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ROBUSTNESS OF RC FRAMES UNDER EARTHQUAKE AND BLAST CHAINED SCENARIOS

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Abstract

In this paper the structural robustness assessment of concrete frame buildings under blast and under earthquake blast hazard chain scenarios is investigated. A deterministic methodology for connecting the robustness with the blast hazard intensity and for conducting the robustness analysis under earthquake-triggered blast is presented and applied to a 3D RC frame building by implementing nonlinear time history analyses considering both plastic behavior and large displacements. The robustness curves (residual structural capacity versus the level of damage occurring in the structure), evaluated both for the blast-only and for the earthquake-blast chained cases, are compared by considering different explosion locations inside the building (location of the blast-induced structural damage). Results show that neglecting the chained load scenarios would lead to the identification of an erroneous location as critical for the structural robustness performance..

Keywords: Structural Robustness; Multi-Hazard; Blast; Earthquakes.

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1 INTRODUCTION

Structural robustness is a global attribute required to structures when subjected to a local damage which, if not confined, can spreads inside the structural system until disproportionated damage or progressive collapse occur with the consequent mining of the global integrity ([1]). The structural robustness is commonly assessed by a well-established procedure called "damage-presumption approach", that is a "hazard-independent" analysis where a certain damage level is assumed for the structure (for a framed structure typically consisting in the removal of a column), and the residual capacity of the structure is then evaluated. Even if widely used in the literature, common damage-presumption approaches, indeed, do not directly and univocally link the presumed damage to a specific hazard and its intensity.

In addition, some natural disasters occurred in recent years shown that most dangerous scenarios for structural robustness performances are those called "hazard-chains" ([2]), where a certain initial hazard strikes on the structure, which can suffer a certain damage from it, and then a subsequent hazard impacts as well, affecting the structure already damaged by the previous one. The initial hazard of the chain has multiple consequences which can affect the structural robustness in the hazard-chain scenario ([3]):

- first, as already said, it can induce a damage who decreases (and in generally changes) the structural robustness of the structure to the subsequent hazard;
- second, the initial hazard can "trigger" the subsequent one. This is the case where a strong seismic shaking can damage electrical system or gas pipes inside the building, causing earthquake-induced fires or explosions.

When, in the literature, the behavior of structures in case of explosions is extensively discussed ([4]), the explosion is therefore not intended triggered by another main hazard.

In this paper, the structural robustness analysis of a RC frame building under blast and under earthquake-induced blast hazard chain scenario is conducted. All the analyses of this paper are conducted in deterministic terms, in case of hazard chains it is assumed that a high intensity earthquake striking the structure causes a subsequent explosion compromising the integrity of the structure.

Specific sensitivity analyses are carried out to highlight the issues arising when the robustness performances must be correctly linked to the considered hazards, by making specific reference to the blast-induced damage, and by introducing novel "blast scenario-dependent robustness curves". Then the design implications related to the consideration of earthquake induced explosion scenarios are highlighted by carrying out robustness analyses under hazard-chain scenarios and providing new insights regarding the multi-hazard structural design.

2 NUMERICAL PROCEDURE FOR STRUCTURAL ROBUSTNESS QUANTIFICATION OF RC FRAMES BY THE DAMAGE PRESUMPTION APPROACH

As already said, in the damage presumption approach ([26]), the dynamic response of the frame under the sudden removal of a number "n" of columns is evaluated by a nonlinear dynamic analysis (NDA) starting from the static equilibrium configuration reached by the non-damaged structure under vertical loads (generally due to the seismic "permanent+0.3*variable" mass combination). By the sudden column removal, the NDA can lead to ([5]): i) a new static equilibrium condition characterized by some residual plastic displacements after an initial damped transitory phase or, ii) the collapse of the structure.

The collapse can be defined to occur when: a) the time-response and/or the load-response diagrams are unconfined by certain boundary limits ([6]) (runaway behavior), or b) the vertical relative drift (DV) between the beam-column nodes located around the removed column reaches the value of 20%, or c) the vertical displacement ratio (largest ratio of nodal vertical displacement to the story height) at the base floor, exceeds a specific threshold of 0.5 ([6]).

If the outcome of the NDA is not the collapse, an incremental static nonlinear analysis of the structure is carried out under lateral forces (pushover) in order to evaluate the residual capacity of the damaged structure. Also if common approaches use the pushdown analysis to assess the residual capacity of the structure under gravity incremental loads ([7]). For example by carrying out a pushover, the damage and the capacity under earthquakes can be coherently considered in the robustness analysis. This is particularly significant in earthquake-induced explosions, where the evaluation of the residual capacity of the blast-damaged structures under earthquake aftershocks is crucial. In this way, each damage level "DL" (i.e., number "n" of simultaneously removed columns) is associated to a residual lateral force capacity (λu) as evaluated by the pushover and expressed as percentage ($\lambda u/\lambda$ %) of the pushover capacity (λ) of the non-damaged structure.

As a result of the aforementioned numerical outcome, the deterministic robustness of the structure can be quantified and efficiently represented by the so-called "robustness curves" as introduced by Olmati et al. (2013) [8]. Robustness curves are represented on a cartesian plane in which on the x-axis there is the DL suffered by the structure ("n" in the text above), while on the y-axis the corresponding residual force capacity percentage ($\lambda u/\lambda$ % in the text above) is reported. See for example the robustness curves represented in Figure 1, where different colors and markers represent different locations along the structure for the presumed damage initiation.

The main indications about the global robustness performances which can be obtained by the robustness curves are: i) the maximum DL that the structure can suffer without collapsing, and ii) the residual capacity of the damaged structure at different DLs. Other considerations can be made by focusing on the steepness of the curve: the steepness of the robustness curve when the DL is incremented, is proportional to the decay of the residual capacity of the structure.

