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Structural robustness of RC frame buildings under threat-independent damage scenarios

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Abstract. This study focuses on a novel procedure for the robustness assessment of reinforced concrete (RC) framed structures under threat-independent damage scenarios. The procedure is derived from coupled dynamic and non-linear static analyses. Two robustness indicators are defined and the method is applied to two RC frame buildings. The first building was designed for gravity load and earthquake resistance in accordance with Eurocode 8. The second was designed according to the tie force (TF) method, one of the design quantitative procedures for enhancing resistance to progressive collapse. In addition, in order to demonstrate the suitability and applicability of the TF method, the structural robustness and resistance to progressive collapse of the two designs is compared.

Keywords: progressive collapse; structural robustness; pushdown analysis; nonlinear FEM; RC framed structures

1. Introduction

Structural robustness is considered as an important feature of the design and the safety assessment of structures. Modern building codes require that a structure be robust and much research on this topic has been carried out in recent years (Bazant and Verdure 2007, Mohamed 2006, EC1 2006, NIST 2007). In light of these studies, several definitions of structural robustness have been reported in the literature, as highlighted in Starossek and Haberland (2010). In the field of structural engineering, robustness is typically considered as the ability to withstand extraordinary events such as impacts, explosions or human errors, without being damaged to an extent which is disproportionate to the original cause (De Biagi and Chiaia 2013, Ellingwood 2006, Starossek 2007, Biondini *et al.* 2008). As a consequence of this definition, two main approaches can be considered to enhance the robustness of a structure. The first approach explicitly provides measures to reduce direct local damage due to extreme events by increasing the strength of key elements (NIST 2007, Starossek 2009). The second approach adopts structural measures intended to prevent the propagation of local damage to a disproportionate extent (progressive collapse resistance) (Dusenberry and Juneja 2003). The present research focuses only on progressive collapse resistance.

Many studies over the past decades have examined and proposed numerical techniques for progressive collapse analysis of reinforced concrete (RC), steel and composites framed structures (Izzuddin *et al.* 2008, Vlassis *et al.* 2008, Kim *et al.* 2009, Brunesi and Nascimbene 2014). Specific design procedures to enhance resistance against progressive collapse phenomena have been discussed in guideline documents (NIST 2007, DoD 2016, ASCE 7-02 2007).

One of the most common approaches to assess structural robustness is the alternate load path (ALP) method, discussed in GSA Guidelines 2013 and DoD 2016. ALP method is a simplified assessment technique that does not explicitly model the loading but rather evaluates the collapse resistance of the system by removing critical structural members. However, ALP method cannot provide information on the proximity to failure of the system since a structure could still have residual capacity to redistribute the loads and avoid collapse. In addition, while most of the aforementioned studies deal with disproportionate collapse, there are no specific methods developed to quantify system robustness.

Besides extensive efforts in design and simulation, the measure of structural robustness to progressive collapse is often controversial, since there are no well-established and generally 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 (De Biagi and Chiaia 2013, Biondini and Restelli 2008, Giuliani 2012).

The aim of this work is to introduce a general method for a consistent and quantitative measure of structural robustness of framed buildings against progressive collapse. In particular, robustness is assessed by comparing the performance of the structure in its original state and in a damaged state as a consequence of a threat-independent damage scenario. The procedure introduced in this study acknowledges the principal merits of the current design and

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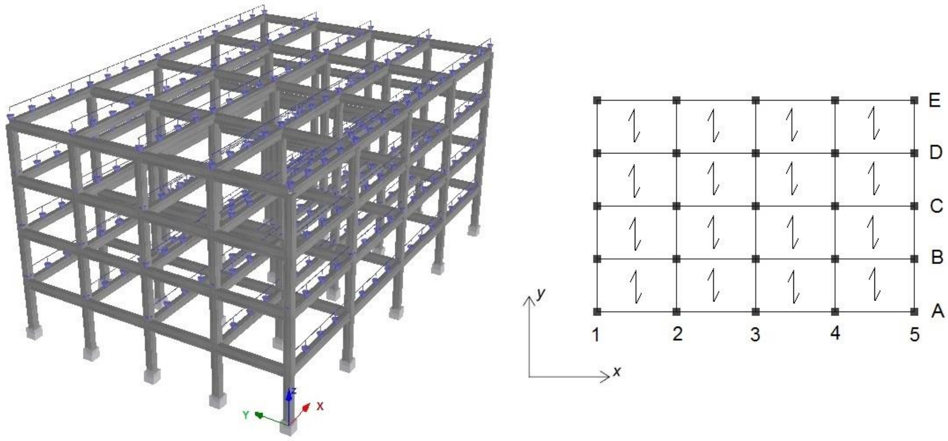


Fig. 1 Perspective and plan views of case-study building model

simulation methods and, at the same time, it is adopted to develop two robustness indicators.

