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OVERSTRENGTH FOR ENHANCING THE ROBUSTNESS OF STRUCTURES

Antonio Ventura[1], **Bernardino Chiaia**[1] **and Valerio De Biagi**[1]

[1] Department of Structural, Geotechnical and Building Engineering, Politecnico di Torino
C.so Duca degli Abruzzi 24, 10129, Torino, Italy
e-mail: {antonio.ventura,bernardino.chiaia,valerio.debiagi}@polito.it

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Abstract. *In the present paper, a methodology able to increase the robustness of a frame structure is proposed. This methodology focuses on the structural behaviour of the key elements, which are endowed with a certain overstrength that can be employed to enhance the capacity of the whole structure to withstand a specific hazard scenario. The "loss of a column" can be an example of the possible scenarios that a robust structure should withstand. The additional resistance due to overstrength can be adopted to reduce direct local damage due to accidental events (local strengthening) and to prevent the propagation of local damage to a disproportionate extent (progressive collapse resistance). Furthermore, this structural measure guarantees the formation of alternative load paths and, thus, the capability of the structure to redistribute the loading due to a local damage among the remaining structural components.*

The paper aims at evaluating the role of overstrength in the design (or in the retrofit) of structural members and in the development of several collapse resisting mechanisms of the whole structure. Those considered for framed structures are: compressive arching, flexural and tensile catenary action developed in the beams adjacent to the initial damage, as well as membrane actions developed in the supported floors. In addition, the influence of the portion of structure adjacent to the area directly affected by the damage is considered.

The procedure proposed in the present paper suggests a strategy that is suitable for a preliminary robustness-oriented design, involving specific solutions in terms of redundancy, local resistance and mechanical properties of the structural members.

1 INTRODUCTION

In recent years, considerable researches have grown up around the theme of structural robustness and the behaviour of structures in presence of extreme actions. In addition, modern building codes require that a structure be robust [1, 2, 3]. Many of these codes specify that structures should be robust in the sense that the consequences of structural failure should not be disproportional to the effect causing the failure.

The term "robust" is often considered as a synonym of the term "strength", but in many fields of science and engineering their meanings are well distinct. Consider a generic system that be able to sustain a generic action load, which can be mechanical load, energy, etc. The strength of the system is the maximum load that can be sustained. On the contrary, robustness is the ability to preserve functionality in extreme scenarios.

A system preserving adequate strength in presence of perturbations due to extreme events is robust toward these events. Robustness is therefore related to the strength S_k of the system subjected to the generic k-th perturbation [4]:

$$S_k = S_0 S_{R,k} = S_0 \frac{1}{S_{L,k}} \tag{1}$$

where S_0 is the strength of the unperturbed system, while $S_{R,k}$ and $S_{L,k}$ are the residual strength and the strength loss fractions associated with the k-th perturbation.

Therefore, robustness can be improved either increasing S_0 in Eq. (1) or minimising the strength loss by optimising the system.

In the past studies, different strategies have been proposed to enhance the robustness of structures [5, 6]. A first strategy consists in guarantee continuity and integrity of structures, this strategy is part of a more general approach that is the Alternate Load Path method. Indeed, continuity and integrity can provide alternatives for a load to be transferred from a point of application to a point of resistance. Compartmentalization of structures is a second approach to enhance structural robustness. In this strategy, the propagation of collapse after an initial local damage is limited to pre-established portions of structure [7]. Compartmentalization prevents the interaction of the collapsing members with the remaining structure, following a contrary line of thought from that of the first strategy. A third strategy foresees a sufficient strength to key structural elements. This is the principle at the base of the Specific Load Resistance strategy for robustness [8, 6]. Increase the resistance of some structural members is a measure supported by different studies, for the simplicity of application and for the multivalent effects, according also to the multi-hazard approach [9, 10].

In the present work a novel methodology able to increase the robustness of a framed structure is proposed. This methodology highlights that giving an overstrength to certain structural members of a frame structure can be enhanced the capacity of the whole structure to withstand a specific hazard scenario. The "loss of a column" can be an example of the possible scenarios that a robust structure should withstand. In the light of the connection between the concepts "robust" and "strength" introduced in Eq. (1), the role of overstrength in a robust design can be adopted in two main plans. The first plan is to increase the strength of the unperturbed system, S_0 in Eq. (1), and thus can be adopted to reduce direct load damage due to accidental events (i.e., local strengthening). Furthermore, the overstrength guarantees the formation of alternate load paths and, thus, the capability of the structure to redistribute the loading due to a local damage among the remaining structural components; according to this second plan can be minimized the strength loss $\frac{1}{S_{L,k}}$. In the present study, the role of overstrength is evaluated for reinforced concrete (RC) frame structures.

In addition, an analytical method for modelling the response of RC frame following column removal, using conventional structural analysis principles, is developed in this paper. It is demonstrated that the new analytical method may well address the principal features of structural behaviour to a similar degree of correctness as detailed numerical models. Careful validation against available experimental test is also implemented. Consequently, it can give advances in evaluating the basic mechanics of the problem. This analytical method is adopted in a strategy suitable to enhance robustness of framed structures involving the role of overstrength.

The methodology introduced in this study states new considerations regarding the role of overstrength and, in the end of this paper, it is adopted to develop an overstrength indicator that can suggest a perspective of robustness-oriented design.

2 ANALYTICAL REPRESENTATION OF THE FRAME STRUCTURE RESPONSE FOLLOWING COLUMN REMOVAL

In a frame structure, the objective of robust design is to avoid the propagation of failure throughout the structural members induced by unbalanced loads after a damage scenario [8, 11]. In an attempt to achieve this objective, the critical state of a frame structure is analysed considering an initial failure of a local member. At the initial phase, when a local element fails, the residual structural components that can provide the resistance to the unbalanced loads are referred to as the “collapse-resisting substructure”. Figure 1 shows a RC frame structure under column loss. In a regular 2D frame following column removal, the resistant substructure contains all the horizontal beams and the vertical columns located above the removed column. If adequate resistance can be provided by this substructure, further failure or collapse of the residual structural members can be prevented. In the opposite case, subsequent damage will occur, which may trigger a propagation of collapse to the entire structure [12]. In light of these aspects, also the rest of the structure (i.e., the lateral parts and the stories under the lost column) can provide a contribute to the whole structure resistance and can be referred as the “indirectly affected part” [13]. In the existing literature different experimental [14] and numerical [15, 16] studies have demonstrated that the propagation of damage, in the initial stage of a column removal event is resisted by a flexural mechanism, also referred as beam mechanism. On the other hand, tensile membrane actions (i.e., catenary mechanism) can significantly improve the resistance to progressive collapse for structures undergoing large deformations [17].

