Proceedings of the World Congress on Engineering 2011 Vol III
WCE 2011, July 6 - 8, 2011, London, U.K.

Aspects Concerning Progressive Collapse of a Reinforced Concrete Frame Structure with Infill Walls

M. Lupoae*,C. Baciu*, D. Constantin*, H. Puscau*

Abstract – **The present paper proposes modern approaches, nonlinear static and dynamic analysis procedures based on the Applied Element Method, to assess a progressive collapse problem of a RC frame structure with infill walls. Comparisons between the results of modeling alternatives for a 6-story building: bare frames, exterior frames with infill full walls, openings or with windows (casement and glass) for two different columns removal approaches (demolition and blast scenarios) were made.**

Index Terms **- applied element method, blast, demolition, progressive collapse.**

I. INTRODUCTION

Progressive collapse became a subject of interest for structure designers starting with the partial collapse of a tower block from Ronan Point – London (May 1968), continuing with the escalation of terrorist activities and culminating with the events of September 2001. Being considered as improbable to happen in the past, the extreme events as blast or impact are now taken into account in various scenarios, having a finite probability of occurrence.

Since then, many experts in the field of structural calculation were concerned with the description, definition, development of a classification of terms, but mostly tried to take into account this phenomenon with as many of its characteristics.

Current codes regarding design standards provide general recommendations for preventing progressive collapse based on providing redundancy, integrity, continuity, ductility and path redistribution, but beyond these recommendations there is a limitation on understanding the phenomenon itself.

Thus in the last three decades, the UK Building Regulations [1] has imposed requirements to avoid disproportionate collapse, which were formulated following the event at Ronan Point and remained unchanged until today.

Eurocode sets different technical regulations relating to those structures, which must be verified to progressive collapse [2].

Among American codes, ASCE - 7 [3] is the only standard contains detailed guidelines on the progressive collapse. Also in U.S. there are a number of rules contained in government documents that provide design direction for progressive collapse resistance of structures. Such documents were provided by General Services Administration [4], Department of Defense [5] and the Interagency Security Committee [6]. In the GSA-2003 there are a series of recommendations on possible failure scenarios in the columns of reinforced concrete structural configurations, the frame structure or structures with thick slabs, scenarios that have been taken by several researchers [7]-[12], in an attempt to quantify the effect on different types of structures.

An analysis of recent data from literature shows that most studies have been conducted on buildings with reinforced concrete frames structure. Thus, in [7] the authors propose a simplified framework for evaluating the progressive collapse of multi-storey structure, considering the instantaneous loss of a column in the design scenario.

Other papers [8]-[10] made assessments of 5-story to 12-story concrete structures, where the columns are removed suddenly in different positions, according to the methodology proposed by GSA. There was used different software and the results show the ability of frame structures, designed to withstand seismic action, to resist progressive collapse.

Paper [11] presents a critical comparative analysis of methods for analysis of progressive collapse phenomenon, starting with a linear static analysis and finishing with a nonlinear dynamic analysis, which takes into account the load produced by the explosion.

The masonry infill walls used as exterior jacket or as partitions were often ignored in design stage, since this type of elements is considered nonstructural architectural elements. On the other hand, it is well-know that the effect of infill walls influences the behavior of frame structure: masonry wall stiffens the frame, reduces the deformations, and allows dissipating energy through nonlinear response for several cycles of deformation, according to many tests performed [13]-[15], in situ measurements [16], observations recorded after medium or sever earthquake.

Manuscript received March 11, 2011

M. Lupoae – is with the Mechatronics Department of Military Technical Academy, no. 81-83, Avenue George Cosbuc, 050141 Bucharest, Romania, phone: +40742731069, fax: +40213355763, e-mail: mlupoae2003@yahoo.com.

