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A Retrofit Method to Mitigate Progressive Collapse in Steel Structures

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Abstract

Accidental events, such as impact loading or explosions, are rare events characterized by a very low probability of occurrence. However, their effects often lead to very high human losses and economic consequences as are likely to trigger the progressive collapse of the buildings. The progressive collapse of structures attracted the attention of many researchers and the topic has been widely investigated in recent years. In addition, increasing interest has been shown also on the definition of retrofit strategies able to increase the robustness of existing structures. The present work investigates the performance and the design of a retrofit solution to increase the robustness of steel moment resisting frames. A case study structure is selected and modelled in OpenSees, including both mechanical and geometrical non-linearities. Non-linear static analyses have been carried out on the frame, simulating a column loss scenario to investigate the subsequent load redistribution. The simulations showed that the case study was unable to redistribute the load and hence retrofitting was required. Among others, a truss system was added at the rooftop level of the building allowing the definition of an alternative load path. The analyses outcomes showed how the proposed retrofit method allows to increase the robustness of the case study structure and allowed for critical remarks on the checks required when this retrofit system is employed.

Keywords

Progressive Collapse, Robustness, Retrofitting, Steel Moment Resisting Frames, Roof-truss.

1 Introduction

Man-made hazards deriving from events such as fire, explosions, impact, the consequences of human error or any kind of event that could produce a sudden loss of a load carrying component gained the attention of many researchers in the last decades because of the possibility of progressive collapse [1, 2]. Progressive collapse of a structure occurs when the failure of a structural component, leads to the collapse of the surrounding members, promoting additional collapse, and modern design codes require this cascading effect of failures to be considered and avoided during the design.

Several disasters, caused by different types of events, made the interest in the response of structures subjected to extreme loads such as impact or blast to continuously grow. Amongst others, well known cases are the collapse of the Ronan Point Building (London, 1968) [3], of the Murrah Federal Building (Oklahoma City, 1995) [4] and of the World Trade Center (New York, 2001) [5]. A significant research effort was made in this topic since 1940 [1, 2, 6 - 20], and provided an increasing understanding of the structural response and the definition of possible design solutions that are nowadays incorporated in several design codes worldwide [21, 22, 23].

Design properties such as stiffness, strength, ductility and stability of a structure are conventionally controlled through codified design procedures against specific design actions in order to meet the design requirements. However, during their life span, structures could be exposed to accidental events that are outside the coverage of normal design processes. These events are unpredictable in terms of cause, probability of occurrence and intensity, and hence it is not feasible and not economical to include their effects in the design procedure. The recognized approach in these cases is to provide the structure with the ability to withstand such events, without being damaged to an extent disproportionate to the original cause [21].

However, whilst many research studies investigated this topic and significantly advanced the level of knowledge on the structural response in case of accidental events, the approaches and procedures focused on the design of new structures and do not address the problem of retrofitting existing buildings to increase their progressive collapse resistance.

Most of the existing buildings worldwide, with a few exceptions, do not incorporate design provisions to achieve structural robustness

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and are susceptible to progressive collapse. In particular, steel structures could be very vulnerable since they are usually optimized with respect to specific design actions and are often characterized by a relatively low level of redundancy. In this context, there is a significant need for the development of effective and efficient retrofit methods against progressive collapse.

Very few research studies have addressed the problem of mitigating progressive collapse in existing structures. Some studies developed local measures to improve the building robustness. In particular, Galal and El-Sawy (2010)[24], Lui (2010) [25] and more recently, Ghorbanzadeh *et al.* (2019) [26] investigated possible retrofit strategies focusing on the increase of the strength and stiffness of beams and/or of beam-column joints with the aim to allow the development of catenary actions. The outcomes confirmed that, in some cases, these solutions could significantly improve the progressive collapse resistance of building structures. However, beam mechanisms are not always the most vulnerable and also requires an inherent degree of structural redundancy which is often not present in steel structures. In addition, in several situations, as consequence of the frame geometry and of the load configuration, the columns could represent the weak components of the system and hence, improving the beams performance would result in an ineffective intervention.