The recent design guidelines (GSA 2013, DoD 2016) allow the structural analysis to be performed adopting linear or nonlinear static or nonlinear dynamic analysis. The disadvantage of linear analysis, both static and dynamic, is the inability to include material and geometrical nonlinearities such as large displacements/rotations, secondary order effects, inelastic behaviour and plastic hinge formation. Nonlinear static analysis is relatively simple and gives a capacity curve that provides insight whether a structure has adequate capacity to resist the extreme loading condition or not. One determining factor in considering a local failure is the highly dynamic effect produced when a structural element is suddenly removed from the building. When a static procedure is adopted, a dynamic increase factor (DIF) is considered to increase the gravity loads acting on the bays that are directly affected by the removed column. This factor approximately compensates for the dynamic effects corresponding to the real load redistribution. Over the last few years, a number of papers covering this subject were proposed in the scientific literature (Mohamed 2015, Liu 2013, McKey *et al.* 2012). However, as demonstrated by Pretlove *et al.* (1991) there are structures which are statically safe, but dynamically unsafe due to the fact that time-dependent overloads, induced by the element removal, may cause the progressive failure of other elements before a new equilibrium state is reached. This requires the nonlinear dynamic behaviour of a structure to be taken into account in progressive collapse simulations. Consequently, these approximate procedures generally involve the application of the DIF, whose estimation in the different possible scenarios represents one of the basic and, at the same time, most controversial aspects of both research and codification on progressive collapse (Ferraioli 2014).

The effectiveness of the proposed strategy is shown by the application to two RC frame buildings, in order to compare their structural robustness and resistance to progressive collapse. One of the buildings was designed for gravity loads and earthquake resistance according to Eurocode 8 (EC8 2004); the other structure has the same EC8-conforming building modified according to the Tie Force (TF) method. In current design philosophy, the TF method is one of the two quantitative approaches for robust design of structures (Li *et al.* 2011, Cormie *et al.* 2009). Recently, amendments and improvements of this method have been recommended by the DoD Guidelines (2016).

In the last part of this study, the reliability of the TF method is verified by the results obtained from the robustness assessment of the two case-study buildings.

2. Modelling considerations

2.1 Structural models

The structural robustness against progressive collapse is assessed in the case of two structures with certain features. Both structures were 4-stories, 4x4 bay RC framed buildings. The first structure was designed according to EC8 (2004) and the second building was the same frame structure re-designed using the TF method according to DoD Guidelines (2016). Fig. 1 shows perspective and plan views of the two buildings under investigation, which were composed of five primary frames connected 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 Fig. 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 was 3 m at each floor. The structures are subjected to the following load combination

$$\Omega_N(1.2DL + 0.5LL) \tag{1}$$

where *DL* represents dead loads and *LL* denotes live loads. This load combination is suggested in the latest version of GSA Guidelines (2013). The investigated buildings were assumed to be designed for housing, dead (DL) and live (LL) loads were assumed to be $3\,\mathrm{kN/m^2}$ and $2\,\mathrm{kN/m^2}$.

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

Building class	Element	Location	Size (mm^2)	Longitudinal reinforcement	Transverse reinforcement	Tie strength force
Seismic design (EC8)	Beam	Any line of floor level: I,II,III,IV	500x300	$3\phi20+4\phi10+3\phi20$	2-leg$\phi10$@80	/
	Column	Any line of floor level: I,II,III,IV	400x400	$12\phi20$	2-leg$\phi10$@50	/
TF-design	Beam	Peripheral x-line of floor level: I,II,III,IV	500x300	$3\phi20+4\phi10+3\phi20$	2-leg$\phi10$@80	$F_p = 166$ kN
	Beam	Internal x-line of floor level: I,II,III,IV	500x300	$3\phi20+4\phi10+3\phi20$	2-leg$\phi10$@80	$F_i = 83$ kN/m
	Beam	Peripheral y-line of floor level: I,II,III,IV	500x300	$3\phi20+4\phi10+3\phi20$	2-leg$\phi10$@80	$F_p = 110$ kN
	Beam	Internal y-line of floor level: I,II,III,IV	500x300	$3\phi20+4\phi10+3\phi20$	2-leg$\phi10$@80	$F_i = 55$ kN/m
	Column	Any line of floor level: I,II,III,IV	400x400	$12\phi20$	2-leg$\phi10$@50	/

