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Progressive Collapse and Robustness of Steel Framed Structures

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Abstract

In this paper the robustness and resistance to progressive collapse of steel framed structures under exceptional actions has been investigated. After some calculation methods proposed by the authors have been presented, a new general robustness assessment technique has been proposed and applied to some case studies, represented by steel structures designed with both the old and the new seismic Italian codes. The robustness of studied structures has been assessed under different column-removal conditions by means of a non linear static analysis approach based on the alternative load path method. The achieved results have allowed a comparison among the examined structures to be made, considering both the location the column removed and the influence of different connection types.

Keywords: progressive collapse, robustness, steel frames, column removal, full and partial strength connections.

1 Introduction

The robustness can be defined as the insensitivity of a structure to undergo local failure independently from the causes and probabilities of initial local failures [1].
It is a structural property which mitigate the susceptibility of structures towards progressive collapse. This phenomenon is caused by the disproportion between the initial damage and the resulting huge collapse of either the structure or its large parts.

After the failure of the Ronan Point apartment tower (1968), in 1970 the UK Building Regulations introduced their *"Fifth Amendment"* in order to provide indications to avoid progressive collapse, which were based on the following requirements [2]:

a) The capacity of buildings to not have disproportionate collapse in case of accidental loads was initially limited to structures with five or more storeys, and subsequently extended to all type of buildings.
b) The requirement for a minimum level of ductility and redundancy throughout a structure was introduced.
c) The requirement for buildings with more than four stories to remain stable after removal of a key element was introduced. If this requirement was not met the element must be designed to resist a pressure of $34kN/m^2$.

Later on, the terrorist attack on the Murrah Federal Office Building (1995), together with the World Trade Centre collapse (2001), have given a new interest towards this subject.

However, nowadays a general theory regarding the study of robustness and progressive (or disproportionate) collapse topics does not exist. In fact, if qualitative study approaches of considered phenomena are very diffused, no general quantitative recommendations to evaluate structural robustness have been yet implemented. In general, there are three alternative approaches to disproportionate collapse resistant design: improved interconnection or continuity, notional element removal and key element design. Nevertheless, no general criteria to quantify these structural evaluation approaches under extreme or unforeseen events have been implemented.

Therefore, in the current paper an innovative approach to evaluate the robustness of steel framed structures is proposed and applied to some case studies.

2 Code provisions

Nowadays, different international codes, namely EN 1991-1-7 (2006) [3], United States Department of Defense (DoD, 2005) [4], the United States General Services Administration (GSA, 2003) [5], UK Building Regulations (BS 6399-1, 1996) [6], starting from the collapse of the Ronan Point building in London (1968), have provided different definitions for *robustness and progressive collapse*, providing at the same time defensive measures for the construction protection.

As an example, according to EN 1991-1-7 [3], the *robustness* is intended as "*the ability of a structure to withstand events like fire, explosions, impacts or the consequence of human error, without being damaged to an extent disproportionate to the original cause*."

On the other hand, different meaning for progressive collapse are used. In general terms, when one or several structural members suddenly fail due to either accident or incidental conditions and subsequently every load redistribution causes in sequence the failure of other structural elements, then the complete failure of the building or of a major part of it occurs and the progressive collapse is attained.

In this framework, all the above codes specify the extent of damage considered as acceptable, by limiting the floor area for which collapse is tolerated after the initial local failure.

More in detail, the United States General Services Administration (GSA) published in June 2003 their guidelines for progressive collapse mitigation to be applied for all federal buildings in USA [5]. The document provides a flow-chart methodology to determine whether constructions require detailed verification for progressive collapse. If the progressive collapse risk deserves to be considered, the document proposes the alternate load path design strategy when a local initial failure happens. The document allows for sophisticated nonlinear static and/or dynamic procedures, but describes in detail only a static linear procedure for progressive collapse mitigation. The combination between dead and live loads, as well as a dynamic amplification factor of 2, is specified for static analyses in order to account for dynamic inertial effects due to the failure of one ground floor column. The GSA static linear guidelines are among the most complete provisions, since they instruct the designer in all steps of the design process.

Only the United States Department of Defense (DoD) guidelines provide details about the nonlinear procedures to be applied [4]. In June 2005, they issued latest guidelines for progressive collapse prevention. Buildings are classified according to the required level of protection. When a very low or low level of protection is required, the safety of the structure is ensured through horizontal and vertical ties, while for higher protection levels an alternate path approach is prescribed in addition to these ties. A step-by-step procedure is provided for linear static and non-linear (static and dynamic) analyses. The load combination, involving dead, live and wind loads, is specified, along with a dynamic amplification factor of 2, for static analyses. The DoD step-by step procedure for linear static analysis is similar to the GSA one in terms of general philosophy. The main differences lie in the choice of the material behaviour used in the simulations, as well as in the fact that the non-linear procedures are detailed in the DoD guidelines only.

The U.K. building regulations required that buildings be designed to resist disproportionate failure by tying together structural elements, adding redundant members and providing sufficient strength to resist abnormal loads [6]. These requirements are considered to produce more robust structures, that is strong and ductile structures capable to redistribute loads. In particular, these specifications are intended to ensure that the structure may withstand a column loss through catenary effects. The load combination between dead, live and wind loads is specified, as well as the area of tolerated damage. However, both computational procedure to estimate the damage extension and dynamic amplification factor are not specified. If the damage amount exceeds the acceptance criterion, the particular key element is designed to resist an additional static pressure of 34 kN/m².

Finally, the EN 1991-1-7 provides a classification of buildings into four classes, based on the consequences of collapse. For the lowest class, no progressive collapse requirements are to be met. For the second class, only horizontal tie force requirements are specified. For the two remaining classes, not only tie requirements should be met, but the structure also needs to be designed for the loss of a vertical load bearing element, with damage not exceeding a specified region. If the damage is too extensive, the vertical load bearing element is considered as a key element and should be designed to withstand an additional pressure of 34 kN/m². Again, no computational procedure is specified for the alternate load path analysis.