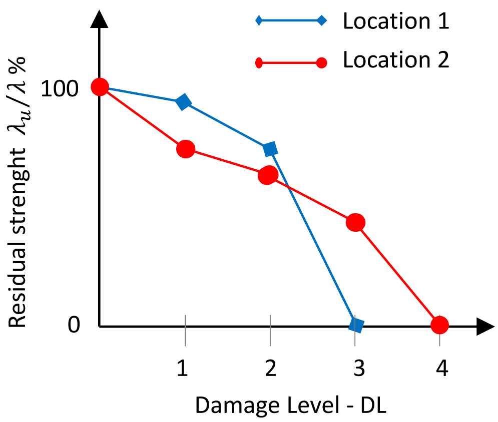


Figure 1: Example of robustness curves.

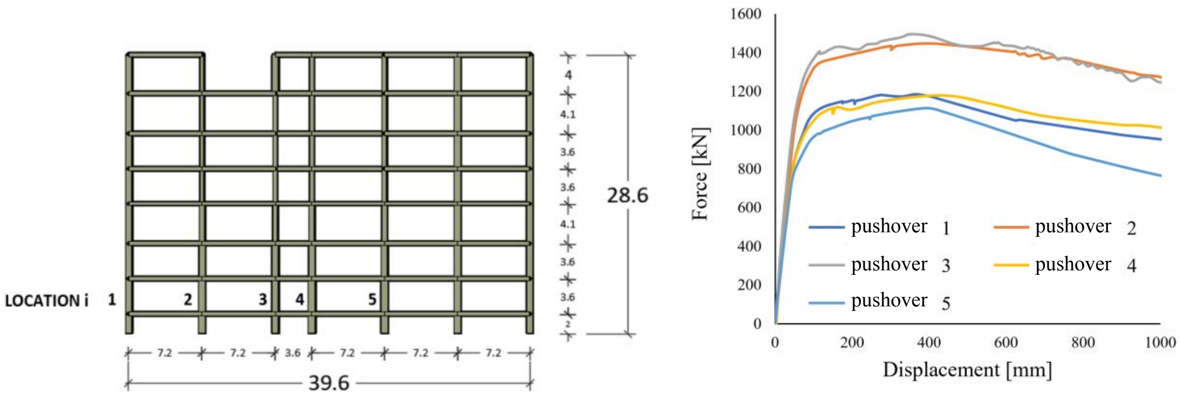
The response to the initial NDA (typically represented by the time history of the vertical displacement of the node at the top of the removed column), the successive pushover analysis and, consequently, the obtained robustness curves, are strongly influenced by several parameters regarding the nature of the damaging hazard, the analysis procedure, or the structural model. One of these parameters is the removal time interval (Δt_d) for the column: the less is the Δt_d, the more severe is the consequent structural response, and the less is the residual capacity obtained by the pushover [5]. In this view, the correct identification or setting of the Δt_d for a certain "n" would be of value.

To understand the importance of the Δt_d parameter related to the procedure described above, the RC 2D frame test structure shown in Figure 2a is considered. The structure is modeled using SAP2000® commercial code. The nonlinear behavior is implemented using the approximation of plastic hinges, which are obtained from the bending moment-rotation relationship (M-θ) evaluated from the equations provided by the Italian Standards NTC2018 (2018) [9] by considering the influence of the axial load. Moreover, geometric nonlinearity is considered by a large displacement analysis formulation.

The sensitivity analysis is carried out by the damage presumption approach, then it contemplates both a global NDA and the subsequent pushover analysis, the last one allowing the evaluation of the residual capacity of the structure after the considered DL (where DL=number of removed columns). The NDA is conducted by the following steps (see Fig. 3):

1. First, a static analysis under vertical loads is conducted to evaluate the reactions at the top of the columns that will be removed in the subsequent step (column-reaction forces). This will allow to develop a "surrogate" FE model of the structure in which the removed columns are substituted by the corresponding column-reaction forces, which reaches the same static equilibrium of the previous model under vertical loads.
2. In the second step of the analysis the surrogate FE model is used to conduct the NDA under sudden the columns removal. Here the two subsequent sub-steps are implemented: i) the vertical loads and the columns-reaction forces are applied by a slow ramp, and; ii) the columns-reaction forces are suddenly removed by a drop-to-zero ramp of duration Δtd (columns removal time-interval);

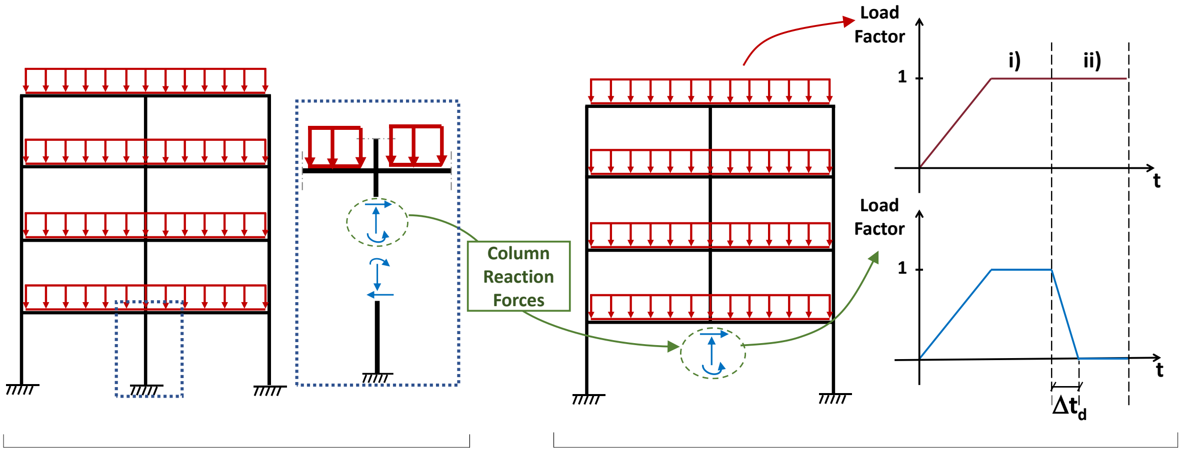
If the structure does not experiment the collapse at step 2 above, the pushover analysis is conducted by implementing triangular forces and by bi-linearizing the response curve as shown in Figure 4. Depending on the location of the damage (location of the first removed column), the results obtained by the lateral pushover analysis are different (Fig. 2b): since the internal columns (n°3, n°4 and n°5) are characterized by a large tributary area for vertical loads, the damage to one of them could cause a bigger reduction of the capacity with respect to the one caused to damages at external columns.