Papavasileiou and Pnevmatikos (2018) [27] investigated a retrofit solution for progressive collapse based on the introduction of steel cables within the frames on a few story levels. The numerical results showed how this system, if properly designed, can effectively provide improved performances by increasing the structure redundancy. However, this approach drastically changes the structural dynamic behaviour under horizontal actions, which could lead to significant detrimental effects on the seismic performance. For this reason, the authors also investigated a strategy where the cables are slack when the structure is in the undeformed configuration, and are not activated for inter-storey drift corresponding to the design-based earthquake intensity. However, the proposed solution requires significant displacements for the activation of the cables also during the progressive collapse scenario which is not optimum.

In addition to the specific limitations highlighted above, the described approaches are invasive and may need of long business interruptions.

The present paper investigates another interesting solution. In this case the structure's robustness is sought by the introduction of a truss at the rooftop level of the building. This 'roof-truss' is connected to the ends of all columns of the last floor and, if properly designed, allows the development of further alternative load paths providing a better redistribution process, without significant influencing the lateral stiffness and the capacity design with regard to seismic actions. The motivations on support of this solution are: *1)* it is a global retrofit measure that can in principle be applied to several structural typologies without relying on high redundant schemes; *2)* the low influence in the seismic response, due to the roof position of the retrofit system and the small added mass; *3)* the effectiveness against the column removal scenario by providing enough stiffness to involve a high number of columns in the alternative load path; *4)* the low invasiveness on the ordinary functions of the building which the intervention would entail, *i.e.*, low business interruption. Mirvalad (2013) [28] already investigated a similar solution consisting of two different rooftop hanging systems: a top beam grid and a top gravity truss. Both solutions aim to compensate the missing vertical stiffness and strength for the building with minimal effect on the building's seismic design. The author studied buildings with different floors number and seismic design actions demonstrating the potential of the retrofit solution in increasing their progressive collapse resistance.

However, the introduction of the 'roof-truss' may entail some issues in the existing structure that needs careful consideration. Amongst others, the column removal may induce tension forces in the upper columns, which may be higher than the yielding tension force of the section and/or of the column joint splices. Moreover, as the 'roof-truss' is able to redistribute the load to the other columns in terms of additional compressive load, they may fail because of buckling. A careful design of the 'roof-truss', able to calibrate both its stiffness and strength, enables the control of the load path generated by the column loss scenario, hence minimizing the local interventions, *i.e.*, strengthening of column splices and measures to prevent buckling. Additional studies are required in this respect.

In the present paper, a case study structure is subjected to a column loss scenario and assessed for progressive collapse. The numerical simulations demonstrated the lack of robustness and the need for retrofitting. Hence, the use of the 'roof-truss' retrofit system was used investigated allowing several considerations about the influence of the 'roof-truss' stiffness and strength on the alternative load path induced in the structure. The paper is organized as follows: in Section 2 the case study and the numerical modelling are outlined; in Section 3 the analysis procedure and the parameters of interest are described, in Section 4 the retrofit method is described and design considerations are made for the 'roof-truss', whilst in Section 5 the results of the numerical simulations are presented. Finally, in Section 6 conclusive remarks and future perspectives are drawn.

2 Case study structure and finite element modelling

A case study structure, already investigated by Gerasimidis *et al.* [29] under several column loss scenarios, have been selected for case study purposes. A Finite Element (FE) model was built in OpenSees [30] and a mixture of element types and analysis procedures were employed balancing accuracy and computational costs. The numerical simulations allowed the evaluation of the effectiveness of the retrofit intervention and the identification of some important aspects to consider for the 'roof-truss' design.

2.1 Case study frame

The case study analysed in this paper is a 9-story Moment Resisting Frame (MRF) building located in Greece and seismically designed according to the Eurocodes [31, 32, 33] considering a peak ground acceleration equal to 0.16g. A single plane frame is analyses and it is composed by 4 bays of 5 m spans and inter-story heights of 3 m for a total height of 27 m. An overview of the elevation, including the main geometric parameters and section members is reported in Figure 1. Sections are oriented with the strong axis within the frame and rigid, full-strength welded beam-column joints were used. The same steel S235 was used for both beams and columns and is characterised by yield strength f_y=235 MPa, Young's modulus E=210000 MPa and Poisson ratio ν=0.3.