3 Recent approaches for robustness evaluation

Contrary to the code development inherent the behaviour of steel structures under catastrophic events, in literature few practical studies oriented towards the robustness quantitative assessment of such constructions have been implemented.

In the following two approaches developed by Authors for quantifying the robustness of steel structures towards exceptional earthquakes and catastrophic events, such as explosions, terroristic attacks and so on, respectively, are illustrated.

Following the first approach, a deterministic definition of robustness is presented [7]. Therefore, instead of a probabilistic approach for modelling both actions and structural properties, a semi-probabilistic or even a deterministic approach is used, it being more affordable, mainly when the random properties of either actions or materials cannot be easily determined.

The first procedure step is to consider a global damage pattern *D*, produced on the structure by an ideal action system *A*, represented by means of the resistance-damage (*R-D*) curve, also called the structural performance curve (SPC). The robustness index I_r can be defined as the ratio between the maximum “direct” energy, which can be absorbed by the structural system, which is associated to the direct damage, and the total energy absorbed by the structure as a consequence of being exposed to a given action, which comprises the one associated to both direct and indirect damage (Fig. 1).

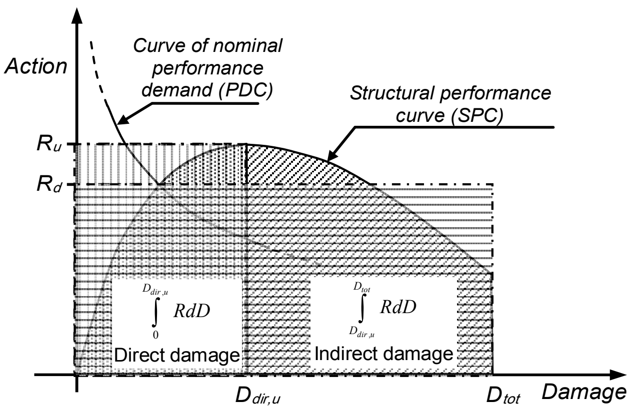


Figure 1. Definition of direct and indirect damage.

The following relationship is given:

$$I_{\mathrm{r}} = \frac{\int_0^{D_{\mathrm{dir,u}}} RdD}{\int_0^{D_{\mathrm{tot}}} RdD} \tag{1}$$

For the purpose of practical calculations, eq. (1) can be also computed in approximate way as:

$$I_{\mathrm{r}} = \frac{\int_0^{D_{\mathrm{dir,u}}} RdD}{\int_0^{D_{\mathrm{tot}}} RdD} \cong \gamma \frac{D_{\mathrm{dir,u}}}{D_{\mathrm{tot}}} \frac{R_{\mathrm{u}}}{R_{\mathrm{d}}} \tag{2}$$

where R_u and R_d are the structural ultimate resistance and the design resistance for a given nominal curve of performance demand (PDC), respectively (Fig.1), and γ is a coefficient depending on the shape of the SPC. In most cases, γ ranges from 1 to 1.3.

If one observes that the ratio $D_{dir,u}/D_{tot}$ represents the ratio of the maximum direct damage, which the structure can withstand ($D_{dir,u}$), to the actual damage undergone due to the loading event (D_{tot}), then a structural integrity index can be conventionally defined as $I_{si} = D_{dir,u}/D_{tot}$. Hence:

$$I_r \cong \gamma \frac{D_{dir,u}}{D_{tot}} \frac{R_u}{R_d} = \gamma I_{Si} \frac{R_u}{R_d} \tag{3}$$

For a given PDC, which can be represented in a general form as shown in Figure 2, three situations are possible:

1) The SPC is below the PDC (Fig. 2a).

This means $I_r < 1$ and $I_{si} < 1$. In fact in this case $D_{tot} = D_{dir,u} + D_{ind}$, hence:

$$\int_0^{D_{tot}} RdD = \int_0^{D_{dir,u}} RdD + \int_{D_{dir,u}}^{D_{tot}} RdD \tag{4}$$

2) The SPC meets the PDC so as $D_{dir,u}= D_{tot}$ (Fig. 2b).

This means $I_r = I_{si} = 1$; $D_{ind} = 0$. In fact in this case, at the intersection of the nominal PDC with the SPC, dA/dD=0, hence:

$$\int_0^{D_{tot}} RdD = \int_0^{D_{dir,u}} RdD \tag{5}$$

3) The SPC is such that $D_{dir,u} > D_{tot}$ (Fig. 2c).

This means $I_r > 1$ and $I_{si} > 1$. In fact in this case, $D_{tot} = D_{dir,d}$, D_{ind}=0 and at the intersection of the nominal PDC with the SPC $dA/dD > 0$, hence:

$$\int_0^{D_{tot}} RdD = \int_0^{D_{dir,d}} RdD \tag{6}$$

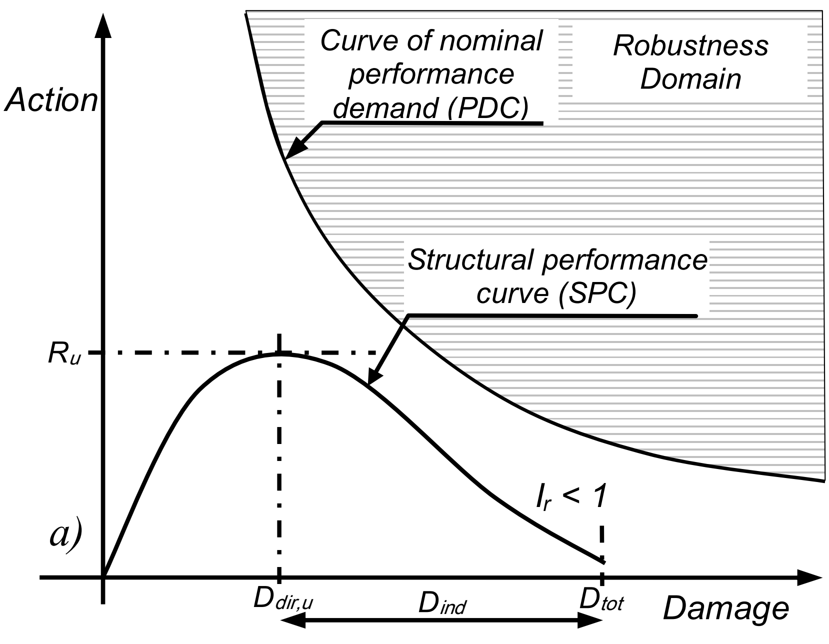
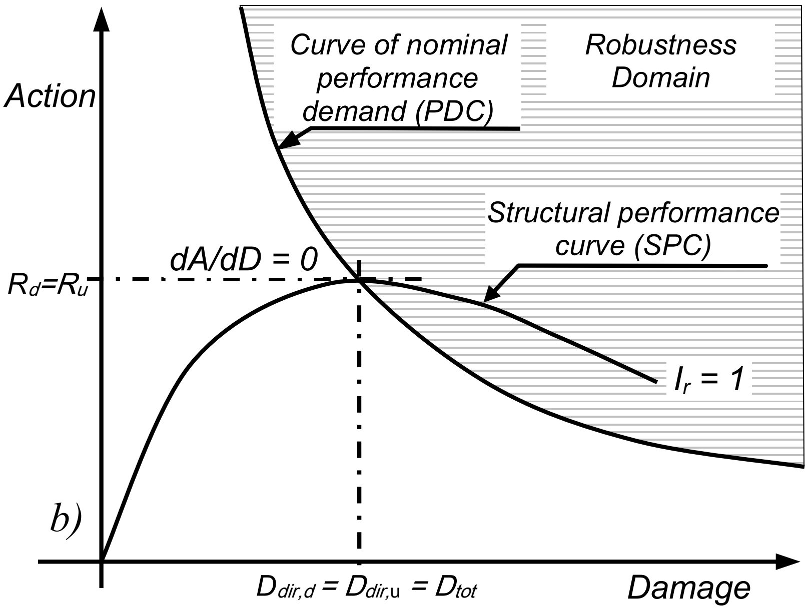


Figure 2. Design situations: a) $I_r < 1$, b) $I_r = 1$, c) $I_r > 1$. (continues)



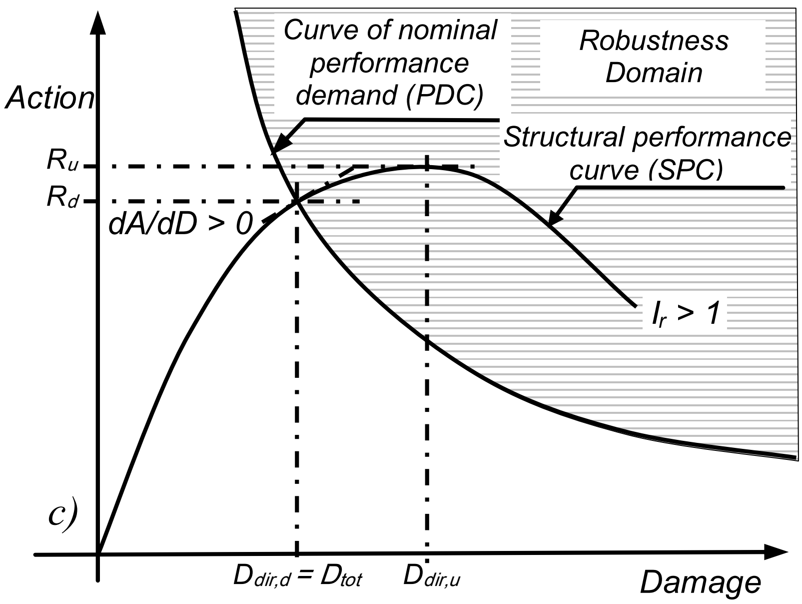


Figure 2. Design situations: a) $I_r < 1$, b) $I_r = 1$, c) $I_r > 1$.

The condition $I_r > 1$ allows possible changes of the PDC due to unexpected or accidental actions to be tolerated with a lower risk to undergo indirect damage.

If the commonly accepted performance levels for construction design are assumed [8], an ideal concept of Robustness-Based Design (RBD) can be defined (Fig. 3), according to which the structural design is carried out considering predetermined levels of robustness, each of them corresponding to a value of the robustness index I_r. As a result, a typical multi-level performance matrix can be set up (Table 1).

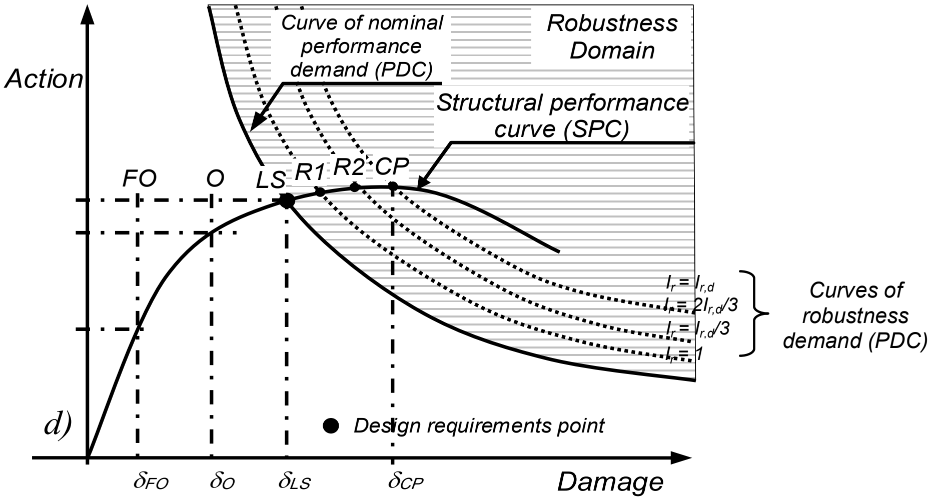


Figure 3. Design situations: the concept of Robustness-Based Design.