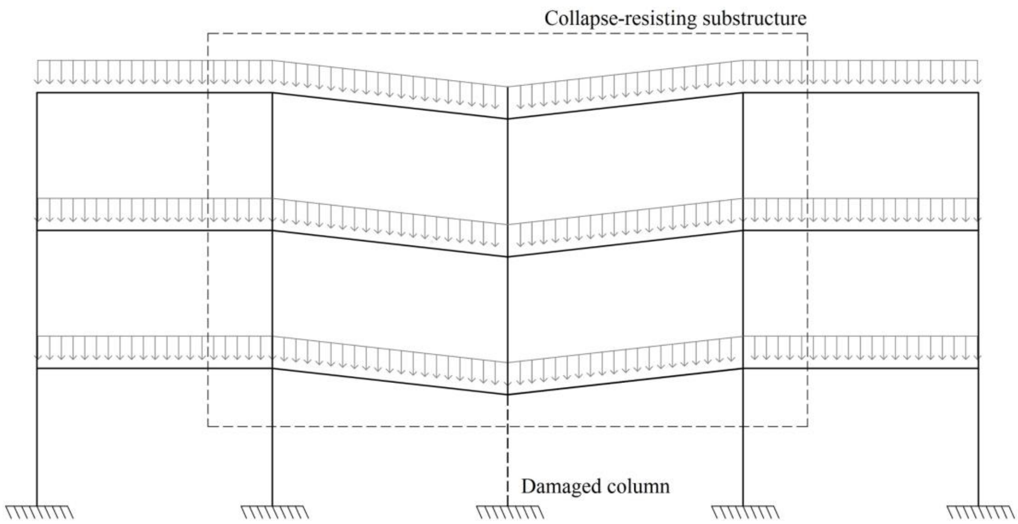


Figure 1: RC frame structure under column removal.

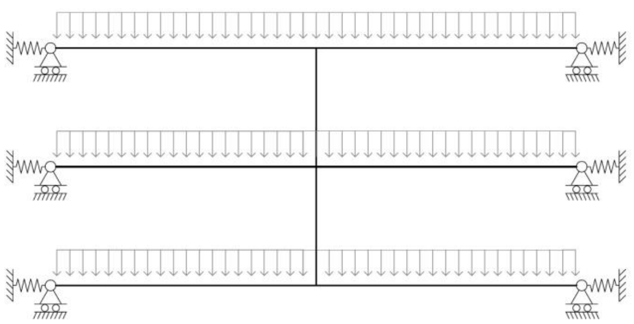


Figure 2: Sub-structural level for collapse evaluation.

An analytical method for describing the nonlinear static response of this mechanism of RC beams and columns following column removal is developed in this section. Figure 2 shows the substructure investigated for collapse evaluation following column removal.

Specified that the substructure is symmetric in terms of geometry and loading with respect to the centreline column as depicted in Figure 2, therefore, consideration may be given only to half of the substructure.

Furthermore, the behaviour of each individual beam of this substructure is modelled as shown in Figure 3. The behaviour of the beam sections is considered as linear elastic and therefore it can be analysed referring to the flexural and axial rigidity (i.e., EI, EA).

Plastic effects are expected to develop only in correspondence of the connections. The behaviour of the adjacent structure is modelled by employing a series of linear elastic boundary springs. In conformity with the position of this substructure into the frame, a degree of axial restraint may be provided to simulate interaction with the surrounding structure. This approach represents an extension of the connection mechanical approach that has been adopted in several recent numerical and experimental studies of progressive collapse [18, 19, 20]. A detailed description of the stiffness provided by the structural components in the adjacent structure is given next.

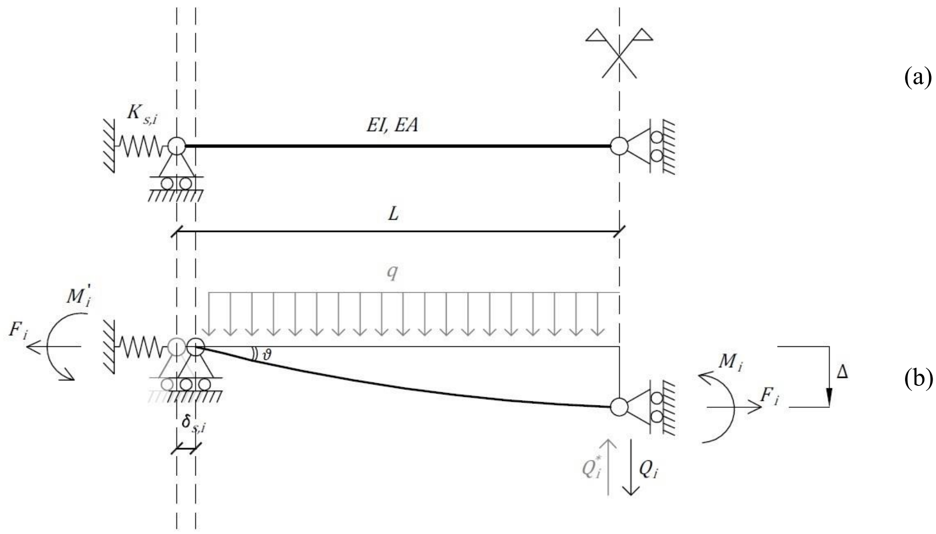


Figure 3: Modelling of single beam behaviour following column loss. (a) Structural representation; (b) Component forces and deformations.

Different schemes of loading are considered including uniformly distributed load (q) and two mid-span point load: Q^* and Q. Q^* is a force that simulates the presence of the structural supporting member in the initial state, before the damage. In addition, a force Q opposite to Q^* is considered. The force Q can be progressively increased from zero following a displacement controlled incremental scheme, this procedure can simulate the member removal [21, 22].

The forces and component deformations developed following the application of the schemes of loading described before are depicted in Figure 3 and are related to the story level i; this story level i is specified as subscript in the terms of the forces and component deformations. Employing conventional structural analysis principles, explicit relationships between the gravity loading (q and Q) and the beam deflection (Δ) as well as analytical expressions linking the component forces (i.e., M, F) and deformations (i.e., δ_s,ϑ) with the gravity loading and beam deflection are derived. The set of those analytical expressions forms an explicit method for describing the complete behaviour; a detailed description of the method is given in the next sections. In details, two principal stages are considered, flexural and catenary mechanism stage respectively [17].

