THEORETICAL AND EXPERIMENTAL RESEARCH ON PROGRESSIVE COLLAPSE OF RC FRAME BUILDINGS

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Abstract. Progressive collapse of the buildings has become an important issue to be studied in recent years due to the catastrophic nature of its effects. This subject can be approached from two different perspectives: one where an ideal collapse of the structure is aimed to be achieved and corresponds to the controlled demolition of buildings and other which treats the mitigation of the potential of progressive collapse of structures. The paper presents the results of theoretical and experimental research conducted by the authors regarding the progressive collapse of RC structures from the two perspectives above mentioned.

Key words: progressive collapse, implosion, detonation, applied element method, column removal

1. Introduction

The particular local failure of Ronan Point building, after a gas explosion (London - 1968), was called "progressive collapse" or "disproportionate collapse, regarding the initial cause" and since then this term is used to designate a phenomenon which due to the catastrophic nature of its consequences has became research topic for many experts in structural design.

Based on such description it was proposed by specialists the following definition, (ASCE, 2001): *progressive collapse - the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it.*

During their lifetime, civil engineering structures could be subjected to natural hazards (earthquakes, hurricanes, tornadoes, floods and fires) or manmade hazards (blast and impact). Because structures are not usually designed for extreme loadings, when such events occur these can lead to catastrophic failure. In last decades, events such as earthquakes (Northridge - 1994, Kobe - 1995, and recent ones of Haiti and Chile 2010) or terrorist attacks (1995 Murrah

Federal building bombing and 2001 attack on the World Trade Center) have led to structural failures and collapse resulting in related loss of life and staggering economic loss.

Nowadays many experts in the field of structural calculation are concerned with the description, definition, development of terms classification, but mostly tried to take into account this phenomenon – progressive collapse 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 has imposed requirements to avoid disproportionate collapse, which were formulated following the event at Ronan Point and remained unchanged until today. Eurocode also sets different technical regulations relating to those structures, which must be verified to progressive collapse.

Among American codes, ASCE 7-05 (ASCE, 2001) 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 (GSA, 2003), Department of Defense (DOD) (UFC, 2005) and the Interagency Security Committee (ISC, 2004).

In these recommendations there are proposed three step analysis procedures for progressive collapse: linear static, nonlinear static and nonlinear dynamic. In the case of static analysis, DOD and GSA recommend a dynamic amplification factor of 2, for both concrete and steel structures, in order to take into account the dynamic effects. This recommendation is considered to be highly conservative by some authors for the analysis of the concrete structures (Tsai and Lin, 2008) or steel structures (Izzudin *et al.*, 2008). Others consider that this factor should have greater values for steel structures, taking values up 3 when inelastic response is considered (Kaewkulchai and Williamson, 2004) or from 3 to 6 (Kim *et al.*, 2009), depending on the modeling technique for failed members.

To evaluate the vulnerability or the robustness of steel structures some authors use an energy-based nonlinear static pushdown analysis (Xu and Ellingwood, 2011; Khandelwala and El-Tawil, 2011).

Lately it is used more and more the nonlinear dynamic analysis because gives the most accurate results, but in the same time it is time consuming and requires considerable skills to implement properly. In the literature there are some papers for the nonlinear dynamic analysis of progressive collapse for concrete (Tsai *et al.*, 2008; Luccioni, 2004; Shi *et al.*, 2010; Salem *et al.*, 2011; Pekau and Cui, 2006) and steel structures (Kaewkulchai and Williamson, 2004; Kim *et al.*, 2009; Kwasniewski, 2010; Feng, 2009).

This paper is structured in two parts. The first part presents the aspects observed during the design and execution of controlled demolition works using explosives that can influence the potential of progressive collapse of a structure. The second part deals with the nonlinear dynamic analysis of a RC frame structure using two initiation scenarios of progressive collapse: one under GSA and DOD recommendations and other by considering explosion as the cause of elements failure.

For the first part there are highlighted the influence of the structure type (reinforced concrete (RC) frame versus load-bearing walls structures). Also, the presence of infill walls and the reinforcement detailing influence on the falling-down of structure on the site and desired direction is presented.

In the second part it is performed a nonlinear dynamics analysis of a RC frame structure, with and without masonry walls, to highlight their importance in reducing or increasing the potential for progressive collapse. Two scenarios were used to initiate the collapse: instantaneous removal of a column as GSA recommendations and a column removal as a result of the detonation of 2700 kg TNT charge, at a stand-off distance of 10 m. For validation the numerical simulations there was performed an experimental test. A column of an industrial warehouse, which was to be demolished, was removed by explosion. Vertical displacement of the node above the damaged column was recorded and compared with simulation results.

2. Implosion vs. progressive collapse

2.1. General considerations

There are many causes that can lead to the progressive collapse of a structure. More often than not this phenomenon is unwanted and more and more specialists are interested in study of it. The most of studies have a purpose of performing buildings less sensitive to progressive collapse. There is a special case when this phenomenon is desired and corresponds to controlled demolition using explosives or implosion. Almost all the situations when a building is demolished, this involves also progressive collapse, regardless of demolition technique chosen.

