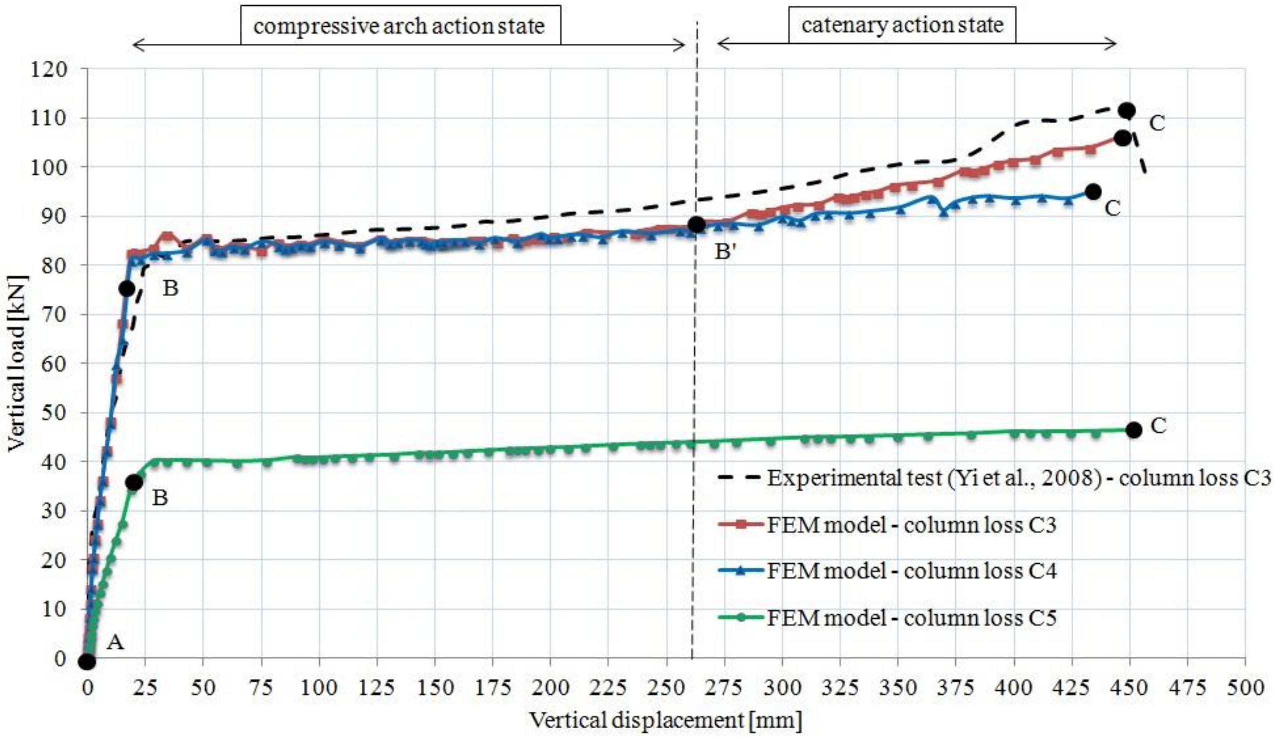



Figure 6. Load-displacement curves of the FEA model subjected to the column removal.

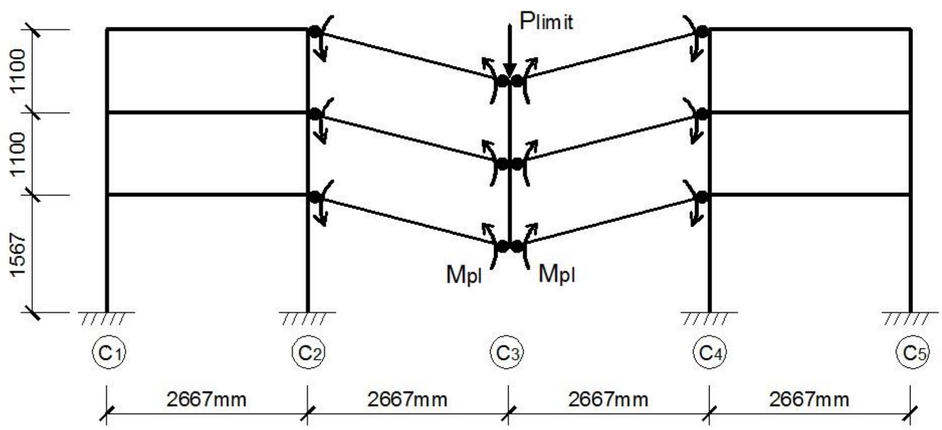



Figure 7. The limit value of the load associated to the three hinge failure mechanism considering the failure of the interior column.