2.1 Flexural mechanism stage

The flexural mechanism stage is exhibited in presence of relatively low deflections (Δ) after the column removal (Figure 3). As consequence of column removal, the support connections of the double-span beams are subjected to hogging bending moments (M') whereas the mid-span connections are subjected to sagging bending moments (M) as depicted in Figure 3. In an attempt to simulate the column removal, the additional load Q was applied progressively to the damaged column's top point. The load Q acting on the substructure produces the bending moment on the beam with the highest value at both ends and in the node beam-middle column. An elastic-perfectly plastic behaviour of the substructure is considered. The plastic limit is considered at the moment that the plastic hinges are formed within the simplified system as illustrated in Figure 4. The limit plastic moment of the beam section is uniform and equals to M_P. Once the plastic moment capacity is reached, the section can rotate and it behaves like a hinge, except with moment M_P at the hinge.

In RC frames, when the bending moment in beam sections reaches the plastic limit, the axial force in the middle column is still less than its resistance, in typical situations. In case this situation is not satisfied, the last resistance value is the fully plastic yield of the middle column under tension, simple modifications of the analytical model are possible.

In presence of relatively low deflection Δ, the bending moment diagram depends both on the connection properties and the flexural stiffness of the beams.

The collapse resisting substructure is presented in Figure 4, its stiffness can be defined by combining the individual members' stiffness. These stiffnesses are connected by parallel or series connections according to the position of individual members. The idea of stiffnesses connections was first proposed by [23] to predict the influence of fire on a frame. In the case model illustrated in Figure 4, the substructure's stiffness is described as the equivalent stiffness of the series of springs in Figure 4(b), indicated as K_{eq}. In simple manner, the bending stiffness of equivalent beams and the axial rigidity of the middle columns are connected.

In the case considered, when the beam sections reach their plastic limit, the bending moment equals M_P and the force acting on the substructure takes on the value:

$$Q^B = \frac{4n_{st}M_p}{L} \tag{2}$$

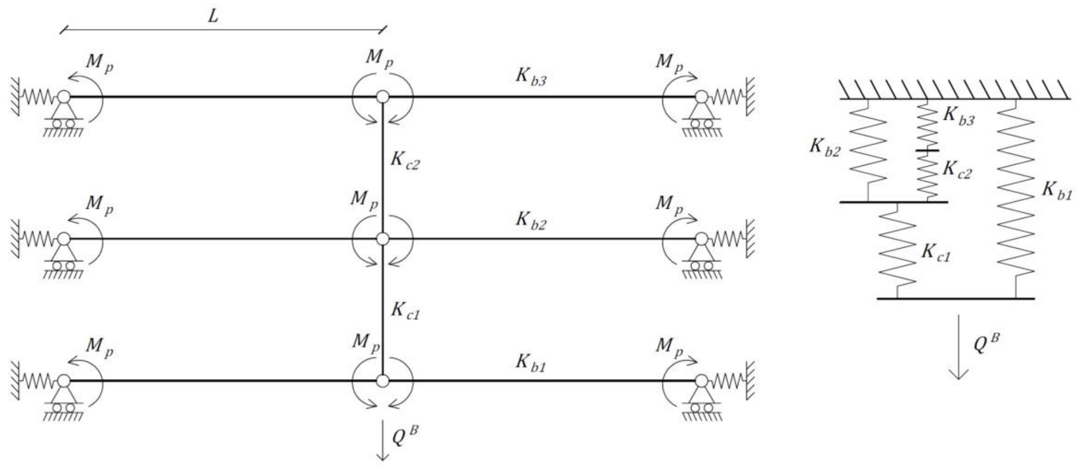
The displacement of the node located on the top of the removed member (i.e., the loaded point) is

$$\Delta^B = \frac{Q^B}{K_{eq}} \tag{3}$$

where n_{st} is the number of stories of the collapse-resisting substructure, Q^B is the whole equivalent beam's resistance, Δ^B is the displacement of the loaded point and M_p is the plastic bending moment of the beam section.

In the light of the elastic-perfectly plastic behaviour of the members, when the bending moment reaches that limit, the section fully yields. In this stage, none of the beam can withstand the additional load anymore, as a result, the structures totally collapse.

The load-deflection relation in the flexural mechanism stage can be represented as depicted in Figure 5.



(a) (b)

Figure 4: (a) Resistance of the collapse-resisting substructure. (b) Equivalent stiffness for the substructure.

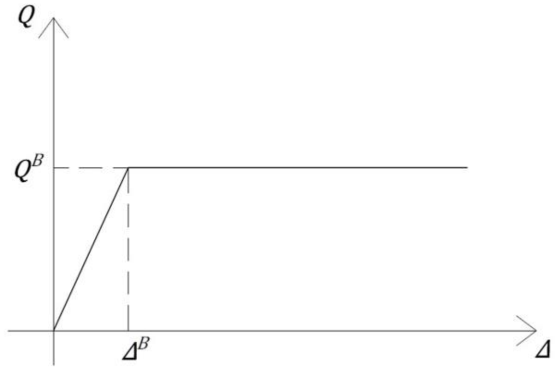


Figure 5: Load-deflection relation in the flexural mechanism stage.

2.2 Catenary action stage

The behaviour of the substructure during the flexural mechanism state is quite simple to derive, as usual limit analysis methods, based on the theorems of plastic failure, can be adopted. On the contrary, during the catenary action stage the analysis of the structural frame become complex as significant second order effects are developing [17]. This stage is influenced by the presence of axial load in the beams at relatively large deflections (Δ). In the present section, the equations needed to predict the defined substructure response during catenary mechanism stage are presented. These equations are related to two distinct parts of the frame: the collapse-resisting substructure and the indirectly affected part. Provided the substructure is symmetric in terms of geometry and loading with respect to the centreline of the removed column, the analytical model for this stage is introduced for a single beam, as that depicted in Figure 6. In one single beam (Figure 6) the following equilibrium equations can be obtained considering force equilibrium and compatibility of displacements. In the equations proposed, the number of stories of the substructure (i.e., the directly affected part) is equal to n_{st}, therefore the equations derived for one beam have to be considered n_{st} times. The story level is indicated with the subscript i.

