

Numerical analyses for the structural assessment of steel buildings under explosions

Pierluigi Olmati^{*}, Francesco Petrini^a and Franco Bontempi^b

Department of Structural and Geotechnical Engineering, Sapienza University of Rome, Rome, Italy

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Abstract. This paper addresses two main issues relevant to the structural assessment of buildings subjected to explosions. The first issue regards the robustness evaluation of steel frame structures: a procedure is provided for computing “robustness curves” and it is applied to a 20-storey steel frame building, describing the residual strength of the (blast) damaged structure under different local damage levels. The second issue regards the precise evaluation of blast pressures acting on structural elements using Computational Fluid Dynamic (CFD) techniques. This last aspect is treated with particular reference to gas explosions, focusing on some critical parameters (room congestion, failure of non-structural walls and ignition point location) which influence the development of the explosion. From the analyses, it can be deduced that, at least for the examined cases, the obtained robustness curves provide a suitable tool that can be used for risk management and assessment purposes. Moreover, the variation of relevant CFD analysis outcomes (e.g., pressure) due to the variation of the analysis parameters is found to be significant.

Keywords: blast engineering; robustness; progressive collapse; blast action; gas explosions; steel buildings

1. Introduction

An explosion event, having a low probability of occurrence but severe consequences, due to a possible structural collapse, is a so-called “Low Probability and High Consequences (LP – HC)” event (Bontempi 2010, Starossek 2009). The high potential losses associated to these events (HSE 2011) make the design of structures for blast loads a key issue. In this respect, three main concepts are particularly relevant: i) the structural robustness evaluation (Bontempi *et al.* 2007, Yagob *et al.* 2009, Ghosn *et al.* 2010); ii) the quantitative assessment of the blast pressure on the structural elements (UFC 2008, Dusenberry 2010); iii) the evaluation of the coupling between the dynamic behavior of the structure and the surrounding air moved by the blast (Fluid-Structure Interaction - FSI) (Subramanian *et al.* 2009, Teich and Gebbeken 2011).

In a damage-based definition (Starossek and Haberland 2011), the structural robustness is the ability of a structure to withstand local damages without suffering disproportionate consequences (see e.g., Giuliani 2009, Brando *et al.* 2012). Strategies and methods for the robustness

^{*}Corresponding author, Ph.D. Candidate, E-mail: pierluigi.olmati@uniroma1.it

^aAssociate Researcher, E-mail: francesco.petrini@uniroma1.it

^bProfessor, E-mail: franco.bontempi@uniroma1.it

achievement are discussed, within a dependability framework adapted from the electronic engineering field, in Gentili *et al.* (2013) and Sgambi *et al.* (2012). In case of sudden loss of one or more structural elements (typical failure in case of explosions), robustness is related to the susceptibility of the system towards progressive or disproportionate collapse (UFC 2009).

Regarding the second topic outlined above, i.e. the assessment of the blast action due to gas explosions, it is important to highlight the fact that the blast pressure on structural members varies significantly depending on some particular environmental characteristics, such as room congestion, failure of non-structural walls and ignition point location. The realistic modeling of the blast load is therefore fundamental for the reliable evaluation of the structural response, something particularly difficult in cases of blast scenarios characterized by a complex geometry. In this respect, a promising tool is provided by Computational Fluid Dynamic (CFD) simulations (Molkov 1999).

The level of room congestion is an important factor, whose effects can be adequately accounted for performing CFD analyses (Pritchard *et al.* 1996). In domestic gas explosions the obstruction provided by furniture and objects in general causes an increase of the flame turbulence and, as a consequence, an increase of the flame velocity, something that leads to an increase in the pressure (Zipf *et al.* 2007).

Concerning the third topic (the evaluation of the FSI), as reported in Teich and Gebbeken (2011) it can be stated that while the coupling effects are negligible for stiff or heavy systems (e.g., reinforced concrete structures) they significantly influence the structural response of flexible and light systems, e.g., membrane structures or glazing facades. For flexible systems, the FSI determines the pressures acting on structural elements and generates an additional (aerodynamic) structural damping, leading to a different structural response than what obtained by neglecting this effect. In general, the FSI effects are more relevant in case of blast loads characterized by low frequencies or low intensity (Subramanian *et al.* 2009, Teich and Gebbeken 2011).

This study deals with both the evaluation of structural robustness under blast damage scenarios and the gas explosion simulation using CFD techniques. To this purpose, numerical analyses for assessing the progressive collapse under blast damage scenarios are shown and detailed simulations of gas explosion are carried out. The FSI effects are not treated in this paper and are being developed in further studies carried out by the authors in the ongoing research project on analysis of structures subjected to blast loads.

The analyses carried out in this paper are part of a framework for the structural risk assessment under blast, since they are useful in the evaluation of some particular aspects of risk (e.g., progressive collapse vulnerability given a local damage). The structural risk under blast can be evaluated by the probability of progressive collapse for the structure after an explosion. Considering a set of blast scenarios, this probability is given by Ellingwood *et al.* (2007)

$$P[C] = \sum_i P[C|LD]P[LD|H_i]P[H_i] \quad (1)$$

Where H_i is the hazard related to the blast scenario “i” (where the scenario is defined by the parameters determining the intensity of the blast action), also known as Intensity Measure (IM) in other engineering fields (Whittaker *et al.* 2003, Ciampoli *et al.* 2011), LD is the structural local damage, C is the collapse event, $P[\cdot|\cdot]$ indicates a conditional probability, $P[\cdot]$ indicates a probability, and the summation \sum is (in theory) extended to all scenarios.