Point C on the load-displacement curve (Fig. 6) is associated to the failure of the RC frame. At this step of loading, the bottom longitudinal rebars from the third-storey beam associated to the removed column fail; this means that the ultimate plastic strain $\varepsilon_{su}$=17% considered in the numerical simulation is attained. Also, large cracks openings in concrete are identified in the critical zones displayed in Fig.9(b) which are very similar with the response of the RC frame during the experimental test performed by Yi et al. (2008) [15] (Fig.9(a)).

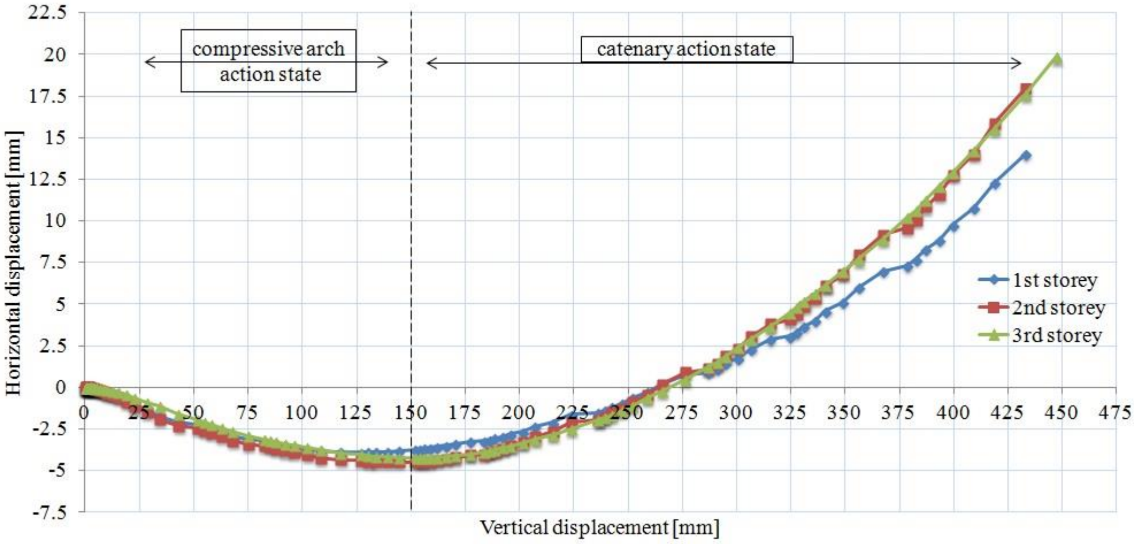
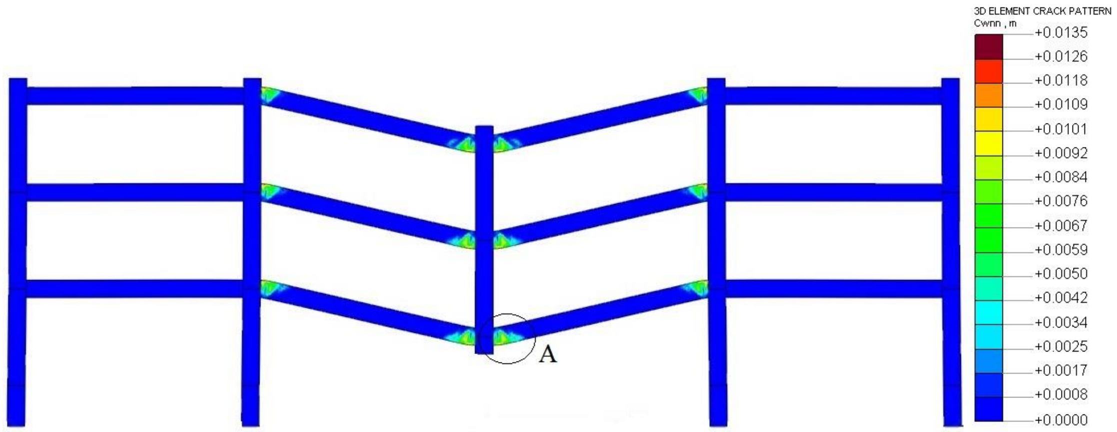
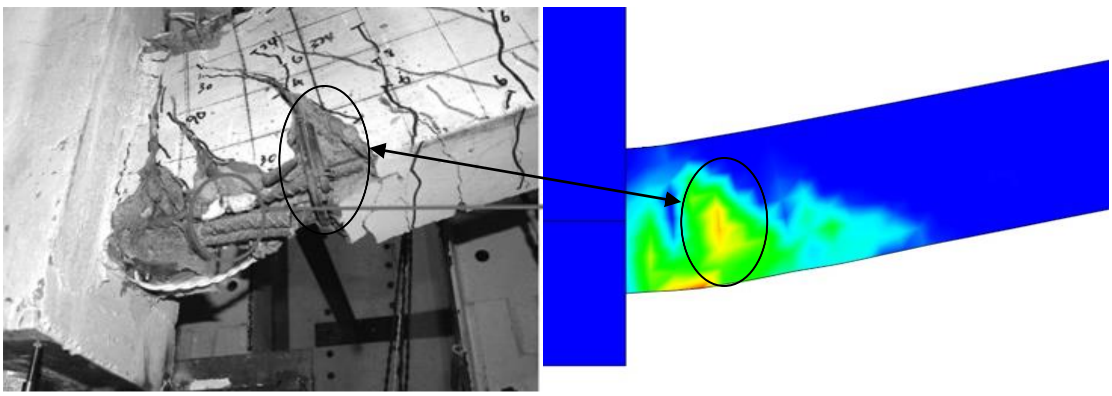