2.2 Load combinations

The total considered Dead Loads (DL) is equal to 5.75 kN/m^2 and is applied on all floors. It includes 3.75 kN/m^2, corresponding to a self-weight of the 15 cm think concrete slab and 2 kN/m^2 related to the weight of the non-structural components. The Live Load (LL) was assumed equal to 2 kN/m^2 and is applied on all floors but not on the roof level where a Snow Load (SL) of 0.69 kN/m^2 is considered.

The following load combination, defined according to the UFC [22] was considered for the progressive collapse analysis:

$$q_d = 1.2DL + 0.5LL + 00SL \quad (1)$$

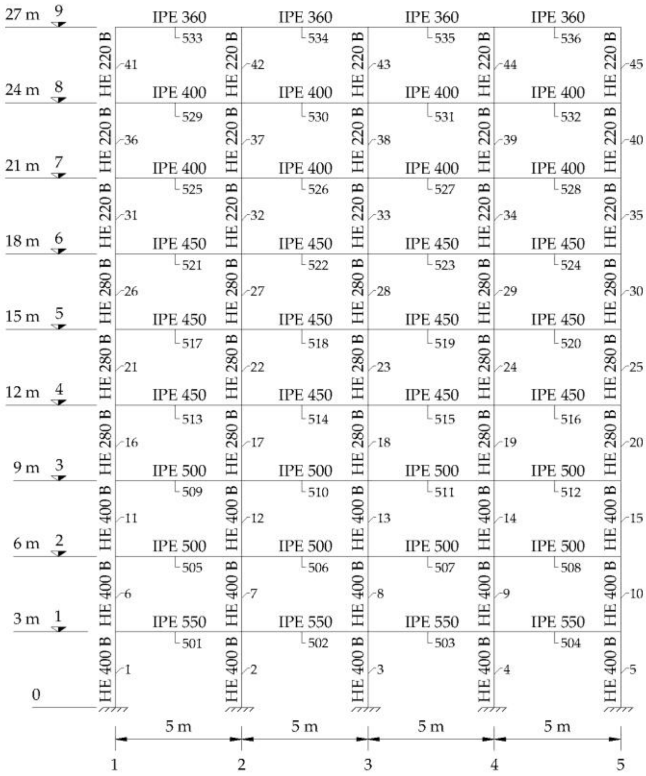


Figure 1 Geometry of the Case study (Adapted from Gerasimidis *et al.* [29]).

In addition, in order to consider the contribution of the dynamic effects, the gravity loads applied on the floor areas above the removed column are amplified by the Dynamic Increase Factor (DIF). The DIF recommended by the UFC [22] was used. This is function of the target structural response level and expected ductility demand of beam elements. In this case the DIF was assumed equal to 1.24.

2.3 Finite element modelling

A 3-D FE model of the plane frame was built in order to account also for the possible out-of-plane flexural buckling about the minor axis of the columns. The columns were modelled through a distributed plasticity approach, *i.e.*, 'force-based beam-column' elements in OpenSees [30] in order to account for the axial-flexural interaction which characterizes the non-linear behaviour of these members. The elastic shear deformations were included through the 'section Aggregator' while both the in-plane and out-of-plane flexural buckling were modelled through the introduction of local and global equivalent imperfections, as recommended by Eurocode 3-1-1 [32].