C. Baciu – is with the Mechatronics Department of Military Technical Academy, no. 81-83, Avenue George Cosbuc, 050141 Bucharest, Romania, phone: +40745603643, e-mail: baciucatalin2001@yahoo.com

D. Constantin – is with the Mechatronics Department of Military Technical Academy, no. 81-83, Avenue George Cosbuc, 050141 Bucharest, Romania, e-mail: dconstantin77@yahoo.com.

H. Puscau is with the Mechatronics Department, Military Technical Academy, no. 81-83, Avenue George Cosbuc, 050141 Bucharest, Romania, e-mail: puscauhorea@gmail.com

ISBN: 978-988-19251-5-2
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)

There is a convenient tend in the current design activity to ignore the presence of the infill walls, simply considering that this approach offers overestimated admitted results. Though, ignoring these walls often conducts to unexpected real structural responses (different from those obtained in the conception and design stage), fact that may lead to important damage of structural elements under sever dynamic loads.

Similarly, in the progressive collapse domain of research, the analysis results could be much different from the real behavior, as the infill walls are ignored.

The study of a real structure - Hotel San Diego – was made by Sasan and Sagiroglu in 2008 [12]. Reinforced concrete 5-story building was assessed when two exterior columns were removed simultaneously and instantaneously by explosion (a corner column and the next one, on the short side of building). Measurements made in situ showed that the maximum displacement of nodes above the destroyed columns was approximately 6.4 mm, the structural system resisting smoothly to progressive collapse.

Also, the GSA Linear Static Analysis procedure is used by Tsai and Huang [17] to assess the effect of interior brick-infill partition on the structure progressive collapse.

II. Applied Element Method – Short Presentation

For modeling the structure, the Applied Element Method was used, which combines features of finite element method and discrete element method. The main advantage of this method is that it can describe the behavior of the structural system from the application of forces, opening and crack propagation, the separation of structural elements to the total collapse [18], [19].

The structure is modeled as an assembly of small elements, with special shape and determined dimensions. These types of elements do not deform, the change of their position is as a rigid medium. AEM elements are connected using the elements entire surface, through a series of connecting springs that adopt all material type and properties, Fig. 1.

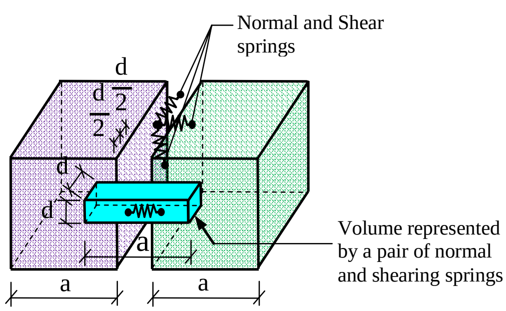


Fig. 1 – The connectivity and separation of the elements in AEM [20]

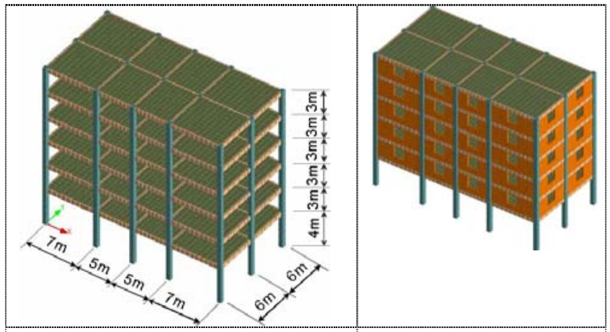
Each group of springs completely represents stresses and deformations of a certain volume and each element has six degree of freedom. The using of this modeling method allowed that the initiation and propagation of cracks and the failure of the structure can be studied using only one initial model. The location of failure is determined during the cycling process.

The computer program used for the analysis in this paper is Extreme Loading for Structures (ELS), developed based on Applied Element Method (AEM).

For modeling of concrete under compression, Maekawa compression model is used [20]. For reinforcement springs, is used the model presented by Ristic *et al.* [20]. The tangent stiffness of reinforcement is calculated based on the strain from the reinforcement spring, loading status (either loading or unloading) and the previous history of steel spring which controls the Bauschinger's effect.

