
NONLINEAR ANALYSIS FOR A REINFORCED CONCRETE FRAME STRUCTURE UNDER EXTREME LOADS

Cătălin BACIU¹, Marin LUPOAE²

¹ PhD, Lecturer, Mechatronics Department of Military Technical Academy, Bucharest, Romania, phone: +40745603643, e-mail: baciucatalin2001@yahoo.com

² PhD, Assoc. Professor, Mechatronics Department of Military Technical Academy, Bucharest, Romania, phone: +40742731069, e-mail: mlupoae2003@yahoo.com

ABSTRACT

Terrorist actions of the last decade obviously determine a reconsideration of the buildings safety, especially for those with special destinations (embassies, military facilities, nuclear plants etc.). Thus, the conception phase should include nowadays scenarios with exceptional actions, other than seismic loads: impact, explosion, failure of supports.

It is well-known that, after earthquakes, the most situations of structural collapse are determined by the terrorist attack using explosive. Even more, it has been assessed that the main reason of loss of life is not the explosion itself, but the partial or total collapse of the exposed building.

In order to secure the structural integrity, collapse analysis should be taken into account as a complex phenomenon, composed of many processes that could be studied separately or as an ensemble. These processes are: the extreme action causes, design requirements, vulnerability of the structure, starting and development of the collapse, its final effects.

Study of these processes leads to additional special measures for design stage and, on the other hand, to valuable knowledge for controlled demolition of tall buildings, as part of bold urbanism projects in densely populated zones.

Considering the above mentioned facts, this paper presents the results of a thorough analysis of a frame structure under explosive loads. The first part of the paper refers to the analysis of a phenomena assembly related to explosion and to the effects resulting from a terrorist attack using explosives (structural elements deterioration and failure, collapse initiation and eventual total failure). The second part of the paper presents an original approach to analyze a building structure under extreme loads, using both classical (pushover) and modern (applied element method).

Keywords: Progressive collapse; blast; nonlinear analysis; Applied Element Method

REZUMAT

Amploarea pe plan mondial a acțiunilor teroriste din ultimul deceniu a condus pe bună dreptate la reconsiderarea riscurilor ce pot apărea pe durata de viață a construcțiilor, mai ales în cazul unor destinații speciale ale acestora (ambasade, puncte de comandă, centrale atomo-electrice etc.). Astfel, în scenariile stabilite în faza de concepție au fost adăugate și cele ce cuprind acțiunile excepționale, altele decât seismul: impactul, explozia, cedarea reazemelor. Se cunoaște faptul că, după cutremure, cele mai întâlnite situații de solicitări extreme, care pot duce la colaps structural, sunt cele provocate de atacurile teroriste cu exploziv. Mai mult, s-a constatat faptul că cele mai multe pierderi de vieți omenești au loc nu datorită exploziei propriu-zise, ci datorită prăbușirii parțiale sau totale a clădirii ce a suferit un astfel de șoc. În vederea garantării siguranței structurale, se impune analiza colapsului structural ca un fenomen complex, compus din mai multe procese, ce pot fi studiate separat sau pot fi tratate ca un ansamblu. Aceste procese sunt: cauzele acțiunii extreme, cerințele de proiectare, vulnerabilitatea structurii, inițierea și dezvoltarea cedării, sfârșitul și urmările colapsului. Studiarea și aprofundarea acestor procese a condus, pe de o parte, la considerarea de măsuri suplimentare, speciale la proiectarea structurilor expuse unor astfel de acțiuni, dar, pe de altă parte, au fost dezvoltate cunoștințele cu privire la demolarea controlată a clădirilor înalte, în cadrul îndrăznețelor proiecte de urbanism din zonele dens populate.

Scopul lucrării de față este de a studia comportamentul unei structuri în cadre supuse unui atac terorist. În prima parte vor fi analizate în ansamblu fenomenele asociate exploziei și efectele produse în urma unui atac terorist cu explozivi (deteriorarea și cedarea elementelor, inițierea și eventual propagarea colapsului). În cea de-a doua parte a lucrării se propune o abordare de analiză a unei structuri supuse unei astfel de acțiuni extreme, utilizând comparativ metode clasice (pushover, time history) și metoda modernă, cea a elementului aplicat.

