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# Robustness Assessment of RC Framed Structures against Progressive Collapse

Antonio Ventura <sup>1</sup>, Bernardino Chiaia <sup>1</sup>, Valerio De Biagi <sup>1</sup>

<sup>1</sup> Politecnico di Torino, Diseg, Corso Duca degli Abruzzi, 24, 10129 Torino, Italy

antonio.ventura@polito.it

**Abstract.** The structural behaviour of buildings under overloading or extraordinary events like impacts, explosions or human errors is extensively admitted to be an influential feature of structural design. Structural robustness is a requirement provided by many current design codes. However, the problem is often recognized in a qualitative manner without referring to a specific process for the evaluation or the achievement of the robustness of constructions. In this paper, a novel procedure derived from dynamic and non-linear static analyses is provided for evaluating and comparing the relative robustness of reinforced concrete (RC) frame buildings against progressive collapse. The developed methodology offers a formal way to compute “robustness curves” following the sudden loss of one or more vertical load carrying member/s. This method suggests a strategy for the definition of the robustness indices, which are applied to two RC frame buildings. The first building was designed for gravity load and earthquake resistance in accordance with Eurocode 8 and the second was the same structure, modified according to the tie force (TF) method. The TF method is one of the major design quantitative procedures for enhancing resistance to progressive collapse and it is currently recommended by the codes of practice. In an attempt to demonstrate the suitability of the procedure, the structural robustness and resistance to progressive collapse of the two schemes is compared.

## 1. Introduction

Structural robustness is considered an important feature of the design and the safety assessment of structures. Modern building codes require that a structure has to be robust and much research on this topic has been carried out in recent years.

In light of these studies, several definitions of structural robustness have been reported in the literature, as highlighted in Starrosek and Haberland [1]. However, in the field of structural engineering, structural robustness is typically considered as the ability to withstand extraordinary events like impacts, explosions or human errors, without being damaged to an extent which is disproportionate to the original cause [2, 3]. As a consequence of this definition, two main approaches can be considered to enhance the robustness of a structure. The first one explicitly provides measures to reduce direct local damage due to extreme events by increasing the strength of key elements. The second approach adopts structural measures intended to prevent the propagation of local damage to a disproportionate extent (progressive collapse resistance). The present research work is focused only on progressive collapse resistance.

While procedures to enhance resistance are discussed in guideline documents, a large amount of studies have examined and proposed numerical techniques for progressive collapse analysis of reinforced concrete (RC), steel and composites framed structures. Besides extensive efforts in design and simulation, the measure of structural robustness to progressive collapse is often controversial, since



there are no generally well established and accepted quantitative methods for the assessment of robustness. Although various approaches for the quantification of robustness have been published, so far, none of these has emerged as distinctly superior and preferable [2, 4].

The aim of this work is to introduce a general method for a consistent and quantitative measure of robustness of RC frame structures against progressive collapse. In particular, robustness is assessed by comparing the performance of the structure in the original state and in a damaged state, as a consequence of a specific hazard scenario. The procedure introduced in this study acknowledges the principal merits of the current design and simulations methods and, at the same time, it is adopted to introduce two robustness indicators. The effectiveness of the proposed strategy is shown through the application to two RC frame buildings, in order to compare their structural robustness and resistance to progressive collapse.

## 2. Modelling considerations

### 2.1. Structural models

The structural robustness against progressive collapse was assessed in the case of two structures with certain features. Both structures were 4-story, 4x4 bay RC framed buildings. The former structure was designed conforming to the prescriptions of Eurocode 8 (EC8) [5] and the latter was the same EC8-conforming building modified according to the Tie Force (TF) method.

Figure 1 shows perspective and plan views of the two buildings under investigation, which were composed of five primary frames connected each other by one-way RC joint slabs and continuous cast-in-situ secondary beams. The plan dimensions were 24 m in x-direction and 16 m in y-direction at any floor, with column spacing of 6 m and 4 m in x- and y-direction respectively. The columns were located in the nodes of a grid, as depicted in Figure 1. The plan position of each column is identified by means of a letter (from A to E) and a number (from 1 to 5). Floor levels are labelled by Roman numbers from I (level at +3.00 m) to IV (top level at +12.00 m). The interstorey height is 3 m at each floor. The structures are subjected to the following load combination