Figure 8. Vertical displacement of the column removed point – $C_3$ *vs.* horizontal displacement of the undamaged column $C_2$.




Detail A



(a) (b)

Figure 9. Cracks openings of the RC frame subjected to the column loss $C_3$ before failure: (a) the experimental test [15]; (b) the FEM model.

Therefore, due to the developing of the two supplementary resisting mechanisms – the compressive arch action and the catenary action, the FEM model subjected to the column loss $C_3$, is capable of resisting for an ultimate load of $F_u$=106.5kN which is 45% higher than the load associated to the formation of the three hinge failure mechanism – $F_y$ (refer to Fig. 10).

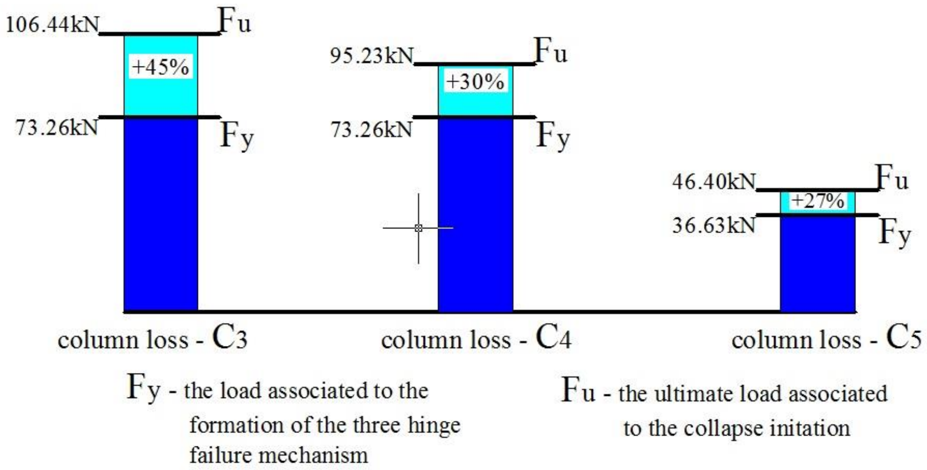
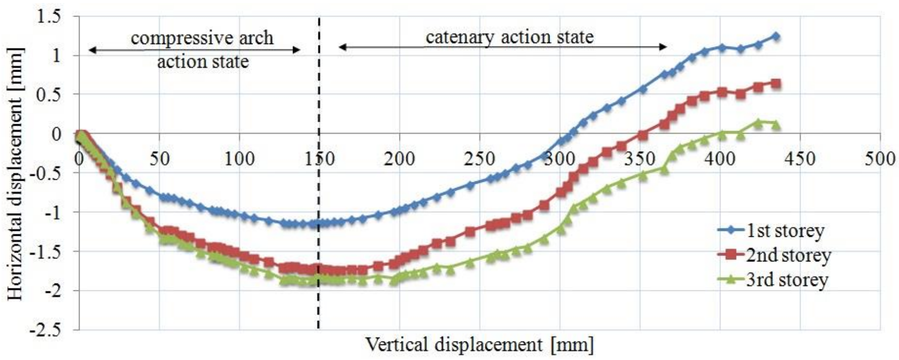