Cuvinte cheie: colaps progresiv; explozie; analiză neliniară; Metoda Elementului Aplicat

1. BUILDING PROGRESSIVE COLLAPSE

During their lifetime, building structures could be exposed to natural extreme phenomena (earthquake, tornados, fire, flooding) and anthropogenic phenomena (blast or impact). Structures are not usually designed for extreme loadings and when such events occur can lead to catastrophic failure. Recently, terrorist attacks aiming important buildings (World Trade Center, Murrah Federal Bureau) led to structural collapse, with important human lives and material loss.

The term of „progressive collapse” refers to the development of an initial local failure as a chain reaction, which could lead to local or total crush. The main characteristic of progressive collapse is the significant disproportion between initial phase and the final state.

The progressive collapse became an interesting topic for building designers and researchers after the partial collapse of the Ronan Point block in London – 1964 and the importance of the subject highly increases with recent terrorist activities all around the world. Extreme events as blast and impact, considered improbable in the past, were moved to credible events, having a finite probability of occurrence.

Thus, nowadays the design activity should have as additional objective the progressive collapse risk mitigation for important buildings. Structural analysis in traditional ways is completed with a new approach, which bring in the conception of the most unfavorable scenarios and then adjust the design processes according to these special situations. The main objective of this kind of approach is to reduce the effects of exceptional events and to mitigate the progressive collapse, performance targets that could be reached even with a partial deterioration of the structure.

1.1. Actual status regarding the progressive collapse usage in structural analysis

The actual building design codes offers general references to prevent progressive

collapse having as support the main characteristics of the structure (redundancy, integrity, continuity, ductility and efforts redistribution), but there are no further recommendations for an analysis that contains the phenomenon.

Furthermore, the usual philosophy of the most of the actual design codes is to protect the structures under conventional loads during their lifetime. So, the structures are not currently designed for exceptional events as blast caused by gas accumulation, impact with a vehicle or a plane, classical explosions. Many codes offer only general recommendations in order to mitigate the effect of progressive collapse.

In the last three decades, the UK Building Regulations (1) imposed requirements for disproportionate collapse prevention, formulated as a result of Ronan Point event. Notes are kept unchanged until these days.

Eurocode established different technical regulations regarding the type of structures that should be supplementary checked to progressive collapse (2).

Of all American codes, ASCE – 7 (3) is the only one that contains detailed guidance for our matter. It is hereby stipulated the necessity of structural protection against extreme events, offering two different approaches: direct method and indirect method. For the direct method, the progressive collapse resistance is considered during the design process using Alternate Path Method and specific local resistance method.

Also, there is a set of governmental documents in the USA, which gives design orientations for structural resistance under extreme loads. Documents are issued by General Services Administration (GSA) (4), Department of Defense (5) and Interagency Security Committee (6). Those published by GSA provide a detailed methodology in order to reduce the possibility of progressive collapse occurrence for new buildings, using Alternate Path Method and to assess the vulnerability level for existing structures, under extreme loading.

GSA document recommends possible scenarios for column failure in different

reinforced concrete structure configuration (frame structure or thick slab structure), scenarios that were assumed by researchers (7), (8), (9), (10), (11), (12).

Fig. 1 presents failure scenarios proposed by GSA. According to their methodology, the evaluation of progressive collapse possibility for a reinforced concrete structure is made analysing the structure behaviour when a vertical element is removed: first floor external column removed (a). All the buildings with underground parking garage opened to public also include a scenario of central column instantaneous removal.

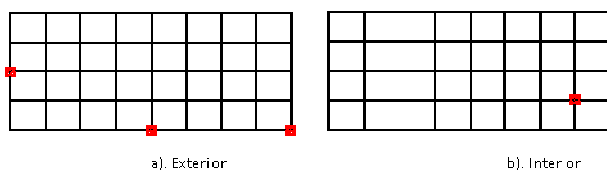


Fig. 1. Scenarios of column loss, according to GSA 2003

A short glance over the papers in this domain gives us the certitude that there is a certain interest in finding the proper approach of progressive collapse evaluation and in determining the starting and development of such phenomenon.