	Nominal design capacity			Robustness capacity			
PERFORMANCE LEVEL	**FO**	**O**	**LS**	**R1**	**R2**	**…**	**CP**
Frequent event	×	Maximum objective					
Occasional event	■	×	Intermediate objective				
Rare event	▲	■	×	Minimum objective			
Very rare or catastrophic event		▲	■	×	×	×	×

Table 1. Performance matrix accounting for robustness levels.

The damage amount corresponding to each performance level (Fully Operational, Operational, Life Safe, R1, R2,… Collapse Prevention) can be determined according to codes, depending on the action under consideration. A SPC has to be obtained by the designer for the structure which meets the nominal PDC at a value of damage at least compatible with protection of human lives (LS), whereas an adequate robustness guarantees the structural integrity until the collapse (CP). In practical applications, the CP performance level is to be related to a given design robustness factor $I_{r,d}$ (Fig. 3), which depends on both the importance of the construction and the type of usage. Account of possible infrequent as well as accidental loadings can be made by means of suitable deterministic or stochastic methods, which may result in required values of the robustness index $I_r >> 1$. If necessary, additional intermediate robustness levels R1, R2, … can be also defined according to particular design requirements. In such a way, at least in principle, the Robustness-Based Design (RBD) is applicable in combination with the traditional approach of the Performance-Based Design (PBD), provided that suitable values of the robustness index I_r are assigned as design requirements.

Two and three levels new steel structures designed according to the old and new seismic Italian codes with randomness of both material and vertical loads were analysed in the non linear filed by using the above procedure in order to evaluate their robustness under different grade earthquakes.

The analyses showed that the new structures, when designed according to the new Italian seismic code for exceptional earthquake actions, provide high level robustness indices, their behaviour being characterized by a global collapse mechanism. Contrary, structures designed according to the old code have shown a deficient robustness level, their behaviour being characterized by a soft-story mechanism at the second level.

More recently the Authors, inspired to some literature works and code guidelines [4, 5], have developed a study approach for evaluating the resistance to progressive collapse of steel framed structures based on the concepts illustrated as follows [9].

When a column is removed from a framed structure, its robustness can be assessed in terms of progressive collapse resistance, intended as the maximum loading capacity to be sustained before failure. In fact, when a building column failed in a sudden way due to an accidental load, an instantaneous vertical loading equal to the one supported by the collapsed column is transferred to the remaining part of the building.

Different analysis types, namely linear static, non linear static and non linear dynamic, are usually performed to evaluate the progressive collapse resistance of framed buildings [10].

First of all, a step-by-step linear static (LS) procedure according to the US General Service Administration [5] and the Department of Defense [4] guidelines can be considered. In the GSA procedure, a step-by-step scheme of inserting moment-release hinges is used to simulate the inelastic structural behaviour. In particular, beam sections attaining a bending moment larger than their yielding one are replaced with hinges to simulate the structural behaviour in plastic range. In this analysis, the vertical loads applied to the structure are gradually increased up to achieve a local flexural failure mechanism resulting into a progressive collapse of the building. Catenary effect is neglected and only flexural failure mode is considered. The load-displacement response from linear static analyses is obtained by putting on the abscissa axis the displacement of the column removed point and on the vertical axis the corresponding applied load. Generally, the buildings have an approximate linear behaviour up to the attainment of the progressive collapse resistance. So, the load-displacement curves are very similar to the response of an elastic-perfectly plastic model. As a consequence, this procedure should be used for elastic analysis only.

Contrary, a displacement control procedure is utilised to carry out non linear static analyses. First dead loads and a percentage of live loads are applied to the building and after a vertical pushover analysis is done. Particularly, a vertical displacement is gradually applied to the column-removed point, up to the attainment of the maximum building resistance. Generally, this analysis type provides a progressive collapse strength lower than the one obtained by linear static procedures. Besides, the response curve reached from the non linear static analysis starts to deviate in a significant way from the static linear one when the structure is considerably pushed into the inelastic field.

However, it is clear that the building behaviour under exceptional actions deriving from a column collapse is a dynamic problem rather than a static one. Therefore, under this circumstance, it is more appropriate to perform non linear dynamic analyses aiming at assessing the real progressive collapse resistance of buildings. Nevertheless, this analysis typology, which generally provides a lower collapse resistance than static analyses one, is time-consuming and result to be too difficult to be carried out for practical applications. As a consequence, an alternative method has been proposed in order to precisely estimate the building collapse resistance under the described exceptional situation instead to perform non linear dynamic analyses [11]. This is illustrated in Figure 4, where, considering that the area below the non linear static load-displacement curve represents the energy stored by the column-removed building under gravity loads, a capacity curve can be obtained by dividing the accumulated energy by its corresponding displacements.

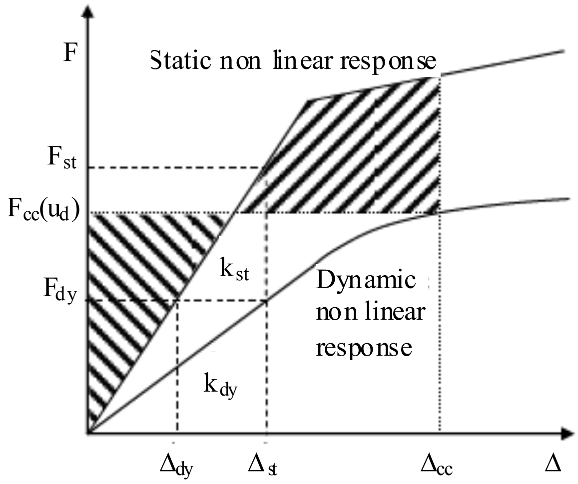


Figure 4. Static non linear response vs. dynamic non linear curve and explanation of the Dynamic Amplification Factor.