III. Case Study

A six storey reinforced concrete frame as show in the Fig. 2 was used as case study. This structure has 2 spans of 6 m and 4 bays (2 bays of 7 m at the extremity and 2 bays of 5 m in the middle). The first storey height is 4 m and all the other levels are 3 m high.



a) frame structure | b) structure with infill walls

Fig. 2 - The ELS model of the RC building

Dimensions of the columns are 60x60 cm, the reinforcement is 4Ø25 mm on a side (represented a total reinforcement ratio of 1.9%). Dimensions of the perimeter beams are 25x55 cm and 30x70 cm for the central beams; the reinforcement ratio is nearly 2%. Thickness of the slab is 15 cm, with 0.5% reinforcement ratio. The elements dimensions and the amount of reinforcement correspond to the Bucharest seismic demand.

The characteristics of the constituent materials are shown in Table I.

Table I - Material characteristics

Material	f_c [N/m^2]	f_y [N/m^2]	E [N/m^2]
Concrete	$30*10^6$		$32.5*10^9$
Steel		$300*10^6$	$210*10^9$
Clay unit	$9.8*10^6$		$19.6*10^9$
Mortar	$9.8*10^6$		$1.96*10^9$

For the case of brick-infill walls, their position was established only on the facades, above the ground floor. The interior walls were supposed to be light partition, considered in analysis only as uniform load on the slabs. In order to capture the effect of the masonry behavior on structure, no walls or window frame were considered at the ground floor.

The ELS program permits to simulate the masonry elements in a real distribution, resulting that the user could model individually the bricks and the mortar being the conection between them. The material characteristics for

ISBN: 978-988-19251-5-2
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)

bricks and mortar are also shown in Table I.

The structure is subjected to a various types of loads: dead load (D) – 1500 N/m^2 on every floor, live load (L) – 2500 N/m^2 on every story except the top floor and snow load (S) – 1500 N/m^2 on the top floor. The combination for the column removing cases:

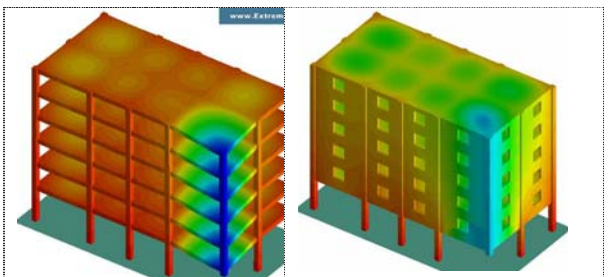
$$\text{Load} = \text{D} + 0{,}4(\text{L} + \text{S}) . \quad (1)$$

IV. Analysis types

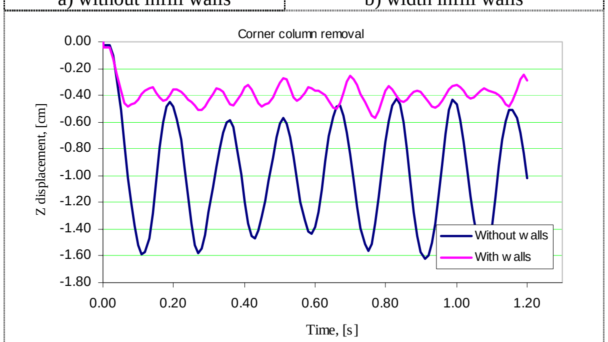
The program can perform the following types of analysis: linear and nonlinear static analysis and nonlinear dynamic analysis. Advantages of the program are represented by the existence of explicit options for simulating phenomena such collapse.