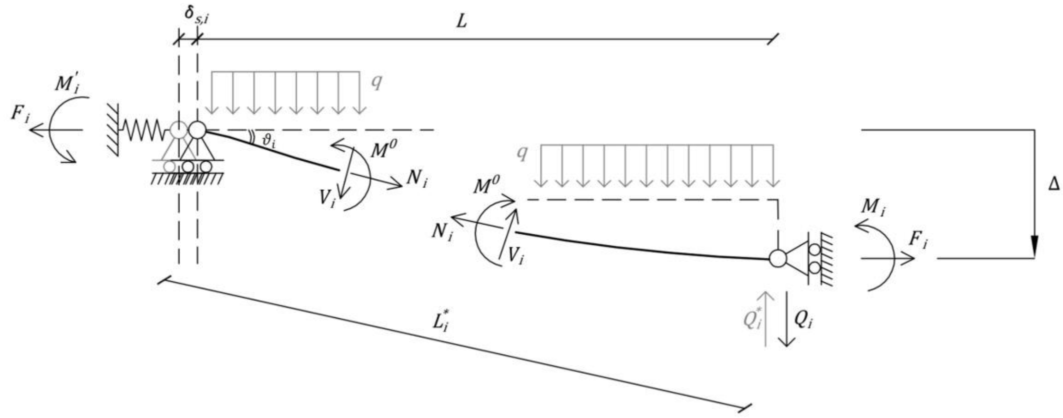


Figure 6: Modelling of beam behaviour in catenary action stage; equilibrium of the system.

$$N_i = F_i cos\vartheta_i + Q_i sin\vartheta_i \tag{4}$$

$$F_i\Delta = Q_i(L - \delta_{s,i}) + V_i L_i^* + \frac{q(L - \delta_{s,i})^2}{2} \tag{5}$$

$$V_i = Q_i cos\vartheta_i - F_i sin\vartheta_i \tag{6}$$

$$L_i^* = (L - \delta_{s,i}) + \frac{\Delta^2}{2L} \tag{7}$$

$$tan\vartheta_i = \frac{\Delta}{L - \delta_{s,i}} \tag{8}$$

$$sin\vartheta_i = \frac{\Delta}{L_i^*} \tag{9}$$

$$\delta_{s,i} = \sum_{j=1}^{n_{st}} c_{ij} F_j \tag{10}$$

$$Q_{tot} = 2 \cdot \sum_{i=1}^{n_{st}} Q_i \tag{11}$$

The simulation accounting for this damage model was implemented by a force Q_i, opposite to the force Q_i^*, that was added to the node at the top of the removed column. Thus force Q_i was gradually increased form this value following a displacement controlled incremental scheme. In these equations, Q_{tot} is the total force applied to the substructure simulating the loss of the column; while, Q_i is the part of that force supported by the storey i-th. In terms of displacements, all the stories have the same vertical displacement, equal to Δ (i.e., it is assumed that the columns elongation between the floors can be neglected) [21, 22].

The equations for the indirectly affected part are displacement compatibility equations. Essentially, the effect of the indirectly affected part is considered through horizontal springs in the external connections of the defined substructure. The displacement $\delta_{s,i}$ at each storey is common to the resisting substructure and to the adjacent structure by the Eq. (10). These springs are considered elastic as the adjacent structure is assumed to be perfectly elastic, in a first approximation. The term c_{ij} is derived from the flexibility matrix of the frame that represents the zone adjacent to the substructure (i.e., the indirectly affected part). In details, c_{ij} is the horizontal displacement at the level i when and unitary force acts at the level j and F_j is the horizontal load applied at storey j. In Eq. (7) the term $\Delta^2/2L$ is the total axial deformation expressed with respect to the beam deflection. This axial deformation is based on the second-order approximation shown in Figure 7 and in accordance to [24].

The initial reaction Q^* is progressively reduced to reproduce column loss until a collapse mechanism occurs. The reaction curve $Q^* - \Delta$ is captured by the proposed analytical model. The objective is to determine a $Q^* - \Delta$ curve reflecting the behaviour of framed structure to estimate the distribution of loads within the structure and finally to check if the structure is able or not to reach the point $Q^* = 0$ that indicates the complete column removal.

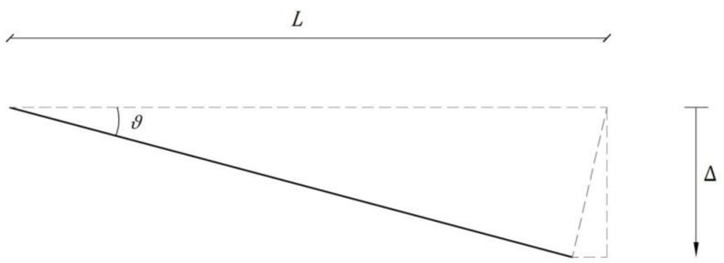


Figure 7: Approximation of second-order geometric effects.

The unknowns and equations obtained from the study of the model in Figure 6 are reported in Table 1. In the same table, the number of equations and unknowns is reported. It can be seen, from the data in Table 1, that the number of equations is equal to the number of unknowns, therefore this system of equations can be numerically solved for different values of displacement Δ.

Equations	Number	Unknowns	Number
$\Delta = input\ data$	1	Δ	1
$N_i = F_i cos\vartheta_i + Q_i sin\vartheta_i$	n_{st}	N_i	n_{st}
$V_i = Q_i cos\vartheta_i - F_i sin\vartheta_i$	n_{st}	V_i	n_{st}
$L_i^* = (L - \delta_{s,i}) + \frac{\Delta^2}{2L}$	n_{st}	L_i^*	n_{st}
$F_i\Delta = Q_i(L - \delta_{s,i}) + V_i L^* + \frac{q(L - \delta_{s,i})^2}{2}$	n_{st}	F_i	n_{st}
$tan\vartheta_i = \frac{\Delta}{L - \delta_{s,i}}$	n_{st}	ϑ_i	n_{st}
$sin\vartheta_i = \frac{\Delta}{L^*}$	n_{st}	Q_i	n_{st}
$\delta_{s,i} = \sum_{j=1}^{n_{st}} c_{ij} F_j$	n_{st}	$\delta_{s,i}$	n_{st}
$Q_{tot} = 2 \cdot \sum_{i=1}^{n_{st}} Q_i$	1	Q_{tot}	1

Table 1: Equations and unknowns for the analytical model of catenary action stage.