It was demonstrated that this capacity curve is able to approximate very well the non linear dynamic behaviour of buildings, when a column collapses. Based on the energy conservation principle, F_{CC} (u_d) in Figure 4 represents the equivalent dynamic loading under the displacement demand u_d. Accordingly, when the building is deprived of a column, the column-removed point attains a maximum displacement such that both the hatched areas of Figure 4 are equal.

So, even if the precision of non linear dynamic results is indubitable, generally more simple analyses, that is the static ones, can be used. In these cases, in order to take into account the dynamic effect due to the removal of a column, the vertical loadings are increased by means of a Dynamic Amplification Factor (DAF), which is defined as the ratio between the dynamic displacement response (Δ_{dy}) of an elastic SDOF system and its static displacement response (Δ_{st}) under the same applied load F (Fig. 4). In the same figure it is apparent that the DAF can be expressed also as the ratio between the static force and the dynamic one under an equal displacement.

The GSA guidelines suggest to use a DAF equal to 2 for considering the behavioural difference between static non linear and dynamic non linear analyses. However, if the load originally supported by the lost column and transferred to the remaining part of the structure provokes an inelastic response, the DAF may assume values different than 2, which depend on the displacement demand.

The investigation carried out on the same steel structures already analysed has shown that the robustness index of buildings designed according to the new code is averagely 10% larger than the one of other buildings satisfying the old seismic provisions. Furthermore, the use of DAFs has been assessed, for considering the dynamic effect due to the column removal, when static analyses are made. The obtained results have shown that the GSA US code provisions are not on the safe side when elastic analyses are performed, because the DAFs values are greater than 2, and that the dynamic amplification in the inelastic field depends on the maximum allowable plastic displacement. In particular, for the 2-storeys and the 3-storeys structure, a mean DAF value of 1.23 and 1.16 is respectively obtained, when the maximum allowable displacement is attained.

4 A new general assessment method

The necessity to have a general methodology for robustness assessment of steel structures under each type of exceptional action has led towards the implementation of a new non-linear analysis approach. The adopted procedure is conceptually similar to the one given by the U.S. Department of Defense, which is a non-linear procedure framed in the category of alternative load paths.

The main difference between the proposed approach and the U.S. one is that the former is not a sequence inversion procedure, that is the computational work does not start with the original FEM model of the structure where a vertical element is removed, but with the numerical structural model where foreseen combination loads are applied.

Later on, in order to simulate the column loss, its stiffness is reduced to zero and, simultaneously, aiming at considering the dynamic inertia effects, in the zones near to the removed elements, loads are amplified with an appropriate Dynamic Amplification Factor (DAF). Therefore, the structure configuration before column removal is taken into account by evaluating the presence of vertical loads; in addition the progressive variation of both the removed element and loads allows to assess in accurate way the force redistribution into structural elements. Such a procedure is the so-called Load History Dependent (LHD) procedure [12].

The procedure is based on a 3D model of the structure with both beam and columns modeled as linear elements and rigid floor diaphragms. Since large displacement analyses (i.e. considering the catenary effect phenomenon) are performed, geometric non linearities have been considered in the FEM model.

For beam and columns concentrated plasticity models of plastic hinges as defined in the FEMA 356 [13] have been adopted. The same U.S. code also specifies three different performance levels, used for the definition of the robustness index, as a function of the yielding rotation θ_y: Immediate Occupancy (0.25 θ_y), Life Safety ($2\theta_y$) and Near Collapse ($3\theta_y$).

The used load combination is the one contemplated in the new seismic Italian code [14].

In order to take into account the dynamic nature of applied loads, a Dynamic Amplification Factor (DAF) is obtained through the following relationship [15]:

$$DAF = 1.08 + \frac{0.76}{\frac{\theta_{pa}}{\theta_y} + 0.83} \tag{7}$$

where θ_{pa} is the allowed plastic rotation. Such a factor, which depends on the selected performance level, assumes values of 1.35 and 1.28 when Life Safety Limit State and Near Collapse one are considered, respectively.

Fully and partially restoring connections in terms of strength and stiffness are used in the numerical model to connect beams and columns. This has allowed to evaluate the connection influence on the robustness of studied structures.

The robustness index I_r, ranging from 0 to 1, is calculated as the ratio between the direct damage and the total one, intended as sum of the direct damage and the indirect one. When the indirect damage is zero, the structure is robust and $I_r = 1$.

The direct damage is the acceptable damage of the structure with reference to a given performance level. The acceptable damage, which is an ideal damage, is function of the allowed plastic rotation of both beams and connections:

$$D_{dir} = \sum_{i=1}^{n} \theta_{ai} + \zeta_{ai} \tag{8}$$

where θ_{ai} is the allowed plastic rotation of the i-th plastic hinge, ζ_{ai} is the allowed plastic rotation of the i-th connection, $n = 2 \ x \ n_b \ x \ n_f$ is the ideal number of plastic hinges activated by the catenary effect in the 3D structural scheme with n_b = number of beams connected to the removed column and n_f = number of floor above the one with the removed column.

The total damage is the real damage produced in the structure, it being defined as follows:

$$D_{tot} = \sum_{i=1}^{n_{Tot}} \theta_i + \zeta_i \tag{9}$$

where θ_i is the allowed plastic rotation of the i-th plastic hinge, ζ_i is the allowed plastic rotation of the i-th connection, n_{Tot} is the real number of activated plastic hinges.

The robustness index I_r is therefore equal to:

$$I_r = \frac{D_{dir}}{D_{tot}} = \frac{\sum_{i=1}^{n} \theta_{ai} + \zeta_{ai}}{\sum_{i=1}^{n_{Tot}} \theta_i + \zeta_i} \tag{10}$$

This index assumes unitary value when $D_{dir} = D_{tot}$, that is the structure is robust with reference to the prefixed performance level. Instead, it assumes values tending to zero when total damage is greater than the direct one, that is when the structure has low robustness. Finally, it is possible to found robustness index greater than one, that is the structural performance is better than the one of the given performance level.