1 Demolition scenario

This analysis is currently used in the cases of blasting and progressive collapse, when the user knows which elements are to damage and cause the collapse of the structure. Under this scenario demolition, the elements to be destroyed are specified and also the time at which the removal is instantaneous. The advantage of using this method is to reduce computational time compared with the blast solution.



a) without infill walls | b) width infill walls

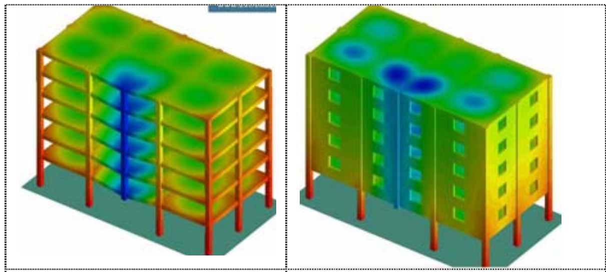


c) The variation of vertical displacements curves for joints in second floor of the two types of structures, above removed columns.

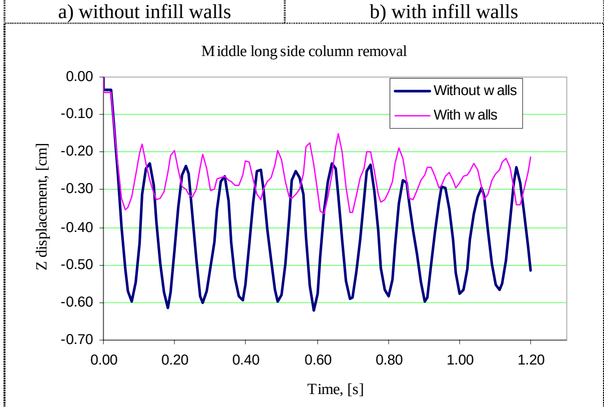
Fig. 3 - The Z displacements of structures for corner column removal

The instantaneous removal of the exterior columns of the structure was performed in accordance with GSA guidelines [4]: a column located at the corner of the building, a column located at the middle of the short side of the building and a column located at the middle of the long side of the building.

For all three cases the loss of the columns was performed instantaneous at time t = 0.025 s and this type of analysis combined with the constitutive material models for concrete, reinforced bars and masonry conduct to a non linear dynamic analysis.



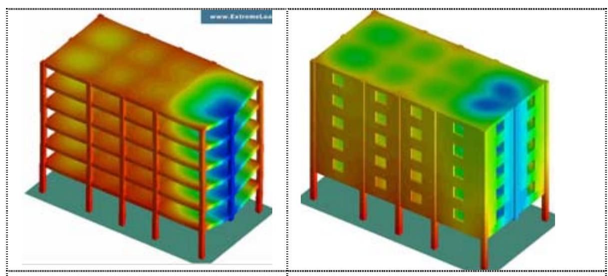
a) without infill walls | b) with infill walls



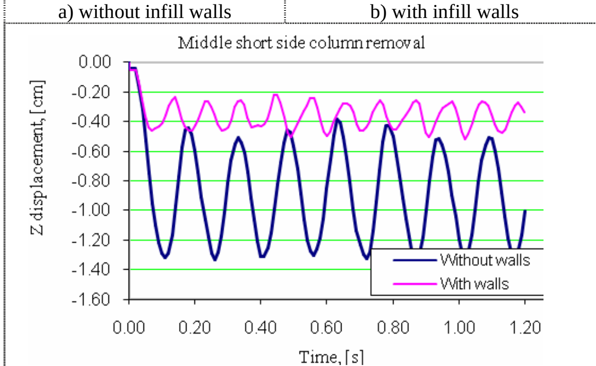
c) The variation of vertical displacements curves for joints in second floor of the two types of structures, above removed columns.

Fig. 4 - The displacements of structures for middle long side column removal

To compare the behavior of structure for different scenarios of column removal and structure configurations, the displacement of the node located directly above the removed column was chosen as a main parameter.



a) without infill walls | b) with infill walls



c) The variation of vertical displacements curves for joints in second floor of the two types of structures, above removed columns.

Fig. 5 - The displacements of structures for middle short side column removal

Figure 3 shows the deformation mode and Z axis displacements for both cases, structure with and without infill walls, for instant removal of the corner column. As can

ISBN: 978-988-19251-5-2
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)

be seen in Fig. 3c and Table II, the use of masonry walls on the perimeter of the structure will reduce the vertical displacement of the nodes above the removed column with 40 to 70%.