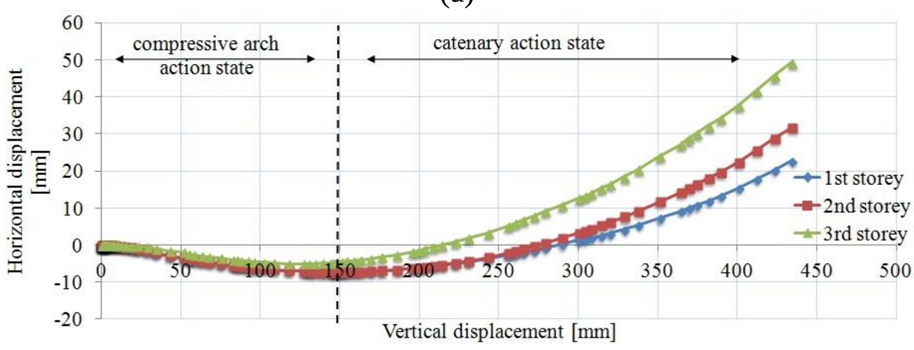

Figure 10. The progressive collapse resistance of the planar frame dependent on the damage case.

### 3.2 Column loss $C_4$

When the FEM model is subjected to the removal of the column $C_4$, its response is similar with the previous case (column loss $C_3$). When the vertical load reaches point B from Fig. 6, a failure mechanism of three hinge type is formed.




(a)




(b)

Figure 11. Vertical displacement of the column removed point – $C_4$ *vs.* horizontal displacement of the undamaged columns: (a) $C_3$, (b) $C_5$.

The limit value for the load associated to this mechanism is determined with Eq. (2): $P_{limit}$=73.26kN considering the failure of an interior column. As in the previous case, two resisting mechanism are developed before the collapse of the frame. The compressive arch action is identified in Fig. 6 until point B'. The catenary action activates when the undamaged columns $C_3$ and $C_5$ start to move toward to the removed column $C_4$. Fig. 11 shows the vertical displacement of the column removed point ($C_4$) versus the horizontal displacement of the undamaged columns $C_3$ (Fig. 11(a)) and $C_5$ (Fig. 11(b)) for each storey. As in the previous case, the horizontal displacement of the column $C_3/C_5$ is changing to an opposite direction when the vertical displacement reaches $\Delta$=150mm ($3/4 \cdot h_{beam}$) indicating the incipient phase of the catenary action. The differences between the horizontal displacements of the undamaged columns $C_3$ and $C_5$ (refer to Fig. 11) is due to the fact that the column $C_3$ is horizontally restrained by the structural bays $C_1$-$C_3$, unlike the column $C_5$ which is located at the corner of the frame. From this step ($\Delta$>150mm), the loads associated to the removed column $C_4$ are transmitted by the critical beams (which exhibit only tension behavior) to the adjacent (undamaged) columns $C_3$ and $C_5$ (Fig. 12). Also, large cracks openings are identified at both ends of the critical beams associated to the removed column $C_4$.

The analysis stops when the vertical displacement reaches $\Delta$=435mm (point C on Fig. 6). As in the previous case, the bottom longitudinal rebars from the third-storey beam associated to the removed column fail and thus the collapse limit state is attained. The FEM model subjected to the column loss $C_4$ is capable of resisting for an ultimate load of $F_u$=95.23kN, which is 30% higher than the load associated to the three hinge failure mechanism – $F_y$ (refer to Fig. 10).