Demolition term is used to define the process of breaking of the building in pieces by destroying its connection system. Controlled demolition consists on breaking the links between structural elements in a precise sequence to conduct the structure in a state of instability. This condition will lead, under the action of its own weight, to the fall in the desired direction and on the predetermined area.
Principle of controlled demolition using explosives consists of the placement of an amount of explosive charge in /on a structural element, specifically chosen. After explosive charges are detonated in a very precise order, the elements are fragmented and the collapse of structure is initiated, the effect being demolition (total or partial) due to the loss of building stability. Instability is induced in the structure by the explosive action in structural weaknesses points carefully identified. Structure collapses under gravity and continues with the crushing of all elements due to deformations that occur during fall and impact with the ground. All these tasks have to be performed using minimum explosive charges in order to reduce unwanted effects: aerial shock waves, fragments

propulsion and seismic type waves (Lupoae, 2004).

2.2. Influence of structure type

Although the way how the collapse of structure it is initiated in controlled demolition with explosives works is not "local failure" from the definition of progressive collapse, it is instructive to follow the design process of demolition for a better understanding of how to mitigate the collapse of a building.

There will be a presentation of only the stages of implosion design and execution that are direct involved in progressive collapse of a structure.

Thus the first factor is the way in which elements are chosen to be destroyed by explosion. Regardless of the type of structure, there will be destroyed first the elements placed at lower floors of building to release the great part of potential energy and accelerate the structure toward the ground. Elements from other levels are then destroyed to help fragment the building debris or control its fall direction and velocity.

In the process of choosing the elements that will be destroyed, an important role has the building type: steel, concrete or masonry. For steel structures, demolitions carried out in our country were limited only to antennas or support structures of reservoirs. In contrast, for concrete and masonry buildings demolition by explosives included almost all types of structures.
A first discussion about the influence of the building type on the behavior of structure after the collapse initiation is related to the difference between the RC frame and load-bearing walls structures. Reinforced concrete frame structures are easier to implode than load-bearing walls or mixed structures and therefore less resistant to progressive failure. For load-bearing walls structures it should be performed preparatory works to transform walls into pseudo-columns as shown in **Fig. 1**. For this type of structure (with load-bearing walls) the proper demolition method is toppling (building falls on one side) because requires a less preparatory works.

In the acceleration space, the walls are removed mechanically and are left only pseudo-columns, **Fig. 2a**; for the rest of the structure, preparatory works (mechanically or with explosives) are performed to create sections to help the structure to move in a right direction or to fragment the building debris. The dimension of this zone shall be at least twice the element thickness to allow the creation of joints around which the structure can pivot **Fig. 2b** and **Fig. 4**.

Preparatory works will be performed so as not to endanger the safety of the structure and workers. That is why usually, the preparatory works that requires the destruction of structural elements (removal of load-bearing walls by creating pseudo – columns or destruction of support elements by performing test blast etc.) are performed just before placing the explosive in the blast holes, limiting to the minimum the time the structure is weakened.

2.3. The influence of preparatory works

One of the important requirements for the RC frame building implosion is the total or partial removal of walls in blast floors. This is necessary to provide enough space for movement and acceleration of the entire structure or just parts of it. If during the process of implosion are destroyed only support elements, without removing infill walls or other non-structural elements this may result in disruption of propagation of collapse or change the trajectory of falling.

This happened when an old building was demolished in Bucharest on a Glucose Factory street. The building had masonry structure with concrete pillars and was made by the Germans during the 40's. According to the demolition design, there were destroyed by the explosion only the concrete pillars; the masonry walls, which in this case were bearing walls, were not removed. As a result of explosive charges detonation, the structure slight tilted and remained supported on the walls. Further intervention, by placing additional explosive into walls caused the collapse of the structure as initial planning. It results here the importance of masonry infill walls on reducing vertical displacement of the structure when a support element was destroyed. This influence will be presented in detail in the second part of the paper.

2.4.The influence of reinforcement bars

Another aspect influencing the collapse of the structure in building demolition works is the reinforcement detailing, especially transverse reinforcement. When tightly stirrups are used, they must be exposed and cut in order to reduce the ability of partially destruction elements to keep their load carrying capacity. Depending on the element type (column, beam, plate), the contribution of longitudinal or transversal reinforcement to the load redistribution can be reduced by removing the concrete cover and cutting the reinforcement, **Fig. 3**, or in some cases just by removing concrete on a certain length, **Fig. 4**. It results that strong confinement provided by stirrups will lead to a greater capacity of RC structure to resist progressive collapse.

It can be seen from **Fig. 3** that a tight transversal reinforcement will increase the resistance of columns to the blast action and consequently will decrease the potential of progressive collapse.

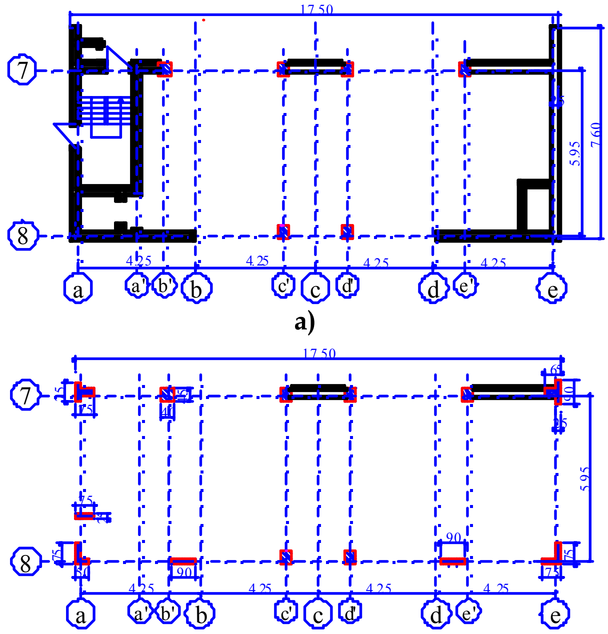
On the other hand, the cutting of the reinforcement bars must be done with care to permit to different parts of the structure to remain tied together and does not interrupt the propagation of collapse.

