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MODELLING THE SLAB FAILURE OF AN OPEN STRUCTURE ACTED BY EXTERNAL BLAST LOADS

George Bogdan NICA – Assistant professor, Technical University of Civil Engineering, Department of Strength of Materials, Bridges and Tunnels, e-mail: george.nica@utcb.ro

Abstract: The explosion of bombs near buildings generally yields severe damages to the structures. Explosion resistant standards and requirements are constantly being developed and upgraded.This paper focuses on the damages which occur toa RC slab due to blast action. The numerical model replicates a ¼ scale experiment. The analysis is conducted using a software based on the recently developed Applied Element Method. This numerical method is able to model accurately all the structural behavior stages up to failure. The results are compared to experimental data available in the literature. The analysis reveals that the slab failure due to uplift pressures may be avoided by some simple reinforcing details, as they are listed in the Romanian National Annex – accidental loads of the Eurocode EN 1991-1-7.

Keywords: slab uplift, vertical displacement, reinforcing details, accidental loads, applied element method

1. Introduction

The explosion of bombs or explosive materials inside or near buildings generally yields to severe damages to the structures. For example, in December 2010, a building located in Bacau undergone severe damages due to a deflagration event which took place in a room situated at the ground floor level [1]. The top concrete slab was completely destroyed, as shown in Fig. 1. The failed top reinforced concrete slab can be seen through the supporting props.

Fig.1 - Blast effects on a four-storey building in Bacau, 2010 [2]   bFailed top slab of the four-storey building in Bacau, 2010 [2]

The methods for predicting blast wave loads from both detonations of high explosives and deflagration of vapor clouds are constantly developing. The blast loads are derived using three classes of techniques: empirical, phenomenological and first principle methods [3].

The most accurate and complex in modeling is the first principle approach. The Computational Fluid Dynamics (CFD) programs are employed, where the governing equations and boundary conditions are modeled, accounting for interaction of the blast wave with the geometry of the domain. The reflection prediction and channeling of the blast wave propagation are calculated.

Because it is so complex and it requires the full modeling of the domain, the first principle approach requires a lot of calculation resources, and the calculation is thus reserved mainly to the research-engineer. The new Applied Element Method developed by the ASI company has already proved is ability in modeling not only the blast loads, but also the structural behavior in accordance with the loads.

The scope of this paper is to reproduce by numerical means an experimental setup in which a ¼ scale model of a reinforced concrete frame structure is loaded by a C4 charge. The blast load is predicted by the software using the UFC guidelines which are mainly employed for design purposes [3]. Thus the pressure applied on the structure is higher than in reality, for safety. The propagation medium is not modeled, thus the reflection of the blast wave cannot be taken into account [4].

2. Experiment data

The numerical model is based on the experiment data presented in papers [7-8]. The experiment is conducted on a ¼ scale reinforced concrete structure. A vertical 1700 kg load acts on the primary column, and a 7.1 kg charge of C4 is placed at a distance of 1.07 m in front of the structure [7-8]. This load is equivalent to a 454 kg C4 charge placed at a distance of 4.28 m in front of the structure. The experimental setup is presented in Fig. 3.

The structure has no cladding or walls, so the blast front travels directly through the structure. The elements which have sufficient large enough areas are loaded with pressure and momentum. In this case the slab above the ground floor is loaded with upward oriented pressure. The slab is commonly detailed with reinforcement for sustaining gravitational loads, so it has a poor capacity for upward oriented loads.

A residualvertical displacement of 64mm is measured at the middle of the slab above the ground floor [7-8]. Fig. 4 shows the deformed shape of the structure [7-8].

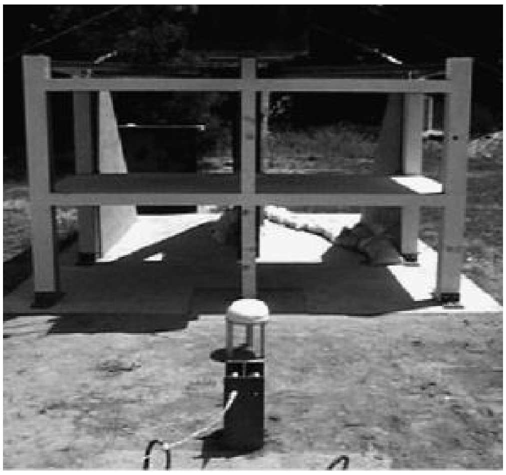

Fig. 3 - ¼ scale structure and setup. [7-8]

Fig. 4 - Residualdeformation of the slab above the ground floor [7-8]

2.1. Theory background of Applied Element Method

The AEM (Applied Element Method) key features are presented in several papers available in the literature (e.g. [4-6]). 3D elements are obtained by virtually dividing the structure. The elements are assumed to be connected by sets of springs distributed around the edges of each element. Each set of springs comprises one normal and two shear springs, which are able to take into account the stresses and deformations of the corresponding volume computed from the influence area.