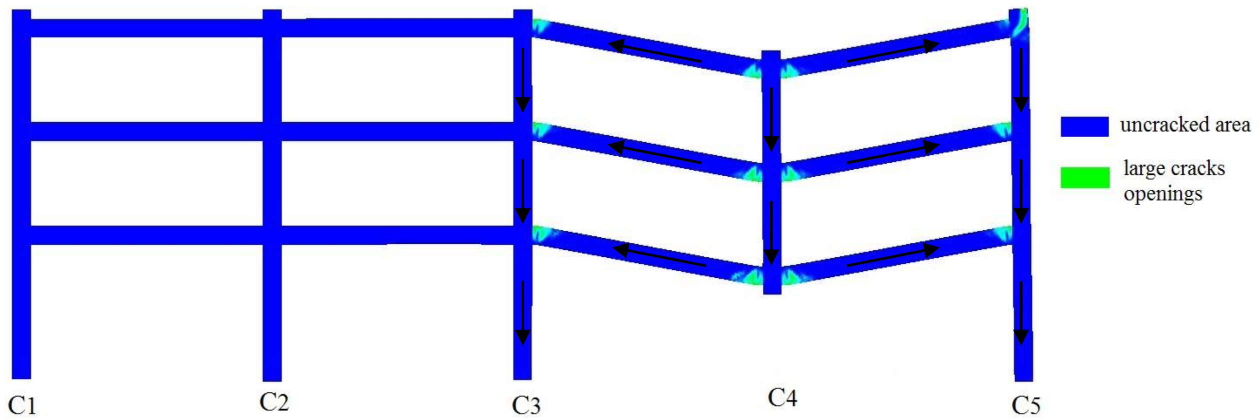



Figure 12. Loads redistribution of the FEM model subjected to the column loss $C_4$ in the large displacement range: $\Delta$>150mm.

### *3.3 Column loss $C_5$*

The response of the FEM model subjected to the corner column removal ($C_5$) is similar with the previous cases (Fig. 6). When the vertical load reaches point B from Fig. 6, a failure mechanism of three hinge type is formed as illustrated in Fig. 13. The limit value for the load ($P_{limit}$) associated to this mechanism is given by the following equation:

$$P_{limit} = 6 \times M_{pl} / L = 36.63\text{kN} \quad (3)$$

Where $M_{pl}$ is the plastic moment of the cross section ($M_{pl}$=15.06kNm) and L is the clear bay dimension (L=2.467m). The analysis stops when the vertical displacement reaches $\Delta$=450mm. The bottom longitudinal rebars from the first-storey beam near the removed column $C_5$ fail indicating the collapse initiation of the planar frame. The ultimate load attained associated to point C on the load-displacement curve from Fig. 6 is $F_u$=46.4kN which is 27% higher than $F_y$ (refer to Fig. 10).

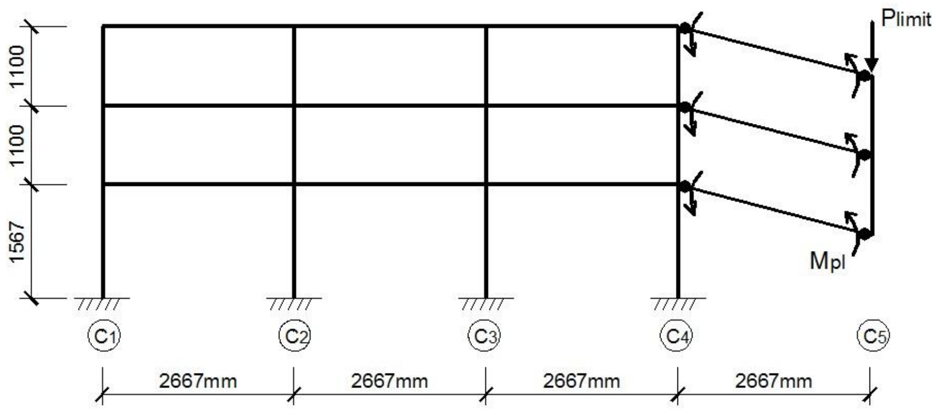



Figure 13. The limit value of the load associated to the three hinge failure mechanism considering the failure of the corner column.

## 4. Concluding remarks

The ultimate load bearing capacity to progressive collapse of RC planar frames dependent on the damage case considered was investigated in this study. A reinforced concrete planar frame modelled in Midas FEA [16] was previously experimentally tested by Yi et al. (2008) [15]. A nonlinear static "push-down" analysis was performed for the FEM model considering three distinct damage cases: the removal of a first-storey column located at the middle ($C_3$), near the middle ($C_4$), respectively at the corner of the frame ($C_5$).
Based on the results obtained herein the following conclusions can be drawn:

- The planar RC frame subjected to the damage case $C_3$ is capable of resisting for a maximum load of 106.44kN which is 12% higher than in the case $C_4$, respectively 129% higher than in the case $C_5$, as indicated in the numerical investigation.
- Irrespective of the damage case considered, the RC frame can support a higher load than the load associated to the development of the failure mechanism of three hinge type before collapse. This is due to the activation of the supplementary resisting mechanisms: the compressive arch action and the catenary action.
- The additional load resistance to progressive collapse due to the initiation of the supplementary resisting mechanisms is 27% for the damage case $C_5$, 30% for the case $C_4$ and 45% for the case $C_3$, with respect to the load associated to the initiation of the three hinge failure mechanism.