5 The method application to case studies

The implemented method has been applied to some case studies reported in [7] and represented by the same framed steel structures designed with the old (DM 96) [16] and the new (NTC 08) [14] seismic Italian codes already analysed with the before illustarated robustness assessment methods.

FEM models of the studied structures, herein called *A* and *B*, have been set-up with the SAP2000 analysis program [17] and are shown in Figure 5. They are made of S275 steel with beams and columns connected by three different connection types: 1) rigid and full strength, 2) semi-rigid and partial strength and 3) semi-rigid and full strength.

For the two structures all possible scenarios of column removing have been considered in order to understand which are the worst conditions.

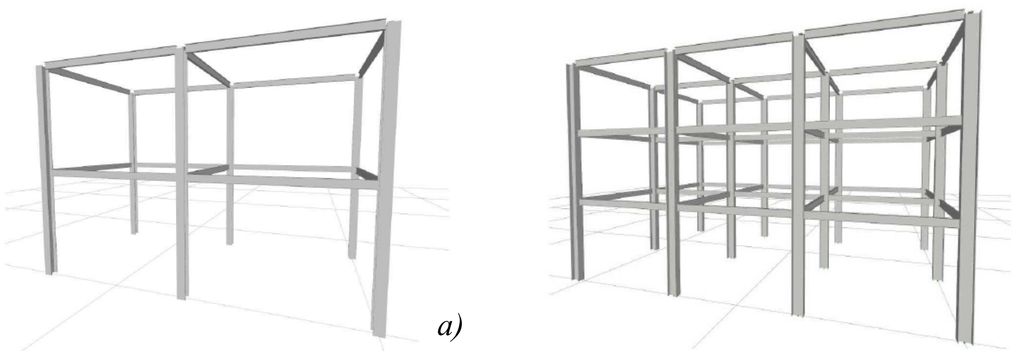


Figure 5. Steel structures type *A* (a) and *B* (b) under investigation.

Robustness indices of examined structures have been calculated considering the Life Safety Limit State as performance level. The achieved results for the two structures under form of histograms are reported in the following.

In Figures 6 and 7 the robustness indices of the structure *A* (with rigid and full strength connections), designed respectively with the new code and the old one, when the position of the removed column changes are reported.

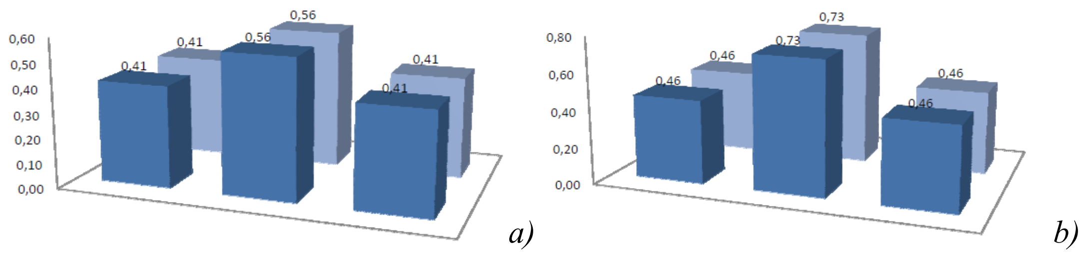


Figure 6. Robustness indices of the NTC 08 structure type *A* with rigid and full strength connections at the first level (a) and the second one (b).

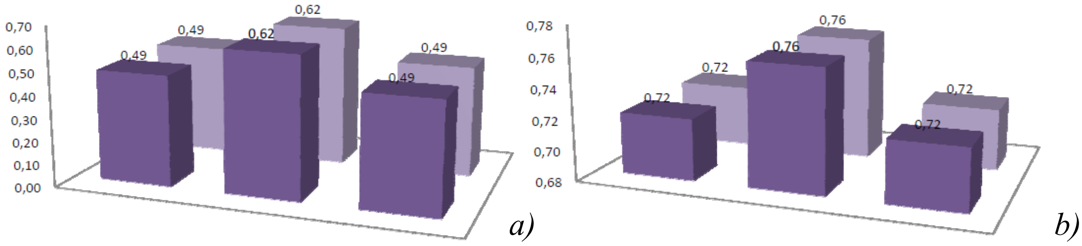


Figure 7. Robustness indices of the DM 96 structure type *A* with rigid and full strength connections at the first level (a) and the second one (b).

The same comparisons for the two structures type A have been performed also considering the presence of semi-rigid and partial strength connections (Figs. 8 and 9) and semi-rigid and full strength connections (Figs. 10 and 11).

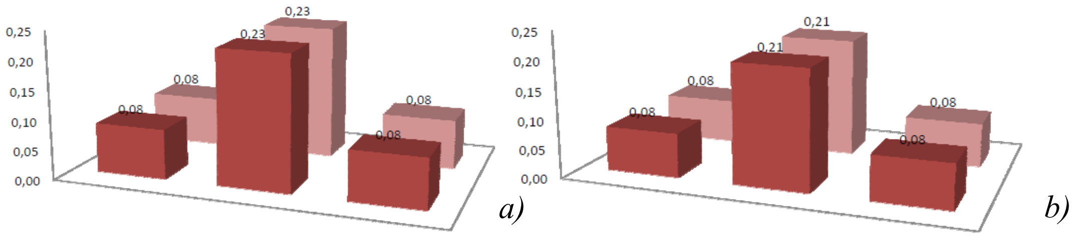


Figure 8. Robustness indices of the NTC 08 structure type *A* with semi-rigid and partial strength connections at the first level (a) and the second one (b).

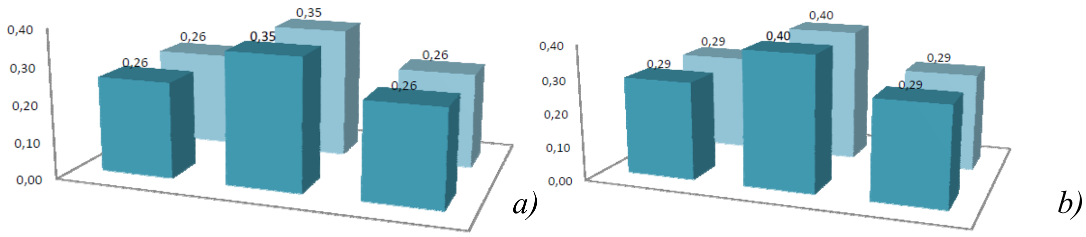


Figure 9. Robustness indices of the DM 96 structure type *A* with semi-rigid and partial strength connections at the first level (a) and the second one (b).