2.5. Numerical simulation of implosion

The studies concerning the response of structures subjected to earthquakes, blast-effects, unexpected impact forces and fire, that are known as extreme loading conditions requires the utilization of computer programs based on finite element or finite difference method. The simulation of the buildings implosion or progressive collapse can be performed by both methods, but the finite difference method is more efficient.

The Applied Element Method (AEM), which combines features from finite element and discrete element methods allow the study of the behavior of the structure under such extreme loadings. The main advantage of this method is that it can track the structural collapse behavior passing through all stages of the application of loads, elastic stage, crack initiation and propagation in tension-weak materials, reinforcement yielding, element separation, element collision (contact), and collision with the ground and with adjacent structures, (Meguro and Tagel-Din, 2002; Tagel-Din and Rahman, 2006).

For exemplification, there will be presented two cases of demolition using explosives: first case corresponds to toppling (building falls on one side) and second to an implosion. For the first case the results for simulation will be compared with the real demolition.



a)

a)

Fig. 5. Floor plan of the structure: a) initial plan; b) plan after transforming bearing-walls in pseudo-columns

The structure, **Fig. 5** was a reinforced concrete building with load-bearing walls and columns. The building had a rectangular shape with 17.50 m and 7.60 m plan dimensions and height of 33.40 m. It had a bay of 5.95 m and five side spans between 2.35 and 4.25 m. The building was placed close to another building with a gap between them of 0.05 m.

The structure consisted in reinforced concrete columns and walls stiffened through slabs. The RC columns, with dimensions of cross section of 0.45x0.40 m, were centrally placed, whereas walls, with thickness of 0.25 m, were placed on contour. The floors consisted in slabs with thickness of 0.15 m and a network of beams with dimensions of 0.25x0.55 m, respectively 0.25x1.00 m.

Grouping of explosives charges, in explosions steps, was established taking into account the collapse trajectory and the limitation of explosive amount per explosive step. One or more support elements can be grouped in an explosion step in order to get the collapse trajectory and the acceleration of structure after collapse initiation. The time intervals among explosion steps were milliseconds range (0.025 s or more) and they were imposed by features of blasting caps used to set off the explosive charges placed into blast holes.

The numerical evaluation of controlled demolition using explosives was performed using Extreme Loadings of Structures (ELS) software. This software use Applied Element Method to simulate progressive collapse of structures. In order to simulate the demolition of building it was necessary to follow these steps:

(a) The geometrical modeling of building, **Fig. 6**;

(b) The establishment of the demolition scenario. This step consists in the specification of the structural elements that will be demolished, sequence and time intervals among explosion steps. The time of analysis and time step should be also set in this stage of analysis. Two values for time step analysis were used: a time step of 0.001 s to see the behavior of structure between two steps of explosion and a step of 0.01 s to verify the collapse trajectory and the level of structure damage;

c) The integrity of structure verification and running analysis;

(d) The verification and interpretation of results.

In terms of collapse trajectory and level of damage of structure, after it hits the ground, the results of simulation are comparable with that obtained in the properly demolition, as it can be seen in **Fig. 6**.

To perform a building implosion of a structure, it can be chosen between a simple vertical knocking down or a combination of vertical demolition of the central part and toppling of the lateral sides, **Fig. 7**.

3. Progressive collapse analyzing

3.1. Introduction

Progressive collapse initiation and propagation assumes a local failure of an element or group of support elements, which can occur as a result of extreme events. Progressive collapse study's main objective is to prevent or reduce the potential for this phenomenon, regardless of the cause leading to its initiation. There are always scenarios that will be able to initiate a collapse unconcerned of other specific design requirements (seismic design, blast and impact design, fire design, etc.).

Based on these considerations General Services Administration (GSA, 2000, 2003) and Department of Defense (UFC, 2005) have published a series of recommendations for minimizing the potential for progressive collapse in the design of new and upgraded buildings, and for assessing the potential for progressive collapse in existing buildings. GSA recommendations are based primarily on Alternative Path Method and include collapse analysis procedures when load-bearing elements are removed.

DOD proposes two methods of analysis: an indirect one – the tie force method and the direct one - the alternative path method. Both GSA and the DOD recommendations use as local failure for the collapse initiation the instantaneous removal of a load-bearing element for one floor above grade, either on the exterior or interior of the structure.