Thus, the paper (7) proposes a simplified approach to evaluate progressive collapse of a multistory structure, considering instantaneous failure of a column as part of design scenario. There are used three different stages: firstly - nonlinear static response of the affected structure under gravitational loads, secondly - simplified dynamic assessment in order to establish the maximum dynamic response after sudden failure of the column and, thirdly - evaluation of connections ductility. There were chosen various positions for the failure column: on the facade of the building, at the ground floor (in the corner / on the short side / on the long side).

Other papers (9), (10) analyze reinforced concrete with 13 floors, where a column is suddenly removed from different positions, according to the GSA documentation. There were used many structural solving software (ETABS, Robot Millennium) and the results

revealed frame structure capacity to resist to progressive collapse.

The paper (8) compares analysis methods of progressive collapse, starting with a linear static analysis and finishing with a nonlinear dynamic one, which is taking into account the blast load. The scenario of the blast consists in detonation of a 125 kilos TNT equivalent explosive, at 5 m distance to the structure and at 1 m height. The obtained results showed that the vertical displacement of the joint just above the removed column using explosion is bigger (1.6 times bigger) than the displacement obtained using the other methods (the biggest displacement is obtained for the nonlinear static analysis).

The case study of the real structure – San Diego Hotel – was conducted by Sasani and Sagioglu in 2008 (11). The 6 story building was assessed for the extreme situation of two exterior column were simultaneously and instantaneously removed (a corner column and the next column of the short side). The column removal was executed using explosive loads introduced in the holes perforated in the structural elements. All the other building elements were protected with special protection materials. In situ measurement showed that the maximum displacement of the joint situated just above the removed columns is 6.4 mm and the structure did not collapse.

2. NONLINEAR ANALYSIS OF THE REINFORCED CONCRETE FRAME STRUCTURES UNDER EXTREME LOADS, SUCH AS IMPACT OR BLAST

The assessment of the possibility of structural collapse under different extreme loadings is well-represented in the domain papers. Thus, there were many approaches used: linear and nonlinear, static and dynamic analysis, with or without blast effects consideration. These analysis were accomplished using Finite Element Method.

Also, there were developed new approaches, based on Applied Element Method (AEM), which combines the advantages of the finite elements method with those of the discrete elements method (2).

2.1. Methods used in the structural analysis

The structural solving software used for the proposed approach is SAP2000 (based on the Finite Element Method) and ELS – Extreme Loadings of Structures (based on Applied Element Method). Nonlinear static and dynamic analyses were run with these software programs. The results of the analyses are comparatively presented in the final section of the present paper.

The limits of ETABS regarding the impossibility of collapse detection are extended using the other program – ELS, which has explicit options to simulate structural collapse: sudden element removal or element destruction, demolition, using the blast effect.

The approach of sudden structural element removal is often used for building demolition works, when it is known for sure which are the elements that will fail and will produce structure collapse. For this demolition scenario there have to be specified the elements which will be destroyed and the time when every one of these elements is suddenly removed (Fig. 2, a). The advantage of this approach is the reduced time for the automatic analysis, compared to the blast loading method.

The second approach – element destruction under blast loading – is used especially to create scenarios that involve terrorist attacks with explosives, the blast effect being automatically generated according to the quantity and the location of the explosive. The blast effect simulation (Fig. 2, b) is obtained using the „blast in free air”

model. The pressure in the blast wave depends on the type and the quantity of the explosive, on the distance to the building and on the moment of time considered. The value of the blast pressure in every moment, for every part of the structure, could be determined using Friedlander’s equation:

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

where:

P_s - value of the blast wave overpressure;

T_s - period of positive stage;

t - period measured until the wave arrival.

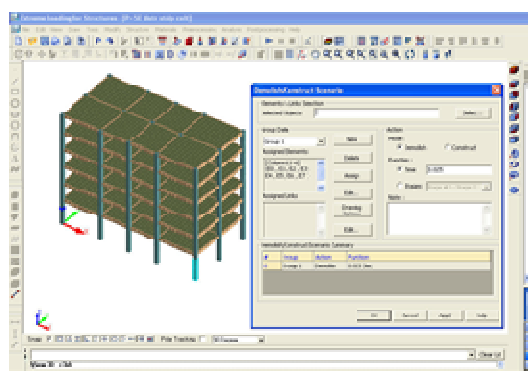
The pressure forces generated by the bomb explosion are obtained using the follow assumptions:

i) for every side of a structural element it is specified whether the imaginary line traced to the bomb position intersects or not other structural elements (the condition of blast exposure);