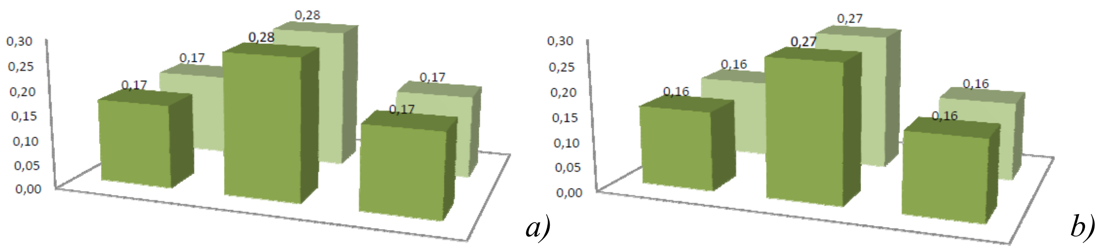


Figure 10. Robustness indices of the NTC 08 structure type *A* with semi-rigid and partial strength connections at the first level (a) and the second one (b).

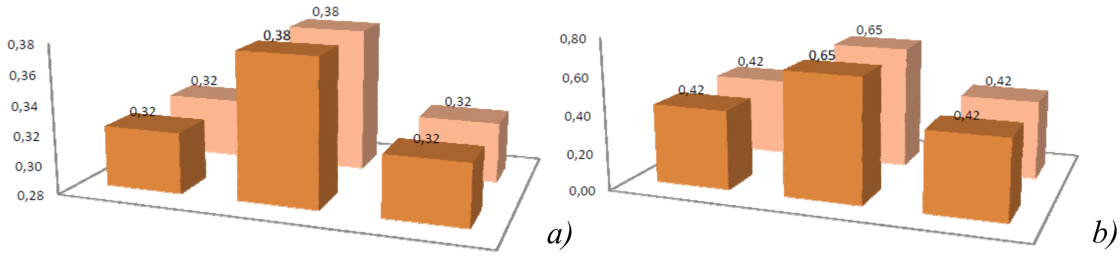


Figure 11. Robustness indices of the DM 96 structure type A with semi-rigid and partial strength connections at the first level (a) and the second one (b).

On the other hand, for structures type B with change of the connection type the robustness indices corresponding to the column removal into a given position of the structure are reported in Figures from 12 to 15. In these figures the following symbols are used: - FR = full restoring of resistance and stiffness; - PR = partial restoring of stiffness; - PRR = partial restoring of resistance; - CRR = complete restoring of resistance.

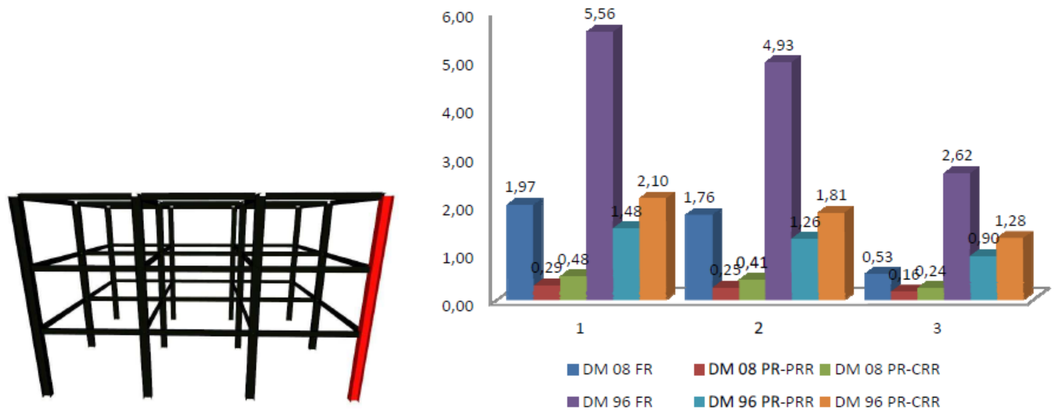


Figure 12. Robustness indices of the structure type *B* with removal of a corner column.

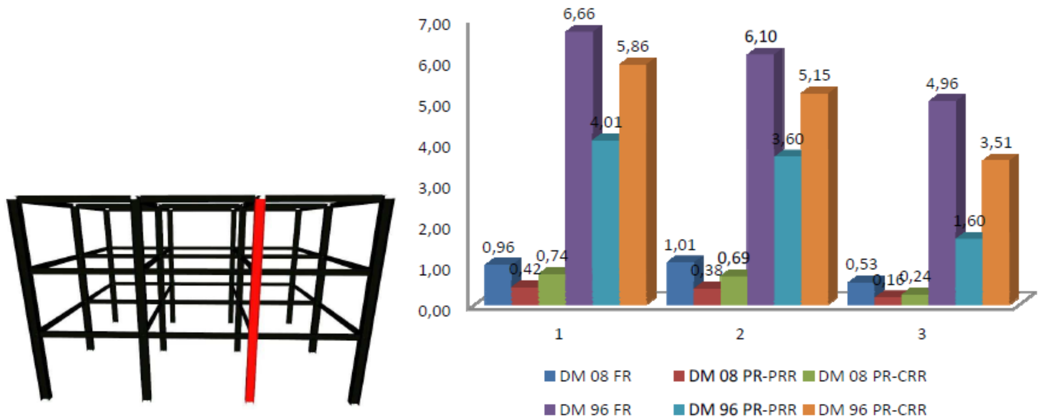


Figure 13. Robustness indices of the structure type *B* with removal of one perimeter column on its long side.