(a) (b)

Figure 2: (a) 2D RC frame structure (sizes in m) and (b) Pushover curves after different locations of damage (please note that the maximum displacements reached in pushover analyses are unrealistic and are obtained by forcing the numerical convergence).

Focusing on location 3 (see Fig. 2a), some sensitivity analyses have been carried out by varying the columns removal time interval Δt_d; the variation of this parameter influences the behavior of the structure in terms of residual displacement after column removal as obtained by the NDA, the ultimate load and deformation as obtained by the pushover analysis, and then it changes the resulting robustness curve. Figure 4 shows the effect of the variation of Δt_d for DL=1 at location 3.



(a) (b)

Figure 3: Two step procedure for the sudden column removal analysis: static analysis under vertical loads (a); NDA under sudden column removal with columns removal time-interval Δt_d.

The Figure 4 shows the bi-linearized results of pushover analyses for location 3 with DL= 1 and how the variation of Δt_d affects the capacity of the damaged structure. Although there are just slight differences between the curves for the considered cases, it is shown that if the Δt_d value decreases, the overall capacity decreases as well. It is important to understand the effect that different values of Δt_d have in terms of decreasing the capacity of the structure because this parameter can be used to simulate the damage induced by different blast scenarios.

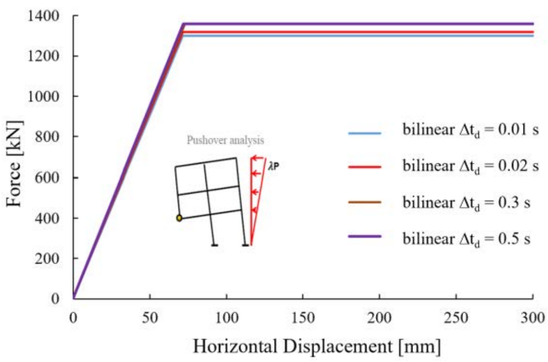


Figure 4. Bi-linearized pushover curve of 2D RC frame structure – DL=1: effect of variation of Δtd (expressed in seconds).

In Figure 5, the effect of the variation of Δt_d on the obtained robustness curves for damage at location 3 is shown: the decreasing of Δt_d determines a decrease of the robustness. The DL= 2 (i.e., two columns simultaneously removed) always determines the progressive collapse of the structure due to the spontaneous collapse of the column adjacent to the initially removed one, while DL=1 causes a drop in initial capacity of about 20%. In the figure, DL=

0.5, corresponds to the loss of the 50% of the column cross-section; this means that the explosion results in a loss of element stiffness and capacity, but not in a collapse.

The decreasing of the pushover residual capacity at shorter Δt_d it is due to the larger damage suffered by the elements adjacent to the removed column (e.g., beams) as a consequence of the large vertical displacement experimented by the nodes at the top of the removed columns.

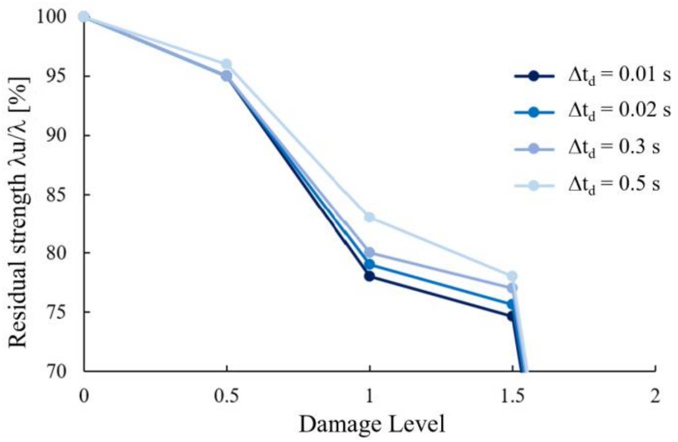


Figure 5. Robustness curves for 2D RC frame structure for location 1: effect of variation of Δtd (expressed in seconds).

In order to take into account for the above-mentioned sensitivity of the robustness to the column removal time interval (Δt_d) and to pertinently connect the robustness with the blast hazard, Francioli et al (2021) [5] proposed a procedure for determining the so-called "blast-scenario dependent robustness (BSR) curves" for RC frame structures, which take into consideration the parameters of the specific blast hazard, like the blast intensity and impulsive rate. The BSR curves are obtained by a two-stage analysis where, as a first step, the vulnerability of the frame's columns is assessed by capacity (pushover) and demand (nonlinear dynamic) analyses carried out by a "local" model (single column under lateral load). By the local NDA of the column, each damage level DL is associated to damage time interval Δt_{local}, the last one being defined by the time interval ranging from the explosion instant to the instant in which the considered damage occurs (column lateral deformation is larger than a fixed threshold value d_U). The damage time interval Δt_{local} is then strictly related with the intensity parameters of the blast: for detonations, pertinent intensity parameters are the equivalent Kgs of TNT joint with the stand-off distance, while for deflagrations, peak pressures acting on the blast-invested elements should be considered as intensity parameter ([10]). In both cases, lower Δt_{local} values can be reasonably associated with higher blast intensities. By assuming that $\Delta t_d = \Delta t_{local}$, if the previous mentioned local analysis is not carried out, the reasonable assumption where lower Δt_d values are associated to larger DLs can be made in conducting the successive global analysis step (described below).