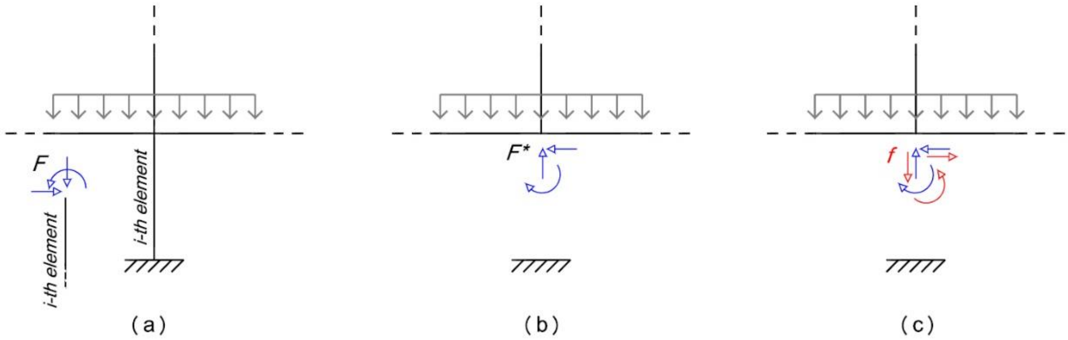


Fig. 2 Illustration of damage model for removal of a column. Details are reported in Section 2.3

The term Ω_N is the dynamic amplification factor that is adopted in the nonlinear static analysis. For framed reinforced concrete structures, Ω_N is equal to

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

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. θ_{pra}/θ_y is the ratio between allowable plastic rotation angle and the yield rotation angle (GSA Guidelines 2013). In Eq. (2), θ_{pra} is the plastic rotation angle given in the acceptance criteria tables in ASCE41 and DoD 2016 for the appropriate structural response level (life safety or collapse prevention), for the particular element, component or connection; θ_y is the yield rotation. For reinforced concrete member, θ_y is determined with the effective stiffness values provided in Table 10-5 in ASCE41. To determine the DIF (Eq. (2)) for the analysis of the entire structure, the smallest ratio θ_{pra}/θ_y is chosen for any primary element, component or connection in the model. Columns are omitted from the determination of the DIF.

The load combination has the term $\Omega_N = 1$ in the nonlinear dynamic analysis. A properly detailed slab was not considered in the building model. Even if a properly detailed slab may potentially improve structural response contributing to the redistribution mechanism of the entire building (Ferraioli 2017, Li and Sesani 2015), a one-way RC floor slab was conservatively assumed to provide no resistance against progressive collapse while its weight and inertia were implicitly included in FE simulations. This approach was also used to account for partitions walls (Brunesi *et al.* 2015).

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 simulated design of EC8-conforming was carried out adopting design strength according to European codes (EC1 2006, EC8 2004); on the contrary, mean strengths were selected for nonlinear analysis. In particular, the design strengths of concrete (f_{cd}) and reinforcing steel (f_{yd}) were derived as $f_{cd} = 0.85 f_{ck}/1.5$ and $f_{yd} = f_{yk}/1.15$, respectively. Mean compressive strength of concrete (f_{cm}) and mean yield strength of reinforcing steel (f_{ym}) were assumed to be $f_{cm} = f_{ck} + 8$ and $f_{ym} = 1.1 f_{yk}$, respectively. 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 restrictions (EC8 2004).

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 (DoD 2016).

The TF method is one of the major design strategies for resisting progressive collapse and is common to different codes and standards worldwide. TF-based design requires the designer to detail the structure such that elements are mechanically tied together.