$$\Omega_N(1.2DL + 0.5LL) \quad (1)$$

where  $DL$  represents dead loads and  $LL$  denotes live loads. This load combination is suggested in GSA [6]. The investigated buildings were assumed to be adopted for housing, dead (DL) and live (LL) loads were assumed to be  $3 \text{ kN/m}^2$  and  $2 \text{ kN/m}^2$ . The term  $\Omega_N$  is the dynamic amplification factor that is adopted in the nonlinear static analysis. For framed RC structures,  $\Omega_N$  is equal to

$$\Omega_N = 1.04 + \frac{0.45}{\theta_{pra}/\theta_y + 0.48} \quad (2)$$

in those bays immediately adjacent to the removed element and at all floors above the removed element, and equal to  $\Omega_N = 1$  in the floor areas away from the removed column.  $\theta_{pra}/\theta_y$  is the ratio between allowable plastic rotation angle and the yield rotation angle.

The load combination has the term  $\Omega_N = 1$  for nonlinear dynamic analysis. A properly detailed slab was not considered in the building model; this is a conservative assumption, while its weight and inertia were implicitly included in the FE simulations. The same approach was also adopted to account for partition walls [7].

Material properties of the structural members were set to 25 MPa for the characteristic compressive cylinder strength of concrete ( $f_{ck}$ ) and 450 MPa for the characteristic yield strength of reinforcement ( $f_{yk}$ ). The structure was designed for medium-high seismicity assuming a peak ground acceleration  $PGA=0.30g$ . The hierarchy of resistance (i.e., capacity design) was implemented in addition to minimum ductility requirements for individual elements. An equivalent viscous damping of 5% was chosen in accordance with current European prescriptions [5].

Table 1 outlines the section properties of beams and columns in terms of member size and reinforcement layout. In both structures investigated, all the design parameters were kept the same for

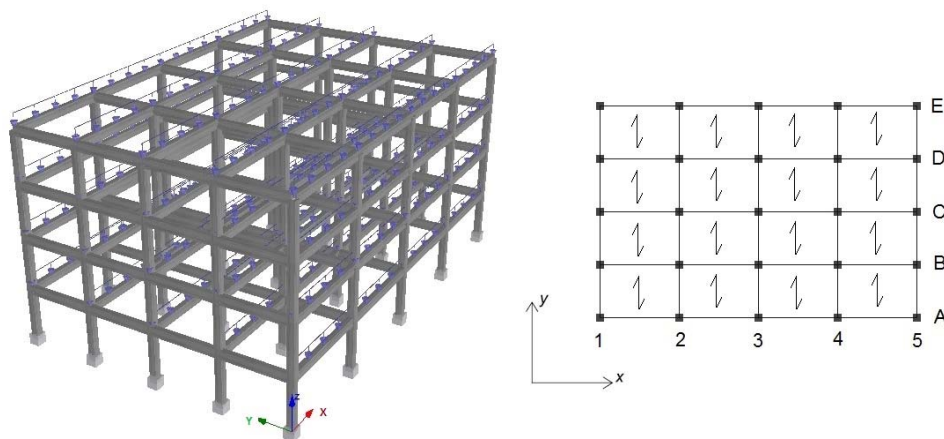
the purpose of comparison. Furthermore, the second building was modified in accordance with the progressive collapse design requirements of the TF method [8]. The structural elements were assumed to be mechanically tied together, thereby enhancing the continuity, ductility, and development of alternate load paths. Tie forces were provided by the existing structural elements, which are designed using conventional design methods to carry the standard loads imposed upon the structure [9].

According to [8], two horizontal ties were provided in this work: internal and peripheral. For the framed building considered, the required tie strengths  $F_i$  and  $F_p$  for internal and peripheral ties respectively, are determined with

$$F_i = 3w_f L_1 \quad (3)$$

$$F_p = 6w_f L_1 + 3W_c \quad (4)$$

where  $w_f$  is the floor load,  $L_1$  is the greater of the distances between the centres of the columns in the direction under consideration, and  $W_c$  is equal to 1.2 times the dead load of cladding over the length of  $L_1$ . The tie forces are calculated with the same formulations in longitudinal and transverse direction, considering the correct geometrical properties of buildings in each case. Vertical ties were provided by ensuring the continuity of the longitudinal reinforcement in the columns. The last column of Table 1 outlines all the tie strength force adopted in the structural members of the buildings.