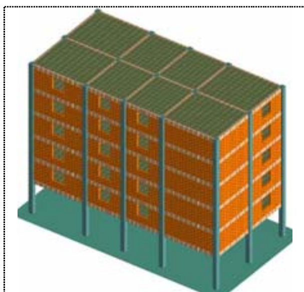
In case of removal of the middle long side column, there is obtained the smallest displacement of all three studied cases, Fig. 4 and Table II.

TABLE II - COMPARISON BETWEEN MAXIMUM Z DISPLACEMNTS

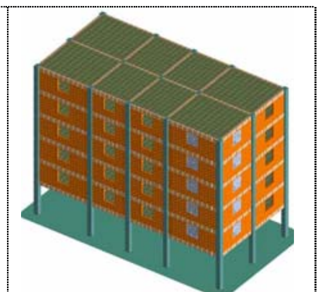
Structure configuration / Case of column removal	Maximum Z displacement, [cm]		
	Without infill walls	With infill walls	Difference, %
Corner column	1.620	0.497	69.40
Middle short side column	1.370	0.486	64.60
Middle long side column	0.622	0.361	42.00

The deformation mode and Z axis displacements of the node located on the first floor for the case of removal of the middle short side column are shown in Fig. 5.

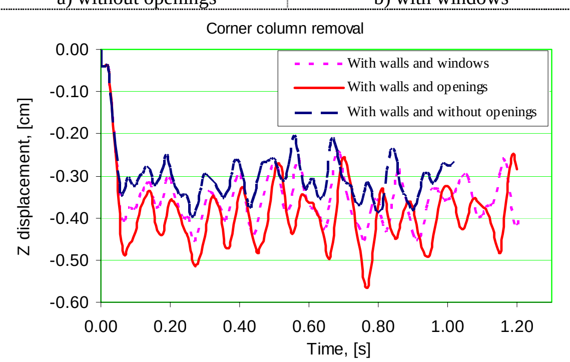
For the case of corner column removal, there were also studied the situations when the two walls adjacent to this column have not openings or have windows with glass and specific frames, Fig. 6. Data analysis demonstrated that the taking into account of windows reduces the Z axis displacement with about 8% compared to the situation when only openings are considered. The walls without openings reduce the displacement with approximately 10% compared to the situation when openings are considered.



a) without openings



b) with windows



c) The variation of vertical displacements curves for joints in second floor of the structures, above removed columns.

Fig. 6 The displacements of structures for corner column removal

2 Blast scenario

Blast effects are modeled using free-field models of blast waves. The pressure resulting from the blast wave is a function of bomb weight, distance to the bomb and time. In order to compute the pressure-time history at any point of the structure is used Friedlander equation:

$$P(t) = P_s\left(1 - \frac{t}{T_s}\right) \quad (2)$$

where: P_s is the peak static overpressure at the wave front, T_s is the duration of positive phase and t is the time measured since wave arrival.

The modeling of blast action on structure using Extreme Loading for Structures software has some advantages: (i) the calculus of the pressure resulting from the blast wave; (ii) the loading of the each element with the corresponding pressure, if there is a direct ray extending from the element face to the bombe source. At the same time it has minuses: (i) the free-field pressure wave models used by ELS does not take into consideration the reflection and refraction of pressure wave at the ground surface and surrounding elements and buildings; (ii) for small stand-off distance the model implemented does not take into account the explosion products effect. Thereby, for such a distance, the blast pressure is concentrated at the expected failed column. As a consequence, the effect of this pressure on the adjacent element is relatively small and is analogously with demolition scenario. For large stand-off distances the effect of blast pressure on the adjacent elements can be very significant.

The energy of the blast load is weighted by the scaled distance:

$$Z = \frac{R}{\sqrt[3]{w}} \quad (3)$$

where: Z is the scaled distance, R is the stand-off distance and w is the charge weight.