Most studies use scenario proposed by GSA and DOD to initiate collapse by instantaneous removal of a column and perform static or dynamic analysis on 2D or 3D structures (Guoqing, Ellingwood 2011; Kim *et al.*, 2009 ; Fu, 2009; Kwasniewski, 2010, Liu, 2010; Izzudin *et al*, 2008; Galal, 2010; Tsai *et al.*, 2008; Salem, 2011; Baciu *et al.*, 2012). Some studies take into question the influence that the explosion causing the removal of the load-bearing element (column) has on the behavior of the structure. Thus Luccioni (Luccioni, 2003) performed an analysis of the structural failure of a RC building caused by a detonation of 400 kg TNT placed at the second floor. All the process from the detonation of the explosive charge to the complete demolition, including the propagation of the blast wave and its interaction with the structure is reproduced. The analysis was carried out with a hydrocode (AUTODYN). Shi (Shi *et al.*, 2010) proposed a three-step method for progressive collapse analysis of RC frame structure, by considering nonzero initial conditions and initial damage to adjacent structural members under blast loadings. Jayasooriya (Jayasooriya *et al.*, 2011) performed an analysis in two stages (first stage in SAP 2000 and the second in LS-DYNA) of a RC frame structure for assessing vulnerability, damage and residual strength capacity of the building frames and component elements subjected to near field blast event (detonation of 500 kg TNT at a 5 m standoff distance).

The overall behavior of RC frame structure and its components under the blast loadings produced by the detonation of an explosive charges placed near the building was analyzed by Lupoae and Bucur (2010) non considering the infill walls and Lupoae and Baciu (2011) taking into account the infill walls.

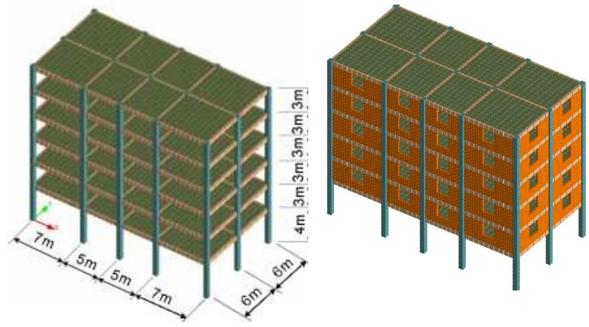
There is presented in the following a comparison between the behavior of a RC frame structure with and without infill walls, for two cases of initiation of collapse: a) instantaneous removal and b) blast removal of a column at the first floor according GSA scenarios.

3.2. Structures with and without infill walls

A six storey reinforced concrete frame as show in the **Fig. 8** 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 height.

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 1**.



a) frame structure b) structure with infill walls

Fig. 8. The ELS model of the RC building

Table 1. Material characteristics

Material	fc [N/m2]	fy [N/m2]	E [N/m2]
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.

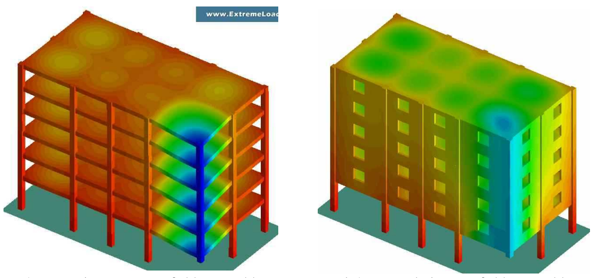
The structure is subjected to a various types of loads: dead load (D) – 1500 N/m2 on every floor, live load (L) – 2500 N/m2 on every story except the top floor and snow load (S) – 1500 N/m2 on the top floor. The combination for the column removing cases:

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

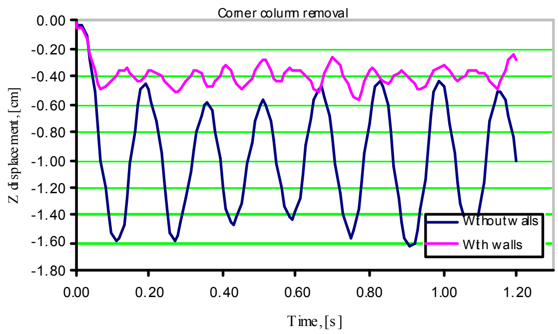
For modeling the structure, the Applied Element Method was used. For modeling of concrete under compression, Maekawa compression model is used (ASI, 2002). For reinforcement springs, is used the model presented by Ristic *et al.*, (ASI, 2002). The tangent stiffness of reinforcement is calculated based 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.

3.2.1 Instantaneous column removal

This analysis is currently used in the cases of blasting and progressive collapse, when the user knows which elements will be damaged and caused the collapse of the structure. Under this scenario, the elements to be destroyed are specified and also the time at which the removal is performed. 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 for joins in second floor of the two types of structures, above removed columns.

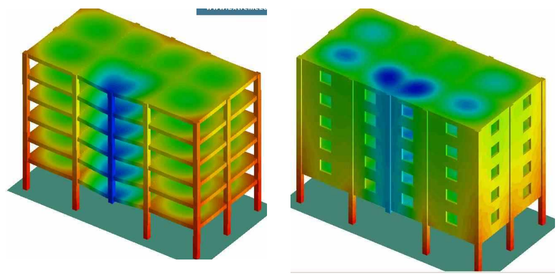
Fig. 9. The Z displacements of structures for corner column removal

The instantaneous removal of the exterior columns of the structure was performed in accordance with GSA guidelines: 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.