**Figure 1.** Perspective and plan views of case-study building model.

## 2.2. Numerical techniques

Numerical techniques developed according to the lines of fiber force-based approaches, have been adopted in the open FE code SeismoStruct [10]. Inelastic force-based fiber elements have been used in an attempt to predict the nonlinear response of the two buildings under investigation, in static and dynamic fashion. These elements were assumed to model the frame members, explicitly including geometric and material nonlinearities. Geometric nonlinearity was accounted using a co-rotational transformation, whose implementation is based on an exact description of the kinematic transformations associated with large displacements and three-dimensional rotations of the beam-column member.

Material nonlinearity was described by a distributed inelasticity approach, in which the sectional stress-strain state of each structural member is obtained through the integration of the uniaxial stress-strain response of the individual fibers. Further, a one-to-six correspondence between structural members and model elements was assumed; these model elements were considered having 5 integration points and 400 fibers [3]. The uniaxial uniform confinement model proposed by Mander et al. [11] was used to represent concrete behavior, while a bi-linear idealization, combined with isotropic strain hardening, was assumed for steel.

The ultimate capacity of the two case-study buildings was defined in terms of steel and concrete strains. The fracture/buckling strain of reinforcing steel was conservatively set to 6%. The ultimate

compressive strain of concrete was obtained in accordance with [11] resulting in 0.8%. In addition, code-compliant shear capacity and chord-rotations verifications were included in the simulations in order to verify whether demand exceeded capacity.

**Table 1.** RC section properties of the case-study building models.

Building class	Element	Location	Size (mm <sup>2</sup> )	Longitudinal reinforcement	Transverse reinforcement	Tie strength force (kN)
Seismic design (EC8)	Beam	Any line of floor level: I,II,III,IV	500x300	3 $\phi$ 20+4 $\phi$ 10+3 $\phi$ 20	2-leg $\phi$ 10@80	/
	Column	Any line of floor level: I,II,III,IV	400x400	12 $\phi$ 20	2-leg $\phi$ 10@50	/
TF-design	Beam	Peripheral x-line of floor level: I,II,III,IV	500x300	3 $\phi$ 20+4 $\phi$ 10+3 $\phi$ 20	2-leg $\phi$ 10@80	$F_p = 166$
	Beam	Internal x-line of floor level: I,II,III,IV	500x300	3 $\phi$ 20+4 $\phi$ 10+3 $\phi$ 20	2-leg $\phi$ 10@80	$F_t = 83$
	Beam	Peripheral y-line of floor level: I,II,III,IV	500x300	3 $\phi$ 20+4 $\phi$ 10+3 $\phi$ 20	2-leg $\phi$ 10@80	$F_p = 110$
	Beam	Internal y-line of floor level: I,II,III,IV	500x300	3 $\phi$ 20+4 $\phi$ 10+3 $\phi$ 20	2-leg $\phi$ 10@80	$F_t = 55$
	Column	Any line of floor level: I,II,III,IV	400x400	12 $\phi$ 20	2-leg $\phi$ 10@50	/

### 2.3. Computational strategies

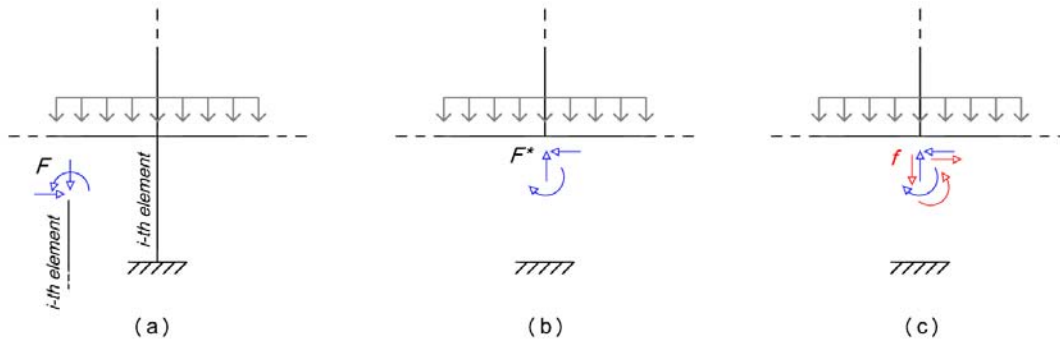
In this study, the well-established concepts in nonlinear static procedures were combined with dynamic analyses. The nonlinear static analysis, also called pushdown analysis, is an incremental nonlinear static procedure in which a downward load of increasing intensity is applied to the structure which has suffered the loss of one or more critical members [12]. The procedure is applied to the building till a collapse condition occurs.