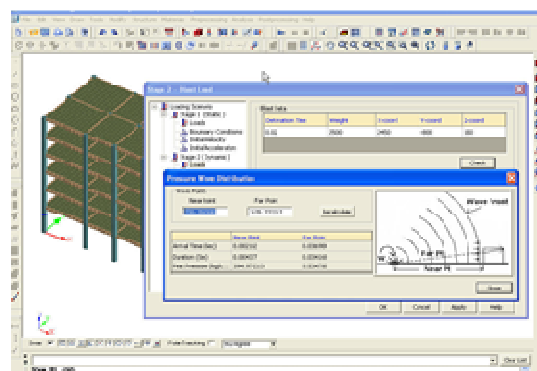
ii) when imaginary line is not intersecting other elements, on the considered side of the element is directly applied the value of the blast wave pressure;

iii) elements are loaded only when the pressure wave comes up to the element sides;

iv) pressure wave acts perpendicularly on the loaded surfaces; the force applied represented the product of the pressure value and the surface area of considered element, and the direction of application is perpendicular to that surface.



a) using demolition scenario



b) using blast loading scenario

Fig. 2. Approaches of column removal using ELS software

There should be added that the approach presented above does not consider the reflection and refraction of the blast wave, to the ground or to the nearby buildings or obstacles.

2.2. The analysis of a frame reinforced concrete structure, under extreme loading as a result of explosive detonation

As case study, there is used a six storey frame reinforced concrete structure, with 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. 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 concrete compressive strength at 28 days is 30 MPa with elastic modulus $E_b = 32.5$ GPa. The yield strength of reinforcement is 300 MPa with elastic modulus $E_a = 210$ GPa.

The structure is subjected to the current types of loads: dead load (D) – 1,75 kPa on every floor, live load (L) – 2,50 kPa on every storey, except the top floor where the snow load (S) – 1,50 kPa is taken into account. The perimeteral and interior walls weight (P) is distributed as uniform load to the beams – (5kN/m). Therefore, the combination for the column removing cases is:

$$D + P + 0,4(L + S) \quad (2)$$

2.2.1. Demolition scenario used for column removal modeling

Using ELS software program, the exterior column removal was made according GSA recommendation (Fig. 1, a): a corner column, one column from short side and one column on

the long side. For all three cases the removal of the column was instantaneously performed at time $t=0,025$ s and this type of analysis combine with the constitutive material models for concrete and reinforced bars conduct to a non linear dynamic analysis.

a). The sudden removal of a corner column

For this case, the maximum vertical displacement of the joint located above the column removed is about 1.62cm.

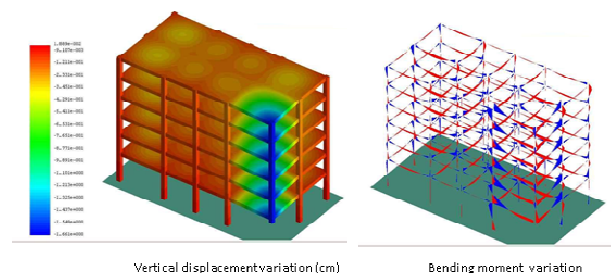


Fig. 3. Case of corner column removal

Fig. 3 presents an image of vertical displacements and also the maximum bending moment variation, developed in the frame elements.

b). The sudden removal of a column situated at the middle of the long side of the building

The maximum vertical displacement of the joint located above the column removed is 0.63 cm and the graphic results for this case are presented in Fig. 4.

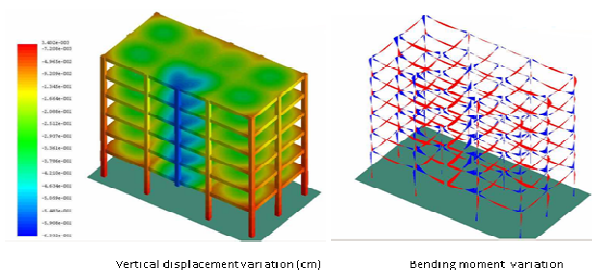


Fig. 4. Middle column of long side of structure removal case

c). The sudden removal of a column situated on a short side of the building

The maximum vertical displacement of the joint located above the column removed is 1.35cm and the graphic results for this case are presented in Fig. 5.