As a second step, the BSR curve is obtained by the damage presumption approach, where each DL is analyzed by a "global" NDA (whole building under sudden column removal) and using a removal time interval (Δt_d) for the columns, previously associated to each of the considered DL. Given a certain DL=n, the DL=n+1 is obtained by removing an additional column which is adjacent to the "n" columns removed at the previous DL. The "n" columns in a certain damage scenario are removed simultaneously by using the $\Delta t_d = \Delta t_{local}$ already associated to that DL=n at the first step. The procedure is carried out for different locations to obtain a set of BSR curves. The BSR curves, on the contrary of those obtained by the classical damage presumption approach, are hazard-dependent, in the sense that each DL is joint to the intensity of the blast hazard, which is directly associable to (and identified by) the parameter Δt_d (= Δt_{local}).

2.1 Robustness analysis hazard-chain scenarios

The scenarios when two chained hazards impact on the structure is one of the most demanding for robustness requirements. A typical chain of hazards is the one implying the sequence of earthquake as triggering action and explosion and/or fire as chained or triggered one. This is what can occur when a big earthquake strikes on the structure: it may cause structural damages and, at the same time, damages to electrical/gas pipes to generate an internal fire or explosion. It would be important to understand to what extent the blast robustness of a structure is changed from the damage suffered by an earthquake, and if the considerations of such a hazard chain scenario provides new design indications. As already said, it is not the goal of this paper to investigate probabilistic aspects of the hazard chain scenarios (e.g., probability for an earthquake to trigger an explosion), while the goal here is to investigate the structural consequences of the earthquake-induced damages on the blast structural robustness of an RC building. For this reason, the robustness analysis under hazard chain scenarios is conducted in deterministic terms. The robustness deterministic analysis under blast after earthquake is similar to the one under the blast hazard only, but it is conducted on the structure when it is already damaged by the earthquake: thus, a nonlinear time history seismic response analysis must be carried out before applying the procedure outlined in previous sections. The robustness analysis under earthquake-triggered hazards make sense only for high-intensity earthquakes, which can induce the damage in the structure before the blast occurs. A fruitful representation of the robustness under hazard chains is on the same graph of the “blast only” robustness curve of the same structure, meaning that the residual capacity percentage $\lambda u/\lambda$ % should be factorized by the residual capacity of the undamaged structure (see Fig. 6). This is something allowing the correct quantification of the robustness decrement due to the earthquake-induced damage.

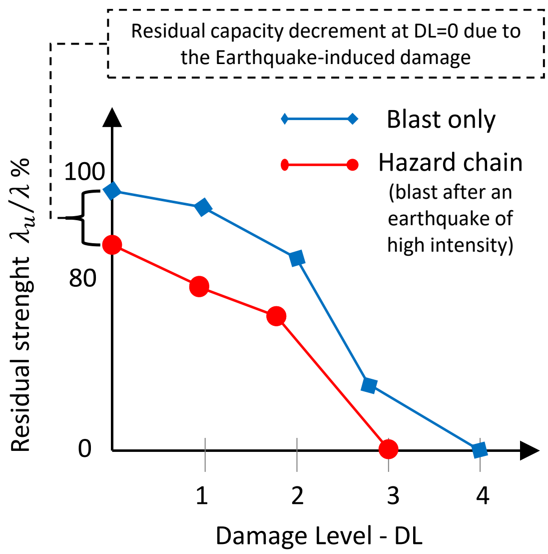


Figure 6. Robustness under hazard-chain.

3 CALCULATION: APPLICATION TO A 3D RC FRAME

The procedure for the BSR curves evaluation has been applied to an existing 3D RC structure serving as hospital building (Fig. 7). The building has a floor extension of about 42x35 meters and a global height of 28 meters (7 floors). It is a RC structure made of concrete C28/35 and steel B450C under the classification of Italian Standards ([9]), the increasing of the concrete elastic modulus (Ec) and of the concrete strength (fc) under impulsive load (strain rate effect) has been taken into account by assuming Ec= 42510 MPa and fc=41.73 MPa for a C28/35 concrete. The steel rebars elastic modulus Es is taken equal to 210000 MPa, while the yielding stress fy (increased under impulsive load) is 510 MPa (B450C steel grade).

The structure has been designed 40 years ago under the Italian Standards both for vertical and seismic loads. The seismicity of the site has been updated in Standards about 20 years after the construction, something that required a successive seismic vulnerability check in consideration of the increased seismic hazard.

As for the demonstrative 2D frame structure analyzed in previous sections, the finite element model has been realized by the commercial structural code SAP2000® using BEAM type element for beams and columns and the material nonlinearity has been taken into account with bending plastic hinges for beams and plastic hinges sensitive to the axial load for columns (modelled coherently with the Moment-Rotation (M-θ) bond required by Italian standards). Different removal time of the column (Δt_d) are considered, while the value of the damping ratio is fixed to 4%.

On the base of architectural drawings, two different locations (i.e., the position of the element removed at DL=1) have been considered, in correspondence to technical premises as shown in Figure 8. Key structural elements (the DL can be identified as the number of the damaged key elements) are the columns at the ground floor which, at the two selected locations have 60x60 and 65x90 (cmxcm) cross-sections.

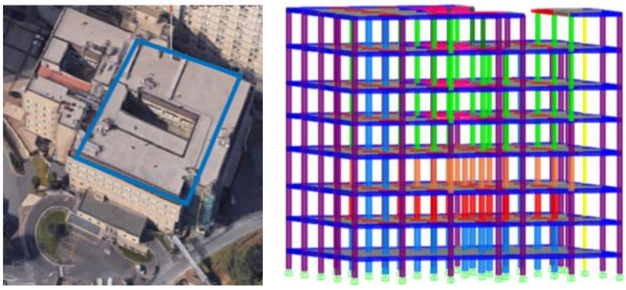


Figure 7. Case study structure: (left) Google Earth® view; (right) FE model of the structure.