The structural analyses carried out in this paper, can be useful in assessing whether a certain local damage is able to activate a global collapse (probabilistically considered in Eq. (1) by the

term $P[C|LD]$), and in assessing the hazard associated with different blast scenarios (probabilistically considered in Eq. (1) by the term $P[H]$). For a realistic evaluation of $P[LD|H]$, FSI models should be implemented, an aspect neglected in this study. This simplification is considered acceptable due to the considerable stiffness of the structure considered as case study, but further studies are needed to verify its validity. The probability of progressive collapse $P[C]$ is not computed in this paper, which focuses on the deterministic numerical analyses that can be useful for the structural risk assessment under blast. The paper has two aims:

- Provide a tool for the global evaluation of the robustness of steel frame structural systems subjected to blast-induced damages in terms of “robustness curves”, by extending the procedure proposed in Giuliani (2009) (not originally related to explosions).
- Quantify the variation of relevant CFD analysis outcomes (e.g., pressure), due to the variation of the analysis parameters (e.g., room congestion), using sensitivity analysis. This is something not well investigated in literature, especially for civil structures.

2. Evaluation of the structural robustness under blast damage

A way to characterize the behavior of buildings subjected to explosion is to compute the dynamic structural response due to a local damage (assumed to be caused by a blast) and consequently assess the robustness of the structure. The structural robustness can be assessed by evaluating the residual load bearing capacity of the damaged configurations as illustrated in Giuliani (2009), where the analysis is based on the assumption of different levels of damage in various locations. The robustness evaluation procedure presented in the following is based on the assumption of a certain damage level caused by a generic load, which is able to instantaneously cut off the contribution of a structural element to the load bearing capacity of the system (Yagob *et al.* 2009, CPNI 2011). Therefore, the method proposed can be used for the design against actions generated both by intentional and accidental explosions and by hazards of different type (e.g., impact). Focusing on steel frame building systems (such as the one studied in this paper, shown in Fig. 1), whose key structural elements are the columns at the ground floor (Parisi and Augenti 2012, Almusallam *et al.* 2010, Valipour and Foster 2010), the local damage level can be identified by the number of the destroyed key elements. It is assumed that the columns directly acted upon by the blast wave are instantaneously destroyed, thus the case of partially damaged key elements is neglected in this study (Crosti *et al.* 2012). Following these assumptions, the first set of damage scenarios is defined by the removal of a single key element (column), the second set by the removal of two key elements, and so on.

Considering the above, two parameters identify the single damage scenario: the location of the first destroyed key element (L), and the local damage level (N) of the scenario (i.e., the number of the removed key elements). The specific local damage scenario is then identified as “D-scenario ($L = i$; $N = j$)”, where capital letters indicate parameters and lower case letters indicate the specific value assumed by the parameters. This means that the generic scenario is obtained by removing a total number of key elements equal to “ j ”, and that the first of these elements was the one positioned in the location number “ i ”. A set of initial N_L damaged key elements defines the D-scenario (L ; 1) with ($L = 1, \dots, N_L$; local damage level $N = 1$). These scenarios (L ; 1) can be chosen a priori by considering the explosion type (e.g., it is realistic to assume that gaseous explosions take place in kitchens or boiler rooms, while intentional explosions take place in the external perimeter of the building). Moreover the generic D-scenario (i ; $j + 1$) corresponding to the

local damage level $N = j + 1$, is heuristically obtained from the previous D-scenario ($i; j$) by removing the most stressed key element (critical element for the D-scenario ($i; j$)) obtained by carrying out a Nonlinear structural Dynamic Analysis (NDA) (Vassilopoulou and Gantes 2011) of the D-scenario ($i; j$). The non-linearity is due to both the inelastic behavior of the steel and the large displacements; the analyses are conducted by considering the effects of live and permanent loads with the associated masses.

As an outcome of the NDA to the D-scenario ($i; j$) the damaged structural configuration may have two kinds of response: a) the critical element for the D-scenario ($i; j$) does not collapse (i.e., it remains under a certain conventional response threshold - “arrested damage response”) or, b) the critical element for the D-scenario ($i; j$) collapses (i.e., it exceeds the conventional response threshold - “propagated damage response”). The sequential steps of the procedure are the following:

- For damaged configurations leading to a “propagated damage response”, the progression of the collapse is presumed, and the computation proceeds by changing the damage location $L = i + 1$. This assumption should need to be verified since, in general, the fact that another element is failing in consequence of an initial damage, it does not necessary mean that a progressive collapse is triggered. However the Authors’ opinion is the indirect failure of a key element should be avoided.
- For damaged configurations leading to an “arrested damage response”, two steps are carried out:

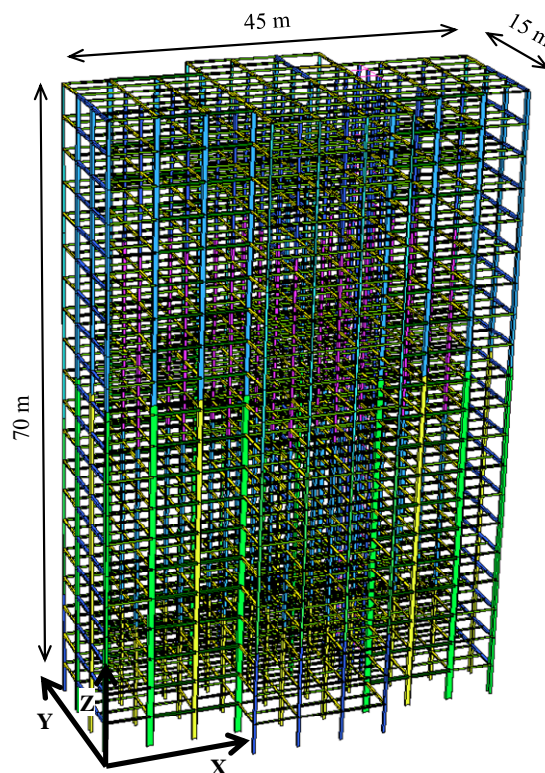


Fig. 1 FE model of the building

- The residual strength is computed, this can be estimated in different ways. Here the so-called “pushover analysis” (Pinho 2007, Kalochairetis and Gantes 2011, Lignos 2011) is used, and the residual strength for the D-scenario (i, j) is identified by the ratio $\lambda_{\%}(i, j)$ between the ultimate load multiplier of the damaged structure and the one corresponding to the original undamaged one. The residual strength (pushover) analysis is carried out under horizontal loads having a triangular distribution along the height of the building. This choice is made with two motivations in mind: i) horizontal loads can activate both horizontal and vertical load bearing structural systems of the building and, ii) direct reference is made to the unlikely eventuality that a seismic aftershock occurs after an explosion. This event is possible in the case that the explosion occurs after a seismic main shock (e.g., hydrogen explosions caused by the Japan 2011 earthquake main shock).
- The increase of the local damage level ($N = j + 1$) as stated before (i.e., by removing the critical element for the D-scenario (i, j)), and performing a new NDA.

After each “propagated damage response” a robustness curve is obtained, defined by the variation of the ratio $\lambda_{\%}(i, j)$ with the local damage level (N). Once all the N_L locations have been analyzed a set of curves describing the robustness of the structure under the considered damage scenarios are obtained (see example of Fig. 2). The whole procedure is summarized in the flowchart of Fig. 3. The outcome of the analysis gives a representation of the structure robustness when it is damaged by a blast in the considered locations. These robustness curves under blast damage scenarios are useful for:

- Risk assessment analysis, if the uncertainties affecting the structural response after a local damage LD (e.g., due to the uncertain structural characteristics) are considered (e.g., by a Monte Carlo analysis, see Petrini and Ciampoli (2012)). The dispersion of the robustness curves for that damage can be useful in evaluating the element $P[C|LD]$ of Eq. (1).
- Risk mitigation analysis, for planning the optimal strategy against the hazard (FEMA 2003), for example by adopting adequate structural or non-structural measures (UFC 2010) focusing on the most critical scenario indicated by the robustness curves (the scenario producing the less robust structural response).

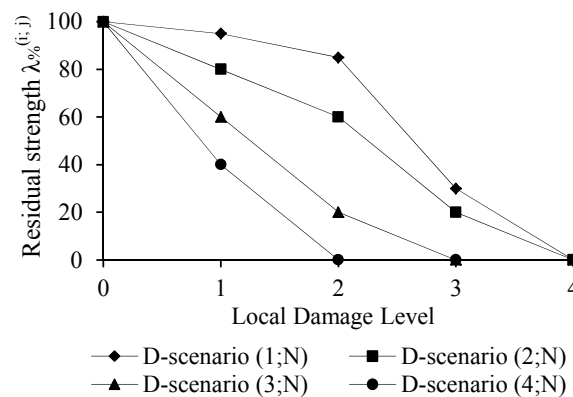
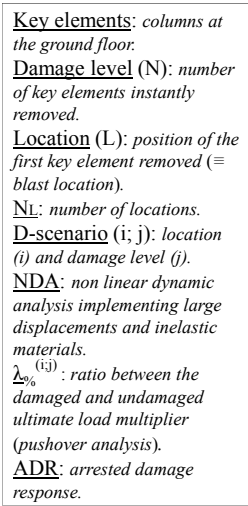


Fig. 2 Examples of robustness curves under blast damage scenarios



3. Gas explosions: main simulation parameters

As stated before the CFD simulation is a powerful tool for obtaining an accurate evaluation of the blast pressure on the structural elements. Several modern CFD codes allow taking into account

accurately the effect of some fundamental phenomena:

- The congestion in the environment. This issue has been extensively studied in industrial facilities and partially explored in civil structures (domestic congestion). The congestion is caused by the presence of all the objects inside a room. In the CFD codes the domestic congestion is implemented in the simulation by modeling solid objects whose effects (e.g., turbulence generation) can be considered as an interaction between flow and objects (Dobashi 1997).
- The failure of non-structural walls. When a gas explosion occurs, the failure of non-structural walls causes the modification of the geometrical scenario and consequently the development of the entire explosion. The non-structural walls are modeled in the CFD simulation by special objects having a cut off pressure level.
- The ignition type and position of the gas cloud ignition, which can vary on a case-by-case basis, and have an important role in the development of the explosion.

