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Robustness assessment approaches for steel framed structures under catastrophic events

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ABSTRACT

The current study deals with the robustness assessment methods of steel framed buildings under catastrophic events. Two steel framed buildings, designed according to old and new seismic Italian codes, have been herein analysed, by considering the uncertainties of both the material strength and the applied loads, through two investigation methods.

First, within the methodologies used for robustness assessment under seismic loads, a deterministic method, framed within the Performance Based Seismic Design (PBSD), has been applied.

Later on, the robustness of studied structures under different column-removed conditions, related to different catastrophic events (blast, impact, fire and so on), has been assessed by means of two force-based analysis techniques (a literature approach and a more advanced procedure) in order to estimate their resistance against progressive collapse.

The application of the two methods has allowed to calculate the robustness index of examined structures, by taking into account the influence of both the catenary effect phenomenon and different beam-to-column joints, with the final aim to show their behavioural difference in terms of robustness.

1. Robustness and progressive collapse

Robustness is one of the main prerequisite of a structure to operate without failure over a time period [1]. It is accomplished when the structure response is proportioned to the actions applied to it. These actions could appear in different ways, e.g. loads exceeding the design ones, accidental loads or damage to members.

A robust structure should be achieved by: (a) either preventing the action or reducing it to an acceptable probability, (b) protecting the building and (c) reducing the building sensitivity to disproportionate collapse [2]. In the latter case, designers are required to certify that removal of any structural building component does not produce a total collapse. Furthermore, any resulting local damage must be confined within the stories above and below the one where the component is removed. Therefore, it is clear that this problem must be incorporated in the design process. In particular, on one hand, significant uncertainties in the problem formulation and, on the other hand, the appropriate assessment of the structure robustness should be considered. With reference to the second aspect, it should be underlined that no general definition of robustness exists.

In the past decades the robustness has been evaluated under two different points of view. Whilst in the first the system performance under exceptional conditions has been appraised, the second has been focused on the application of normally random conditions. According to the first perspective, a structure can be declared as robust when either collapse is not sudden or the resistance is not substantially lost although the deformations exceed the serviceability level. Those developments also appear in the context of structural vulnerability with an explicit investigation of the structural performance in comparison to critical failure scenarios. From the second perspective, a robust system can withstand either occasional or frequent changes of environmental conditions without noticeable effects in terms of serviceability limit state.

The assessment of robustness is of increasing interest in structural design. It is aimed to achieve robust structures, which possess optimum performance under environmental oscillations. So, in accordance with traditional design methods, the robust structural design is made by solving an optimization problem. Commonly, all variable parameters are considered as random quantities, which allow to assess structural robustness in probabilistic way. The corresponding optimization problem for achieving robustness

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generally aims at both an optimum mean and a minimum variance of the structural responses with respect to input variations. Nevertheless, optimal design solutions are not often adopted in practice because, even if they can be considered as satisfactory from economic point of view, they are surely lacking in robustness. As a whole, the two viewpoints in understanding structural robustness do not contradict each other but complement one another and even can be coupled.

When civil structures are not robust, the final failure state is disproportionately greater than the one initiating the collapse. So, the so called progressive collapse is attained, it being defined as "*the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it*" [3]. Thus, in structures susceptible to progressive collapse, small events can have catastrophic consequences. According to this, the degree of "progressivity" of a collapse can be defined as the collapsed volume (or area) over the same quantity directly destroyed by the event.

The concept of progressive collapse can be illustrated by the famous failure of the Ronan Point building in London in 1968. This building, 22 stories high and made of precast concrete bearing walls, suffered a gas explosion in a corner at the 18th floor which produced, by means of a chain reaction, the collapse of the corner bay of the building from top to bottom (Fig. 1a).

More recently, the terrorist attacks on World Trade Center buildings on 2001 September 11th have represented a clear example of progressive collapse. A Boeing 767 crashed into the tower at high speed and this caused structural damage near the impact point, producing also an intense fire within the building (Fig. 1b). Then, as a result of the combination of impact damages and fire damages, both buildings collapsed, since the weight and impact of the collapsing upper part of the towers caused a progression of failure extending down to the ground.

Starting from these failure experiences under exceptional actions, the interest on assessing structural robustness has increased in a large way in the recent years. In this framework, some efforts have been made by a large number of researchers, who have setup several code provisions (see Section 3). Nevertheless, practice and reliable criteria either in measuring robustness or determining whether the robustness level of a system is acceptable have not yet been given in detail. Therefore, based on some studies performed by Authors on robustness of existing buildings [4], the first step of the research within this field is to establish a reliable methodology for evaluating the performance of structures subjected to exceptional actions.

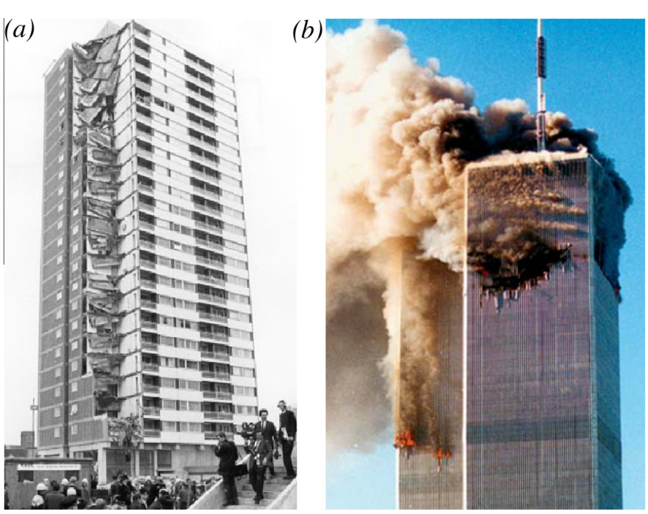

Fig. 1. Collapse of the Ronan Point building (a) and the World Trade Center towers (b).

So, in the present paper, starting from a general review of a conference paper written by Authors [5], two methods for assessing the robustness of structures under exceptional earthquakes and accidental loads (blast, impact and so on) have been implemented.

First of all, within the deterministic methodologies used for robustness assessment under seismic loads, a method framed within the Performance Based Seismic Design has been conceived. In this case the robustness under exceptional earthquakes considered in the new technical Italian code has been evaluated by using an energetic approach. The method, based on the determination of direct and indirect damages suffered by structures under extreme earthquakes, has allowed to evaluate the structural robustness by comparing the structure capacity curve with the demand spectrum related to an exceptional seismic event. In this case the vulnerability evaluation has been intended as the relationship between structural integrity and robustness, in the sense that 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 load application should be prevented and the indirect one should be really limited in order to avoid the global structural collapse.

Later on, the robustness under different column-removed conditions, related to various catastrophic events, has been assessed by means of a new non-linear analysis method based on a Load History Dependent (LHD) procedure able to take into account both the catenary effect and the behaviour of connections. Also in this case the robustness index is defined as the ratio between the direct damage and the total one and can assume different values changing from zero (no robustness) to values greater than one (high robustness, that is structural performance better than a given performance level).

2. Progressive collapse mitigation options

The progressive collapse design strategies have been classified, according to the recent literature sources [6,7], into the following three approaches:

(a) Specific local resistance and non-structural protective measures (event control).
(b) Alternate load path.
(c) Prescriptive design rules.