The robustness evaluation procedure presented in the following is based on the assumption of a certain damage scenario caused by a generic extreme event, which is able to instantaneously remove the contribution of a structural member to the load bearing capacity of the system [13]. Pushdown analysis was implemented in three principal steps (Figure 2), according to De Biagi et al. [14].

First, the undamaged structure was loaded with the external loads with the load combination in Eq.(1). A nonlinear solver was considered and the forces and displacements in the elements were evaluated. In particular, the forces acting in the potentially damaged element were recorded, i.e., the forces in Figure 2(a). In the next step, the damaged element was removed and a set of external forces,  $F^*$ , were added to the scheme (Figure 2(b)); such forces are opposite to the ones of the previous step, i.e.,  $F^* = -F$ . A nonlinear run was made and nodal displacement were read. They were approximately equal to the ones evaluated in the first step (Figure 2(a)); it came out that the system of forces (i.e.,  $F^*$ ) correctly simulate the presence of the structural member. In the third step, a system of forces  $f$  opposite to the forces  $F^*$  was added on the node, as sketched in red in Figure 2(c). Thus, forces  $f$  were progressively increased from zero following a displacement controlled incremental scheme.

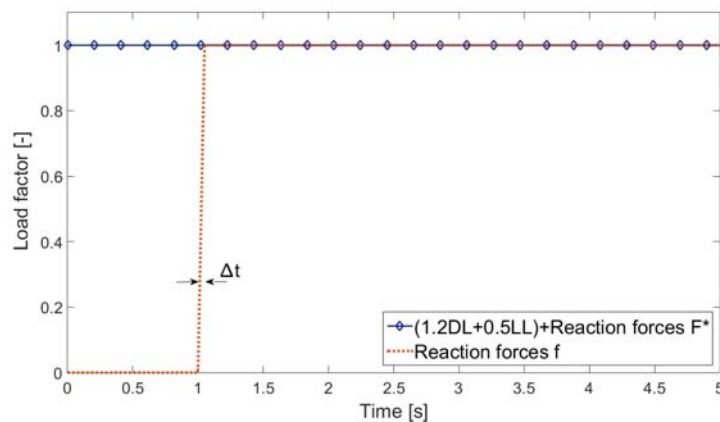
At the end of these simulations, a load multiplier was computed as the ratio of the push-down loads i.e.,  $f$  and the forces acting on the member before its removal (i.e.,  $F^*$ ). The outputs of pushdown analysis are reported in load-displacement capacity curves (pushdown curves). In these curves, the load multiplier is plotted against the vertical displacement at the top node of the removed vertical element.

In nonlinear static analyses, a unique value of the dynamic amplification factor ( $\theta_{pra}/\theta_y = 2$ , thus  $\Omega_N = 1.2$ ) was adopted for the bays immediately adjacent to the removed element and at all floors above the removed element.



**Figure 2.** Illustration of damage model for removal of a column. Details are reported in Section 2.3.

In the present paper, the pushdown procedures were allied with dynamic analyses. Despite nonlinear dynamic analyses are time consuming, they are the most appropriate method to simulate the effects of the sudden loss of one or more structural members. In these procedure, the buildings were subjected to the load combination in Eq. (1) with a dynamic amplification factor  $\Omega_N = 1$  in all the floor area of the structure. These analyses were carried out following an approach similar to that of the pushdown procedures. An initial undamaged situation was considered by replacing a column by its reaction forces  $F^*$  (Figure 2 (b)) and a system of forces  $f$  opposite to the forces  $F^*$  was added on the node, as sketched in red in Figure 2(c). The system of forces  $f$  were suddenly applied to simulate the damage with a time interval ( $\Delta t$ ) smaller than  $1/10$  of the fundamental period associated with the pertinent vertical modal shape of the damaged structure [8]. The implemented load factor time history is shown in Figure 3.