This requirement results in an enhanced degree of ductility, continuity and development of alternate load paths (Cormie *et al.* 2011). Tie forces were provided by the existing structural elements, which are designed using conventional design techniques to sustain the standard loads imposed upon the structure (Li *et al.* 2011).

In accordance to (DoD 2016), two types of horizontal ties are provided in this work: internal and peripheral. For the framed building considered, the required tie strengths F_i and F_p for internal and peripherical ties respectively, are determined with

$$F_i = 3w_f\, L_1 \tag{3}$$

$$F_p = 6w_f\, L_1 L_p + 3W_c \tag{4}$$

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. L_p is equal to 1 m. 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 beam-column nodes. The last column of Table 1 outlines all the tie strength forces adopted in the structural members of the buildings.

2.2 Numerical techniques

Numerical techniques developed according to fiber force-based approaches (Brunesi and Nascimbene 2014, Spacone *et al.* 1996) are adopted in the FE code SeismoStruct. In particular, inelastic force-based fiber elements were used in an attempt to predict the nonlinear response of the two buildings under investigation, in both static and dynamic conditions. These elements were implemented to model the frame members, explicitly including geometric and material nonlinearities.

Geometric nonlinearity was accounted for by a co-rotational transformation, which is implemented using 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.

Although a force-based formulation does not necessarily require element discretization (Spacone et al 1996), 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 (Brunesi and Nascimbene 2014). Furthermore, the uniaxial uniform confinement model proposed by Mander *et al.* (1988) was used to represent concrete behaviour, 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 Brunesi *et al.* (2015) 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. The structural response to extreme loading conditions is expected to be dynamic since it occurs in a short time and can involve strain rate effects. These effects may modify the failure mechanism of structures and influence the collapse capacity. The effects of strain rate are generally approximated adopting suitable amplification factors on the static strain rate (GSA Guidelines 2013). In accordance to Ferraioli (2016), the present study is carried out under the hypothesis that the strain rates are in the seismic loading range, i.e., rather low, which justifies not accounting for strain rate effects in the analysis model. Thus, the strain rate effects are neglected in this study and should be investigated in the future.

In the numerical model, rigid offsets are included in the beam and column elements to ensure adequate alignment of all structural members.

2.3 Computational strategies

In this paper, 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 (Khandelwal and El-Tawil 2011, Wang *et al.* 2014).

The robustness evaluation procedure presented in the following is threat-independent, i.e., the cause of damage is unknown, thus encompasses a broad range of loading scenarios and extreme events. An instantaneous removal of the contribution of a structural member to the load bearing capacity of the system (Olmati *et al.* 2013) was considered and implemented through a three-steps pushdown analysis (Fig. 2), according to De Biagi *et al.* (2017).

First, the undamaged structure was loaded with the external loads with the load combination of Eq. (1). A nonlinear solver was considered and the forces and displacements in the elements were evaluated. In particular, the end forces acting in the potentially damaged element were recorded, i.e., the forces in Fig. 2(a).

In the next step, the damaged element was removed and a set of external forces, F^*, were added to the scheme (Fig. 2(b)); such forces are opposite to the ones of the previous step, i.e., $F^* = -F$. A nonlinear run was made and nodal

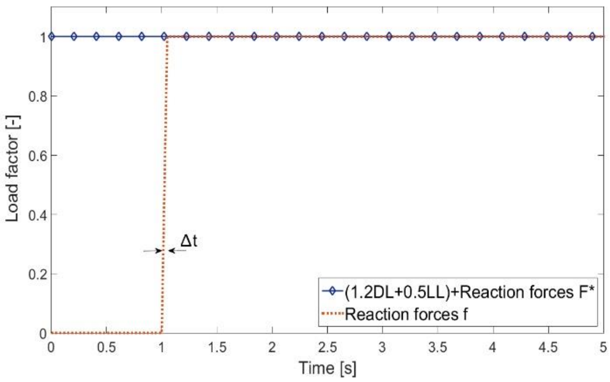


Fig. 3 Time-history function of applied load for dynamic analysis

displacement were read. They were approximately equal to the ones evaluated in the first step (Fig. 2(a)); it came out that the system of forces (i.e., F^*) correctly simulates 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 Fig. 2(c).