In AEM the element separation can be easily simulated because each corner of an element may have different displacements, compared to Finite Element Method (FEM), in which full displacement compatibility at the nodes is assumed. Some springs may fail while others are still effective during analysis. Thus a partial connectivity is easily simulated. There is also no need for transition elements, which are commonly used to switch from large sized elements to smaller ones. The mesh connectivity requirements and interface requirements are brief.

Three reinforcement springs, one normal and two shear springs, are inserted at the exact location of the steel bars. These springs break if the reinforcement bar stresses satisfy the steel failure criteria or if the separation strain limit is reached [4-6]. For the reinforcement springs the Ristic et al. [4-6] model is used.

In ELS ® software, the concrete material is represented by “matrix springs”. The concrete in compression is modeled through the Maekawa [4-6] model. For concrete springs subjected to tension, initial stiffness is assumed until the cracking point, and after cracking the spring stiffness in tension is set to zero. Linear shear stress - shear strain relation is assumed until reaching the cracking point, and a drop down of shear stresses depending on friction and aggregate interlock is assumed after that.

2.2. Numerical model

The C4 mass is equated to a TNT mass according to paper [7], respectively a 1 kg of C4 produces the same momentum as a 1.19 kg of TNT. The 7.1 kg C4 charge is modeled as a 8.5 kg TNT. The arrival time, positive phase duration and overpressure are calculated by ELS ® software according to UFC guidelines [3] and are tabulated in Table 1.

Table 1

Blast load characteristics calculated in ELS ® software

Arrival time	0.361 ms
Positive phase duration	0.757 ms
Peak overpressure	257.6 kgf/cm^2

The values presented in Table 1 are in accordance withthe reference values presented in papers [7-8]. For example, the free-field overpressure, measured at 1.12 m behind the C4 charge, is $2800\,psi \cong 200\,kgf/cm^2$ at 0.35 ms [7-8]. The reflected overpressure, measured on the incident face, at the middle height of the front column, is $3800\,psi \cong 267\,kgf/cm^2$ [7-8].

The ¼ scale model has a 41 mm thick reinforced concrete slab [7-8]. The slab rests on a beam in the façade, and is having drop panels of 30x30 cm plan dimensions in the back [7-8]. The drop panels have an overall thickness of 52mm [7-8]. For simplicity, the numerical model replaces the back drop panels by another beam, similar to the façade one, as depicted in Fig. 5.

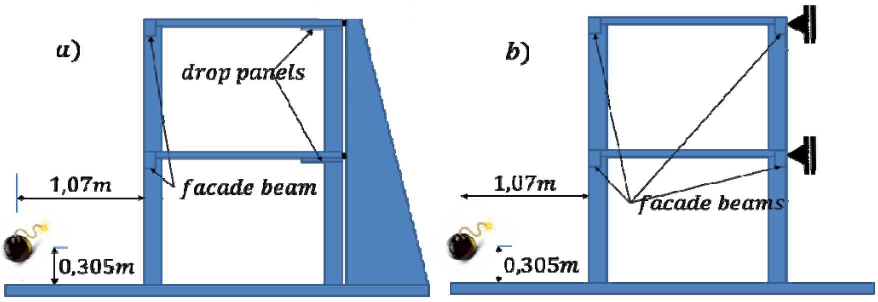

Fig. 5 - a) Experiment setup b) numerical model

The modeling of the reinforcement of the slab follows the experimental setup, with continuous bottom reinforcement and top reinforcement for negative bending moment, connecting the slab to the façade beams. The slab is divided in 15 elementsinthe direction of the blast front and 22 elements along the beams. The total number of elements used to model the structure is 1494, a much smaller one compared to 4 million elements used in papers [7-8] to model a half of the structure. The model described in papers [7-8] consists of the blast front propagation medium as well as the structure itself, so it's a more refined one.

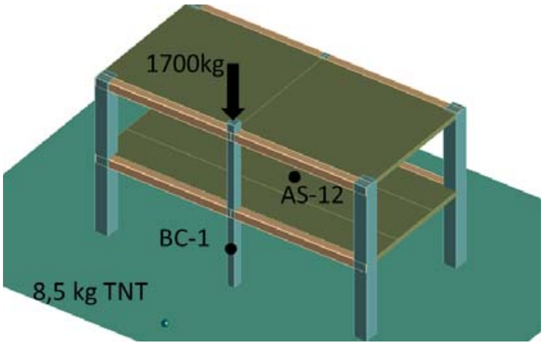

Fig. 6 - Numerical model developed in ELS ® software

The integration step recommended in [4] is 1/50.. 1/100 from the positive phase duration, in this case being considered $3 \cdot 10^{-5}$ s. In order to obtain the structural response in a reasonable analysis time with limited calculation resources, the load is applied in two stages having differentiated time steps, as tabulated in table 2.