The aleatory uncertainty related to the above-mentioned issues is one of the principal reasons causing the variability of the intensity and direction of the blast action. Of course, a significant dispersion of results, especially in cases of complex numerical models such as the ones used for the simulation of gas explosions, is due to the epistemic uncertainty (e.g., model uncertainty). The issues related to this epistemic uncertainty are not addressed in this paper.

In the case of gas explosions, the “gas region” is defined as the volume containing the cloud of gas, usually assumed as homogeneous. This gas region can be characterized by the so-called “Equivalent Ratio” (ER) (GexCon-AS 2009)

$$ER = \frac{\left(\frac{V_{\text{fuel}}}{V_{\text{O}_2}} \right)_{\text{actual}}}{\left(\frac{V_{\text{fuel}}}{V_{\text{O}_2}} \right)_{\text{stoichiometric}}} \quad (2)$$

Where O_2 indicates the Oxygen molecule, V_i ($i = \text{“fuel”}$ or $\text{“O}_2\text{”}$) indicates the volume of the cloud component “i” at normal atmospheric conditions and the term “fuel” is referred to the flammable component of the gas cloud (e.g., methane, hydrogen, etc.). The term “actual” refers to the exploding cloud, while the term “stoichiometric” refers to the quantities of fuel and Oxygen needed for a balanced chemical reaction. In Eq. (2), the value of the denominator can be derived from literature, while the value of the numerator in the design phase can be chosen with reference to experimental data (see for example SFPE 2002), in order to maximize the initial (laminar) burning velocity, which represents an important aspect for the determination of the severity of the explosion.

4. Application on a steel building

The procedure introduced above for evaluating structural robustness under blast damage has been applied to a case study building. On the same building, a number of CFD analyses have been then performed in order to evaluate the effects of the previously mentioned environmental parameters (see section 3).

The case study building is an office structure 70 meters high for a total of 20 story, each one being 3.5 meters high. The layout is rectangular with two protruding edges on the longest side and

is globally delimited in a 45×25 square meters area (see Fig. 1 and Fig. 4). Columns and beams have European HE cross-sections. The beam-column and beam-beam connections are made by double angle cleat connections (shear-resisting), while the column-column connections are moment-resisting welded and bolted connections. In addition to the previous sub-systems, appropriate braced walls are present in order to support the horizontal loads, these having beam-column moment resisting connections, and diagonal tension members that consist in 2L 100x50/8 profiles; the position of the bracing systems is shown in Fig. 5. The floors have a horizontal braced system that is formed by a set of members having an L 100x50/8 profile. The column cross-section shapes are shown in Fig. 5 and classified and grouped in Table 1; for each column type (A, B, C, D) the size of the cross section decreases through the building height. The slab is a steel ribbed slab, spanning North to South, simply supported by girders and beams (see Fig. 5). The girders cross-sections are HEA 240, spanning North to South, while the floor beams cross-sections are HEA 200, spanning West to East. The girders and beams belonging to the braced walls have a HEB 300 shape. The girders and beams have a span of 5 meters and are placed at 5 meters and 2.5 meters steps respectively. A grade S235 steel is adopted, with a yielding (f_{yk}) and ultimate (f_{uk}) stress equal to 235 and 360 N/mm² respectively.

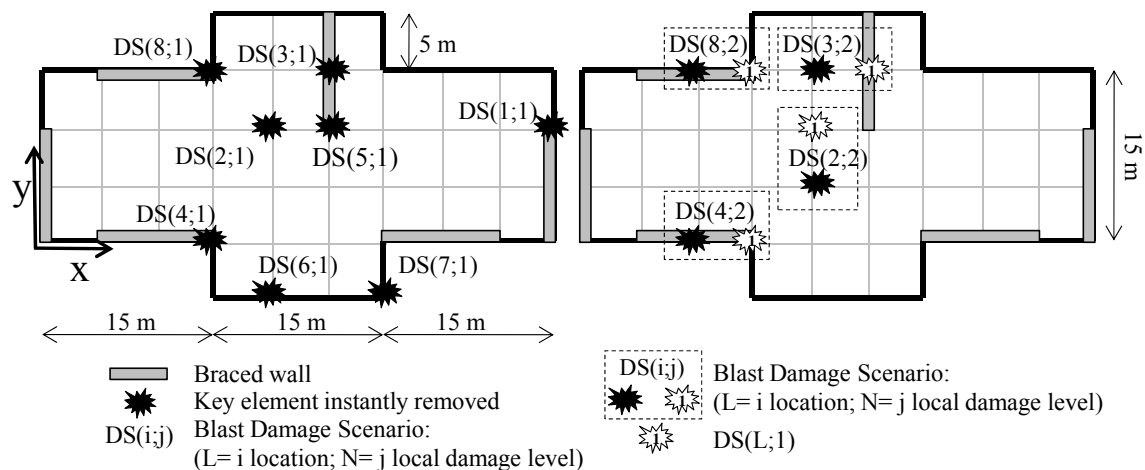


Fig. 4 Damage scenarios (L; 1), and Damage scenarios (L; 2)

Table 1 Column cross sections

Quote [m]	Column type			
	A	B	C	D
0 - 12	HEM 550	HEX 700×356	HEB 300	HEM 550
12 - 33	HEB 550	HEX 700×356	HEB 260	HEM 550
33 - 39.5	HEB 320	HEX 700×356	HEB 240	HEM 550
39.5 - 70	HEB 300	HEB 550	HEB 240	HEB 550