**Figure 3.** Time-history function of applied load for dynamic analysis.

### 3. Evaluation method of the structural robustness

In this section, a method for evaluating the robustness of RC frame structures is proposed. According to Giuliani [15], robustness can be assessed by considering the structural behaviour of the damaged configurations of a system. The method presented in this study can be classified as a damaged based method. Thus, robustness is accounted as the capability of a structure to withstand a limited degradation of its performance as a consequence of a damage increment. In light of the recognized concepts of the damaged based methods, the structural performance is evaluated as the ultimate resistance of the structure; the number of failed structural member are considered as criterion for the quantification of damage level. The following steps describe the proposed procedure.



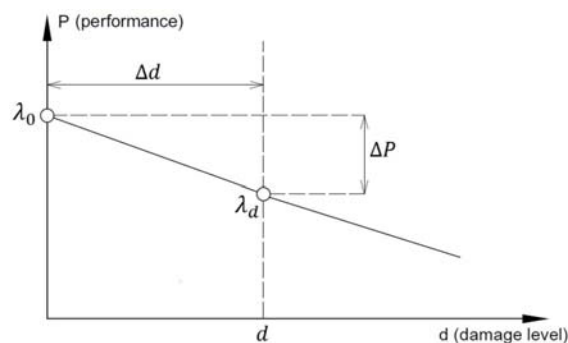
The first consists of assessing the structural performance of the structures in their undamaged state (i.e., for a damage level that can be assumed equal to zero). The performance is computed with a pushdown analysis, i.e., incrementing all the vertical uniform loads up to the failure of the structure.

The failure of the structure is supposed to occur when one of the ultimate conditions presented in Section 2.2 are reached. In the following, the ultimate resistance of the structure is identified with the load multiplier  $\lambda$ . In detail, the ultimate resistance related to the undamaged structure is indicated as  $\lambda_0$  and the ultimate resistance for a specific local damage level ( $d$ ) as  $\lambda_d$ . Different sets of damage scenarios are defined by the removal of a single vertical member, by two members, and so on.

In the second step, each column at the ground floor level of the building is independently removed. In particular, only external columns were considered in this work, these damage scenarios can be chosen *a priori* by considering the potential damage scenario (e.g., it is realistic that intentional explosions or impact take place in the external perimeter of the structure). Due to the symmetry of structural layout, only a limited number of member was removed. Each removal is accomplished by a nonlinear dynamic analysis (NDA) that simulates the sudden loss of the respective column. Two damage responses are expected: *i*) the sudden removal of selected columns leads to an unbounded response, indicating progressive collapse and lack of residual strength. In this case, ultimate resistance is assumed equal to 0 (i.e.,  $\lambda_d = 0$ ); *ii*) the critical member does not collapse (i.e., sudden removal does not lead to an unbounded response), then the damage response is arrested and the ultimate resistance is computed by a pushdown analysis. The ultimate resistance is identified by the ultimate load multiplier obtained from the pushdown analysis. In the latter case, the local damage is increased. This increment is heuristically obtained by alternatively removing the adjacent columns on the selected floor and relative to the directly damaged frame. The second step is repeated for a new damage configuration and the corresponding structural response is evaluated.

If progressive collapse is triggered by the selected local damage, a robustness curve can be obtained reporting in a diagram the ultimate resistance as a function of the considered damage level. The procedure ends when all the damage scenarios have been analyzed; at this point, a set of curves describing the robustness of the structure under the considered damage scenarios are obtained.

An example of robustness curve is depicted in Figure 4. In this graph, the structural performance (i.e., ultimate resistance  $\lambda$ ) is represented on the y-axis, while the x-axis indicated the amount of damages. The whole procedure is described in the flowchart of Figure 5.