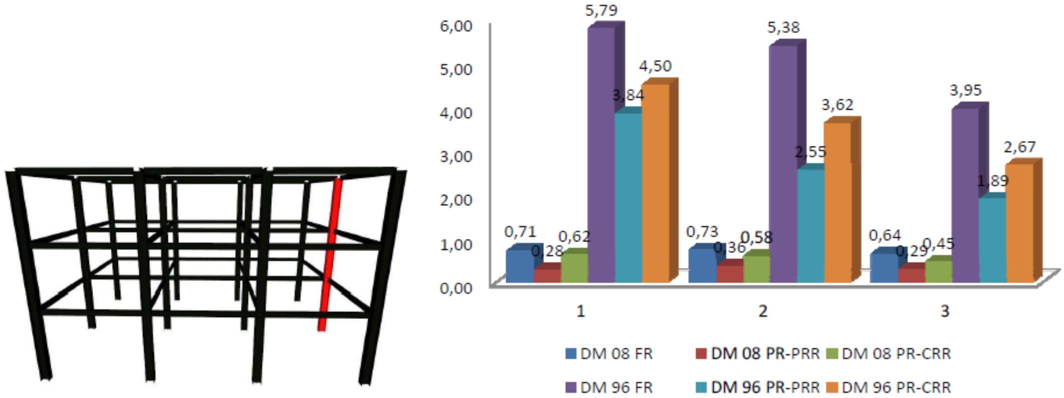


Figure 14. Robustness indices of the structure type *B* with removal of one perimeter column on its short side.

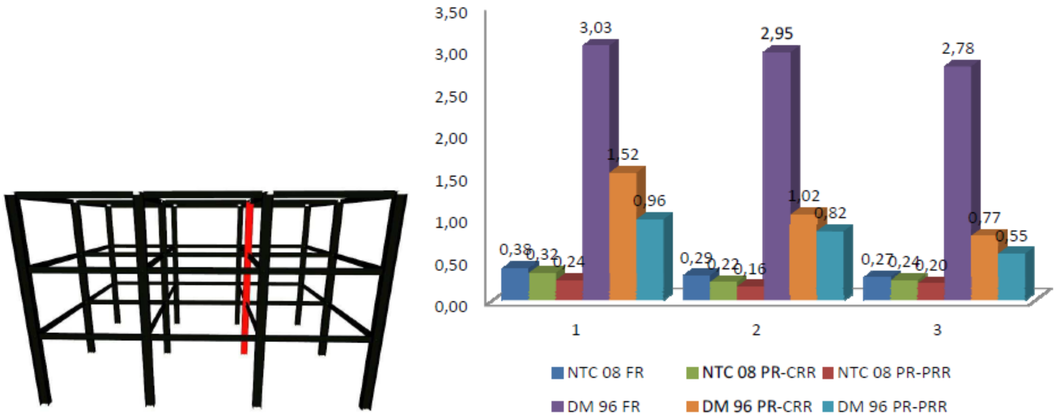


Figure 15. Robustness indices of the structure type *B* with removal of a corner column.

From results achieved for both structures (type *A* and *B*) it is apparent that those designed with the old code have robustness indices greater than the ones designed with NTC 08 thanks to the use of beams with greater flexural stiffness.

Analyses also shown clearly the influence of the connection type. In fact, full strength and rigid connections allow to achieve high robustness levels, whereas semi-rigid ones exhibit less performance, showing a better behaviour when they are of full strength type.

In addition, indications about the worst scenarios of column removal have been given. Bad situations are those connected to the loss of internal columns, immediately followed by the loss of the corner column. Analyses have also shown that the scenario hazard increases as the structure level number increases. This is in agreement with the provisions of the U.S. Department of Defense [15], which foresees as obligatory scenario the removal of the top storey column.

5 Conclusions

This study deals with the robustness assessment methods of steel framed buildings under catastrophic events. Such a topic is of wide interest when exceptional actions, that is either loads not considered in the design phase or loads greater than the design ones, are applied on constructions. In this case the vulnerability evaluation of these structures can be intended as the relationship between structural integrity and robustness. In particular, the robustness reserve of the structure has to be exploited in order to preserve its structural integrity. As a consequence, the direct damage deriving from the loads application should be prevented and the indirect one should be really limited in order to avoid the global structural collapse.

According to these premises, two steel framed buildings (2-storeys 1-bay – type *A* and 3-storeys 3-bays – type *B*), designed according to old and new Italian seismic codes, have been herein analysed.

The robustness of studied structures has been assessed under different column-removed conditions, related to different catastrophic events (blast, impact, fire, etc.), by means of a new non linear static analysis approach based on the alternative load path method in order to estimate their resistance against progressive collapse. In particular, the computational model starts with the whole structural model where the loads are applied. Afterwards, both the structural stiffness is decreased for taking into account the column loss and the applied loads are increased by means of a Dynamic Amplification Factor (DAF) for considering the dynamic nature of the phenomenon. This allows to assess in a more precise way the stress redistribution into structural elements, so leading towards a load history dependent procedure. Finally the structure robustness index is determined as ratio between the direct damage caused by the exceptional event and the total damage, equal to the sum of the direct damage and the indirect one.

The analyses performed has allowed to evaluate the robustness performance of study structures, by considering the variability of the joint types (full strength and partial strength), as well as to make a comparison among structures designed by the new seismic Italian code and the old one. The achieved results have shown the best behaviour of structures designed by the old normative code due to the presence of more robust beams able to offer a better catenary effect.

Analyses also clearly showed the influence of the connection type. In fact, full strength and rigid connections allow to achieve high robustness levels, whereas semi-rigid ones exhibit less performance, showing a better behaviour when they are of full strength type.

In addition, indications about the worst scenarios of column removal have been given. Bad situations are those connected to the loss of internal columns, immediately followed by the loss of the corner column.

Finally, analyses have also shown that the scenario hazard increases as the structure level number increases.

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