The methodology used to assess the robustness performances of the structure is the one explained in Section 2: in a certain location, starting from a certain DL the structural response is evaluated by the NDA. If failure (as defined in section 2.1) doesn't occur spontaneously, first, the residual capacity of the structure is identified by a pushover analysis, then the damage level is increased (i.e., the previously removed plus an additional column are simultaneously removed) and the structural response is re-evaluated.

As already stated above, during the pushover analysis the criteria identifying the residual lateral force capacity (λu) are: i) the occurring of the "run-away" behaviour observed in the vertical displacement time history of the nodes around the removed column; ii) the occurrence of a vertical drift ratio (DV) bigger than 20%; iii) a vertical displacement ratio bigger than 0.5.

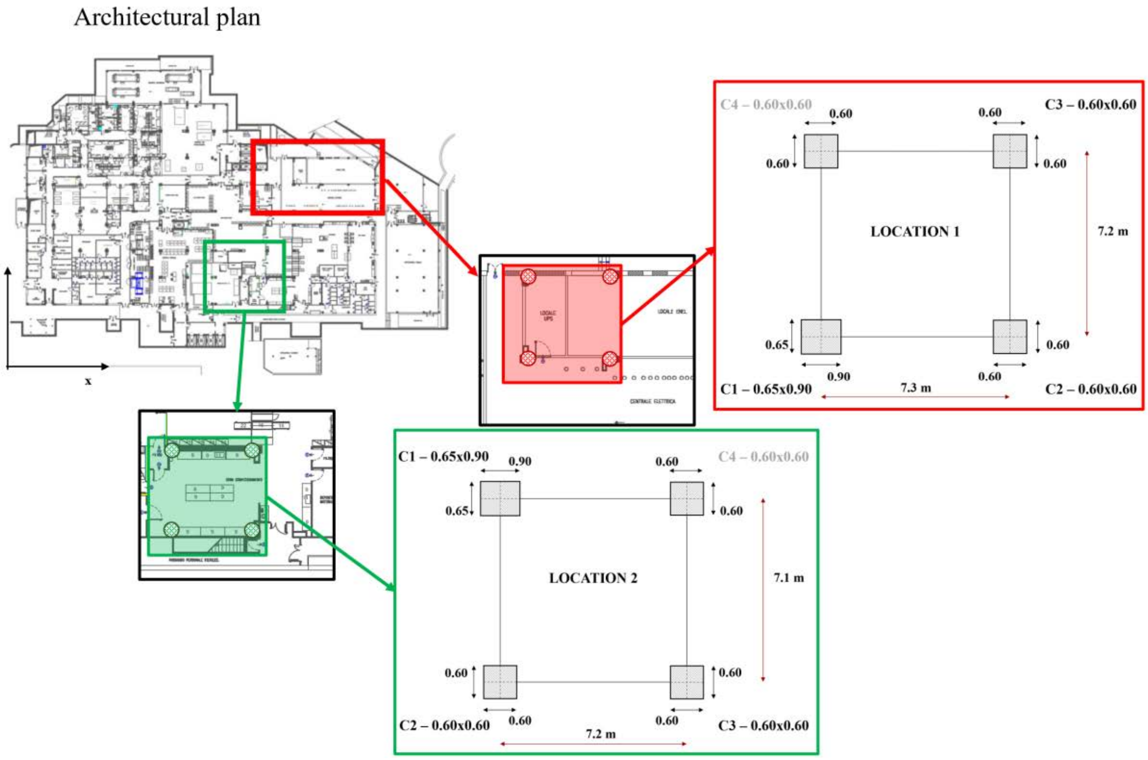


Figure 8. Architectural specifications: parts of the first floor with high concentration of components potentially subjected at risk of explosion; first (red) and second (green).

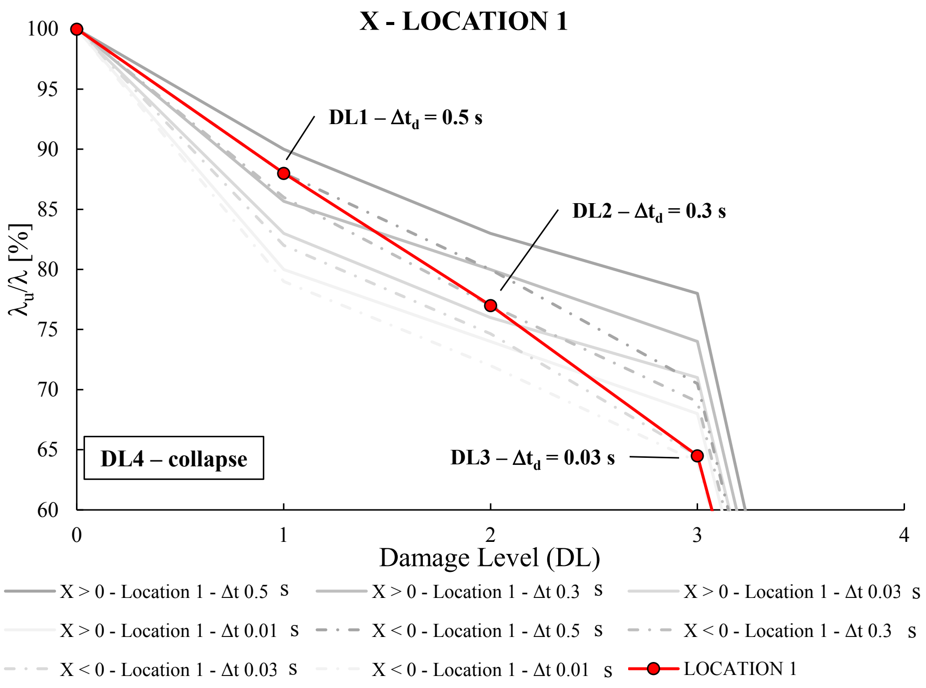
4 RESULTS AND DISCUSSION

4.1 BSR curves

As said, the explosion is assumed to occur in technical areas inside the building, then the structure is subjected to deflagrative explosions (of gaseous material). The deflagration overpressure peak P_0 can reach maximum values of 8 - 20 bar, while the duration of the positive pressure phase (Δt_p) may vary depending on the circumstances, with values ranging between a few milliseconds and hundreds of milliseconds ([11]). The typical pressure wave time history of deflagrative explosions it is usually obtained by ad hoc Computational Fluid Dynamic (CFD) analyses, which is not developed here, to explore the local effect (pressure time history on the columns) of the explosion.