**Figure 4.** Qualitative representation of robustness curve.

The robustness curves can be utilized to develop two measures of structural robustness. The first proposed index is a local indicator that represent the limitation of the decrement of the resistance for a given increment of damage amount:

$$Ir = \frac{1 - \Delta P / \lambda_0}{\Delta d} = \frac{\lambda_0 - (\lambda_0 - \lambda_d)}{\lambda_0 \Delta d} = \frac{\lambda_d / \lambda_0}{\Delta d} \quad (5)$$

In this approach, the performance indicators are used as state variables and the obtained robustness indices are dimensionless functions of these variables varying in the range [0, 1]. The index can be

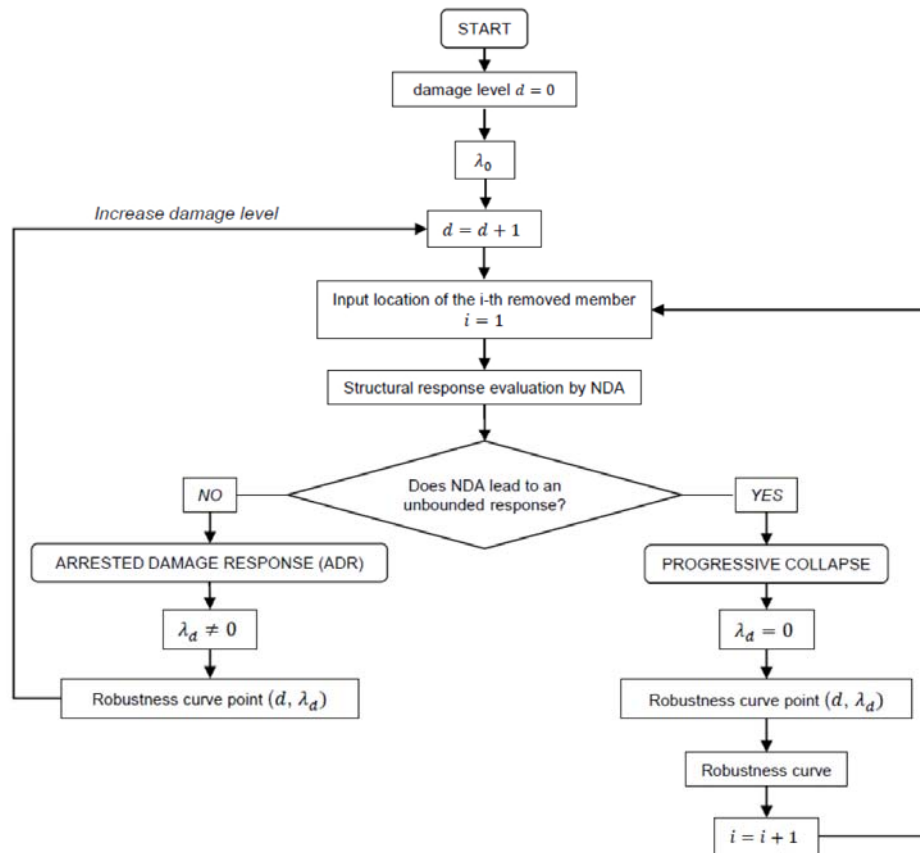


defined for each damage level (i.e.,  $Ir_d$ ), and has value  $Ir = 0$  and  $Ir = 1$  for a collapse and a robust situation, respectively.

A second index that provides a quantitative measure after all the damage configurations is defined as follows:

$$IR = \sum_{d=1}^{n_d} d \times Ir_d \quad (6)$$

where  $Ir_d$  is the first index referred to the damage level  $d$ ,  $n_d$  is the total number of damage scenarios. When  $IR$  is major then 0, the structure is proportionally more robust.