Approaches (a) and (b) are referred to as "direct", since they are based on analytical computations for specific load cases, while approach (c) is referred to as "indirect", since it consists in applying design rules to increase the overall robustness of a structure, without performing an analytical computation for a specific load case.

In the specific local resistance approach, key vertical load bearing elements are designed to resist anticipated threats, such as blast, loads or fire.

The alternate load path approach consists in designing the structure so that stresses can be redistributed after the loss of a vertical bearing element.

The prescriptive design rules approach includes 'best practice' rules, such as continuous reinforcement, minimum joint resistance and ductility, redundant structural systems and so on. The tie force provisions adopted by the current codes are also included in this approach aiming at insuring a sufficient tying between horizontal and vertical building components, so that the structure may sustain the loss of a column through catenary (i.e. membrane) effects.

The notional member removal provisions are applied with conventional design checks and, therefore, they ignore the beneficial effects of the catenary action nonlinear phenomenon. Consequently, this can lead to the prediction of an unrealistically large damage area exceeding the prescribed limits. Also, a substantial amount of local damage due to notional member removal is allowed, but no guidance on debris resulting from such a damage on other building areas, which could potentially lead to the structure progressive collapse, is given.

A further significant shortcoming of the notional member removal provisions is the assumption of a static structural response rather than a highly dynamic phenomenon. In this context, sudden column loss represents a more appropriate design scenario, also considered by two most recent USA guidelines [8,9] for progressive collapse mitigation, which includes the dynamic effect of the event. Although such a scenario has not the same dynamic effect with respect to the column damage resulting from impact or blast, it is able to assess the influence of the column failure over a short time on the structure response.

In this paper, after some literature methodologies are applied to quantify the robustness of some steel framed structures, a detailed but simplified new procedure for their progressive collapse assessment is proposed considering sudden column loss as a design scenario. This *modus operandi* offers a quantitative approach for considering important issues in this field, such as ductility, redundancy and energy absorption of structures. The simplicity of the proposed method is such that it can be directly applied in the design practice, so to transform the problem of structural robustness from general to quantifiable.

3. Code provisions

The tragic failure of structures under exceptional actions occurred in the past years has pushed the scientific community to find the way to reduce the occurrence of the progressive (or disproportionate) collapse, which is related to the structural robustness improvement under extreme accidental and natural events.

Nowadays, different international codes (EN 1991-1-7 [10], United States Department of Defense [8], the United States General Services Administration [9] and UK Building Regulations [11]) have provided different definitions for the terms robustness and progressive collapse, providing at the same time defensive measures for the construction protection.

As an example, according to EN 1991-1-7, 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 attack 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 where collapse is tolerated after the initial local failure.

More in detail, the United States General Services Administration (GSA) released in June 2003 their guidelines for progressive collapse mitigation to be applied for all USA federal buildings. The document provides a flow-chart methodology to determine whether constructions require detailed verifications against 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 two, 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, whose last version was delivered in 2005, provide details about the nonlinear procedures for progressive collapse prevention to be applied. Buildings are classified according to the required protection level. When a very low or low level of protection is required, the structure safety is ensured through horizontal and vertical ties, while for higher protection levels an alternate path approach is additionally prescribed. 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 two, 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. These requirements lead to more robust structures, that is strong and ductile constructions 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 no computational procedure to estimate the damage extension and no dynamic amplification factor are specified. If the damage amount exceeds the acceptance criterion, the particular key element is designed to resist an additional static pressure of 34 kN/m^2.

Finally, the Eurocode 1 – part 1.7 provides a classification of buildings into four classes, based on the collapse consequences. For the lowest class, no progressive collapse requirements should 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/m^2. Also in this case no computational procedure is specified for the alternate load path analysis.

4. Robustness assessment methodologies

4.1. Under seismic actions

The most effective probabilistic method for robustness assessment is based on the work developed by the Joint Committee on Structural Safety [12], represented by the event tree shown in Fig. 2.

First, modeling of exposures (EX) is done, they having the capacity to damage structural parts [13]. They include extreme values of design loads, deterioration processes and also human errors during the whole structure life. When exposures occur, the structural system components can be either undamaged ($\overline{D}$)or damaged (D) according to several damage states which can lead to the failure (F) or not ($\overline{F}$). Consequences are classified as either direct (C_{Dir}) or indirect (C_{Ind}). The first ones are represented from damage to structural system parts. The second ones are due to either loss or

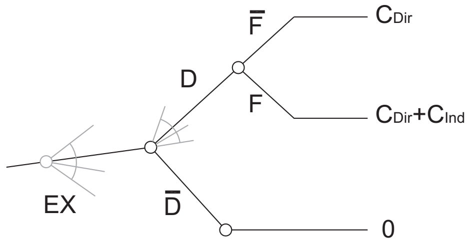

Fig. 2. Robustness quantification through an event tree.

failure of the system functionality and can be attributed to lack of robustness. Consequences are generally represented by inconvenience to system users, injuries, fatalities and/or monetary costs. In order to make a comparison among these effects, they can be combined into a scalar measure of consequences, often called utility (or disutility in case of negative consequences).

With the event tree defined in Fig. 2, it is possible to compute the system risk due to each possible event scenario. This is done by multiplying the consequence of each scenario by its probability of occurrence and then integrating over all of the random variables in the event tree. The risk corresponding to each branch is:

$$R_{Dir} = \int_x \int_y C_{Dir} P(\bar{F}|D = y) P(D = y|EX_{BD} = x) P(EX_{BD} = x) dydx \quad (1)$$

$$R_{Ind} = \int_x \int_y C_{Ind} P(F|D = y) P(D = y|EX_{BD} = x) P(EX_{BD} = x) dydx \quad (2)$$

A system is considered to be robust if indirect risks do not contribute significantly to the total risk. Consequently, the following index of robustness I_r is proposed, it measuring the fraction of total risk resulting from direct consequences:

$$I_r = \frac{R_{Dir}}{R_{Dir} + R_{Ind}} \quad (3)$$

The index can assume values between zero and one depending upon the risk source. If the system is completely robust and there are no risks related to indirect consequences, then $I_r = 1$. At the other extreme, if all risks are due to indirect consequences, then $I_r = 0$.

Based on Eq. (3), a deterministic procedure to define the structure robustness is needed aiming at evaluating how much this reserve should be exploited in order to preserve the structural integrity [14–16].