3 ASSESSMENT OF COLLAPSE RESISTANCE DEMAND

The procedure proposed in the present section suggests a strategy for defining a certain contribute of overstrength in order to satisfy the collapse resistance demands of a frame structure. In details, in the regular substructure shown in Figure 2, when the collapse resistance demands of the beams are satisfied, the demands of the substructure is also satisfied. Furthermore, the collapse resistance demand of the columns and part of frame immediately next to the damaged area needs to be satisfied. If these adjacent parts have low level of strength, the overloads due to the initial damage can not be carried and therefore collapse can not be arrested. In presence of a column removal from the structure, the loads are applied to the connecting beams, which act at as an alternative load path in transferring these loads to the adjacent columns and portion of structure. If the elements that are involved in this load path have a certain overstrength, they can withstand these loads in addition to their existing loads, the collapse is arrested and the structure is stable in its damaged condition. On the other hand, if these elements do not have sufficient strength and residual capacity to withstand the additional demand, they also fail and the collapse propagates.

The considered members are key elements in the frame structure. Furthermore, the overstrength ensures that the effects of the loss of a column are not disproportionate, at the same time, the key elements are hardened to withstand the threat and ensure it is not allowed to fail. In this work two resisting mechanisms are considered fundamental to the robustness problem, catenary mechanism in the structural frame and the behaviour of the columns and portion of structure immediately next to the damaged area.

3.1 Resistance curve for RC frames under catenary mechanism

In an attempt to evaluate the demands of the beam elements under catenary mechanisms, the two framed beams of the first level are isolated from the substructure. These beams are presented in Figure 8. A progressive damage in the framed beams can be resisted by the structural resistance R^c and the elemental resistance; which is, the axial tensile forces (F_{C1} , F_{C2}) in the beams. The framed beams are considered subjected to a concentrated load; in practice, when a damage occurs in a frame structures due to the local failure of a vertical element, a large concentrated load is expected on the top of the missing element. When the frame loses one of its column is possible to derive the evolution of the reaction force, i.e., Q^*, respect to the vertical displacement Δ at the top of the removed column (i.e., $Q^* - \Delta$ reaction curve).

Applying the method proposed above is known the total force applied to the substructure simulating the loss of a column, Q_{tot}, in function of the displacement Δ. In details, when a damage occurs in the intermediate floors of a multi-story building due to the local failure of a vertical member, a large concentrated load from the upper stories columns is expected on the top of the missing element.

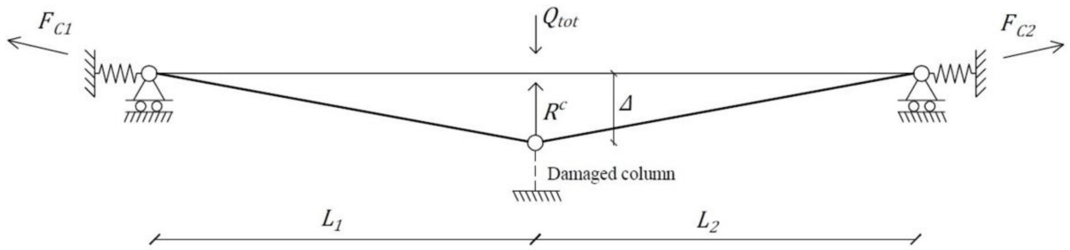


Figure 8: Schematic diagram of framed beam under catenary mechanism.

According to [25] the straight-type catenary mechanism shown in Figure 8, R^c, F_{C1} , F_{C2} satisfy the following equations:

$$R^c = \frac{(L_1 + L_2)\,\Delta}{L_1 L_2} F_{C1} \tag{12}$$

$$F_{C1} \approx F_{C2} \text{ when } \Delta \leq 0.2 \times \min(L_1, L_2) \tag{13}$$

The discrepancy in the axial forces in the two beams is less than 2% if the beam length ratio becomes as large as 10, which is rarely practical [25]. Under the straight-type catenary mechanism, the structural resistance under catenary mechanism R_L^c before the beams yield is

$$R_L^c = \frac{(L_1 + L_2)\text{EA}}{2L_1^3 L_2} \Delta^3 \tag{14}$$

Corresponding to the expression in Eq. (14), the curve OA in Figure 9 represents the behaviour of the catenary action before the beams yield. After yielding of the longitudinal reinforcing bars in the beams, the axial force F_{C1} remains the same as the yield force F_y. The structural resistance under the catenary mechanism R_N^c after yielding of the beams is the following:

$$R_N^c = \frac{(L_1 + L_2)F_y}{L_1 L_2} \Delta \tag{15}$$

The relationship in Eq. (15) refers to the line AB in Figure 9, which represents the catenary action following tension yielding in the beams. Applying the method proposed above is known the total force applied to the substructure simulating the loss of a column, Q_{tot}, in function of the displacement Δ. In the graph in Figure 9, the value R_{eq}^c, represented by Point B, satisfies the equilibrium condition under the unbalanced load Q_{tot}. Therefore, if adequate resistance can be provided by the beams, the collapse resistance demand of the substructure is satisfied, further failure or collapse of the residual structural members can be prevented.