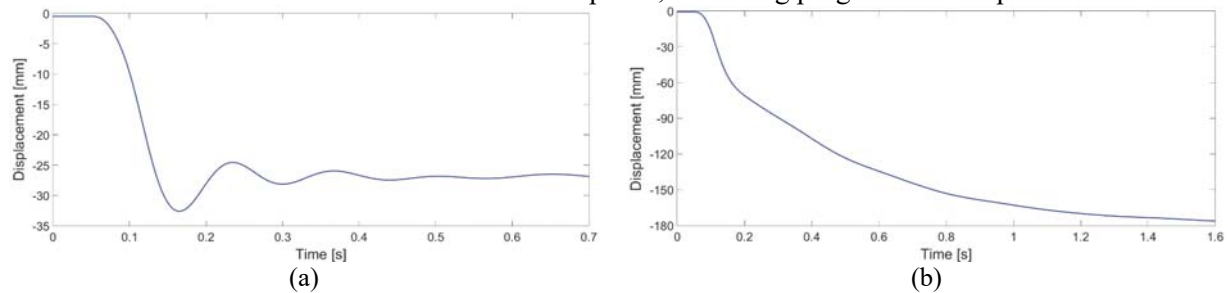


**Figure 5.** Flowchart of the procedure to evaluate the structural robustness.

### 3.1. Robustness assessment

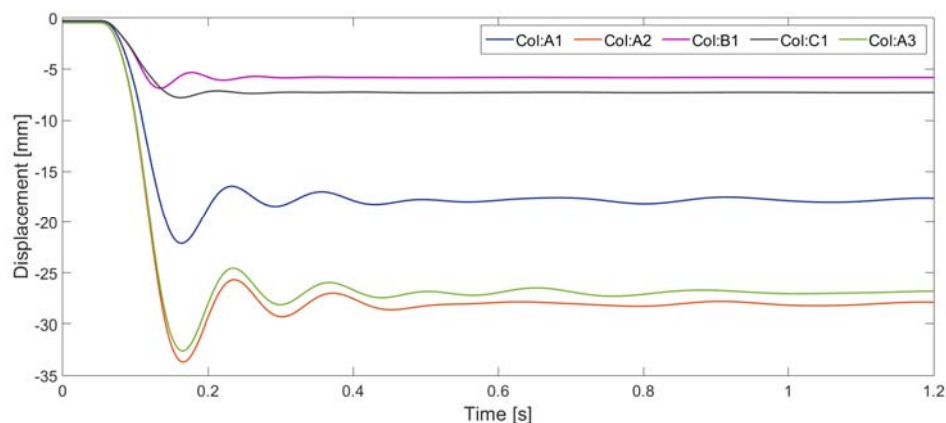
In this section, the method reported above has been applied to the two case-study buildings. In the following, direct reference is made to the steps of the procedure illustrated above and to the flowchart of Figure 5. As previously stated, the first step resulted in the evaluation of the performance of the undamaged structure. In detail,  $\lambda_0=4.7$  and  $\lambda_0=4.9$  are obtained in the buildings designed according to EC8 and TF method, respectively. Subsequently, supposing damages scenarios due to vehicle impact against the building, the columns at the ground floor have been considered as key elements and the numerical investigations were carried out removing the key element by NDA. The buildings are doubly symmetrical in plan; thus, the second phase of methodology was only applied to columns: A1, A2, A3, B1 and C1. The typical vertical displacement time history for a node located on the top of the removed key member is shown in Figure 6, for both the arrested damage response and progressive collapse situation, with local damage level  $d = 1$ . The first case (Figure 6(a)) shows a bounded response with an

initial oscillation and after a decaying response (damped oscillation). Figure 6(b) shows that the removal of the selected column leads to an unbounded response, indicating progressive collapse.

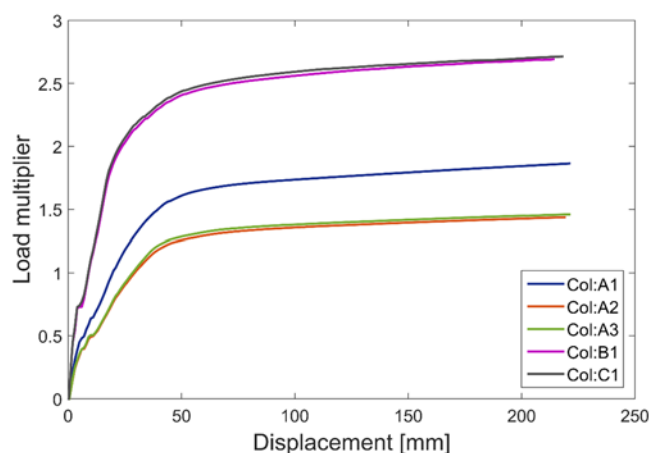


**Figure 6.** Response time-history for a node on the top of the removed column: (a) arrested damage response and (b) progressive collapse.

In the local damage level equal to one, each of the column removals resulted in a stable dynamic response indicating that the structure will survive the individual removal of any of the columns of this selected damage scenario. Figure 7 depicts the vertical displacement time-histories obtained by NDA on the EC8-conforming building. Furthermore, Figure 8 shows the different curves obtained from the pushdown analyses following the removal of the selected columns. The other steps of the procedure proposed were implemented, considering the other damage scenarios. The assessment methodology was then applied to the second building designed in accordance with TF method.