With reference to an ideal action system *A*, producing a global damage pattern *D* on the structure represented by means of the so called Structural Performance Curve (SPC) (resistance *R* – damage *D* curve), the robustness index I_r can be defined. It is calculated as the ratio between the maximum "direct" energy which can be absorbed by the structural system, that is associated with direct damage, and the total energy, associated to both direct and indirect damages (Fig. 3), absorbed by the structure as a consequence of being exposed to a given action. Therefore, the following relationship is given:

$$I_r = \frac{\int_0^{D_{dir,u}} RdD}{\int_0^{D_{tot}} RdD} \quad (4)$$

For the purposes of practical calculations, Eq. (4) can be also computed in approximate way as:

$$I_r = \frac{\int_0^{D_{dir,u}} RdD}{\int_0^{D_{tot}} RdD} \cong \gamma \frac{D_{dir,u}}{D_{tot}} \frac{R_u}{R_d} \quad (5)$$

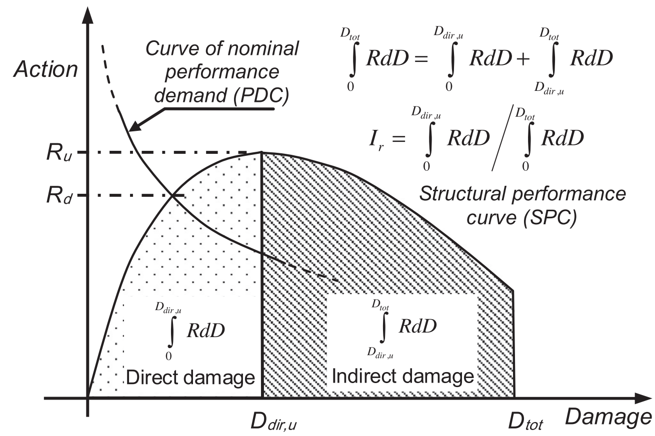

Fig. 3. Definition of direct and indirect damaged.

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. 3), and γ, ranging in most cases from 1.1 to 1.3, is a coefficient depending on the shape of the SPC.

If one observes that the ratio $D_{dir,u}/D_{tot}$ represents the ratio of the maximum direct damage that 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:

$$I_r \cong \gamma \frac{D_{dir,u}}{D_{tot}} \frac{R_u}{R_d} = \gamma I_{si} \frac{R_u}{R_d} \quad (6)$$

For a given PDC, three situations can occur:

(1) The SPC is below the PDC, which means $I_r < 1$ and $I_{si} < 1$; in this case $D_{tot} = D_{dir,u} + D_{ind}$, hence:

$$\int_0^{D_{tot}} RdD = \int_0^{D_{dir,u}} RdD + \int_{D_{dir,u}}^{D_{tot}} RdD \quad (7)$$

(2) The SPC meets the PDC so as $D_{dir,u} = D_{tot}$, which means $I_r = -I_{si} = 1$; in this case at the intersection of the nominal PDC with the SPC $dA/dD = 0$, hence:

$$\int_0^{D_{tot}} RdD = \int_0^{D_{dir,u}} RdD \quad (8)$$

(3) The SPC is such that $D_{dir,u} > D_{tot}$, which means $I_r > 1$ and $I_{si} > 1$; in this case $D_{tot} = D_{dir,d}$ and at the intersection of the nominal PDC with the SPC $dA/dD > 0$, hence:

$$\int_0^{D_{tot}} RdD = \int_0^{D_{dir,d}} RdD \quad (9)$$

The condition $I_r > 1$ allows for 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, an ideal concept of Robustness-Based Design (RBD) can be defined, in which the structural design is carried out according to 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 setup (Table 1).

4.2. Under column-removed conditions

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

Table 1
Performance matrix accounting for robustness levels.

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

to an accidental load, an instantaneous vertical loading equal to the one supported by the collapsed column is transferred to the remaining building part.

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 [17].

First of all, a step-by-step Linear Static (LS) procedure according to the US General Service Administration (GSA) and the Department of Defense (DoD) 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 a structural behaviour in the plastic range. In this analysis, the vertical loads applied to the structure are gradually increased up to achieve a local flexural failure mechanism resulting in the building progressive collapse. Catenary effect is neglected and only flexural failure mode is considered. The load–displacement response from LS analyses is obtained by putting on the abscissa axis the displacement of the column removed point and on the ordinate 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.

Instead, a displacement control procedure is utilised to carry out Non Linear Static (NLS) analyses. First dead loads and a percentage of live loads are applied to the building and, consequently, a vertical pushover analysis is executed. Particularly, a vertical displacement is gradually applied to the column-removed point up to the maximum building resistance attainment. Generally, this analysis type provides a progressive collapse strength lower than the one obtained with 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 (NLD) 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 design applications. As a consequence, instead to perform NLD analyses, an alternative method has been proposed in literature in order to estimate precisely the building collapse resistance under the described exceptional situation [18]. This is illustrated in Fig. 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 accomplished by dividing the accumulated energy by its corresponding displacements.

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} (Δ_{CC}) in Fig. 4 represents the equivalent dynamic loading under the displacement demand Δ_{CC}. Accordingly, when the building is deprived of a column, the column-removed point attain a maximum displacement such that both the hatched areas of Fig. 4 are equal.

So, even if the precision of non linear dynamic results is unquestionable, generally more simple analyses, that is static ones, are 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 two for considering the behavioural difference between static non linear analyses and dynamic non linear ones. 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 two and depending on the displacement demand. This is investigated in the present paper with reference to the frame structures of Section 5.

The necessity to have a more general methodology for robustness assessment of steel structures under each type of exceptional actions has led towards the implementation of a new non-linear analysis approach able to take into account both the catenary effect and the behaviour of connections. 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 path approaches.

The main difference between the proposed approach and the U.S. one is that in the former the computational work does not start with the original FEM model of the structure with a vertical element removed, but with the numerical structural model subjected to the design load combination. Therefore, the structure configuration before column removal is taken into account by evaluating the presence of vertical loads. 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 DAF. 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 method, called Load History Dependent (LHD) procedure [19], is based on a 3D structure model with both rigid floor diaphragms and beams and columns modeled as linear elements. Since large displacement analyses (i.e. considering the catenary effect phenomenon) are performed, geometric non-linearity have been considered in the FEM model.

For beams and columns concentrated plasticity hinges, as defined in the FEMA 356 [20], have been adopted. For the definition of the robustness index, the same U.S. code also specifies three different performance levels 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 Italian seismic code [21].

In order to take into account the dynamic nature of applied loads, a DAF is obtained through the following relationship [22]:

$$DAF = 1.08 + \frac{0.76}{\frac{\theta_{pa}}{\theta_y} + 0.83} \tag{10}$$

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 considered in the numerical FEM models to connect beams and columns. These design variables have allowed to estimate the connection influence on the inspected structures robustness.

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

$$D_{dir} = \sum_{i=1}^{n} \theta_{ai} + \zeta_{ai} \tag{11}$$

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, $n = 2 \times n_b \times n_f$ 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 column removed.

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

$$D_{tot} = \sum_{i=1}^{n_{Tot}} \theta_i + \zeta_i \tag{12}$$

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 and n_{Tot} is the real number of activated plastic hinges.

The robustness index I_r is therefore equal to:

$$I_r = \frac{D_{dir}}{D_{tot}} = \frac{\sum_{i=1}^{n} \theta_{ai} + \zeta_{ai}}{\sum_{i=1}^{n_{Tot}} \theta_i + \zeta_i} \tag{13}$$

When the indirect damage is zero, the structure is robust with reference to a prefixed performance level and $I_r = 1$. 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 that of a given performance level.