The using of an explosive charge of 2700 kg TNT, placed at a height of 1.5 m above the ground and at a stand-off distance of 10 m from the face of the corner column, conducts to the separation and propulsion of a part of the column, when a structure without infill walls is considered. The amount of explosive charge corresponds to a vehicle bomb attack and the stand-off distance of 10 m was chosen in accordance with minimum defended stand-off distances in order to respect the medium ISC level of protection for reinforced concrete construction [6].

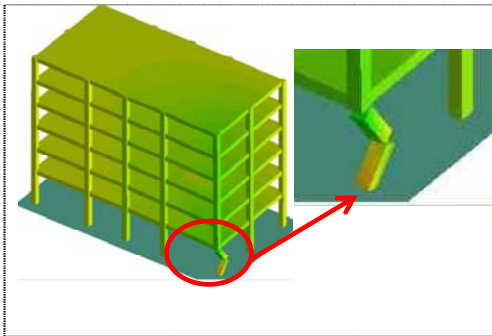
The blast wave propagation from explosive charge is performed as a concentric wave, with center in explosive charge place. As a result, almost all elements of the structure are loaded by the blast wave, each of them in a different proportion, depending on the position and the distance from the explosion source.

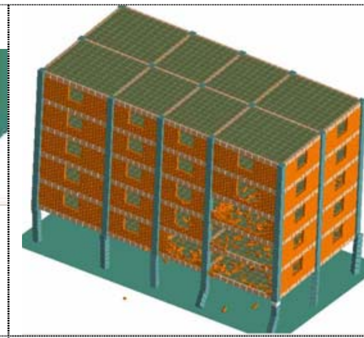
The analysis of the vertical displacement variation with time of the joint of the second floor above the damage column for the structure without infill walls, shows that in the first stage the structure is moving upward in the shock wave direction, because of the value of overpressure, and only after that the structure is moving down to the ground and then the column is damaged and thrown, Fig. 7a.

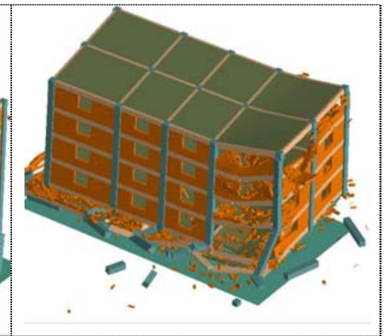
The maximum value of the vertical displacement of the joint above column destroyed by blast is 22 times greater than in case when the column is removed using demolition scenario, for structure without infill walls.

In case of infill walls, Fig. 7b, the effect of blast wave increases because of the larger surface exposed. As a result

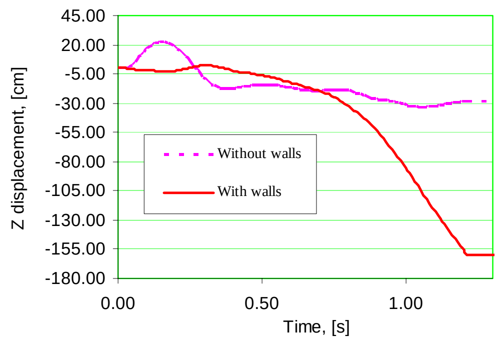
ISBN: 978-988-19251-5-2
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)


a) corner column damage – no collapse


b) columns and infill walls damage


c) collapse of the structure

Structure without infill walls | Structure with infill walls



c) The variation of vertical displacements curves for joints in second floor of the two types of structures, above corner column.

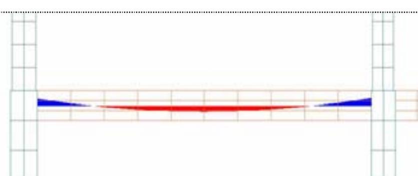
Fig. 7 - The Z displacements of the second floor above the corner column and the damage of the structures under blast action

of the shock wave on the surface of the exterior walls, the corner column and also the neighbor columns are entirely damaged (Fig. 7b), inducing the structure collapse (Fig. 7c).