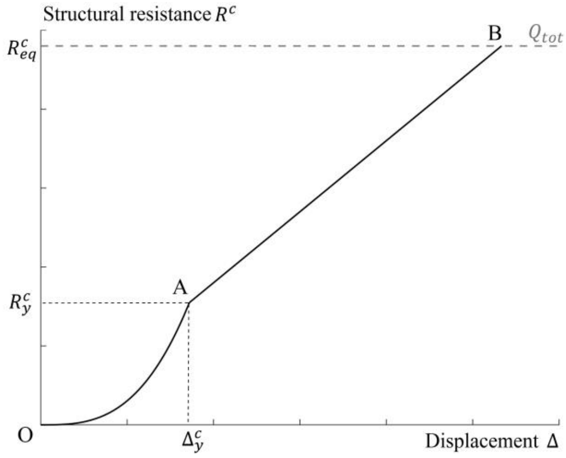
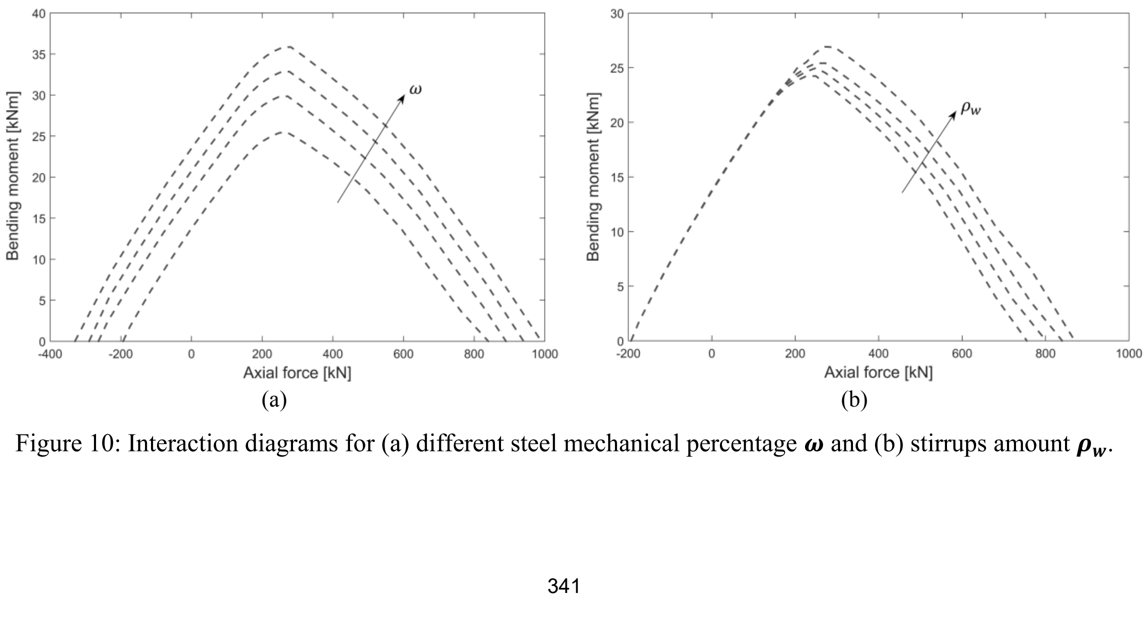


Figure 9: Resistance demands of RC frame structures under catenary mechanism.

3.2 The role of indirectly affected part and adjacent columns

The collapse-resisting substructure supporting the load transfers this load to the columns at the end of the beams [26]. In the light of the formation of an alternate load path, the additional load acting on the directly affected part is fully transferred to the adjacent columns. On each floor, the system of beams composing the collapse-resisting substructure deform after a column removal. Then the reaction forces are transferred to the adjacent columns. In each floor, the adjacent columns section must support a combined load of compression and bending moment. Therefore, the adjacent column in compression and bending is a dangerous situation that have been identified. The adjacent columns can be retrofit, additional longitudinal and transversal reinforcement can be guarantee in order to resist to the combined load of compression and bending. For a practical point of view, the design (or retrofit) of RC adjacent columns is performed by mean of interaction diagrams. These diagrams are convex domains, loci of the pairs of resistant axial force and resistant bending moment. The behaviour of the cross section of the adjacent columns is defined in function of the strength of the two materials (i.e., concrete and steel), of the geometrical sizes, the quantity of longitudinal reinforcement (i.e., steel mechanical percentage ω) and to the confinement effect due to the stirrups (i.e., ρ_w). Then the design of the section is performed and an additional strength is specified for these structural members. In the designing of the adjacent columns an adequate level of ductility is also guaranteed. Typical interaction diagrams are shown in Figure 10; these diagrams are calculated for different steel mechanical percentage and stirrups amount to find the best structural design solution.



(a) (b)

Figure 10: Interaction diagrams for (a) different steel mechanical percentage $\boldsymbol{\omega}$ and (b) stirrups amount $\boldsymbol{\rho_w}$.

4 VERIFICATION STUDY

The analytical method presented in the previous section is verified herein against experimental observations and detailed numerical models. In details, numerical techniques developed according to the line of fiber force-based approaches, have been adopted in the open FE code SeismoStruct [27]. Furthermore, the strategy to increase the robustness of frame structures is applied to the case-study frame.

4.1 Experimental benchmark

In this section, the validation of the proposed model has been done through a comparison with the experimental observation in Yi et al. [14]. The experimental setup in [14] consists of a plane frame made of reinforced concrete (Figure 11). Columns are square in section ($200\text{x}200\ \text{mm}^2$), beams are rectangular ($200\text{x}100\ \text{mm}^2$). The position of each column is identified by means of a letter (from A to E). Floor levels are labelled by Roman numbers from I (level at +1.567 m) to III (top level at 3.767). The interstorey height is 1.567 m at first floor and 1.1 m in the other floors. The strength and ultimate strain of concrete and steel are specified in [14]. The mid-column at the first floor is replaced by jack that provides an upward vertical force Q^*. In the middle of the top floor, a servo-hydraulic actuator applies a constant downward vertical force $N = 109\ \text{kN}$, to represent the self-weight of upper stories. Initially $N = Q^* = 109\ \text{kN}$, and then it is progressively reduced to reproduce quasi-static column loss, until a mechanism triggers collapse (see Figure 11(b)). During the experiments, the increasing values of the mid-span inflection Δ is plotted against Q^*, to get the force-displacement reaction curve.

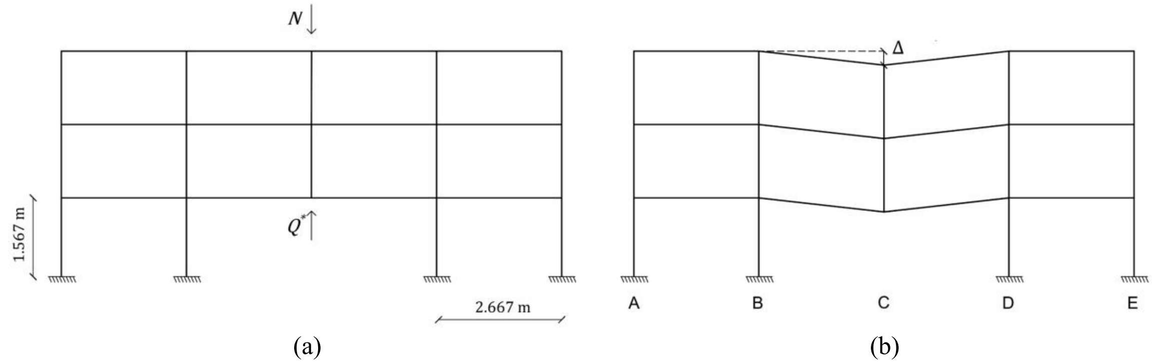


Figure 11: (a) Geometry of the experimental model frame and (b) generic collapse mechanism.