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 (GSA Guidelines 2013, Ferraioli *et al.* 2014). The pushdown procedures were combined with dynamic analyses. Despite nonlinear dynamic analyses (NDA) are time consuming, they are the most appropriate method to simulate the effects of the sudden loss of one or more structural members (Wang *et al.* 2014, Bao *et al.* 2008, Kwasniewsky 2010). In these procedures, 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 (DoD 2016).

The dynamic analyses were carried out following an approach similar to that of the pushdown procedures described above. An initial undamaged situation was considered by replacing a column by its reaction forces F^* (Fig. 2 (b)) and a system of forces f opposite to the forces F^* was added on the node, as sketched in red in Fig. 2(c). The system of forces f were suddenly applied to simulate the damage with a time interval (Δt) smaller than 1/10 of the fundamental period associated with the pertinent vertical modal shape of the damaged structure (DoD 2016). The implemented load factor time history is shown in Fig. 3. The sudden removal of a column in a framed structure causes the remaining damaged building to vibrate vertically and horizontally; this behaviour was analysed to determine if alternate load path and residual capacity exists to avoid propagation of the damage (i.e., progressive collapse) (Fascetti *et al.* 2015).

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 (2012), robustness can be assessed by considering the structural behaviour of the damaged configurations of the 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 procedure, which is illustrated in the flowchart of Fig. 4.

The first step 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). In this case no damages are considered, thus the pushdown procedure described above cannot be performed. In an attempt to define a performance indicator of the structure in its undamaged state, the entire vertical loads, i.e., $\lambda(1.2DL + 0.5LL)$ are increased by the multiplier λ until the failure of the building is reached. The failure of the structure is considered to occur when one of the ultimate conditions presented in Section 2.2 is reached and it is impossible to apply additional loads on the building. In the following, the performance of the undamaged structure is identified with the load multiplier λ at which the ultimate condition is reached. In detail, the performance related to the undamaged structure is indicated as λ_0.

In the second step, columns at the ground floor level of the building are alternatively removed. In particular, the location of the first damaged columns can be chosen *a priori*. In this work, external columns are initially considered (e.g., it is realistic that impact events or explosions take place in the external perimeter of the structure).

Various sets of damage scenarios are defined by the removal of a single vertical member, by two members, and so on. Parameter d specifies the number of removed columns and the performance of the building for a specific local damage level (d) is indicated as λ_d. 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 distinct 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: in this case, the vertical displacement time history for a node located on the top of the removed key element after the extinction of the initial high frequency oscillation shows a damped oscillation. Therefore, the damage response is arrested and the ultimate resistance is evaluated for the specific damage level (d). The ultimate resistance is computed with the

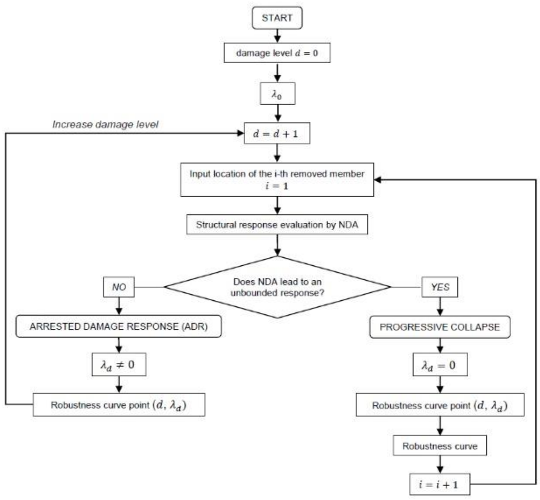


Fig. 4 Flowchart of the procedure to evaluate the structural robustness

three-steps pushdown analysis previously described. In this approach, the load multiplier is computed as the ratio of pushdown loads and forces acting on the member before its removal. The ultimate load multiplier identifies the performance of the building for the damage level d (i.e., λ_d). If case *ii*) is observed, the local damage is increased. This increment is heuristically obtained by alternatively removing, by a NDA that simulates the sudden loss, 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. Typically, it is necessary to only remove a limited subset of members since symmetry, structure layout and observations during each element removal will provide information on sequence of removals. 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 analysed; 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 Fig. 5. In this graph, the structural performance (i.e., ultimate resistance) in represented on the y-axis, while the x-axis indicates the amount of damage intended as the number of removed elements. In addition, Fig. 5 shows in graphical form the decrement of the performance ΔP and relative damage amount Δd.