V. DISCUSSION AND COMMENTS

A structure subjected to an action that has as effect a strength element removal, may have the ability to redistribute the additional efforts. The effort redistribution can be made through three mechanisms [12]:

- catenary action of slabs and beams, which allows gravity system to include also adjacent elements;
- Vierendeel mechanism of the frame situated above the removed element;
- capacity of infill walls to support extra gravity loads that result from the partial destruction.


a) before the column removal

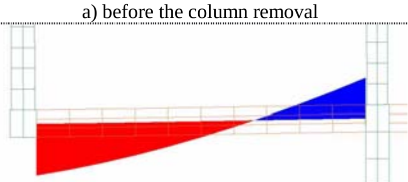

b)after the column removal

Fig. 8 – Bending moment before and after the column removal

The catenary action mechanism explains the lower value of the vertical displacement when the middle long side column is removed, compared to the situation of the middle short side column removal (Table II, Figures 4 and 5). The span obtained after the middle long side column destruction is 10 m and the surface of adjacent slabs is 2*5m*6m = 60 m^2, compared to the second situation, when the span is 12 m and the adjacent slabs surface is 2*6m*7m = 84 m^2.

Vierendeel type mechanism for redistributing efforts consists of a vertical relative motion between the ends of the beam and corresponds to a beam deformation as double curvature type. Such deformation causes shear forces in beams, that fact ensuring the vertical efforts redistribution after the column removal (Fig. 8).

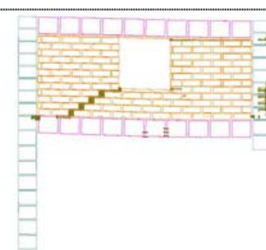


Fig. 9 – The crack propagation in the infill wall with opening , after the corner column removal

As can be seen in Fig. 8, the beam bending moment diagram before the column removal has the classic form that means top compression at the middle and bottom stretching to the ends of the beam. After the removal of the ground

ISBN: 978-988-19251-5-2
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)

floor column, the form, the direction and the values of the bending moment are changed, allowing a redistribution of the additional effort.

Analyzing the data from Table II, results that the influence of the exterior infill walls is important. After the vertical element removal, the exterior infill wall supports extra gravity loads, inducing the cracks development, Fig. 9. Note that the main part of that extra gravity loads is supported by the walls situated above, not by the lateral adjacent infill.

The situation changes when is come to blast case. For the bare frames, the overpressure affects adjacent elements (corner column) and large surface elements (slabs), resulting that in the first stage the structure is moving upward in the shock wave direction and only after that the structure is moving down to the ground and then the column is damaged and thrown, Fig. 7a. Although the vertical displacements have high values and the cracks propagation is important in all strength elements, especially in slabs and beams, the collapse does not occur, Fig. 10.

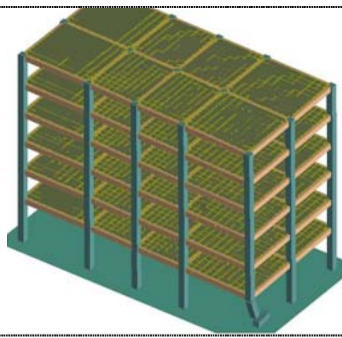


Fig. 10 – The crack propagation in the bare frames structure, in the blast case

When infill walls are taken into account, they act as a barrier against the shock wave propagation. Even if part of these walls is seriously damaged, their surface still contributes to increasing the blast action surface. As the structure is moving in the shock wave direction, the closest columns are destroyed (the corner and the adjacent columns), Fig. 7b, and then the structure collapse occurs, Fig. 7c.

ACKNOWLEDGMENT

The authors acknowledge the Academic License of the *Extreme Loading for Structures* software from Applied Science International LLC.