The experimental results are compared to the analysis of progressive collapse according on the proposed analytical model and finite element method (FEM). The numerical validation is accomplished using the nonlinear finite element program SeismoStruct.

In details, inelastic force-based fiber elements have been used in an attempt to predict the nonlinear response of the frame under investigation. 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 of 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.
Furthermore, 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 [22, 28]. The uniaxial uniform confinement model proposed by [29] was used to represent concrete

behaviour, while a bi-linear idealization, combined with isotropic strain hardening, was assumed for steel. Figure 12 presents a comparison of the test results with numerical and analytical solutions. It is observable that the prediction is accurate.

Throughout the comparison with the test results, the reliability of using the analytical and FE model in collapse simulation is verified. Therefore, the methodology discussed above is implemented. The resistance demand of the frame structure under the catenary mechanism is evaluated. At the same time, the collapse resisting demand of the columns and part of frame immediately next to the damaged area is considered.

The diagrams of the resistance demands evaluated in this case-study are reported in Figure 13. The elements that are involved in this loadpath are re-designed with a supplemental strength and residual capacity to withstand the additional demands in consequence of a damage scenario. These elements are endowed with a certain overstrength, this methodology is referred to as overstrength design.

Table 2 outlines the section properties of beams and columns in terms of member size and reinforcement layout for the initial design. In the same table, the results of the overstrength design are summarized.

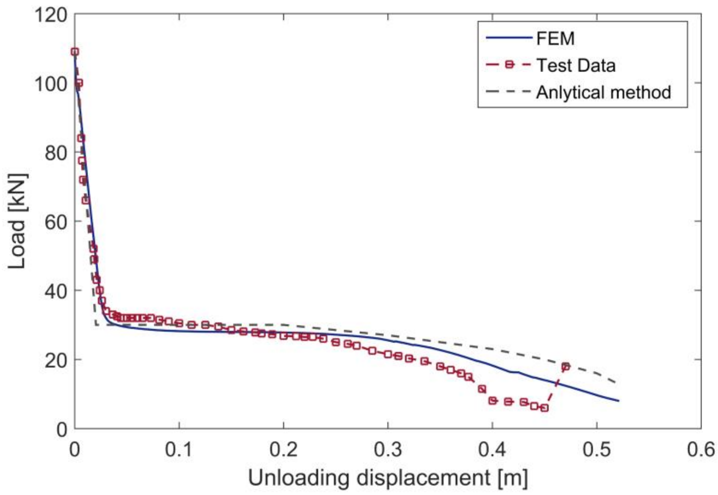
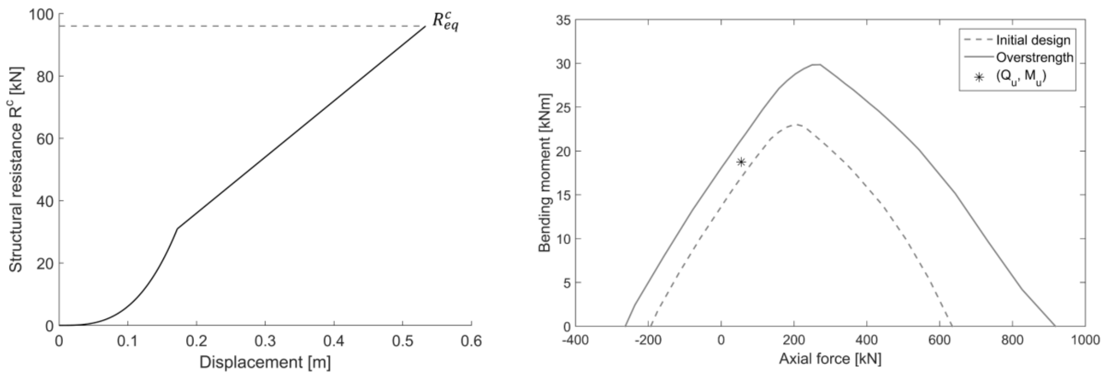


Figure 12: Experimental reaction curve and predictions from analytical method and FEM.



(a) (b)

Figure 13: (a) Diagram of collapse resisting demand under catenary mechanism; (b) Diagram of collapse resisting demand of adjacent columns.

	Element	Location	Size (mm^2)	Longitudinal reinforcement	Transverse reinforcement
Initial design	Beam	Any line of floor level: I,II,III	200x100	$4\phi12$	2-leg$\phi6$@150
	Column	Any line of floor level: I,II,III	200x200	$4\phi12$	2-leg$\phi6$@150
Overstrength design	Beam	Any line of floor level: II,III	200x100	$4\phi12$	2-leg$\phi6$@150
	Beam	Any line of floor level: I	200x100	$6\phi12$	2-leg$\phi6$@150
	Column	Line A,C,E of floor level: I,II,III	200x200	$4\phi12$	2-leg$\phi6$@150
	Column	Line B,D of floor level: I,II,III	200x200	$4\phi14$	2-leg$\phi8$@100

Table 2: Section properties from initial and overstrength designs.

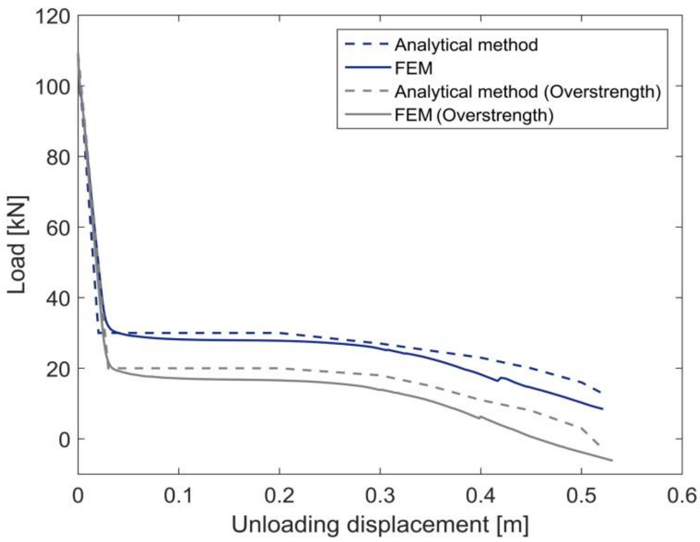


Figure 14: Reaction curves in the case of initial and overstrength designs derived from analytical and numerical methods.