5. The investigated structures

Considering the exceptional nature of last Italian seismic events [23,24], two different types of steel framed structures have been analysed aiming at evaluating their robustness under outstanding earthquake actions having a return period of 2475 years and a

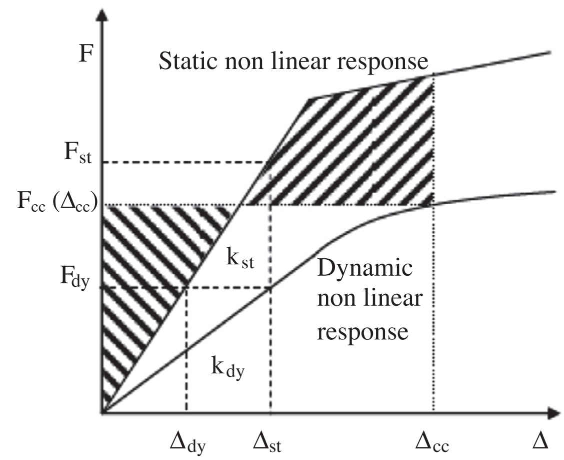

Fig. 4. Static non linear response curve vs. dynamic non linear one and explanation of the DAF.

probability to be exceeded of 2% during their service life [21]. These structures have been located in the historical centre of Naples on a soil type B. They are composed of three frames made of S275JR steel profiles, spaced 5 m each other, subjected to a permanent and variable loads of 5.15 kN m^{-2} and 2 kN m^{-2}, respectively. The first structure (type A) is a plane frame with a single 5 m bay on two levels with inter-story height of 3.5 m. The second plane frame (type B) has three levels (H = 3.50 m at 1st floor and H = 3.00 m at 2nd and 3rd floors) with three 5 m wide bays. Both structures have been designed according to the old (M.D. 96) [25] and the new (M.D. 08) [21] Italian seismic codes (see Figs. 5 and 6, where the used profiles are indicated) [26].

For these structures, the randomness of both materials (coefficient of variation *COV* of 3–5–7%) and vertical loads (coefficient of variation *COV* of 10–20–30%) have been considered at the light of a semi-probabilistic approach to be used for the robustness analysis of new structures. Therefore, the combination of the above *COVs* lead to nine analysis, which have been performed on each of the four examined structures.

Finally, different beam-to-column connection types, namely rigid and full strength, semi-rigid and partial strength and semi-rigid and full strength, have been considered only when the new robustness assessment method has been applied.

6. Analysis results

6.1. Under seismic actions

The robustness assessment under earthquakes has been performed by means of pushover analyses on the 2D FEM models of the examined structures, implemented through the SAP2000 non linear analysis program [27]. For each of the nine analyses, two different lateral load distributions have been considered, namely constant and inverted triangular type, so leading in total to seventy-two pushover curves (18 for each frame). Major details on the performed analyses are available in [28,29].

The gotten curves have been transformed into the Acceleration-Displacement Response Spectra (ADRS) plane in order to be compared with the demand spectrum given by the considered exceptional earthquake.

Therefore, according to Eq. (4) and Fig. 3, the robustness index has been determined for each examined structure, leading to the mean values of Table 2.

The achieved results have shown that frames designed according to old prescriptions are not robust, since they have a soft-story

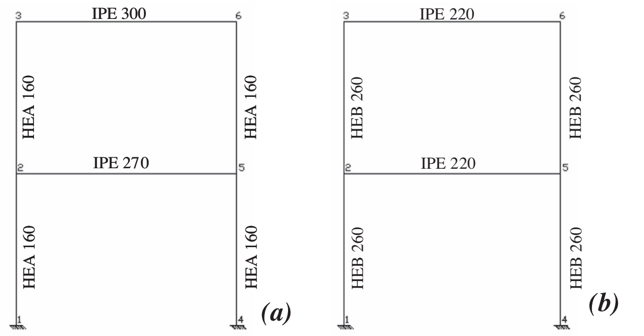

Fig. 5. 2-Story frame designed according to M.D. 96 (a) and M.D. 08 (b) Italian codes.

mechanism (both at the 2nd story) under lateral forces. On the other hand, frames designed according to M.D. 08 are extremely robust, they showing a global collapse mechanism. However, it is apparent that the new code provides results too on the safe side in designing new steel structures. In addition, by considering the structure characteristic curves, intended as the ones having the probability of being minored of 5%, six different target displacements can be identified as a function of both the yielding S_{dy} and the ultimate S_{du} spectral displacements of the same curves. Such targets are called as Fully Operational (FO = 1/3 S_{dy}), Operational (O = 2/3 S_{dy}), Life Safety (LS = S_{dy}), Robustness 1 (R1 = 1/3 S_{du} + 2/3 S_{dy}), Robustness 2 (R2 = 2/3 S_{du} + 1/3 S_{dy}) and Collapse Prevention (CP = S_{du} = maximum acceleration). For the sake of example, the characteristic pushover curve of the two-story frame designed according to M.D. 08, together with the above targets, is reported in Fig. 7.

Therefore, by considering five different seismic demand spectra, represented by the four earthquakes given in M.D. 08 (DLS, OLS, LLS, CLS) plus the exceptional one having the probability to be exceeded of 2% (ExLS), associated to the above target displacements, a robustness matrix can be built for each frame. For the sake of representation, such matrixes are reported for the 2-story frame only (Tables 3 and 4).

First of all, it appears that the old Italian code (M. D. 96) provides a very low level of robustness, whose indices are almost always below one. Also, in case of an exceptional earthquake, the structure designed according to M.D. 08 has a robustness index from 7 to 11 times greater than the one designed according to M.D. 96 (Table 5).

6.2. Under column-removed conditions

Firstly, the robustness of analysed structures has been assessed in terms of progressive collapse resistance by using the three analysis types of the first evaluation method described in the Section 4.2.

Two and six threat-independent column-removed conditions have been considered for the two level structure and the three level one, respectively. In the 2-story building, the first and the second level columns of the central frame have been removed separately from the structure. On the other hand, in the 3-story building, the columns of the 1st, 2nd and 3rd level belonging to the external and internal alignments of vertical elements have been removed one by one from the central frame.

So, for each of the examined 2-story framed structure, also considering the randomness of both materials and loads, 18 × 3 analysis type (LS, NLS and NLD), that is a total of 54 analyses, have been performed. Instead, for each of the 3-story framed structures, by fixing a COV_m = 7%, only load *COV*s have been changed and, therefore, 6 × 3 analysis types (18 analyses in total) have been carried out. As a result, a total number of 144 analyses has been executed.

For the sake of example, in Figs. 8 and 9 the behavioural curves of the 2-story building designed according to old and new seismic codes, respectively, when the 1st story column and the 2nd story one are separately removed from the central frame, are reported.

In the same picture, modified LS analyses accounting for the catenary effect have been also plotted. From the analysis results it is apparent that the modified LS curves are able to assess the real building behaviour in terms of stored energy, since the area under these curves is equal to the one enclosed under the NLD curves. So, a simplified way to evaluate the building behaviour under dynamic conditions due to the column loss has been found.

From Figs. 8 and 9, the robustness index of structures can be achieved for each NLD curve by making the ratio between the progressive collapse strength and the strength corresponding to the applied loads. If this index is larger than one, then the structure is robust; contrary, the structure is not able to sustain exceptional actions.

The analysis results have shown that: 1) examined structures, except the one with COV_m = 7% and COV_l = 10%, are not robust; 2) the robustness index of the building designed according to the new code is larger than 10% than that of buildings satisfying the old seismic provisions; 3) the robustness index of tested structures is slightly larger when the failure of the upper column occurs.