REFERENCES

[1] *** Office of the Deputy Prime Minister, “The building regulations 2000, *Part A, Schedule 1: A3, Disproportionate collapse*”, London (UK), 2004
[2] C. Bucur, V. Bucur, M. Lupoae, “Numerical simulation of progressive collapse”, *The 9th Session of scientific seminars - SIMEC 2010, pp. 9-16,* Bucharest (in Romanian).
[3] *** ASCE Standard 7-05, “Minimum Design Loads for Buildings and Other Structures” *(ASCE 7-05/ANSI A58) American Society of Civil Engineers*, Reston, VA.,2005
[4] *** U.S. General Service Administration (GSA 2003), “Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects”, Washington, D.C.
[5] *** Unified Facilities Criteria (UFC 2005), “Design of buildings to resist progressive collapse”, Department of Defense, Washington, D.C.
[6] *** Interagency Security Committee (ISC) (2004), “ISC Security Design Criteria for New Federal Office Buildings and Major Modernization Projects”, Washington, DC.
[7] B.A. Izzuddin, A.G. Vlassis, A.Y. Elghazouli, D.A. Nethercot, “Progressive collapse of multi-storey due to suden column loss, Part I”, Simplified assessment framework, *Engineering Structures,* vol 30, pp 1308 – 1318, 2008,
[8] A. Ioani, L. Cucu, C. Mircea, “Assessment of the potential for Progressive Collapse in RC frame”, *Ovidius University Annals Series*, vol 1, no 9, may 2007
[9] M. Lupoae, C. Bucur, C. Baciu, “The behaviour analysis of RC frame under explosion loadings, *34th IABSE Symposium*, Venice 2010
[10] S.M. Baldrige, F.K. Humay, “Preventing Progressive Collapse in Concrete Buildings”, *Concrete International*, vol 25, no 11, pp 73-79, nov 2005,
[11] P. Mendis, N. Lam, T. Ngo, “Progressive collapse analysis of RC frame subjected to blast loading”, *Australian Journal of Structural Engineering*, Vol.7, No. 1 – 2006
[12] M. Sasani, S. Sagiroglu, “Progressive Collapse Resistance of Hotel San Diego”, *Journal of Structural Engineering* – march 2008, pp 478-488
[13] J. Centeno, “In-Plane Shake Table Testing Of Gravity Load Designed Reinforced Concrete Frames With Unreinforced Masonry Infill Walls”, *The University Of British Columbia*, Vancouver - Canada, 2009
[14] C. S. Dragomir, C. Matei, D. Dobre, E. S. Georgescu - “The Behaviour of Masonry Structural Elements under Extreme Loading”, *INCERC, Bucharest* - Romania, 2009 (in Romanian)
[15] J. Dorji, “Seismic Performance of Brick Infilled RC Frame Structures in Low and Medium Rise Building in Bhutan”, *Queensland University of Technology*, Brisbane - Australia, 2009
[16] G.C. Manos, B.Yasin, J. Thaumpta, M. Triamataki, P. Skalkos, M. Demosthenous, “The Dynamic Response of 5-story of R.C. Structures in-situ at the European Test Site at Volvi”, *Aristotle University Thessaloniki*, Greece, 1997
[17] M. Tsai, T. Huang –”Effect of Interior Brick-infill Partitions on the Progressive Collapse Potential of a RC Building: Linear Static Analysis Results”, *International Journal of Applied Science*, Engineering and Technology, 6:1, 2010
[18] K. Meguro, H.S. Tagel-Din, “Applied Element Method Used for Large Displacement Structural Analysis”, *Journal of Natural Disaster Science*, vol. 24, No 1, 2002, pp 25-34
[19] H. Tagel-Din, N.A. Rahman, “The Applied Element Method: The Ultimate Analysis of Progressive Collapse”, Structure Magazine, No 4, April 2006, pp 30-33.
[20] *** ASI – Extreme Loading for Structures - Technical Manual

ISBN: 978-988-19251-5-2
ISSN: 2078-0958 (Print); ISSN: 2078-0966 (Online)