5 DEFINITION OF AN OVERSTRENGTH INDICATOR

In this section, an indicator of overstrength is defined. The procedure reported above is executed and the performance of the building in the initial design and after the application of certain overstrength (i.e., overstrength design) is compared. The comparison is performed in terms of force-displacement reaction curve. The curves in the two cases are depicted in Figure 15, which outlines a typical situation. In conformity with the reaction curves depicted in Figure 15, an overstrength index is proposed:

$$\Omega = 1 - \frac{a}{b} \tag{16}$$

The proposed index is a local measure of overstrength for a given damage amount and is a dimensionless factor varying in the range [0,1]. The index has a value $\Omega = 1$ if the overstrength makes the structure robust. On the contrary, $\Omega = 0$ states that structure does not have sufficient overstrength to avoid collapse.

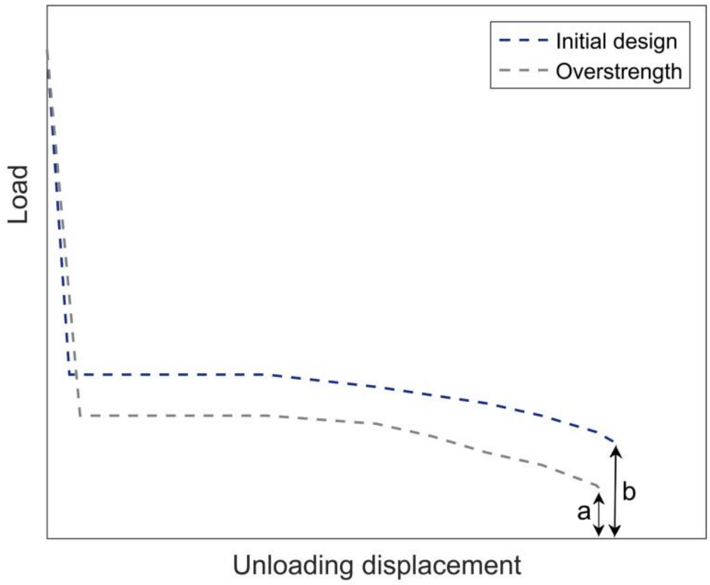


Figure 15:Typical reaction curve for initial and overstrength design.

In this section, it is now considered a real building in contrast to the test structure reported in the previous section; a EC-8 conforming frame is considered. A parametric analysis is performed. The controlling parameter is represented by a shape factor ξ, that is $\xi = L/h$; where, h is the length of the columns and L is the distance between columns. The value of ξ ranges from 0.033 to 4. In other words, saying that h is kept constant at 3 m, $L_{min} = 0.1$ m and $L_{max} = 12$ m. In this parametric analysis the monitored quantity is the Overstrength index Ω.

In Figure 16, the overstrength indicator of the framed structure is plotted as function of ξ. As can be seen, for $\xi \to 4$, that is as the distance between the columns increases, the frame tends to have overstrength index equal to 0 (i.e., collapse situation). This tendency highlights the fact that the additional overstrength of the key members is not able to guarantee the efficient redistribution of the loads in the resisting part of structure. As illustrated, as the distance between the columns reduces, i.e., $\xi \to 0$, the overstrength index is always equal to 1.

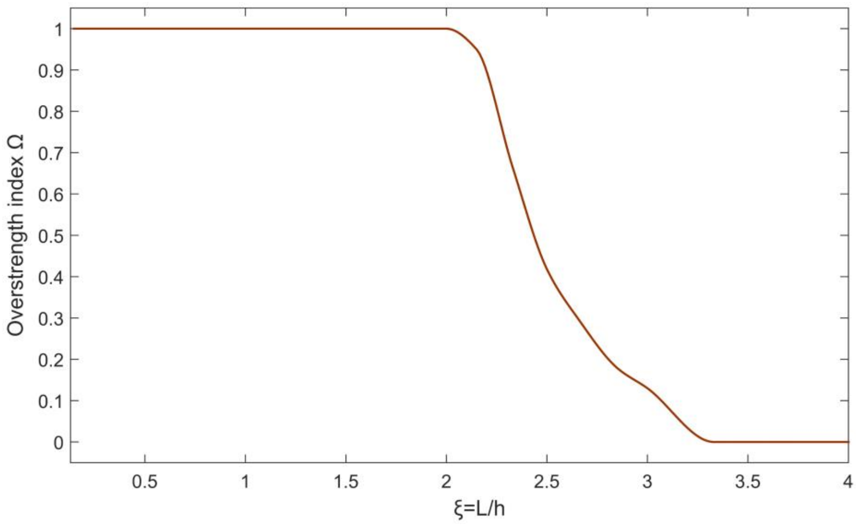


Figure 16: Overstrength index as function of the ξ ratio.

6 CONCLUSIONS

According to the findings shown in the previous sections, the following conclusions can be drawn:

- The analytical method presented in this paper allows predicting the response of a reinforced concrete frame subjected to a column removal. This method has been validated with experimental and numerical results.
- The results from studying the behaviour of building frames subject to column removal are utilized in order to identify an effective method for enhancing structural robustness.
- A strategy is proposed that is suitable for a preliminary robustness-oriented design, adopting specific solutions in terms of local resistance, develop of alternate load-paths and mechanical properties of the structural members. This strategy is applied in order to enhance the robustness of RC frame structures and validated by numerical models.
- The results found in the framework of this study on structural behaviour can be suitable in the pursuit of a methodology for increasing the robustness of structures. Furthermore, this framework proposes an effective procedure for protecting the structures from extreme events leading to element removal, such as impact [30, 31] or explosions.
- A measure for quantifying the overstrength of a frame structure has been herein proposed. This overstrength indicator permits to put into evidence the capability of frame structures to redistribute the overload due to a damage scenario.
- In this work plane frame structures are considered. In 3D, the collapse mechanisms and propagation of collapse can differ significantly. 2D models underestimate the collapse loads compared to 3D analyses, due to disregarding load redistributions in three dimensions. However, the aim of this work is to develop a simplified tool that allows to consider the collapse mechanisms associated with extreme events explicitly as a part of the design, at least as a preliminary phase to choose the more suitable structural solution. This tool can be applied complementary to detailed structural models for evaluating robustness of building, as reported in the work [22].

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