From the same Figs. 8 and 9, the following DAFs can be defined:

$$DAF1 = \frac{F_{\max,LS}}{F_{\max,NLD}} \tag{14}$$

$$DAF2 = \frac{F_{\max,NLS}}{F_{\max,NLD}} \tag{15}$$

In Figs. 10 and 11 the variation of DAFs with the displacement demand is plotted for the 2-story buildings (COV_m = 7% and COV_l = 10%).

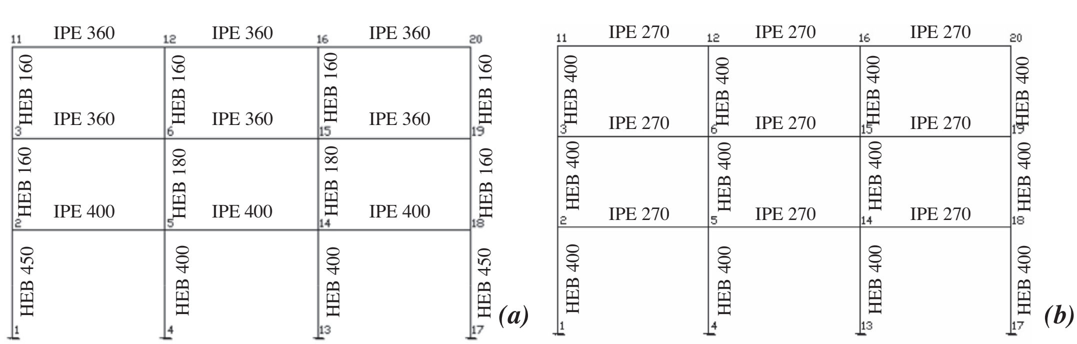

Fig. 6. 3-Story frame designed according to M.D. 96 (a) and M.D. 08 (b) Italian codes.

Table 2
Mean robustness indices of the study frames.

Frame	M.D. 96	M.D. 08
2-Story	0.34	4.21
3-Story	0.40	3.55

From the above figures it is apparent that: (1) DAF1 values are larger than DAF2 ones, with difference more marked when the 1st story column is removed; (2) in both cases DAF1 assumes a mean value of about 2.10 in the elastic field and is within the range [1.25–1.90] in the plastic one; (3) in both cases DAF2 assumes a mean value of about 2.03 in the elastic field and is within the range [1.22–1.80] in the plastic one.

From these results it is apparent that: (1) the GSA provisions are not on the safe side when elastic analyses are performed, since DAF1 is larger than DAF2; (2) the dynamic amplification in the inelastic field depends on the maximum allowable plastic displacement.

With reference to the 3-story structures, the results have shown that the robustness index of the structure designed with M.D. 08 is 10% larger than that of the old code structure. Differently from the previous case, the structures are always robust except when the column is removed from the 3rd story. In addition, loss of internal columns provides robustness index lower than the cases where columns are removed from external alignments. In this case, DAFs have been calculated at the ultimate condition, that is with reference to the strength corresponding to the maximum allowable displacement (Fig. 12).

From the above results it is evident that, when upper columns are removed, lower values of DAF are achieved. In all cases, a decreasing behaviour of DAF with the increasing structure height is noticed. As for the previous case, DAF1 values are larger than DAF2 ones. Finally, DAF1 values are decreasing when number of floors increases, with values comprised between 1.13 and 1.31 for M.D. 08 and 1.36 and 1.66 for M.D. 96. Contrary, DAF2 is almost constant when the number of floor is increased, with mean values of 1.07 and 1.04 for M. D. 08 and M.D. 96, respectively.

Secondly, the new robustness evaluation method has been applied to 3D FEM models of two investigated structures (Fig. 13).

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

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

Table 3
Robustness matrix of the 2-story frame designed according to M. D. 96.

	FO	O	LS	R1	R2	CP
OLS	0.46					
DLS	0.21	1.14				
LLS	0.02	0.11	0.27			
CLS		0.07	0.17	0.21		
ExLS			0.11	0.14	0.16	0.19

Table 4
Robustness matrix of the 2-story frame designed according to M.D. 08.

	FO	O	LS	R1	R2	CP
OLS	3.01					
DLS	1.47	9.17				
LLS	0.16	0.97	2.48			
CLS		0.57	1.46	2.20		
ExLS			0.80	1.20	1.63	2.09

Table 5
Robustness index ratios between the 2-story frame designed according to M.D. 08 and the one designed according to M. D. 96.

	FO	O	LS	R1	R2	CP
OLS	6.54					
DLS	7.00	8.04				
LLS	8.00	8.82	9.18			
CLS		8.14	8.59	10.48		
ExLS			7.27	8.57	10.19	11.00

In Figs. 14 and 15 the robustness indices of the structure type *A* (with rigid and full strength connections), designed respectively with the new code and the old one, are reported when the removed column position changes.

The same comparisons for the two structures type A have been performed also considering the presence of semi-rigid and partial strength connections (Figs. 16 and 17) and semi-rigid and full strength connections (Figs. 18 and 19).

On the other hand, for structures type *B* with connection type variation, the robustness indices corresponding to different column removals are reported in Figs. 20–23, where the symbols FR (resistance and stiffness full restoring), PR (stiffness partial restoring), PRR (resistance partial restoring) and CRR (resistance complete restoring) are used.

From achieved results, it is apparent that:

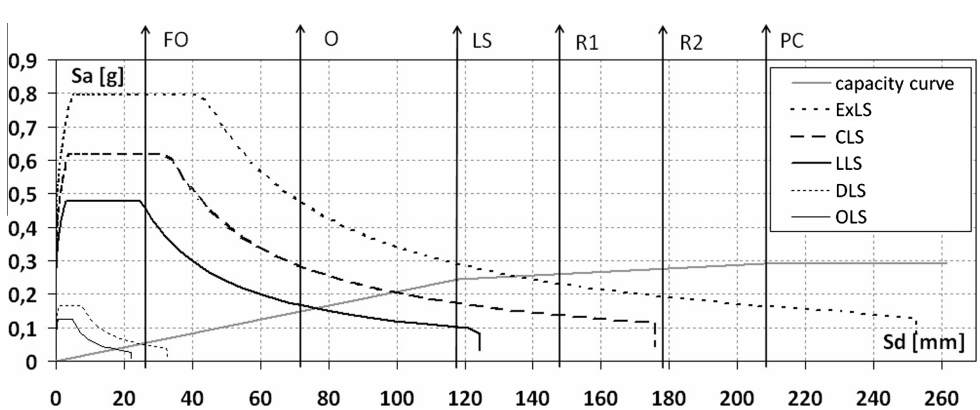

Fig. 7. Characteristic pushover curve of the 2-story frame designed according to M.D. 08.