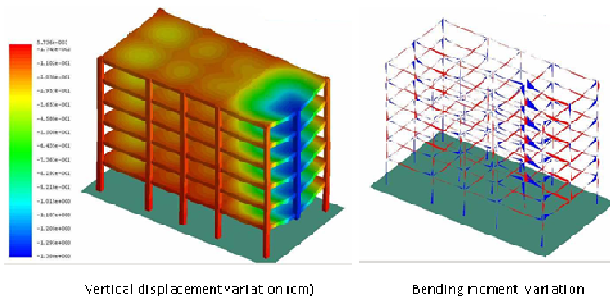


Fig. 5. Middle column of short side of structure removal case

The variation of vertical displacements curves for joints in second floor, above removed columns are shown in Fig. 6. After the curves of vertical displacements of joints above removed columns were analyzed, it can be seen that the oscillations of the structure are higher for corner and short side columns than for column located on the long side of the structure. The maximum vertical displacements are comparable as order of magnitude to those measured by Sasani after the removing of two adjacent columns of the building of Hotel San Diego, which was finally controlled demolished (11).

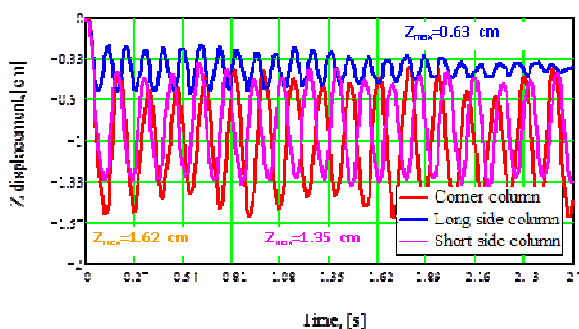


Fig. 6. The variation of vertical displacements curves for the joint located just above the removed columns

2.2.2. Blast scenario used for column removal modeling

The modeling of blast action on structure using Extreme Loading for Structures software has important 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. This approach has also disadvantages: (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. For small stand-off distance, the blast pressure is concentrated on the expected failed column (14), (15). As a consequence, the effect of this pressure on the adjacent element is relatively small and is the same with demolition scenario. For large stand-off distances the effects of blast pressure act on the adjacent elements.

The energy of the blast load could be assessed depending on the scaled distance (13):

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

where:

Z - scaled distance;

R - stand-off distance;

w - charge weight.

According to the above relation, it is assumed that the energy transferred to a target is identical for the same scale distance. The energy released by 1000 kg TNT to a target situated at 10 m stand-off distance is the same with the energy released by 8 kg TNT to a target situated at 2 m, both having the same scaled distance of.

Using an explosive charge of 2700 kg TNT, placed at 1.5 m above the ground level and at 10 m stand-off distance to the corner column, causes the separation and propulsion of a part of the column. The amount of explosive charge corresponds to a vehicle bomb attack and the stand-off distance of 10 m was chosen in accordance with the minimum safe stand-off distance in order to respect the medium ISC level of protection for reinforced concrete construction (4).

The blast wave propagation from explosive charge is formed as a concentric wave, with center in explosive charge place, Fig. 7. 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.

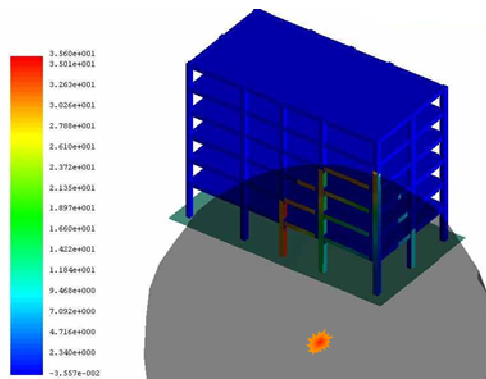


Fig. 7. The propagation and the action of the shock wave on structure (pressure measured in kgf/cm^2)

The software has the capacity of automatically adjust material characteristics in order to model the real behavior of the reinforced concrete elements under extreme loadings applied at very high velocity.