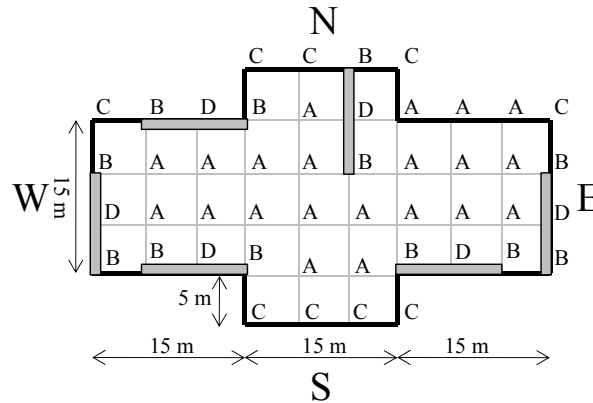


Fig. 5 Column types

4.1 Robustness assessment

In what follows, direct reference is made to the flowchart of Fig. 3. Eight locations (L) have been considered ($N_L = 8$) defining the blast damage scenarios, as indicated in Fig. 4. As previously stated, the columns at the ground floor have been considered as key elements and the numerical investigations are carried out removing instantaneously the key element by NDA. In Fig. 1 the finite element model of the structure developed with Straus7[®] (G+D Computing HSH 2004) is shown. Only the frame system is explicitly modeled, both the floors and the live load are taken into account by considering additional (fictitious) mass density on the beams. The building is subjected to gravity and the structural properties of the cladding system are not considered. The structural response to the D-scenario (i; j) is evaluated by carrying out non-linear Lagrangian (Bontempi and Faravelli 1998) dynamic implicit FE analyses. Explicit FE solver (more capable in evaluating triggering effects due to local collapses) has been avoided in order to limit the computational efforts. The use of implicit method is also justified by the fact that, in this specific structure, the failure (assessed by implicit analyses) of some structural key elements can be conceptually associated to the propagation of the collapse to other structural parts supported by the key elements.

An initial damage is considered in the FE model by replacing a column by its reaction force (computed with the Dead + Permanent + 0.3 Live load combination). In order to minimize inertial effects caused by this loading phase, a sufficiently slow load ramp is provided. Moreover, a successive oscillation extinction phase is added (see Fig. 6) where the load factor is maintained equal to 1. After that the reaction of the key element is suddenly removed to simulate the damage with a time interval (Δt) smaller than 1/10 of the fundamental time period associated with the pertinent vertical modal shape of the damaged structure (UFC 2008). The implemented load factor time history is shown in Fig. 6.

Concerning the material and geometrical nonlinearities, the distributed plasticity model along the length of the beams and the large displacements assumption are adopted. In Fig. 7 some moment-curvature diagrams for girders and beams are shown, the softening behavior is not implemented.

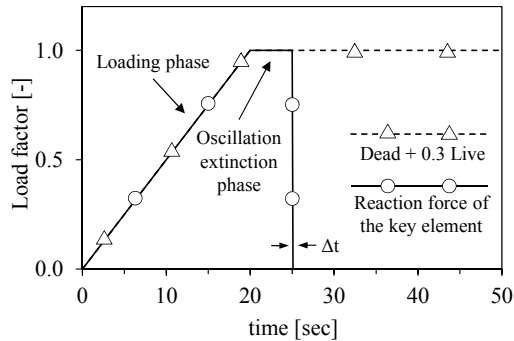


Fig. 6 Load factor time history chart

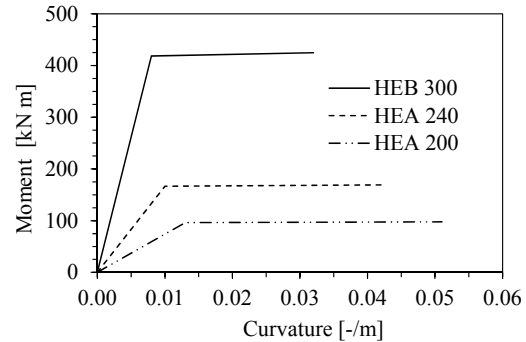


Fig. 7 Moment-Curvature diagrams

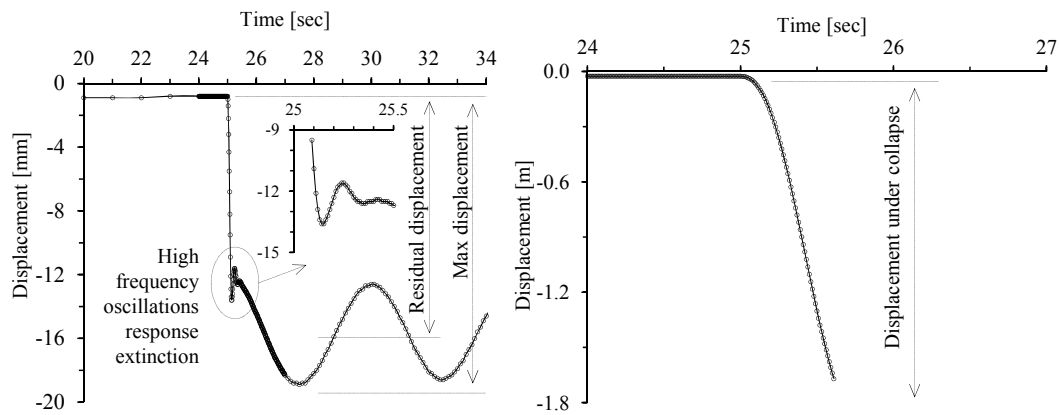


Fig. 8 Response time history for a node on the top of the removed key element, D-scenario (5;1) (left, arrested damage response) and D-scenario (6;1) (right, propagated damage response)