The robustness curves are utilized to develop two indicators of structural robustness. The performance indicators proposed herein are used as state variables and the obtained robustness indices are dimensionless functions of these variables varying in the range [0, 1]. The first proposed index, Ir, is a local measure that expresses the decrement of the resistance for a given increment of damage amount

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

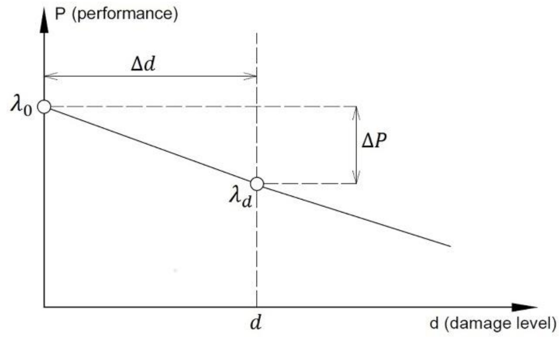


Fig. 5 Qualitative representation of robustness curve

The index can be defined for each damage level (i.e., Ir_d), and ranges from value $Ir = 0$ to $Ir = 1$ for a collapse and a robust situation, respectively.

A second index, IR, provides a quantitative measure after all the damage configurations and is defined as follows

$$IR = \sum_{d=1}^{n_d} Ir_d \tag{6}$$

where Ir_d is the first index referred to the damage level d, n_d is the total number of damage scenarios. An IR larger than 0 indicates that the structure has a certain amount of robustness.

Although the robustness already plays a fundamental role in design and represents a modern research topic in the field of structural engineering, there is no a unique method described in the literature to quantify it by means of a rational procedure and a corresponding index definition.

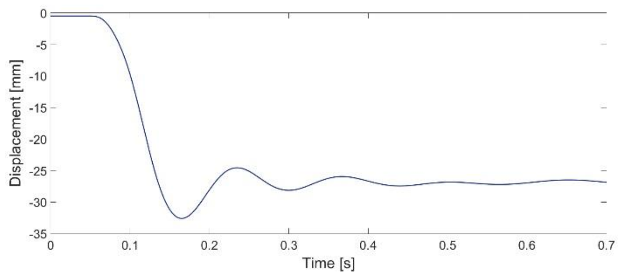
As above-mentioned, the robustness indicators proposed can be included in the formulation of damaged-based measures of robustness.

In these measures, since the definition of robustness reflects the ability of the structure to respond to damage, the comparison between the structural performance of the system in the original state and in a perturbated state is applied. For instance, Khandelwal *et al.* (2011) propose pushdown method to assess the residual capacity of the frame structures and define the overload factor, which is suggested as a robustness measure. Recently, Fascetti *et al.* (2015) define an index that provides a more quantitative measure of robustness, it is defined in function of the ultimate load multipliers.

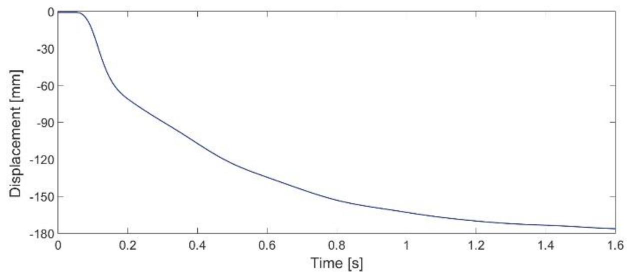
3.1 Robustness assessment

In this section, the method for evaluating the structural robustness 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 Fig. 4.

As previously stated, the first step resulted in the evaluation of the performance of the undamaged structure. Subsequently, supposing damages scenarios limited to the external ground floor elements, 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.



(a)



(b)

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

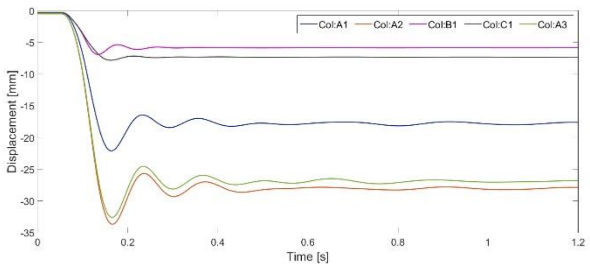


Fig. 7 Vertical displacement time-history for a node of the top of the removed column

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 Fig. 6, for both the arrested damage response and progressive collapse situation, with local damage level $d = 1$.