The analysis of the vertical displacement time-variation for the joint of the second floor above the removed column, Fig. 8, shows that in the first stage the structure is moving backwards in the shock wave direction because of the value of overpressure, and only after that the structure is moving downwards as the column is destroyed, Fig. 9.



Fig. 8. Vertical displacement history for the joint above the removed column

The maximum value of the vertical displacement of the joint above column removed using blast scenario is appreciatively 36 cm, 22 times greater than in case when the column is removed using demolition scenario.

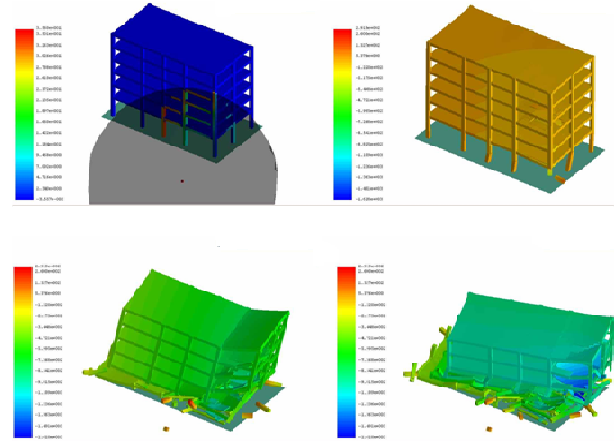


Fig. 9. Shock wave propagation, damage and collapse evolution in case of an increased explosive charge

When an explosive charge of 3000 kg TNT is detonated at a stand-off distance of 12 m to the corner column and at 1.5 m above the ground, the structure is collapsed because of the damage of many vertical (columns) and horizontal (slabs and girders) bearing elements. The shock wave propagation and the damages evolution are presented in Fig. 9.

The conclusion of the analysis using ELS software is that there could be obtained accurate results in a short period of time. On the other hand, the current structural analysis programs (ETABS, SAP2000) do not have the capability to model the blast effects and the sudden removal of an element involves complicated laborious approaches. Nevertheless, the next section presents an analysis of the same structure carried out using SAP2000.

2.2.3. Removal of column using classical approaches (SAP2000)

In order to capture into account the dynamic effect of column removal, a mix of linear static analysis, nonlinear static analysis (pushover) and time-history method (16) will be used.

a) Linear static analysis

The simple, direct corner column removal, without taking into account the dynamic effect and material nonlinearity, offers the following results:

- the maximum vertical displacement of the joint located just above the column removed is 14 mm;
- periods of the first three modes of vibration are nearly closed to those of initial unaffected structure: $T_{transv} = 0,75$ s; $T_{long} = 0,73$ s; $T_{torsion} = 0,69$ s.

It could be easily noticed that the maximum vertical displacement of the considered joint is lower than the values obtained before, using ELS software.

b) Dynamic analysis (time-history approach)

The steps of this approach are:

i). The structure completely modeled, with all its elements having initial rigidity ($E_c I_c$) is loaded by a long-term combination. Efforts (N, T, M) are determined for the considered joint (above the column that is to be removed) in the integral structure analysis.

ii). The column is simply directly removed. This status corresponds to the model from the linear static analysis. The displacements and efforts increase for the adjacent elements, but, without the dynamic effect taken into account, these results are far from the real behavior.

Thus, for keeping the initial state of deformation and stresses, after the column removal, in the released joint there are introduced corresponding forces (N, T_x , T_y , M_x , M_y , M_t) with the same values but having inverse directions. Therefore the structure is not practically influenced by the column removal.

iii). The dynamic effect is introduced on the structure from the above step, without the column removed, but with initial deformations and stresses. Now, the corresponding forces are dynamically applied as linear time-variation functions (starting at 0 and having the maximum values at $t = 0.025$ s). This very short period of time is the same as the period used in the approach using ELS program.

The complex method offers the possibility for SAP2000 user to obtain dynamic effect of column removal. The results could be read at every step of time (at every 0.03 seconds), for all 300 steps, from 0 to 9 seconds.

Conclusions of the results analysis:

i) The values of the vertical displacements are quite the same with those obtained with initial approach (section 2.2.1.a). The structural deformation, showed in the next figure, is the same as using ELS software - Fig. 3.