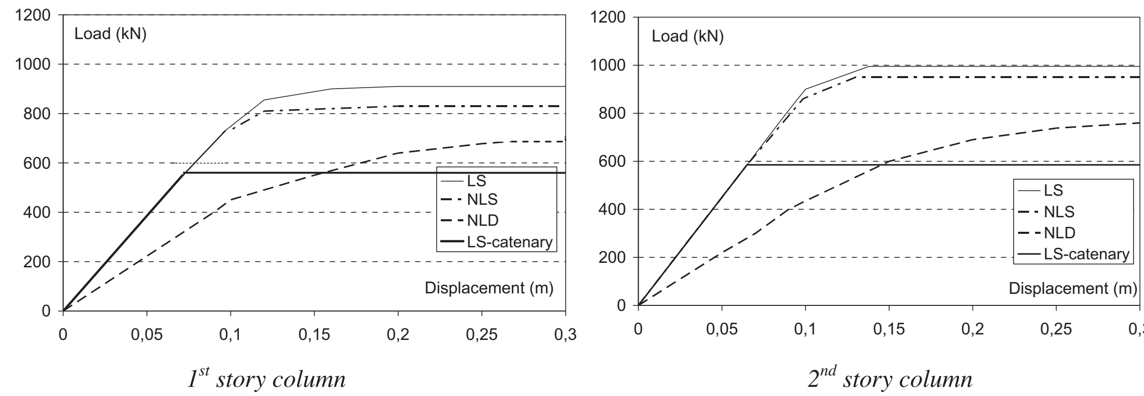

Fig. 8. Progressive collapse resistance of the 2-story structure (COV_m = 7% and COV_l = 10%) designed according to M.D. 96.

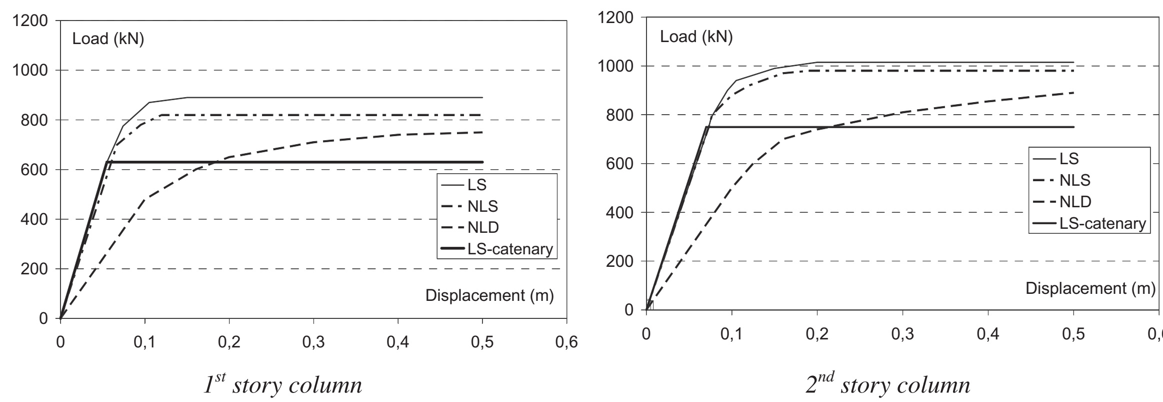

Fig. 9. Progressive collapse resistance of the 2-story structure (COV_m = 7% and COV_l = 10%) designed according to M.D. 08.

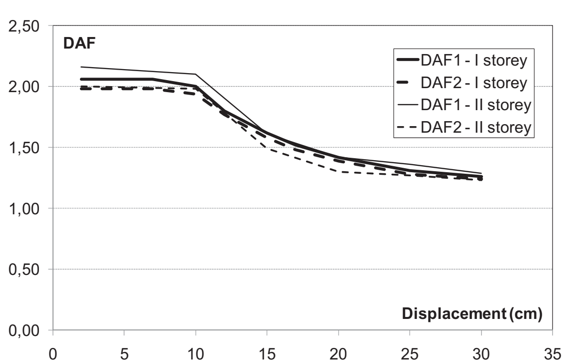

Fig. 10. DAF vs. displacement demand for the 2-story structure designed according to M.D. 96.

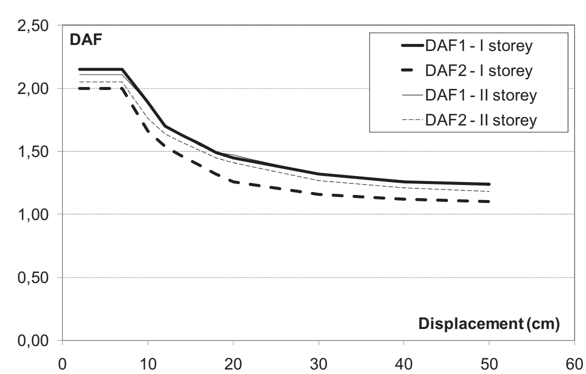

Fig. 11. DAF vs. displacement demand for the 2-story structure designed according to M.D. 08.

(1) Structures designed with the old code, due to beams with high flexural stiffness, have robustness indices greater than those of NTC 08 frames.
(2) The connection type is an influent design parameter. 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 a full strength type.
(3) About the worst scenarios of column removal, bad situations are those connected to the loss of internal columns, immediately followed by the loss of the corner column.
(4) The hazard scenario increases as the structure level number amplifies. This is in agreement with the provisions of the U.S. Department of Defense [22], which foresees as obligatory scenario the removal of the top story column.

7. Concluding remarks

In the current paper the robustness assessment of steel framed buildings designed according to the old and the new Italian seismic codes has been performed.

First, after recalling the definitions of robustness and progressive collapse, a general overview on the methodologies used for evaluating the structural robustness has been given. In particular, within the methods used for robustness assessment under earthquakes, a new deterministic approach, framed within the Performance Based Design, has been applied for evaluating the case studies performances. For these frames, the implemented

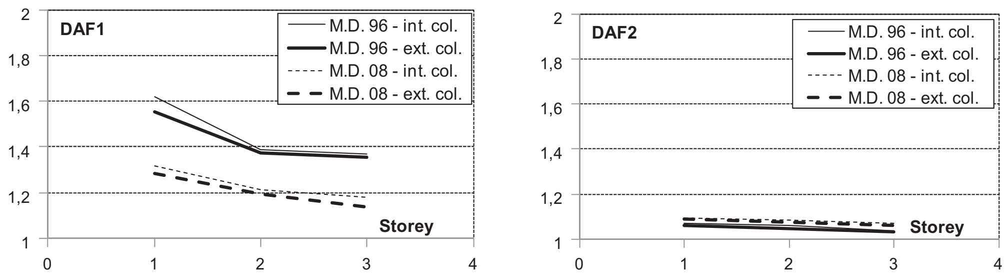

Fig. 12. Change of DAFs with respect to the column removal at different stories.

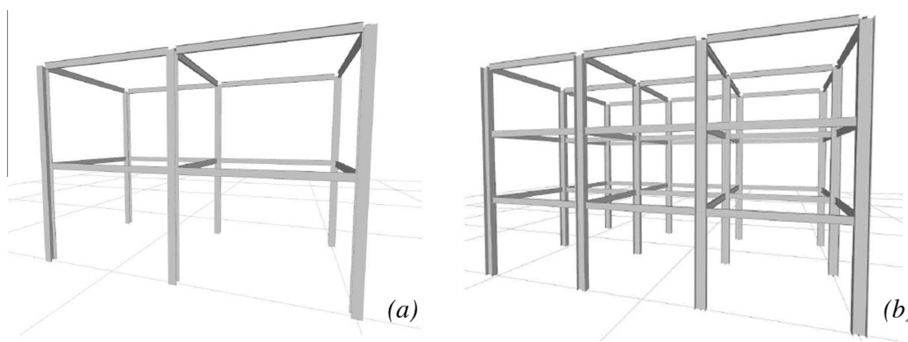

Fig. 13. 3D FEM models of type *A* (a) and type *B* (b) structures.