The first case (Fig. 6(a)) shows a bounded response with an initial oscillation and after a decaying response (damped oscillation). Fig. 6(b) shows that the removal of the selected column leads to an unbounded response, indicating progressive collapse. In case the local damage level equal to one (i.e., $d = 1$), each of the column removals resulted in a stable dynamic response. This aspect indicates that the structure will survive after the individual removal of any of the columns of this selected damage scenario. Fig. 7 depicts the vertical displacement time-histories obtained by NDA on the EC8-conforming building and damage level $d = 1$. After the dynamic analysis following the sudden removal of a column, pushdown investigation was applied.

The subsequent steps of the proposed procedures were implemented, considering the other damage scenarios. The assessment methodology was then applied to the second building designed in accordance with TF method.

Fig. 8 shows the different curves obtained from the pushdown analyses following the removal of the selected columns on the lowest floor of the building and contemplating all the damage scenarios.

In the legend of the Fig. 8 is indicated the sequence of column removals.

The computed load multipliers for each removed column in both buildings are summarized in Table 2.

The obtained robustness curves are shown in Figs. 9(a) and 9(b) 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 lowest envelope of the ultimate resistance for each damage level is highlighted with a bold line. These curves of minima are employed 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 computed robustness indicators (Ir and IR) implementing the two proposed strategies for each building are reported in Tables 3 and 4, respectively. 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).

In addition, the robustness curves and indicators highlight that no significative 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.

4. Conclusions

This paper proposes a methodology suitable for characterising the behaviour of RC frame structures in terms of progressive collapse resistance. Furthermore, important considerations for simulating the large displacement inelastic response of frame buildings, subjected to sudden column loss, are presented.

This methodology is carried out combining the most appropriate types of analyses when sudden column removals occurs, i.e., nonlinear static coupled with dynamic analyses. Robustness curves determined from the proposed method are compared and employed to define a measure of the structural robustness of a framed structure.

In addition, the proposed procedure can be easily implemented on a finite element code and is effective in terms of computational costs. These features offer the professional engineer an easy system to take into account robustness against extreme actions during the design stage. In this light, the critical member of a building can be clearly identified observing the lower robustness curve and targeted

Table 2 Summary of load multipliers for each removed column in both buildings

	Damage Level 0	Damage Level 1		Damage Level 2		Damage Level 3	
	λ_0	Identification code	λ_d	Identification code	λ_d	Identification code	λ_d
Building 1 (EC8-conforming)	4.7	col: A1	1.86	col: A1-A2	1.08		
				col: A1-B1	1.19		
		col: A2	1.44	col: A2-A3	1.08		
				col: A2-A1	1.11		
		col: A3	1.46	col: A3-A2	1.08		
		col: B1	2.69	col: B1-A1	1.32		
				col: B1-C1	2	col: B1-C1-A1	1.06
		col: C1	2.71	col: C1-B1	2.08		
Building 2 (TF design)	4.9	col: A1	2.08	col: A1-A2	1.19		
				col: A1-B1	1.31		
		col: A2	1.59	col: A2-A3	1.2		
				col: A2-A1	1.28		
		col: A3	1.61	col: A3-A2	1.2		
		col: B1	2.91	col: B1-A1	1.56		
				col: B1-C1	2.2	col: B1-C1-A1	1.28
		col: C1	2.93	col: C1-B1	2.2		

Table 3 The computed structural robustness indices using Eq. (5) (Envelope of the minima)

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

Table 4 The computed structural robustness indices using Eq. (6)