The vertical displacements for the considered joint (located just above the removed column), without considering the structural damping are shown in Fig. 11. The undamped oscillations form an unrealistic ideal movement of the structure after the column removal. The maximum value is 2.6 cm, closed to the value obtained using ELS software – 1.62 cm.

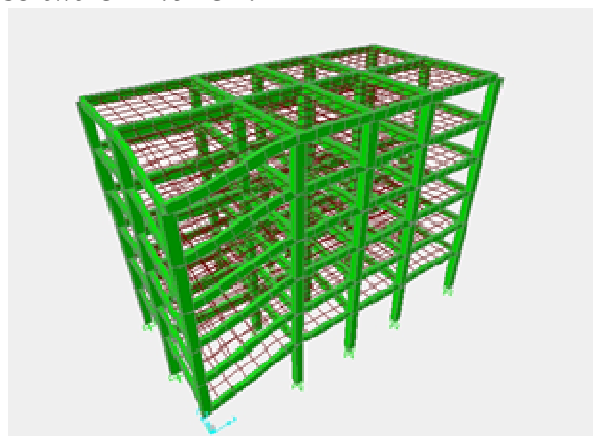


Fig. 10. Structural deformation after corner column removal (using SAP2000 program)

The real behaviour of the structure depends on the damping characteristics of the materials, sections, elements and structure as a whole. For frame reinforced concrete structures, the value for critical damping ratio is 0.05. Taking into account the realistic damping level, there is obtained different graph, with oscillations that tend to fade away – Fig. 12. It could be noticed that the vertical movement of the joint is evidently damped, and after 2 seconds the steady value of the displacement is 2.1 cm, closed to that obtained earlier.

The influence of dynamic effect on structural response is evident, comparing the maximum vertical displacements for the two cases: proposed time-history approach (2.1 cm) and the static method (1.4 cm).

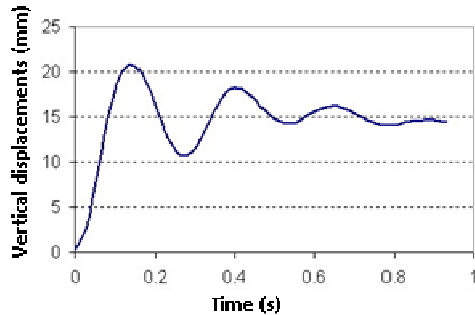


Fig. 12. Vertical displacements of the joint with the damping effect considered

ii) Another important conclusion refers to uniform vertical deformation along the structure height, on the removed column line:

the beams and the slabs located at every storey have almost the same deformations, because of the axial rigidity as the column that binds together the floors.

iii) After the removal of the column, the efforts that were initial undertaken by this column are redistributed to the adjacent beams through the slab and then to the next columns. The bending moments in the beams connected to the considered column, having initial negative value, become positive after the removal event. Along with the slab, the beams redistribute part of the extra loads to the adjacent columns.

The next table contains axial force values for columns before and after the event. In order to mark out the redistribution effect, the table also offers the percent of axial loads increasing.

Table 1. Redistribution of the axial force to adjacent columns

Adjacent columns	Axial force before the event (kN)	Axial force after the event (kN)	Extra load (kN)	Increasing percent
Corner column (that is to remove)	837	-	-	-
Adjacent column – longitudinal direction	1513	2300	787	52%
Adjacent column – transversal direction	1787	2805	1018	57%
Adjacent column – diagonal direction	3030	3256	226	7.5%

After the column removal, the axial force values correspond to the measured maximum displacements, not to the final balance bending moment.

The sum of the percent in the table above exceeds 100% because the dynamic effect. The axial force values correspond to the maximum displacements, when the structure oscillates, lowering to the removed column. Therefore, the adjacent columns are overloaded and those situated at the opposite extremity are discharged.

3. CONCLUSIONS

The main goal of the paper refers to the progressive collapse modeling, analyzing a

frame reinforced structure using a new numerical calculus –Applied Element Method, when the structure is loaded with a shock wave produced by an explosion. The results were compared with those obtained using the classical structural program – SAP2000.

To evaluate the occurrence and the development of the structural collapse, two scenarios were set up using ELS software: (i) demolition scenario to simply remove the columns, (ii) blast scenario to destroy the vertical elements.