Table 2

Time steps used in the numerical model developed in this paper

Stage	Duration	Time step
1 – Loading of the structure with overpressure	$0 \rightarrow 0.04$ s	$3 \cdot 10^{-5}$ s
2 – Vibration of the structure near a new equilibrium position	$0.04 \rightarrow 1.04$ s	$1 \cdot 10^{-3}$ s

3. Numerical results

The results are compared in terms of displacements measured at the half-height of the central (primary) column situated in façade of the ground floor (point BC-1), and point AS-12, situated at the half-span of the above ground floor slab. Fig. 7 depicts the deformed shape at different time intervals.

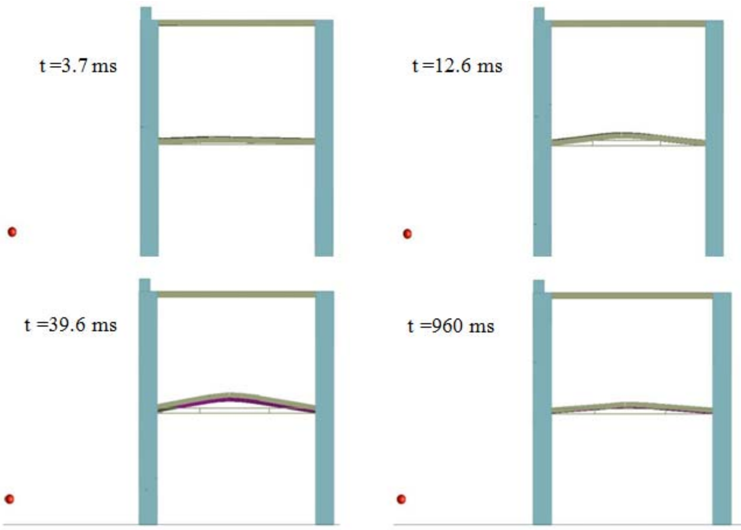

Fig. 7 - Deformed shape at different time intervals

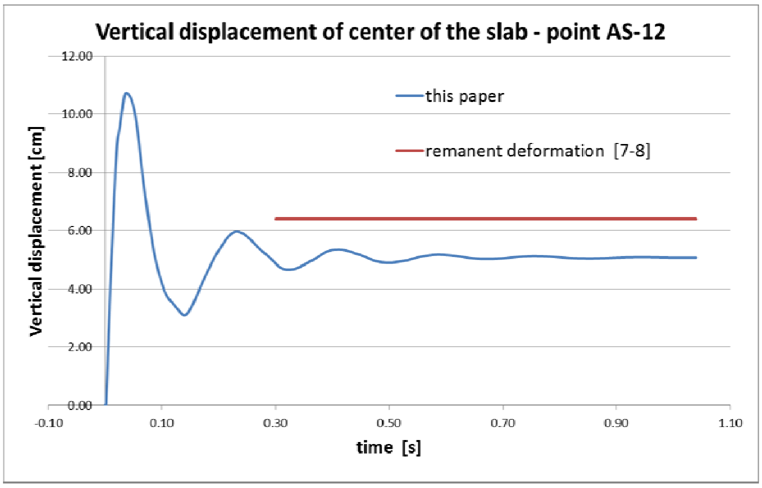

Fig. 8 - Variation of the vertical displacement of the center of the slab

Fig. 8 depicts the variation of the vertical displacement of the center of the slab both numerical and experimental. Papers [7-8] report a vertical residual displacement of 64 mm while in the numerical model a residual displacement of 52 mm is calculated. The 18% difference is a result of replacing the drop panels with a beam similar to the façade one, as shown in Fig. 5. The beam is having a larger bending and torsion stiffness than the drop panels, conducting to a stiffer slab-beam assembly.

Papers [7-8] note that the primary column, i.e. the central column located in ground floor façade, is subjected to bending, having a residual horizontal displacement of 6.3 mm. The maximum horizontal displacement is 0.5 inch ≅ 13 mm, as shown in Fig. 9. Although the numerical model mimics the instant increase of horizontal displacement over the 0..100 ms time interval, thus being in accordance with the experiment, the torsional stiffness of the beam-slab assembly is higher than the experiment and it does not allow the horizontal residual displacement to stabilize itself. The horizontal displacement of section BC-1 obtained numerical in this study is depicted in Fig. 10.