	Robustness index IR
Building 1 (EC8-conforming)	0.42
Building 2 (TF design)	0.45

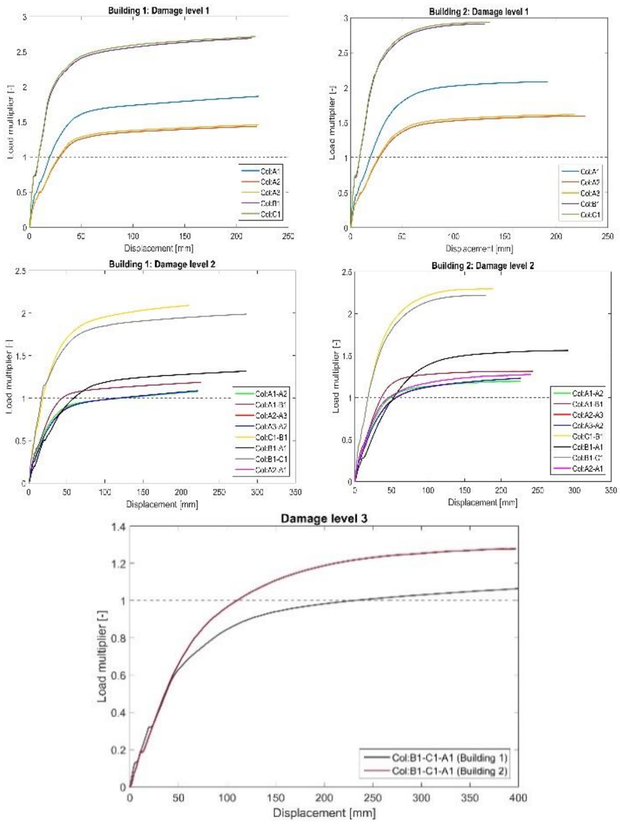


Fig. 8 Pushdown curves for all the damage levels in both buildings

solutions in the structural design can be adopted.

A design procedure that can optimise the structure is a suitable strategy in an attempt to avoid global failure in structures (De Biagi 2016).

Another significant practical implication of this research is the possibility of using the method described here for risk assessment and control procedures.

In this work, the assumption of damage scenarios at the external element level appears to be congruent with the frame structures and with the type of extreme events considered, leading to element removal, such as impact (Ventura *et al.* 2017), explosions. However, in the future, improvements to the proposed procedure could involve extending the damage at internal element level of buildings, considering other extreme events that can endanger buildings, such as fire or consequences of human error.

It is important to notice that the response of the structure to a loss of column also depends on various aspects of the structural layout. First of all, the directionality of floor slabs affects the distribution of gravity loads on primary elements of the structure (De Biagi *et al.* 2017). Additional redistribution capabilities may be researched in a properly designed and detailed floor slab system, whose resistance,

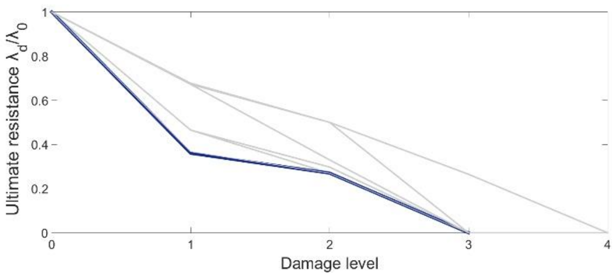


Fig. 9(a) Robustness curves for building designed according to EC8

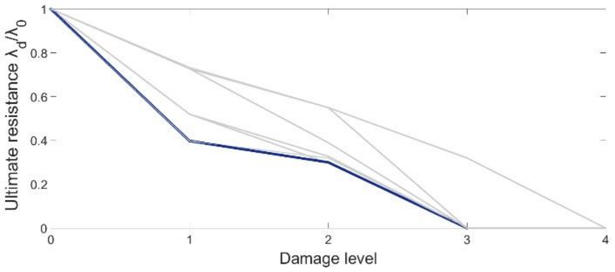


Fig. 9(b) Robustness curves for building designed according to TF method

conservatively omitted in the analyses presented in this study, is expected to provide a significant potential for alternative load paths. The second aspect is the distribution of stiffness in the frame. In the buildings analysed in this paper, no RC walls were present. Such structural elements increase the stiffness of the overall structure, enhancing its tolerance to damage. In addition, referring to the shape of the building, irregularities in plan and in elevation can affect the residual strength of the structure. These effects could also be further investigated. In spite of the aforementioned simplifying assumptions, the observed results are expected to be reproduced in a complete and more detailed approach.

Finally, the application of the proposed method on two case-study structures has highlighted some open problems in using TF method for improving the robustness of RC frame buildings. Generally, the cause of the collapse resides in the inadequacy of the structure to support the catenary effect within the unsupported bays, proving that a correct design has to be carried in order to prevent the propagation of local damage to overall collapse. Future developments of this research will deal with the definition of a new procedure that can increase the robustness of frame structures.

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