Using the option of column removal in the demolition scenario, the vertical displacements of the joints above the removed column were obtained, these values being compared to those from other papers in the literature. The geometrical configuration of the model and the

reinforcement position allowed for efforts redistribution, so that the collapse was avoided. The most unfavorable case is the removal of the corner column, because of the small number of alternative ways to redistribute the efforts, in contrast with the other cases (removal of the column located at the middle of the short side of the building or of the column located at the middle of the long side of the building).

The option of blast destruction of the vertical elements is closer to reality in case of the large stand-off distance (larger than the range of explosion products action), because, like in a real case, the greater number of structural elements are affected and because of the free-field pressure wave models used by ELS. The analyze of vertical displacements of the joints above removed or blast destroyed columns showed that the maximum value of the displacement in case of blasted column is 17 times higher than the case when the column is removed using demolition scenario. There was also conclude that, for small stand-off distances, the pressure wave model does not take into consideration the explosion gases action and also the reflection and refractions of pressure wave at the ground surface and surrounding elements and buildings.

The results obtained using demolition scenario were compared with those determined using a complex approach, but fully capable to introduce the dynamic effect of the real removal, using structural analysis program – SAP2000. These results were practically the same, in the case of taking into account the structural damping.

On the other hand, there was analyzed the redistribution of the efforts within structural elements after the column removal: the bending moments at the end of the beams changed the sign and the adjacent columns integrally took the axial force of the removed vertical element. Once again, the results confirmed the dynamic method advantage, obtaining an extra axial load in the adjacent columns bigger than the initial static load of column to be removed. Thus, the structure behavior using these dynamic approaches is

far closer to the real behavior, compared to the case of static approach.

REFERENCES

1. *** - Office of the Deputy Prime Minister. The building regulations 2000, Part A, Schedule 1: A3, Disproportionate collapse, London (UK), 2004.
2. BUCUR, C., BUCUR, V., LUPOAE, M., *Simularea numerică a prăbuşirii progresive* - a IX-a Sesiuni de comunicări ştiinţifice SIMEC 2010
3. *** - ASCE Standard 7-05, Minimum Design Loads for Buildings and Other Structures (ASCE 7-05/ANSI A58) (2005), American Society of Civil Engineers, Reston, VA.
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 Facilites 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. IZZDDIN, B.A., VLASSIS, A.G., ELGHAZOULI, A.Y., NETHERCOT, D.A., *Progressive collapse of multi-storey due to suden column loss – Part I: Simplified assessment framework*, Engineering Structures, vol. 30, 2008.
8. ELVILA, MENDIS P., LAM, N., NGO, T. (2006), *Progressive collapse analysis of RC frame subjected to blast loading*, Australian Journal of Structural Engineering, vol.7, nr. 1 – 2006.
9. IOANI, A., CUCU, L., MIRCEA, C., *Assessment of the potential for Progressive Collapse in RC frames*, Ovidius Universitz Annals Series, vol. 1, nr. 9, 2009.
10. BALDRIGE, S.M., HUMAY, F.K., *Preventing Progressive Collapse in Concrete Buildings*, Concrete International, vol. 25, nr. 11, 2005.
11. SASANI, M., SAGIROGLU, S., *Progressive Collapse Resistance of Hotel San Diego* – Journal of Structural Engineering, 2008.
12. LUPOAE, M., BUCUR, C., BACIU, C., *The Behaviour Analysis of RC Frame Structure under Explosion Loading*, 34th IABSE Symposium on „Large Structures and Infrastructures of Enviromentally Constrained and Urbanized Areas”, Veneţia - Italia, 2010.

13. KINNEY, G., *Explosive shocks in air*, Springer-Verlag, 1985
14. LUPOAE, M., BACIU, C., *Aspecte privind distrugerea prin explozie a elementelor din beton armat*, Simpozionul național “Noi reglementări pentru beton”, Universitatea Tehnică de Construcții București, octombrie 2009.
15. LUPOAE, M., BACIU, C., *The mechanism of RC elements destruction under detonation*, International Conference „Construction 2008”, Technical University Cluj-Napoca, 2008
16. BACIU, C., *Pushover analysis for reinforced concrete structure*, International Conference on Military Technologies, Brno - Cehia, 2007.