The typical vertical displacement time history for a node located on the top of the removed key element is shown in Fig. 8 for both the “arrested damage response” and “propagated damage response” with local damage level $N = 1$. The first one, after the extinction of the initial high frequency oscillation, shows a decaying response (damped oscillation), while the second one shows an unbounded response. The computed robustness curves are shown in Fig. 9. It results that for the selected scenarios, with the local damage level equal to two (D-scenario (L; 2)), the structure always shows a “propagated damage response” (i.e., a progressive collapse is presumed). In some cases (D-scenarios (6; 1) and (7; 1)) the collapse progression occurs even at the first level of local damage. This behavior occurs in all cases where the local damage is located in the external columns of the building that are not part of braced walls (see Fig. 4). In fact, in all other D-scenarios, for local damage level $N = 1$ the load originally carried by the removed key element is re-distributed to a number of adjacent key elements positioned in both floor directions (x and y in Fig. 4) with respect to the removed one, this allows the development of a double catenary effect (in x and y directions), something that is not realized in the critical D-scenarios (6; 1) and (7; 1).

As stated before, the results of the robustness analysis under blast damage scenarios can be useful in planning an optimal risk mitigation strategy. In the present case for example, the measures of mitigation should primary focus to the external columns.

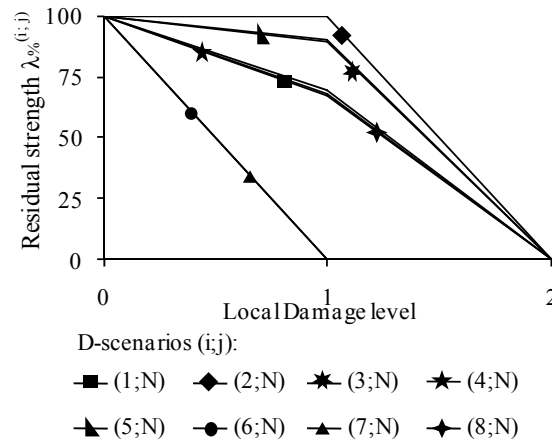


Fig. 9 Robustness curves under blast damage

4.2 Blast action evaluation by CFD

With reference to the three fundamental phenomena outlined in section 3 (congestion of the environment, failure of non-structural walls, ignition type and position), a number of CFD analyses is carried out in order to evaluate the effects of the scenario parameters on the blast load. The analyses are carried out using the CFD commercial code Flacs[®] (GexCon-AS 2009). A set of gas explosions at the ground floor are modeled where the presence of some commercial activities and one restaurant are hypothesized, including the kitchen (where the ignition point and the gas region are located). Methane is assumed to be the fuel and the equivalent ratio (ER), see Eq. (2), is assumed equal to 1.12, in order to maximize the initial laminar burning velocity. The main features of the blast scenario are shown in Fig. 10. Different CFD models are considered and they are resumed in Table 2.

The room congestion is realized by rigid furniture, modeled by still filled blocks in the uncongested room. Only two room congestion cases have been considered, indicated respectively as “free room” (where the room is considered without furniture) and as “congested room” (where furniture is present, see Fig. 11)

When the failure of non-structural walls is considered (“frangible wall” cases), the walls are modeled by cut-off pressure panels that are able to increase the porosity of the walls from 0 (undamaged wall) to 1 (completely damaged wall) after the crossing of a threshold pressure level (wall strength). In the “non-frangible walls” cases the walls are undamaged (porosity equal to 0) in all the simulation. The parameters of the cut-off pressure panels are reported in Table 3 (Lees 1980). Six different ignition locations inside the kitchen have been considered and the ignition locations are shown in Fig. 12.

In order to obtain realistic results in a CFD explosion analysis, the adoption of an appropriate mesh grid is fundamental. The details of the mesh grid are the following: inside the building ground floor the edge of the cubical cells is always 0.20 meters, while outside the building the mesh grid is stretched in order to reduce the total number of cells. The max aspect ratio (the longest side of the control volume divided by the shortest one) is equal to 5.59, thus lower than 10, as recommended by GexCon-AS (2009), while the total number of cells is about one million. By

using a 3.6 GHz CPU computer with 4 Gigabytes of RAM the analysis time of the single scenario is approximately 12 hours.

Fig. 13 shows the effect of the domestic congestion. Simulations I and II have a different pressure peak due to the interaction between the flow and the objects inside the building. In the congested room case (simulation II) the turbulence of the flows increases, consequently both the burning velocity (see section 2) and the pressure increase as well, inducing more turbulence. By referring to a certain monitoring point inside the kitchen, the domestic congestion (simulation II) causes a 43% increase of the pressure peak and a 28% increase of the pressure impulse (area under the curves reported in Fig. 15) with respect to the uncongested case (simulation I).

In simulation III both failing walls and room congestion are considered. Comparing the results obtained in simulations II and III the percentage decrements in terms of pressure peak and pressure impulse (see Fig. 15) are equal to 66% and 77% respectively (Fig. 14). Moreover, the two explosions are completely different in the spatial development. These results indicate that the failure of the walls significantly modifies the explosion development.