In cases where such a local analysis is not conducted, it is a reasonable approximation to assume that larger DLs are associated to lower Δt_d values. In other words, the lower considered Δt_d is associated to the DL that involves the removal of the highest number of columns, while at the maximum Δt_d is linked the one that causes the removal of the lowest number of columns. With this criterion, in order to obtain the BSR curves, pertinent points are selected from the set of damage presumption robustness curves shown above (obtained for all the considered Δt_d and all pushing-over directions). Namely, since $\Delta t_{d\_1}<\Delta t_{d\_2}<\Delta t_{d\_3}<\Delta t_{d\_4}$, the Δt_{d_1} is associated to DL=4, while Δt_{d_2}, Δt_{d_3} and Δt_{d_4} are associated to DL=3, DL=2 and DL=1 respectively.

In the present paper, a local model (single column under lateral load due to the explosion) has been developed in SAP2000® to associate different values of the Δt_d to different blast intensities. T. In addition, since the structure is characterized by different residual capacities in both X direction ($\lambda_{u,\,X>0} \neq \lambda_{u,\,X<0}$) and Y direction ($\lambda_{u,\,Y>0} \neq \lambda_{u,\,Y<0}$), for each DL the lower capacity is chosen between the two alternative results in each direction. The resulting BSR curves are shown in Figure 9 for the X direction (similar graphs, here neglected for sake of simplicity, are obtained for the Y direction).



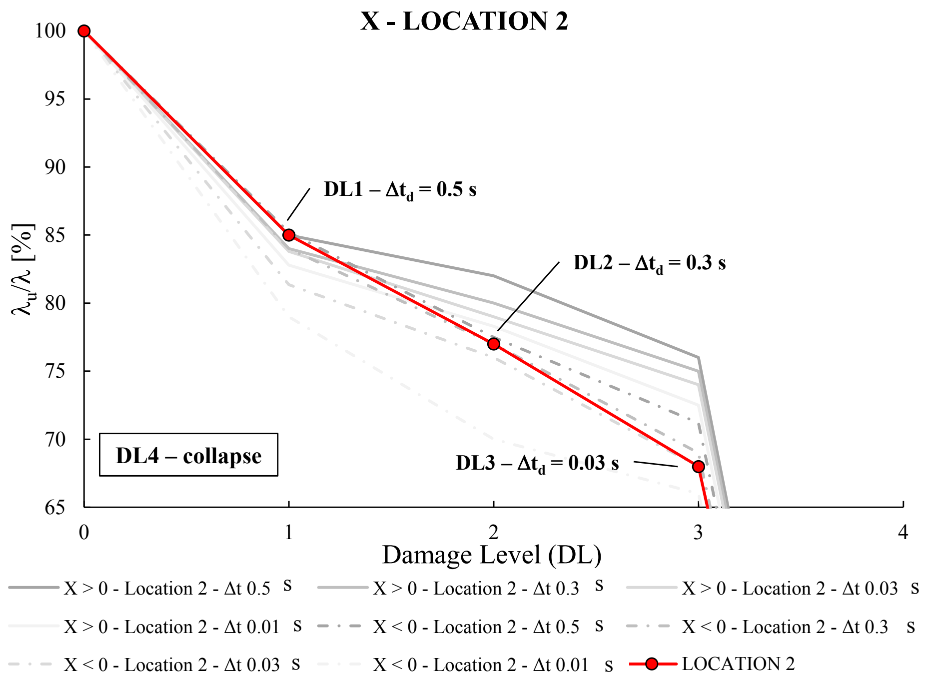
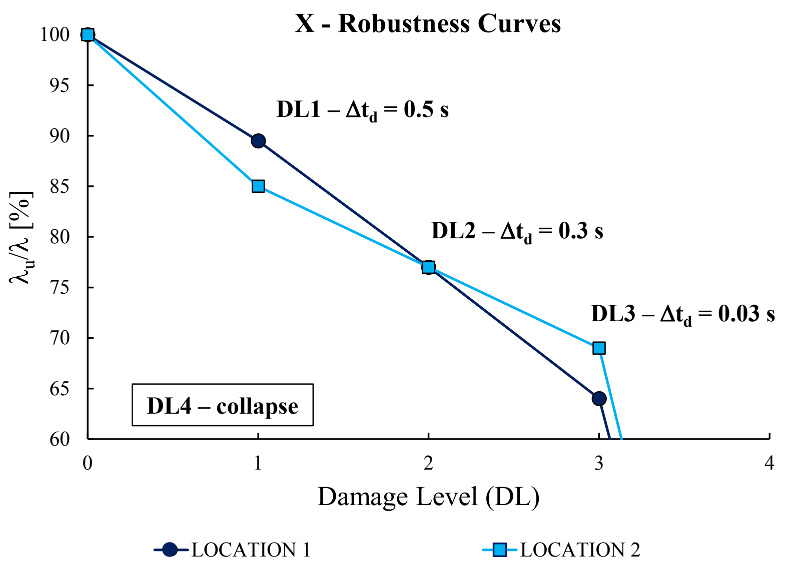
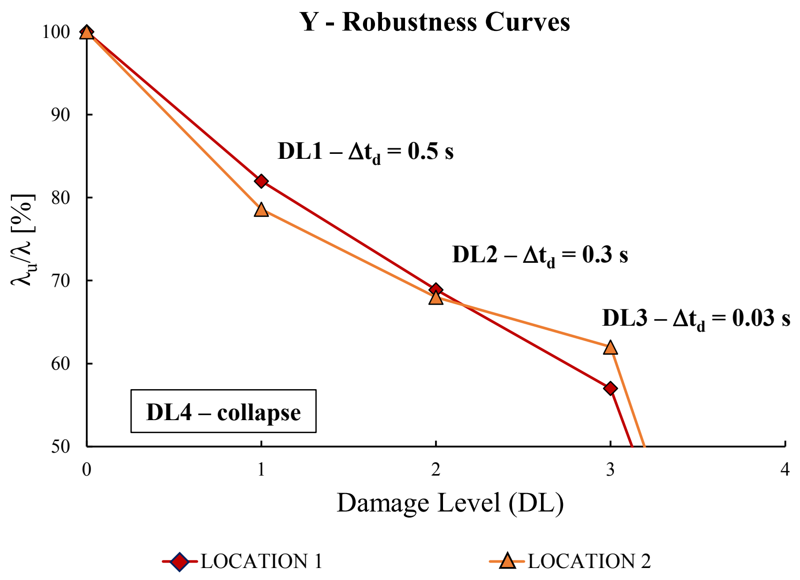