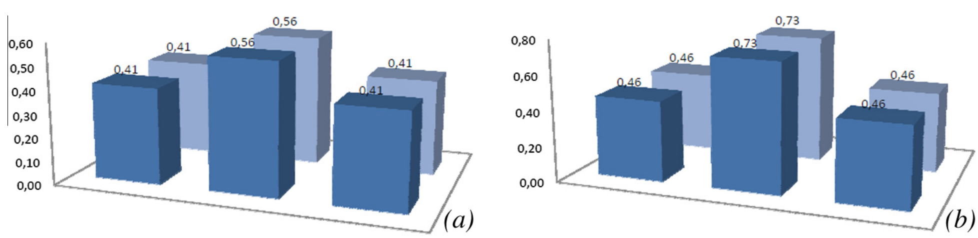

Fig. 14. Robustness indices of the NTC 08 structure type *A* with rigid and full strength connections at the first level (a) and the second one (b).

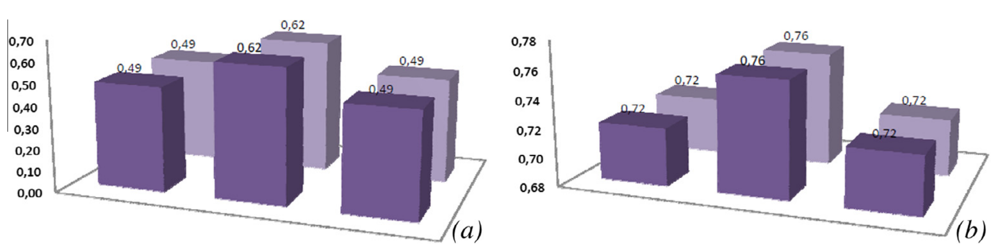

Fig. 15. Robustness indices of the DM 96 structure type *A* with rigid and full strength connections at the first level (a) and the second one (b).

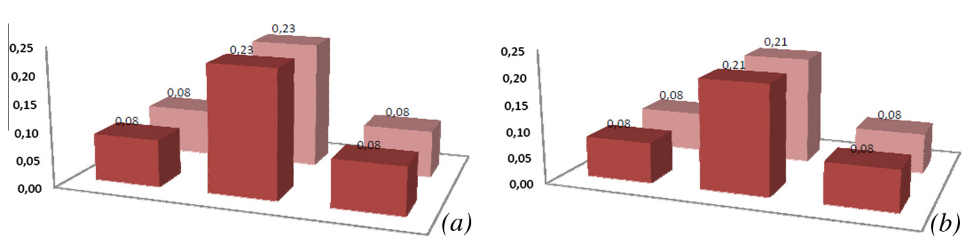

Fig. 16. Robustness indices of the NTC 08 structure type *A* with semi-rigid and partial strength connections at the first level (a) and the second one (b).

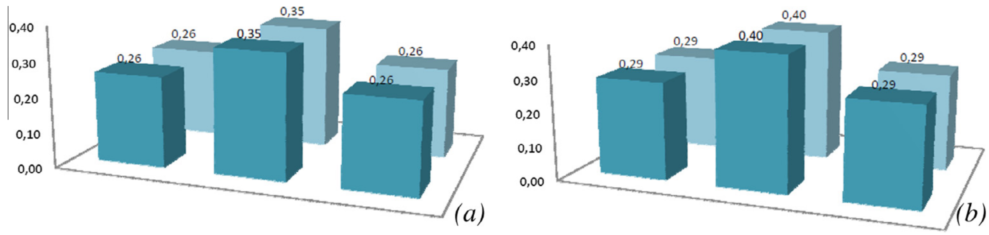

Fig. 17. Robustness indices of the DM 96 structure type *A* with semi-rigid and partial strength connections at the first level (a) and the second one (b).

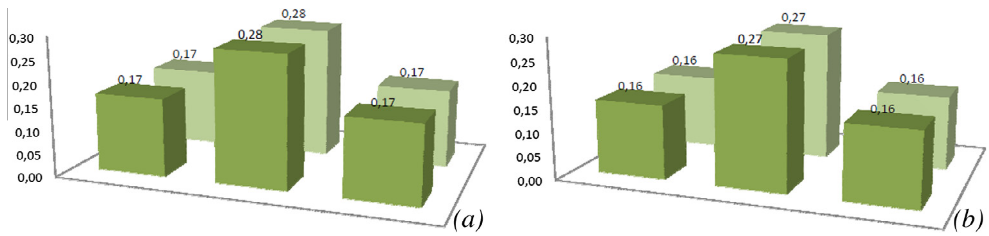

Fig. 18. Robustness indices of the NTC 08 structure type *A* with semi-rigid and partial strength connections at the first level (a) and the second one (b).

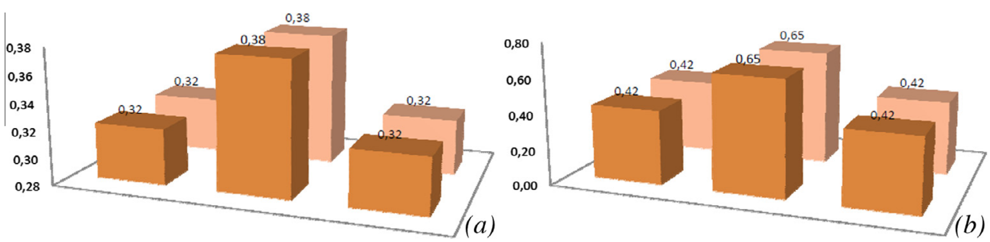

Fig. 19. Robustness indices of the DM 96 structure type A with semi-rigid and partial strength connections at the first level (a) and the second one (b).

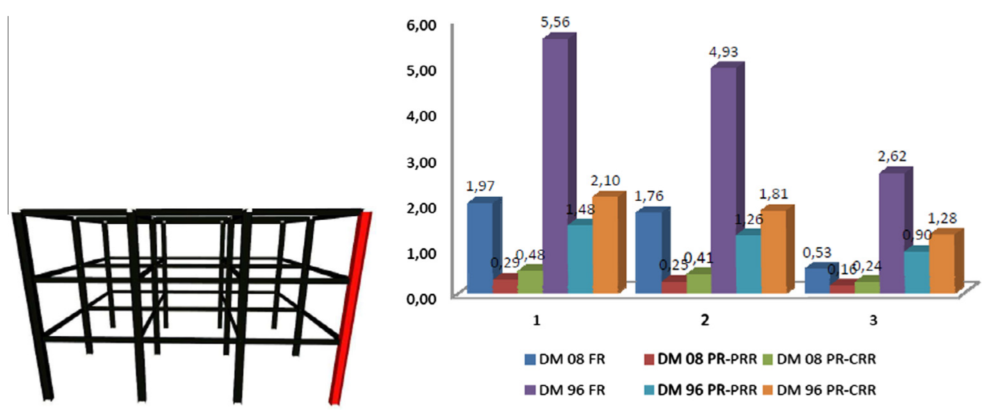

Fig. 20. Robustness indices of the structure type *B* with removal of a corner column.

method has allowed to calculate the robustness indices corresponding to specified targets, providing multi-level performance matrices. The analyses have shown that the old Italian frames have a very low robustness level, with both indices almost always below one and a soft-story mechanism at the second level. On the contrary, the investigated structures, when designed according to the new Italian seismic code, have provided high robustness indices under exceptional actions, their behaviour being characterised by a global collapse mechanism. Moreover, for the 2-story frame subjected to an exceptional earthquake, it has been noticed that, for CP limit state, the new seismic resistant structure has a robustness index 11 times greater that of the old seismic structure.