Consequently, the suddenly removal of the first-storey column located at the middle (case $C_3$) when subjected to an abnormal loading assumes a lower risk for progressive collapse for the RC frame than for the other damage cases ($C_4$ and $C_5$). Therefore, a higher supplementary load resistance is obtained with respect to the limit value of the load associated to the three hinge failure mechanism. When the planar frame is subjected to the corner column removal (damage case $C_5$) a lower supplementary load resistance is obtained unlike the other damage cases considered herein.

## 6. References

[1] General Service Administration (GSA), *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*, GSA, Washington, U.S.A, 2003.
[2] General Service Administration (GSA), *Alternate Path Analysis and Design Guidelines for Progressive Collapse Resistance*, GSA, Washington, U.S.A, 2013.
[3] Department of Defense (DoD), *Design of Building to Resist Progressive Collapse*, Unified Facility Criteria, UFC-4-023-03, Washington, U.S.A, 2013.

[4] SR EN 1998-1:2004/NA:2008 (Eurocode 8), *Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings*, ASRO, Bucharest, Romania, 2008 (in Romanian).

[5] ASCE 41-06, *Seismic Rehabilitation of Existing Buildings,* American Society of Civil Engineers, ISBN 970-0-7844-0884-1, Reston, Virginia, USA, 2006.

[6] P100-1/2013, *Seismic design code – Part I: design Rules for Buildings,* MTCT, Bucharest, Romania, 2013 (in Romanian).

[7] Baldridge S., Humay F., *Preventing Progressive in Concrete Buildings*, Concrete International, **Vol. 25**, 73-79 (2005).

[8] Ioani A.M., Cucu H.L., Mircea C., *Seismic design vs. progressive collapse: a reinforced concrete framed structure case study*, Proceedings of ISEC-4, Melbourne, Australia, 2007.

[9] Tsai M.H., Lin B.H., *Investigation of Progressive Collapse Resistance and Inelastic Response for an Earthquake-resistant RC Building Subjected to Column Failure*, Engineering Structures, **Vol. 30**, 3619-3628 (2008).

[10] Marchis A.G., Moldovan T.S., Ioani A.M., *The Influence of the Seismic Design on the Progressive Collapse Resistance of Mid-Rise RC Framed Structures,* Acta Technica Napocensis: Civil Engineering & Architecture, **Vol. 56**, No. 2, pp. 222-234, 2013.

[11] Choi H., Kim, J., *Progressive collapse-resisting capacity of RC beam-column sub-assemblage*, Magazine of Concrete Research, **Vol. 63**, No. 4, 2011.

[12] Sadek F., Main J.A., Lew H.S., Bao Y., *Testing and Analysis of Steel and Concrete Beam-Column Assemblies under a Column Removal Scenario,* Journal of Structural Engineering, **Vol. 137**, No. 9, pp. 881-892, 2011.

[13] Yap S.L., Li B., *Experimental Investigation of Reinforced Concrete Exterior Beam-Column Subassemblages for Progressive Collapse,* ACI Structural Journal, **Vol. 108**, No. 5, pp. 542-552, 2011.

[14] Yu J., Tan K.H., *Experimental and numerical investigation on progressive collapse resistance of reinforced concrete beam column sub-assemblages,* Engineering Structures, **Vol. 55**, pp. 90-106, 2013.

[15] Yi, W.J., He, Q.F., Xiao, Y., Kunnath, S.K., *Experimental study on Progressive Collapse-resistant behavior of reinforced concrete frame structures,* ACI Structural Journal, **Vol. 105**, No. 4, pp.433-438, 2008.

[16] MIDAS FEA, *Advanced Nonlinear and Detail Analysis System: Analysis and Algorithm,* Midas Information Technology Co., Ltd.

[17] Moldovan T.S., Marchis A.G., Ioani A.M., *Progressive Collapse Analysis of an old RC structure subjected to extreme loading,* Proceedings of the International Conference People, Buildings and Environment 2014, Kromeriz, Czech Republic, 15-17 October, 2014.

[18] ACI 318-08, *Building Code Requirements for Structural Concrete*, American Concrete Institute, 2011.