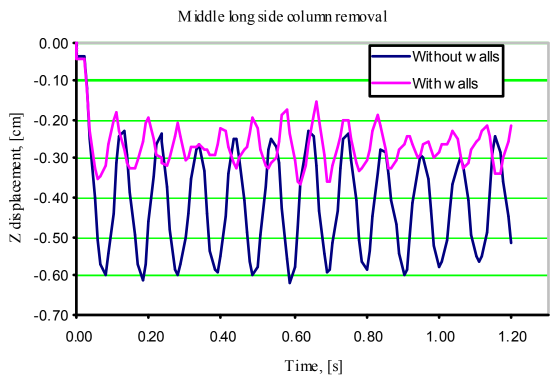
For all three cases the loss of the columns was performed instantaneously at time t=0.025 s and this type of analysis combined with the constitutive material models for concrete, reinforcement bars and masonry conduct to a non linear dynamic analysis.

Table 2. 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



a) without infill walls b) with infill walls



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

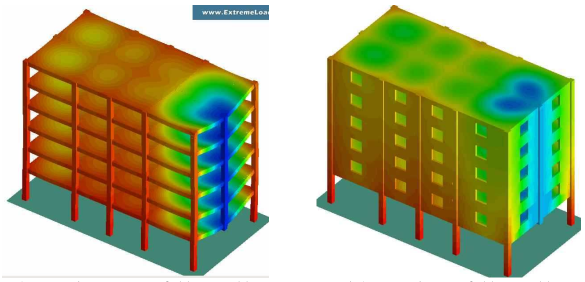
Fig. 10. The displacements of structures for middle long side column removal

To compare the behavior of the 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.

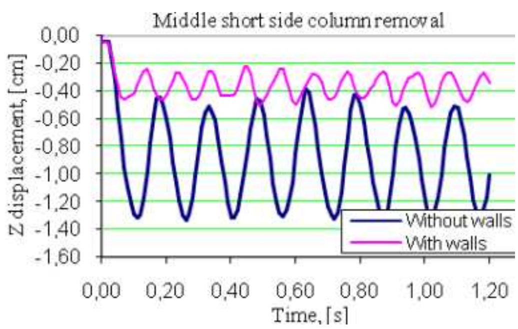
Figures 9, 10 and 11 show the deformation mode and Z axis displacements for both cases, structure with and without infill walls, for instantaneous removal of the columns.
As can be seen in these figures and **Table 2**, the use of masonry walls on the perimeter of the structure will reduce the vertical displacement of the nodes above the removed column with 40 to 70%.

3.2.2 Column removal by blast

The case when the column is destroyed and removed as a result of an explosive charge detonation is different for instantaneous column removal scenario.



a) without infill walls b) with infill walls



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

Fig. 11. The displacements of structures for middle short side column removal

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. The Friedlander equation is used to compute the pressure-time history at any point of the structure:

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

where: P_s is the peak static overpressure at the wave front, T_s is the duration of positive phase, θ describes the decay of the curve and t is the time measured since wave arrival. In this case the ambient air pressure is the reference pressure.

In order to destroy a corner column it was used 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. 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. The parameters of the blast loads acting on the structure are presented in **Table 3** and the graphical representation of the pressure and impulse can be seen in **Fig. 12**.

Table 3. Parameters of blast loads acting on the structure

Parameter	Value
Peak incident overpressure, kPa	2622
Normally reflected pressure, kPa	18590
Positive phase duration, msec	8.189
Incident impulse, kPa*msec	2592
Reflected impulse, kPa*msec	19460

In the ELS software, the free-field pressure wave model does not take into consideration the reflection and refraction of pressure wave at the ground surface and surrounding buildings and also the explosion products effects for small stand-off distance. Thereby, for small distances, 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 instantaneous column removal scenario. For large stand-off distances the effect of blast pressure on the adjacent elements can be very significant.

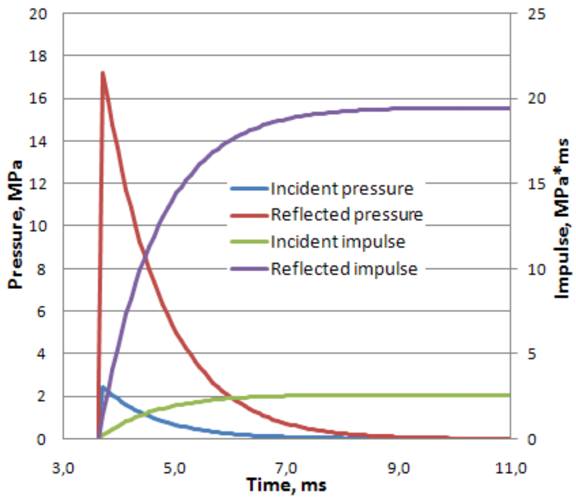


Fig. 12. The time history for incident / reflected pressure and impulse for 2700 kg TNT and 10 m stand-off distance

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 join on 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 the column is damaged and thrown, **Fig. 13a**. 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, **Fig. 13d**.

In case of infill walls, **Fig. 13b**, the effect of blast wave increases because of the larger surface exposed. As a result of the shock wave action on the surface of the exterior walls, the corner column and also the neighbor columns are entirely damaged (**Fig. 13b**) and this induce the structure collapse (**Fig. 13c**).