Beams were modelled through a lumped plasticity approach, combining 'elasticBeamColumn' elements with the 'Parallel Plastic Hinge' (PPH) model proposed by Lee *et al.* (2009) [34]. This model was developed to simulate progressive collapse scenarios and allows to account for both the flexural and axial actions rising in the beams' for increasing values of the vertical displacement [35]. An illustration of the beam-column joint and the PPHs used in the beams is reported in Figure 2. The PPH model [34] aims to provide an efficient macro-model for practical progressive collapse analyses. A set of parametric models were developed for the definition of the mechanical parameters of the springs and, in the present work such a system was calibrated and validated against experimental results [36] in order to increase the confidence on the results of the numerical model. The validation phase is reported in Figure 3, where the results of the PPH model are compared with the experimental data. It is possible to observe that the pushdown curve is in good agreement with the experimental results provided in [36].

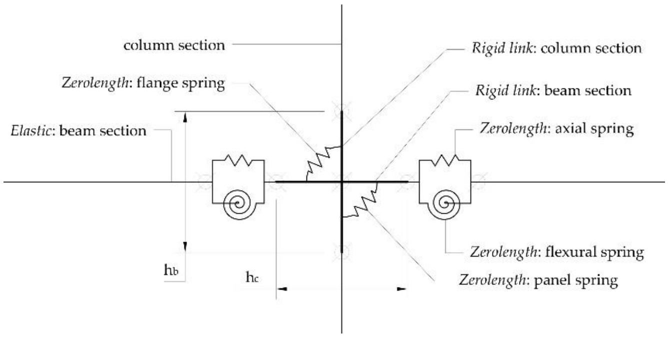


Figure 2 Model of the Beam-column joint.

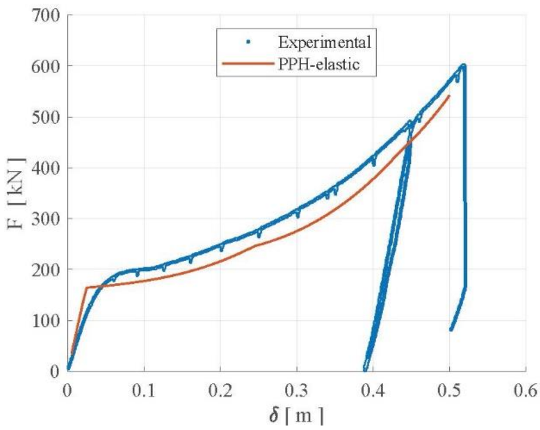


Figure 3 Experimental model calibration results.

In addition, as illustrated in Figure 2, the deformation of the panel zone of the beam-column joints was included by using the 'Scissors Model' [37]. This model consists of a set of two independent flexural springs, which simulate respectively the deformability of the web panel and the flanges of the column in the node. These springs connect two orthogonal rigid links whose extension is consistent with the physical dimensions of the node. The factors which govern the mechanical behaviour of this system were evaluated following specifications of recent studies on this particular model [37].

In order to simulate resisting mechanisms occurring in large displacement analyses, such as the catenary actions, the 'Corotational' formulation was employed in the analyses.

3 Progressive Collapse Analysis

3.1 Analysis procedure

The assessment of the progressive collapse performance in building frames is conventionally done by linear or non-linear static analysis, as suggested by the UFC [22]. The initial damage is introduced through the static removal of a column; thus the assessment of the progressive collapse resistance is carried out through the alternate load path method. The column removal was conducted in a quasi-static way, carrying out non-linear static analyses. However, the dynamic nature of the event was accounted for through the introduction of the DIF that amplifies the loads on the bays above the removed column as already discussed in Section 2.

In order to simulate the column loss scenario, the following procedure was used as illustrated in Figure 4.

In the "Analysis 1", standard static analysis of the undamaged structure (Figure 4(a)) are performed allowing the evaluation of the vertical load carried from the column which is meant to be removed.