Figure 9. BSR curves associated to the X direction.

It is noted that, if four columns are simultaneously removed (DL=4), at least one of the threshold values of the adopted failure criteria are exceeded independently of the pushing direction, of the blast location, and of the value of the time interval Δt_d. The comparison of the BSR curves for the two considered locations in each direction is reported (Figure 10 (a) and (b)). It appears a similar behaviour between the two locations, either in X or Y directions; in both cases the location 1 appears more robust than location 2 for low DLs, while for a DL equal to 3, location 1 is less performing than location 2. As said, the DL =4, here associated to the Δt_d = 0.01 s, always determines the collapse of the structure. As general conclusion, it can be said that the critical location for the blast robustness of the considered structure is the location 1, with λ_u/λ %=64% for DL=3.



(a)



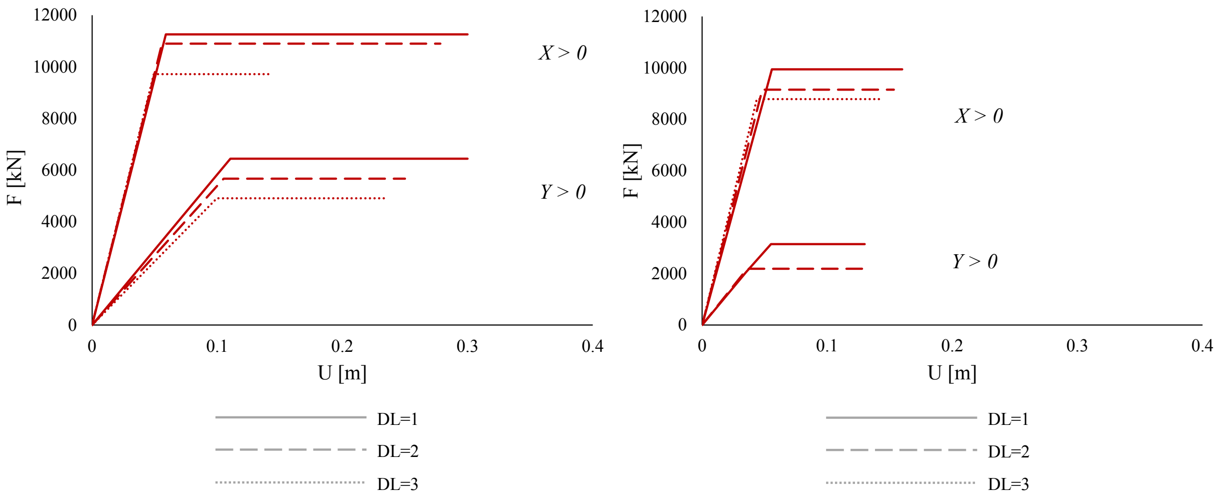
(b)

Figure 10. BSR curves associated to the X and Y directions.

4.2 Robustness under hazard chain scenarios

To highlights the consequences of an initial earthquake damage on the blast-robustness performances, a single earthquake is considered here, characterized in intensity for the considered structure by the spectral acceleration at the first natural period $Sa(T_1)$ of 0.37g, and a return period of 475 years.

The effect of the earthquake induced damage on the structural capacity is evident from the comparison of the bi-linear capacity curves obtained by the pushover analysis of the structure at different blast induced damages (DLs) obtained without considering the earthquake or after the earthquake-induced damage (Fig. 11 for the location 1). The capacity curves shown herein refer to one of the Δt_d values considered in the analyses (Δt_d = 0.3s).