3.3 Experimental investigation

Experimental tests aimed to measure vertical displacement of the node immediately above the column that was removed. According to GSA scenario the column removed was placed at the middle of long side of an industrial warehouse, in the first (ground) floor. Because the building was to be demolished and adjacent structures were at enough distance to be safe, the column suddenly removal was performed by explosion. Explosive charges were placed into blast holes, drilled into the column, **Fig 14a**. The explosive charges were computed so that the explosion would be thrown entirely concrete among reinforcement bars, in order not to influence the vertical movement of the structure.

The measurement of the global vertical displacement of the testing structure was facilitated by the presence of an auxiliary structure, the gap between the two buildings being 5 cm, **Fig. 14a**.

Following the detonation of explosive charges, concrete was shattered and complete thrown, except from the neighboring building column, **Fig. 8c**.

Stirrups were straightened and some thrown and also longitudinal reinforcement were bent. The longitudinal reinforcement bending occurs due to action of the shock wave and the propulsion of concrete fragments and not as a result of vertical displacement of the structure after the blasting of the column. There were used linear potentiometers to measure global vertical displacement of the joint above the column removed. The main part of the potententiometers was fixed on a column of adjacent structure and the mobile part of sensors was mounted on a L shape fixed above the column that was to be destroyed by explosion, **Fig. 14b**.

Because the explosive charges were placed in blast holes drilled into column and their total weight was about 0.6 kg TNT equivalent, blast effects on other elements of the building were negligible. After the column was removed, the structure moved vertically up to the maximum displacement and then oscillated around the final movement. **Fig. 15** shows the vertical displacement history of the join. The figure displays a maximum downward vertical displacement of 10 mm at 0.045 s. A permanent vertical displacement of about 7 mm is recorded at the end of vibration, which is different from the maximum displacement.

The structure returned to the equilibrum state after about 0.5 s, without the destroying column leading to the initiation of the collapse.

After geometrical modeling of the structure in ELS and using instantaneous removal scenario, we obtained history of vertical displacement of join above the removed column, **Fig. 15**. As can be seen the maximum value obtained by simulation is only 9.2 mm compared to 10 mm obtained from experimental tests, but the profile curve of the vertical displacement obtained by simulation ranges between trends of variation of displacements obtained from the two sensors.

4. Conclusions

Study of the design and execution of controlled demolition works using explosives can lead to obtain important information for the study of potential for progressive collapse. It has been highlighted, with examples, the advantages of bearing walls structures versus reinforced concrete frame structures, in terms of progressive collapse initiation and propagation. Thus, wall-bearing structures need considerable preparatory works in order to be made to the state that they can be demolishd using explosives. This aspect reveals their greater resistance to progressive collapse than RC frame structures. Another aspect showed in the first part of the paper was the special importance of stirrups in the process of reducing the possibility of collapse initiation.

Also other preparatory works such as removing infill-walls on blasting floors or exposing and cutting reinforcement bars or performing openings in elements are aspects which influence on structural collapse should be considered. The narrow openings in beams and plates are designed primarily to stop the redistribution of additional efforts occurred after the destruction of a support element and secondary to help structure to move in a right direction and to fragment the building debris.

The process of instantaneous removal of load-bearing elements according to GSA and DOD recommendations, instead of considering the real scenario (blast or impact) can lead to different results due to the extension of blast loadings to a greater number of adjacent elements.

Using a simplified modeling of the building (without considering infill walls, for example) leads to significant differences in vertical displacement from the real model (between 40 and 70%) in the case of GSA scenario and conduct to the collapse of structure when using blast scenario.

Experimental tests aimed to measure the vertical displacement of the join above the column removed by explosion, according to GSA scenario. Comparison of vertical displacements recorded and the values obtained by numerical simulation showed that there is a good agreement between them, thus validating both the method and material models used for progressive collapse analysis of reinforced concrete buildings.

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a) b) c) d)

Fig. 1. Example of transforming a structure with load-bearing walls: a) initial structure, b) structure after transforming the bearing-walls in pseudo-columns, c) side view, d) front view

b) b)

Fig. 2. Preparatory works for the concrete mixed structures a) the removing of bearing-walls from the acceleration zone of structure b) the creation of auxiliary failure sections.

a) b) c) d)

Fig. 3. The influence of transversal reinforcement about failure mode of a column: a) şi b) the stirrups were not cut ; c) cutting the stirrups; d) increasing the distructive effect (Dykon, 2005)

a) b) c)

Fig. 4. Concrete removal in preparatory works a), b) mechanical removal) and c) blast removal.