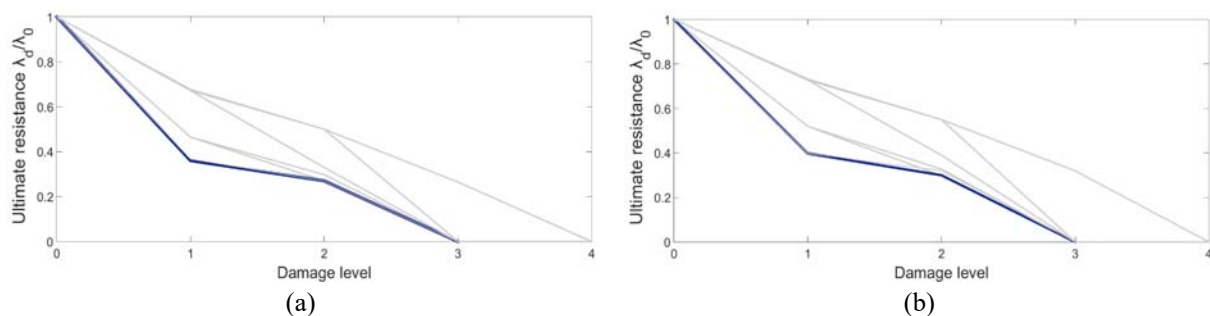
**Figure 7.** Vertical displacement time-history for a node of the top of the removed column.



**Figure 8.** Pushdown curves.

The computed robustness curves are shown in Figure 9 for the two buildings investigated. In these figures, for each damage level the ultimate resistance of all different damage configurations is stored. In the graph, the configuration corresponding to the lowest value of ultimate resistance for each damage level is highlighted with bold lines. Then, the curve of minima is considered, which is adopted for deriving the robustness indices. For a better presentation, the y-axes of these curves are made dimensionless by scaling the ultimate resistance values to the ultimate resistance of the integer structure.

The dimensionless load multiplier corresponding to the load combination  $(1.2DL + 0.25LL)$  is equal to  $\lambda_d/\lambda_0 = 0.21$  in the building designed according to EC8 and  $\lambda_d/\lambda_0 = 0.20$  in the building designed according to TF method. The computed robustness indicators, implementing the two proposed strategies for each building are reported in Table 2 and 3. The procedure indicates that the same removal sequence is valid for the two buildings. It has been shown that for the selected scenarios, with the damage level equal to three, the structure can lead to partial or global collapse in all cases but one (i.e., a progressive collapse). In one case only, the collapse progression occurs at the fourth damage level (removal sequence of columns: B1, C1, A1, A2). The robustness curves and indicators, highlight that no significant differences in terms of relative robustness are present in the two case study buildings. It can be found that the TF method is unable to enhance the progressive collapse resistance of the RC frame structures.



**Figure 9.** Robustness curves for building designed according to (a) EC8 and (b) TF method.

**Table 2.** The computed structural robustness indices using Eq. (5).

Damage level	Robustness index $I_r$	
	Building 1 (EC8-conforming)	Building 2 (TF design)
1	0.31	0.32
2	0.11	0.13
3	0	0

**Table 3.** The computed structural robustness indices using Eq. (6).

Robustness index $IR$	
Building 1 (EC8-conforming)	0.54
Building 2 (TF design)	0.59

#### 4. Conclusions

The paper proposes a methodology able to evaluate the robustness of RC frame structures to progressive collapse. Furthermore, important considerations in the large displacement inelastic response analysis of 3D frame buildings, subjected to sudden column loss, have been investigated. The obtained curves are suitable to characterise the behaviour of frame structures in terms of progressive collapse resistance. Two indicators of structural robustness aimed to give a quantitative measure of the degradation of the ultimate resistance of the structure due to the increasing of the initial damage has been proposed. The implementation of such method gives a clear identification of the critical members in a structural system; thus, it can be employed in a robust oriented design procedure able both to tackle

different damage scenarios and to optimize the structure [16, 17]. In addition, the results of this research can be useful for risk assessment and control.

Finally, the application of the proposed method on two case study structures has highlighted the inadequacies of the current TF method for improving the robustness of RC frame buildings.

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