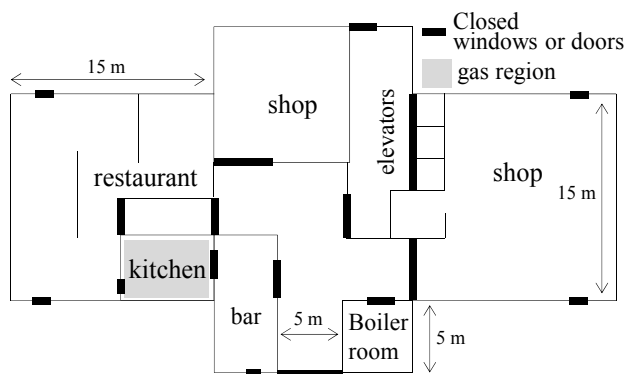


Fig. 10 Main features of the blast scenario

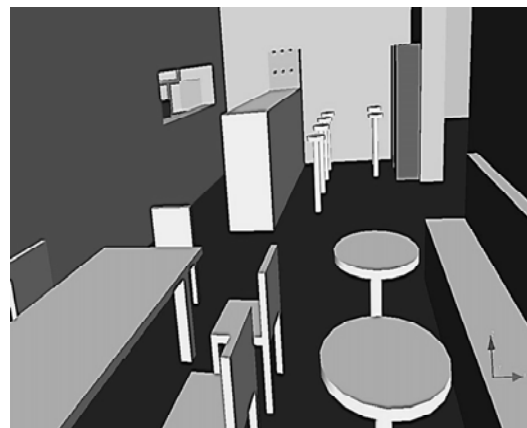


Fig. 11 Congested room model

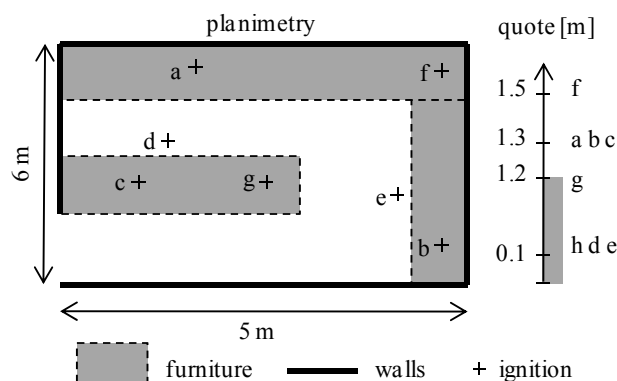


Fig. 12 Position of the ignition points

Table 2 Performed CFD analysis

Simulation	Type of walls	Room congestion	Ignition (Fig. 12)
I	Non-frangible	none	a
II	Non-frangible	yes	a
III	Frangible	yes	a
IV	Frangible	yes	b
V	Frangible	yes	c
VI	Frangible	yes	d
VII	Frangible	yes	e
VIII	Frangible	yes	f
IX	Frangible	yes	g

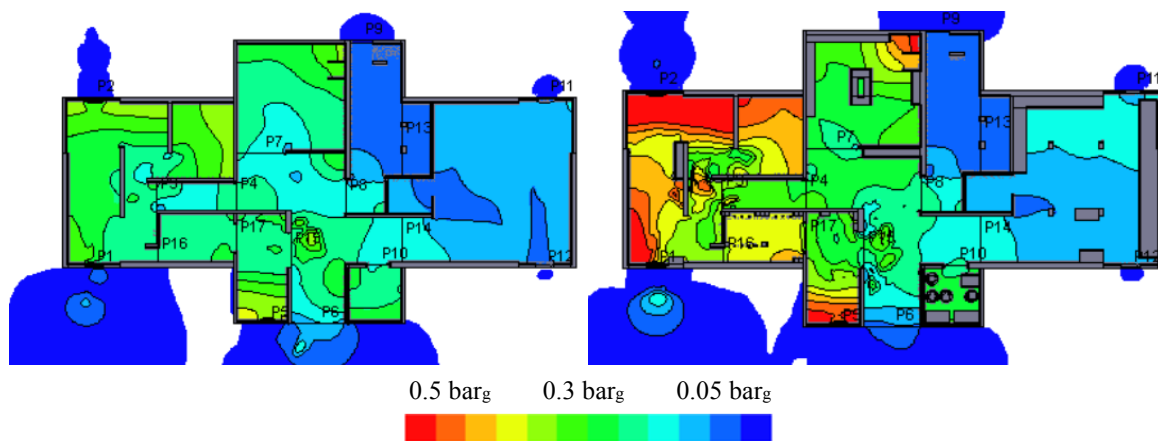


Fig. 13 Max pressures. Effect of the domestic congestion; simulation I on the left and simulation II on the right

In simulation III both failing walls and room congestion are considered. Comparing the results obtained in simulations II and III the percentage decrements in terms of pressure peak and pressure impulse (see Fig. 15) are equal to 66% and 77% respectively (Fig. 14). Moreover, the two explosions are completely different in the spatial development. These results indicate that the failure of the walls significantly modifies the explosion development.

Fig. 15 shows a comparison between the pressure time histories, in the same monitoring point, obtained by simulations I, II and III, where the previously mentioned differences can be appreciated. Moreover, Fig. 15 shows that both the pressure gradient and the time instant corresponding to the pressure peak are highly influenced by the congestion level. The increase of the congestion level produces an increase in the pressure gradient, thus anticipating the occurrence of the pressure peak.