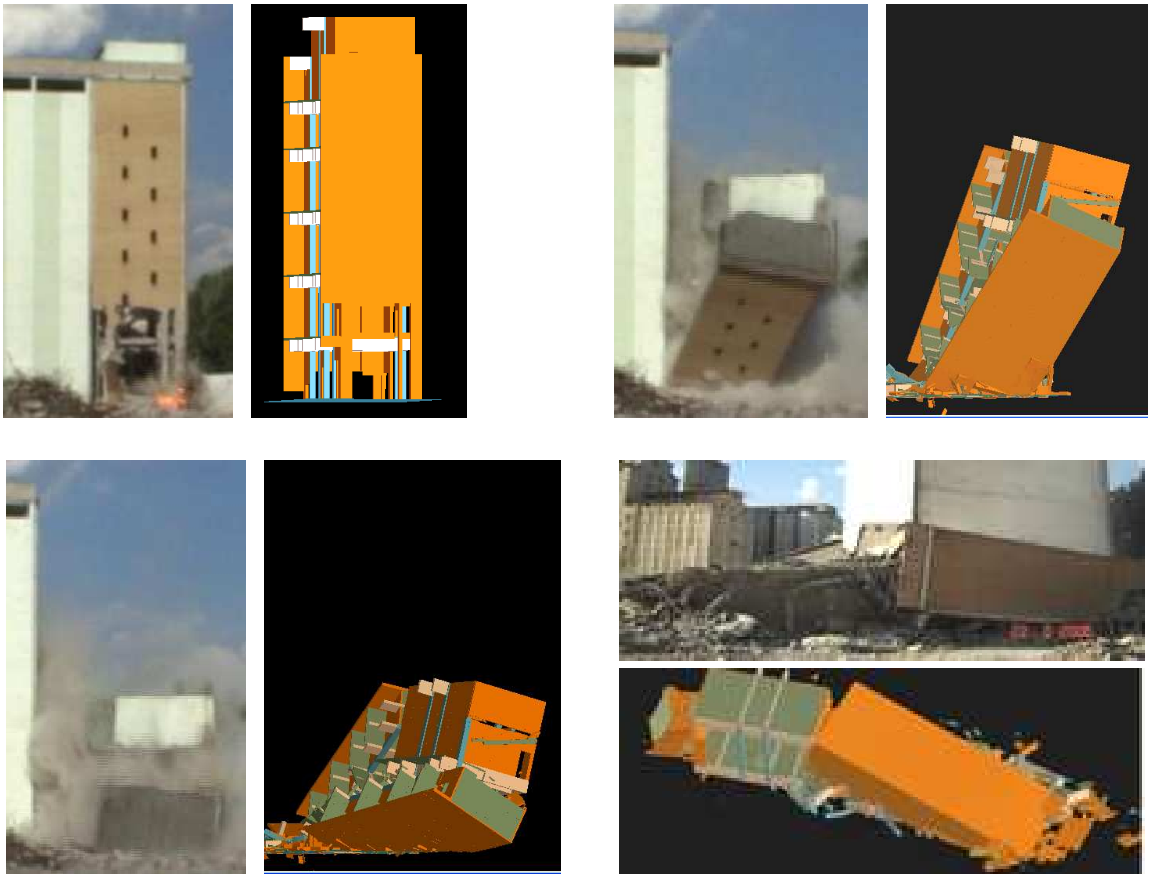

Fig. 6. Implosion and numerical simulation of lateral demolition (Lupoae, Bucur 2009)

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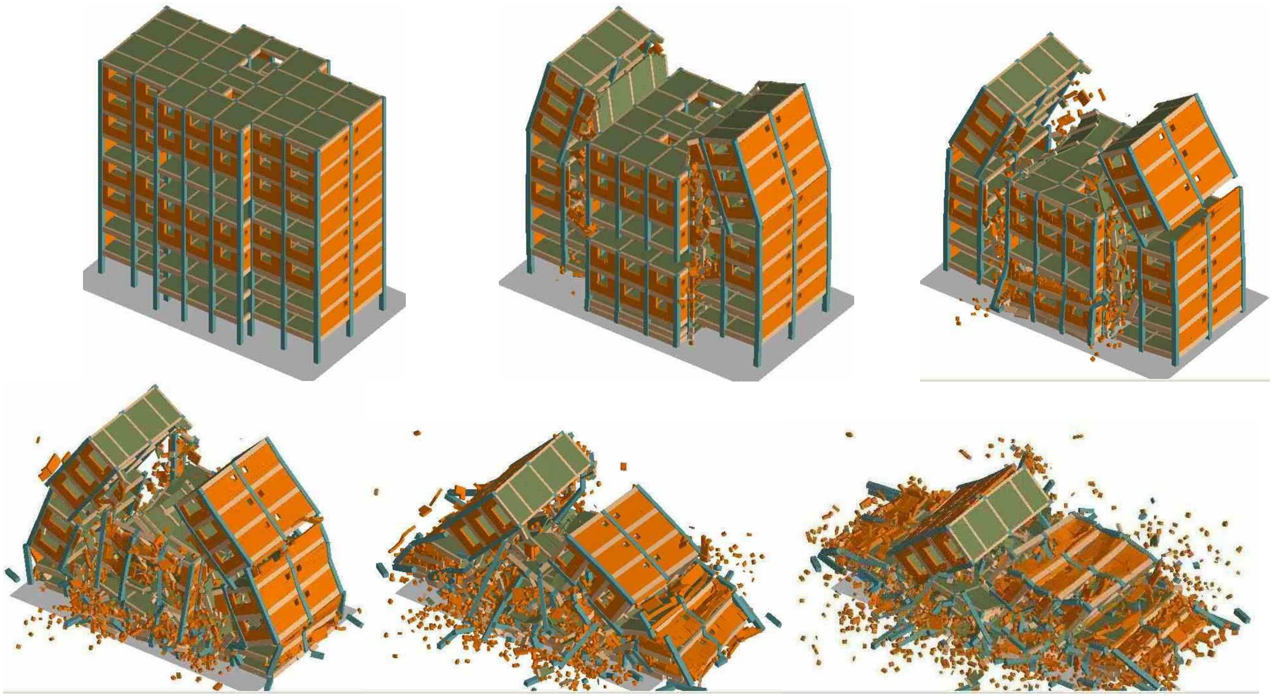
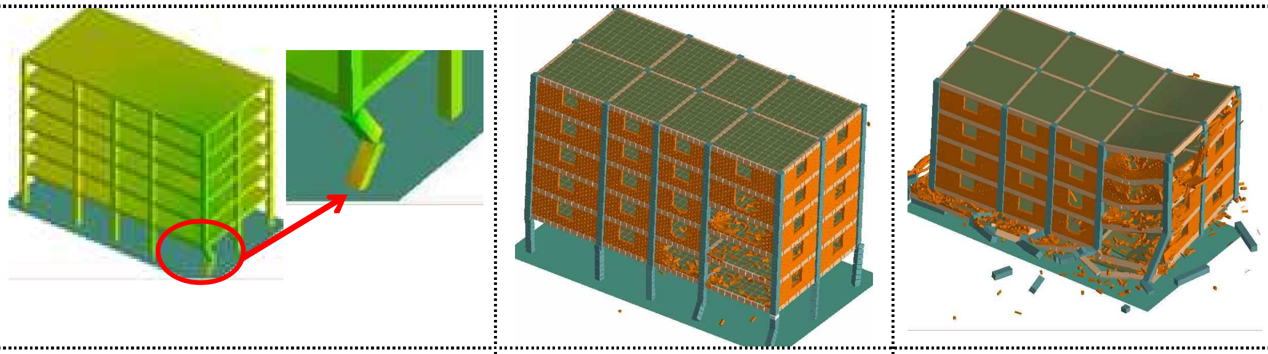
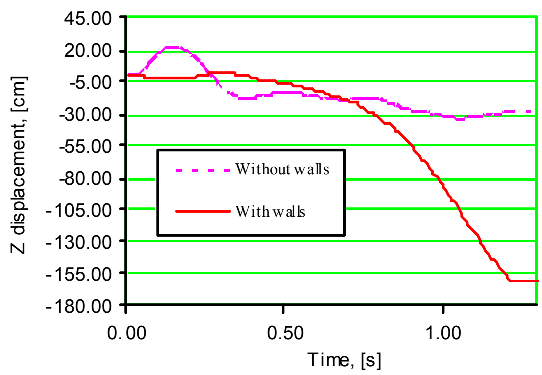