Second, the resistance to progressive collapse of steel framed structures designed according to the old and the new Italian seismic codes has been initially assessed by using linear static, non linear static and non linear dynamic analyses. The linear static analyses has been used when the column-removed structure is substantially elastic. Instead, in the plastic field, the collapse resistance has been well estimated from the capacity curves, which are used to simulate the structure NLD behaviour by exploiting the energy conservation principle. Also, linear static analyses

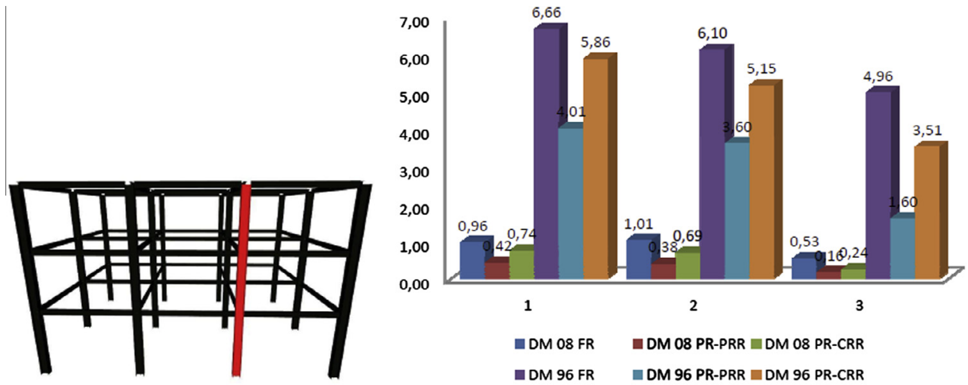

Fig. 21. Robustness indices of the structure type *B* with removal of one perimeter column on its long side.

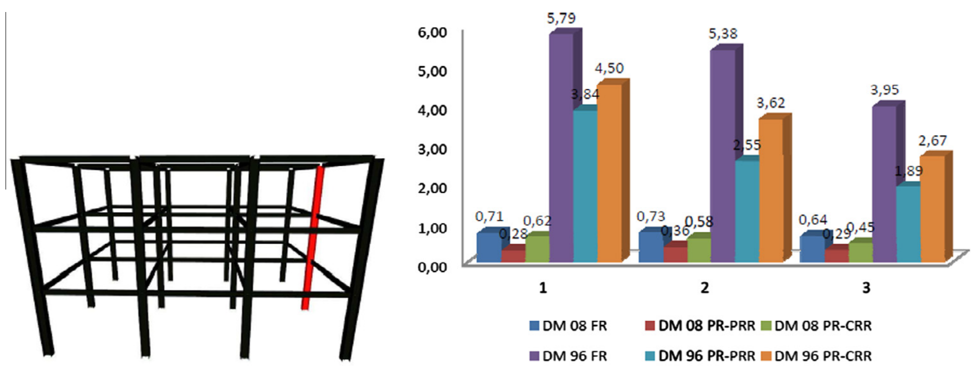

Fig. 22. Robustness indices of the structure type *B* with removal of one perimeter column on its short side.

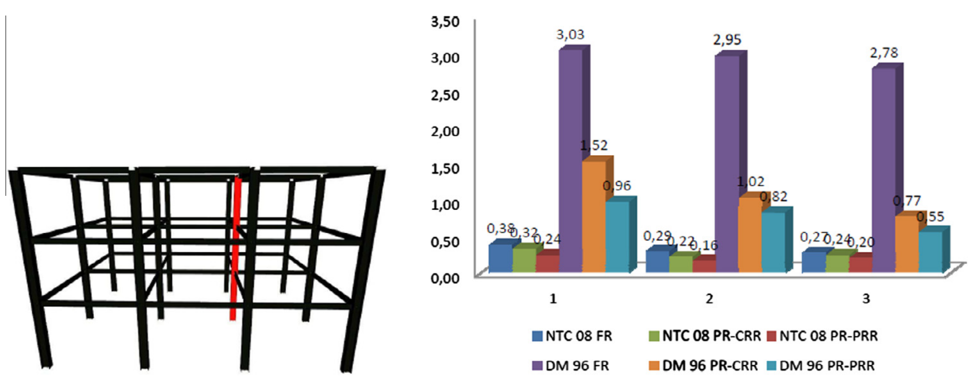

Fig. 23. Robustness indices of the structure type *B* with removal of a central column.

accounting for the catenary effect have been also performed, they being able to assess in a simple way the real building behaviour in terms of stored energy.

The analyses have shown that the robustness index of new code buildings is averagely 10% larger than that of structures satisfying old seismic provisions. Furthermore, the Dynamic Amplification Factors (DAFs) accounting for the dynamic effect due to the column removal when static analyses are executed have been assessed. From the results it has been shown that the GSA US code provisions are not on the safe side when elastic analyses are performed, since achieved DAFs are larger than two, and that the dynamic amplification in the inelastic field depends on the maximum allowable plastic displacement. In particular, for the 2-story and the 3-story structure, a mean DAF value of 1.23 and 1.16 is respectively achieved when the maximum allowable displacement is attained.

Finally, a new non linear static analysis method based on the alternative load path approach has been proposed in order to estimate the resistance against progressive collapse of examined structures. In particular, the computational model has been started with the whole structural model where gravity loads are applied. Afterwards, both the structural stiffness is decreased for taking into account the column loss and applied loads are increased through a DAF for considering the phenomenon dynamic nature. This allows to assess in a more precise way the stress redistribution into structural elements, so leading towards a Load History Dependent procedure. Therefore, the structure robustness index has been 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 the presence of geometric non linearity due to the catenary effect phenomenon. The achieved results have shown the best behaviour of structures designed by the old normative code due to more robust beams able to offer a better catenary effect, which is one of the most important parameters to resist to progressive collapse. Another fundamental robustness factor is 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. Adverse situations are those connected to the loss of internal columns, followed in order by the corner column loss and the perimeter columns lack.

In conclusion, the main achieved results, to be validated through further applications on real case studies, can be generalised and summarised as follows:

– By following a PBSD approach, old Italian framed structures show low ductility and do not satisfy robustness requirements, whereas the new ones exhibit high robustness levels, even if they are not explicitly designed against exceptional actions.
– Contrary, on the basis of a force-based analysis approach established on column-removed conditions, it can be affirmed that structures designed by the old code have better behaviour than new ones thanks to higher stiffness beams able to offer an effective catenary effect, indispensable to avoid structural collapses. Therefore, a new beam-to-column strength hierarchy criterion should be defined for robustness analysis of new framed structures. Furthermore, by analysing the influence of the connection type among members, the best performance are exhibited by full strength and rigid connections, which are recommended for steel moment resisting frames. Finally, it is advisable that both the hazard scenario increases as the structure level number increases and the worst circumstance of column removal is generally represented by the internal column loss.

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