All these results obtained by the CFD computations, clearly shown that the structural design against such type of explosions cannot be conducted carefully without a specific evaluation of the action. In terms of design practice, or design standards for civil buildings, these kinds of simulations can be useful for defining parametric equations with the aim of appropriately defining the blast load on structural elements, also by taking into account the above described phenomena.

For risk assessment purposes, due to the high variability of the action with the considered parameters, specific studies aiming in assessing plausible probability distributions for those parameters are needed for a correct evaluation of the hazard.

Table 3 Frangible objects characteristics

Panel	Mass [kg/m ²]	Pressure of opening [bar _g]
exterior walls	250	0.05
interior walls	100	0.03
windows	20	0.015
doors	2	0.001

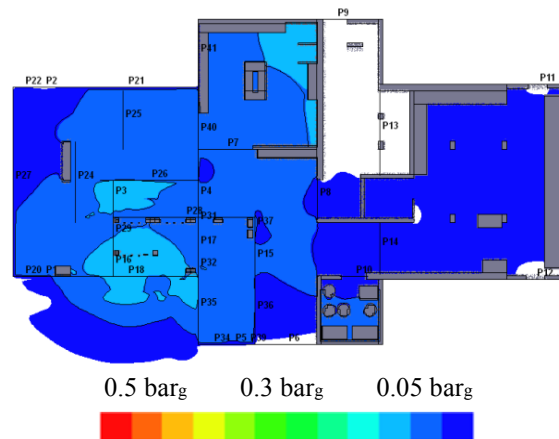


Fig. 14 Max pressures. Effect of the frangible walls: simulation III

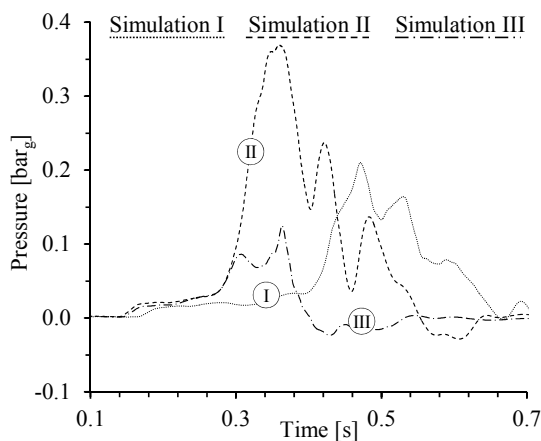


Fig. 15 Pressure time history in the kitchen (the gas region) for the three different simulations: I, II, and III

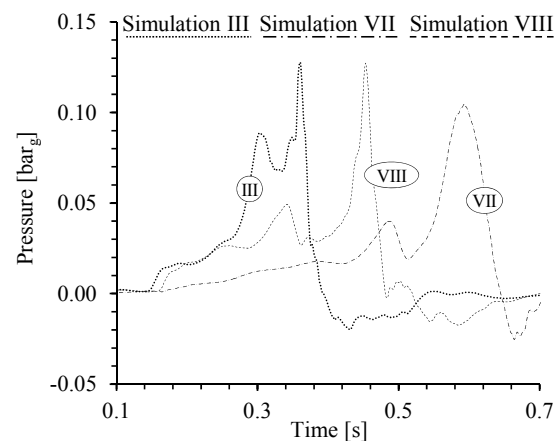


Fig. 16 Pressure time history inside the kitchen for three analyses with different ignition locations

5. Conclusions

The robustness curves obtained in this study form a suitable tool that can be helpful for risk management and assessment. The procedure can be employed to handle different hazards, such as terrorist attacks or accidental explosions. In particular, the proposed procedure for robustness assessment is based on the assumption that the structural members directly involved in the blast fail instantaneously, without any prior evaluation on the blast intensity. Since in the recent years a number of intentional explosions were caused by truck bombs near the buildings, leading to the failure of some columns, the previous mentioned assumption seems particularly reliable in case of intentional explosions. The same approach could be extended for computing the robustness curves in case of structures subjected to impact of ships and vehicles that engage key structural elements.

The approach based on the removal of key structural elements and on the subsequent investigation of the dynamic structural nonlinear behavior has been adopted by different authors (see for example Yagob *et al.* (2009), Purasinghe *et al.* (2012), Weerheijm *et al.* (2009), and Sasani *et al.* (2011)), and has been implemented in guidelines (e.g., UFC 2009). To this regard, the novelty offered by this paper consists in describing the results of this analysis in a synthetic and fruitful manner, provided by the computation of the robustness curves. Moreover, the proposed method of evaluation by means of robustness curves, takes into account the dynamic effects of the structural initial damage by evaluating the structural behavior under impulsive loads.

This can be particularly useful for the structural risk assessment against intentional explosions. With respect to natural explosions (e.g., gas) instead, a useful tool for the action intensity evaluation is given by the CFD analyses illustrated in this study.

In the case of gas explosions, the intensity of the blast load depends principally on the activities inside the building and the explosion occurs mainly in well-identifiable locations, therefore advanced CFD analyses are particularly suitable to provide realistic assessment of the blast action which is essential for the assessment of the consequent local damage. From the analyses developed in this study emerges that, at least for the examined cases, the variation of relevant CFD analysis outcomes (e.g., pressure) with the parameters mentioned above is indeed significant.

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