Fig. 7. Sequence of demolition stages for an implosion.



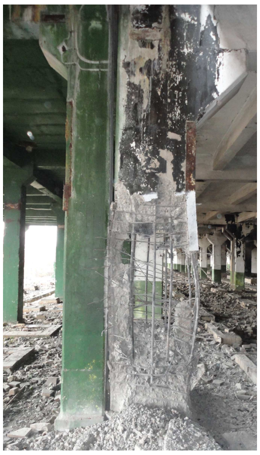
a) corner column damage – no collapse	b) columns and infill walls damage	c) collapse of the structure
Structure without infill walls	Structure with infill walls	



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

Fig. 13. The Z displacements of the second floor above the corner column and the damage of the structures under blast action





a) drilling blast holes b) placement of sesnsors c) effects

Fig. 14. Blasting of a RC column and vertical displacement measurement

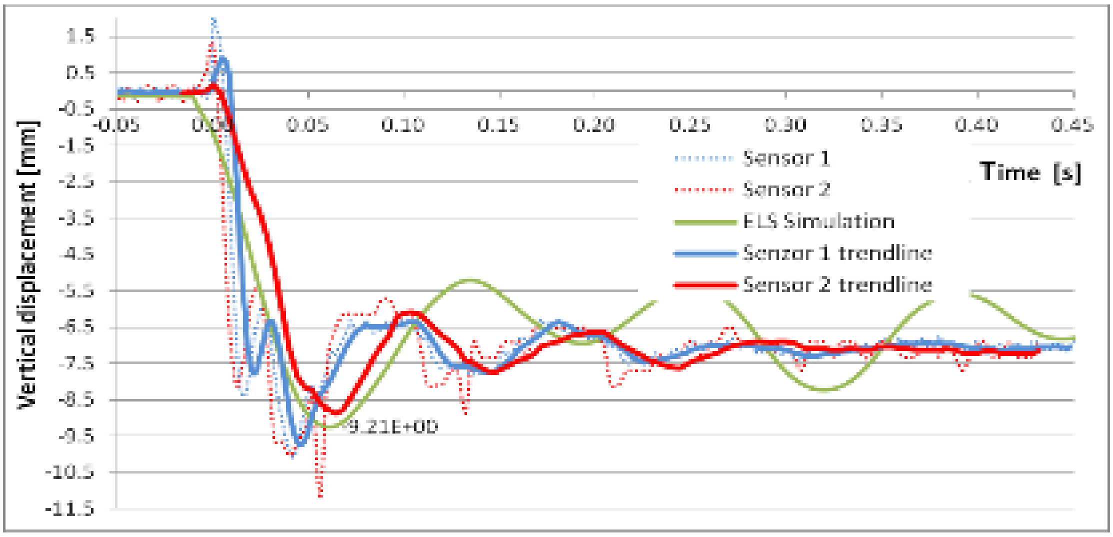


Fig. 15. Comparison between experiment and simulation