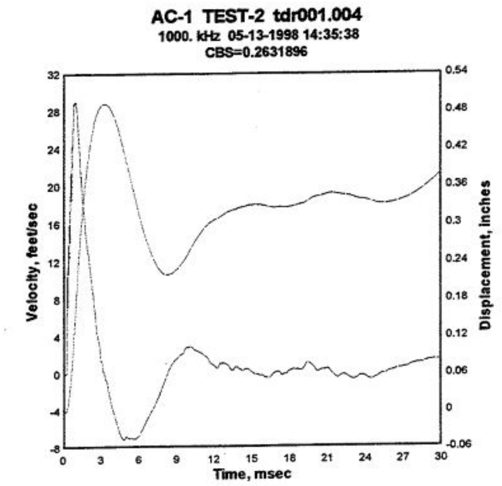

Fig. 9 - Horizontal displacement of section BC-1, located at the half-height of primary column [7-8]

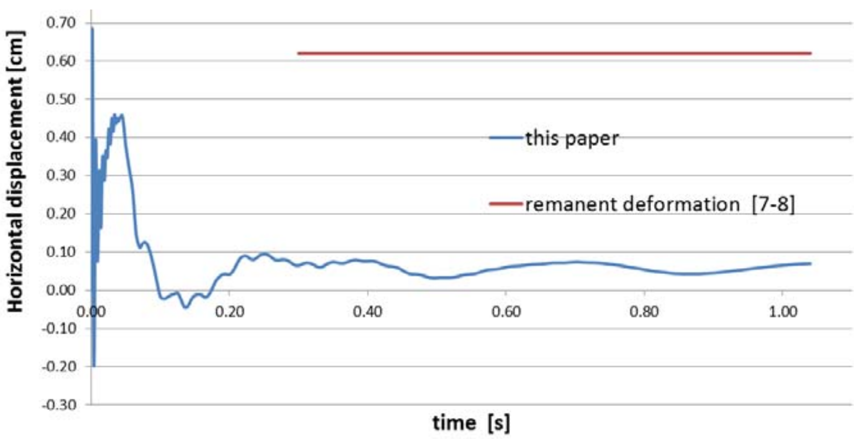

Fig. 10 - Variation of the horizontal displacement of section BC-1 obtained in this study

4. Conclusions

This short paper presents the numerical modeling of the slab failure for an open RC structure acted by blast loads. The blast front passes through the structure deforming the slab above the ground floor. The uplift vertical displacement obtained numerically is in accordance with the experimental data. The small difference is the result of replacing the back drop panels of the slab with a beam similar to the façade one. The residual displacement of the middle-height cross-section of the primary column is not captured by the numerical model.

The severe uplift of the slab may be avoided by reinforcing details as described in Romanian National Annex – accidental loads of the Eurocode EN 1991-1-7, namely top reinforcing bars should be placed in the span of the slab.

This work can be continued by modeling the experimental setup no. 3 described in the papers [7-8], in which the same structure with façade masonry walls is acted by the same amount of C4 charge as in this paper. The experiment described in [7-8] shows that the masonry walls reflect the blast front, thus protecting the slab above the ground floor. This reflection leads to complete failure of the masonry walls which become secondary missile objects that can injure the people inside the structure. Also, by reflecting the blast front, a reflected overpressure acts on the incident face of the primary column, leading to the complete failure of it.

Acknowledgements

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References

[1] Nica, G.B., (2013), Mathematical modelling of shock waves generated by blast events and their effect on concrete structures, 4th conference of young researchers from TUCEB , ISSN 2069-1793
[2] Nica, G.B., (2016), Studiul efectelor exploziilor la structuri din beton armat, Phd Thesis, Technical University of Civil Engineering, Bucharest.
[3] Cormie, D., Mays, G., (2009), *Blast effects on buildings*- second edition, Thomas Telford.
[4] Extreme Loads for Structures, (2013), Technical Manual
[5] Salem, H.M., El-Fouly, A.K., Tagel-Din, H.S., (2011), Toward an economic design of reinforced concrete structures against progressive collapse. EngStruct; 33:3341-3350.
[6] Salem, H.M., Helmy, H.M., (2014), Numerical investigation of collapse of the Minnesota I-35W bridge. EngStruct; 59:635-645
[7] Woodson, S.C., Baylot. J.T., (1999), Structural Collapse: Quarter-scale Model Experiments - US Army Corps of Engineers, Engineer Research and Development Center, Technical Report SL-99-8 august 1999
[8] Baylot, J.T., Bevins T.L., (2007), Effect of responding and failing structural components on the airblast pressures and loads on and inside of the structure - Computers and Structures 85 (2007) pages 891–910