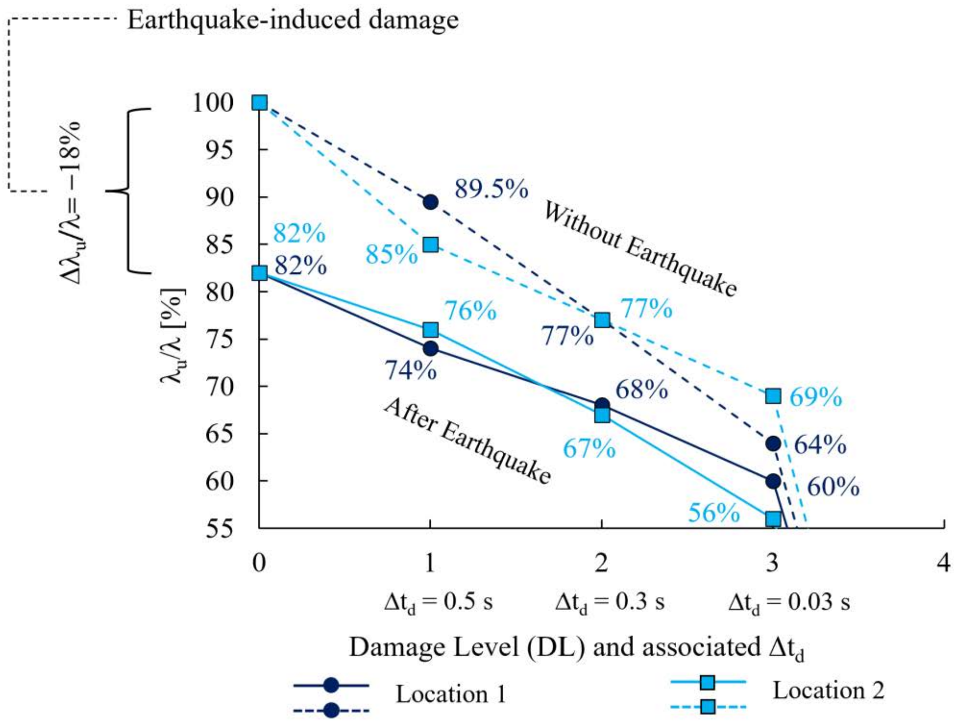
(a) (b)

Figure 11. Pushover response curves of the 3D frame at different DLs (Dtd = 0.3 s); Location 1: (a) without considering earthquake damage, (b) with earthquake damage (DL=3 for Y>0 is null).

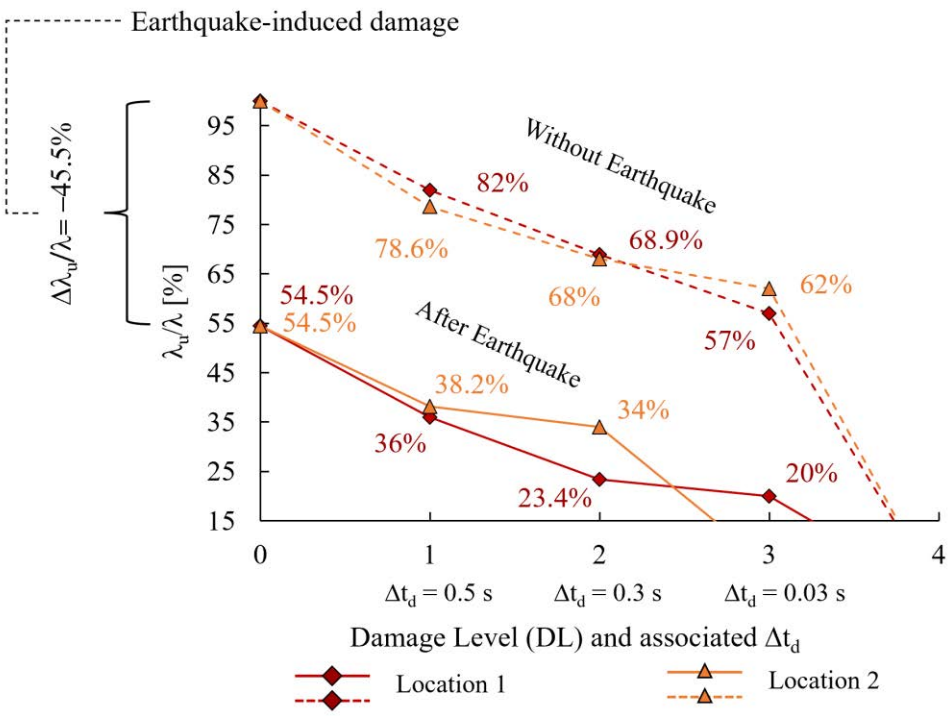
The effect of the earthquake induced damage on the structural robustness is also noticeable in Figure 12, where the BSR curves obtained for the directions X and Y for the structure when damaged or not by the earthquake are shown.

5 CONCLUSION

Looking at Figure 12, it can be highlighted that the earthquake-induced damage is quantified to be 18% and 45.5% in X direction and Y direction respectively (with residual capacities after earthquake at DL=0 being 82% and 54.5% respectively). In addition there is an important result to highlights, which can give new insights about the robustness design of RC frames against blast: when the hazard chain scenario is not considered, the critical location is the one identified as location 1, suggesting that eventual provisions for enhancing the structural robustness should focus on the blast-strength improvement at that location or, alternatively, they should focus on some management measures to decrease the blast hazard at that location, for example by keeping the technical premises away from location 1 and placing them at location 2.



(a)



(b)

Figure 1. Effect of hazard chain on the BSR curves: (a) X direction, (b) Y direction.

On the contrary, if the hazard chain scenarios are considered in the robustness analysis, they lead to the identification of the location 2 as the critical one for robustness performances, meaning that moving technical premises at location 2 (one of the above-mentioned design provisions) is not appropriate because this can increase the risk of progressive collapse. The change of critical location in terms of robustness when the hazard-chain scenarios are considered with respect to the blast-only scenario suggests that the effect of the initial earthquake-induced damage cannot be ignored in structural design, at least for important buildings located in seismic regions.

The result of this analysis show that the blast robustness of a previously earthquake-damaged structure dcreases as obvious, but most of that the analyses show that the critical location inside the structure for the blast-induced damage in terms of robustness performances is different if the hazard-chain scenario is taken into account or not, something indicating that, when buildings are located in seismic regions, these types of hazard chain scenarios cannot be neglected in the design of RC frames for robustness. All the analyses of the paper are deterministic. No references are made to the probabilistic occurrence of the hazard chain scenarios or to the probability for a fixed intensity of the earthquake to trigger explosions, a high intensity earthquake is rather deterministically assumed to cause an explosion.

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