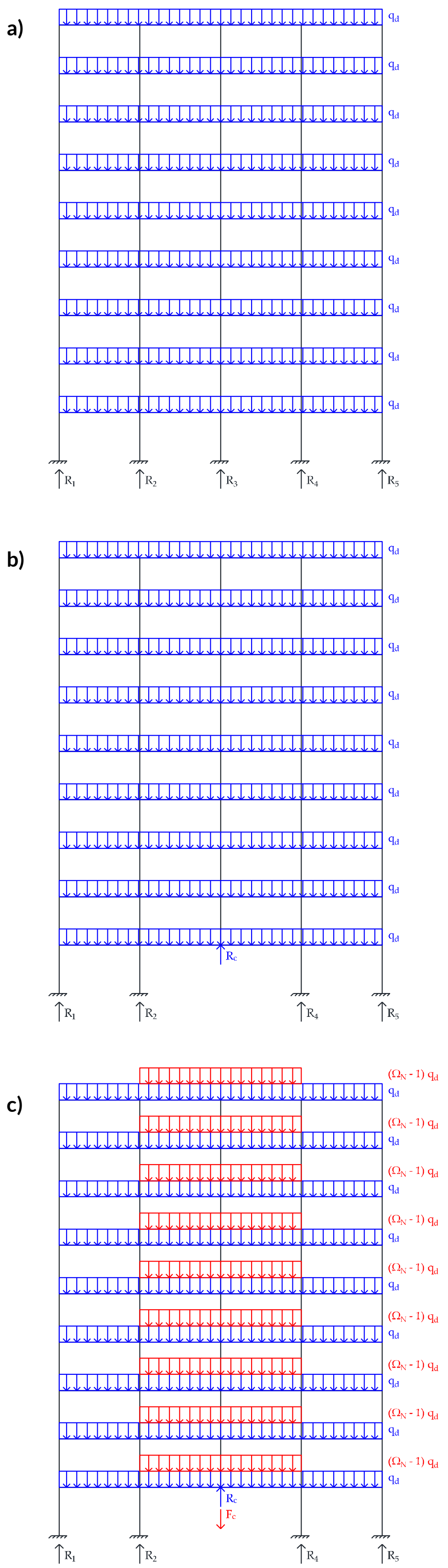


Figure 4 Analysis procedure for a central removal column scenario.

During the "Analysis 2", the column removal is simulated and this analysis is performed in two phases. The Phase 1, represented in Figure 4(b), allows the simulation of the column before the removal. An equivalent upward force R_c is applied to the frame, which entity corresponds to the vertical load previously detected on that column. In this Phase the gravity loads and the force R_c are monotonically increased until the target value. In the Phase 2, represented in Figure 4(c), a downward force F_c is applied to the same node of R_c, simulating the column removal. These two forces are equal in terms of values and opposite in terms of direction. Moreover, the loads on the beams adjacent to the removal are amplified with the DIF. This second set of loads are thus gradually applied. In order to simulate the initial condition, *i.e.*, presence of the column, in the second analysis, only the vertical force is applied to simulate the vertical reaction. Other reactions, such as shear force and moments where zero due to the symmetry of the structure.

3.2 Monitored parameters

The state of progress of the removal event is monitored by introducing the Load Factor coefficient (λ) which is defined as:

$$\lambda = \frac{\sum_{i=1}^{N} R_i}{Q_{tg}} \tag{2}$$

where $\sum_{i=1}^{N} R_i$ is the sum of the base reactions of the frame and Q_{tg} is the load target the structure is supposed to bear in the specified situation. When the load factor reaches the unitary value, all the loads applied have found an alternative path to the ground and the removal event is completed. If any failure is detected in the structure before this point, this means that the structure is not able to redistribute the load for the damage scenario investigated, highlighting the need for retrofit measures.

Moreover, to monitor the performance of the columns, the Work Ratio coefficient (WR) was defined as the ratio from the axial force N and the value which causes failure (*i.e.*, yielding in tension or buckling in compression) in that element N_b. The relative horizontal displacements u_x and u_z of the columns' middle nodes have also been monitored. For the beams, the maximum chord rotation (θ_t) defined according to the UFC [22] acceptance criteria for plastic rotation of primary beams are considered. The specific values for the different situations are reported in Table 5-2 of the code and for the present case study, which uses welded unreinforced flanges (WUF) connections, the maximum plastic allowable rotation is of $\vartheta_{pra} = 0.0284 - 0.0004\,d$, where *d* represents the beam deep.

4 Retrofit method

The retrofit intervention consists in the construction of a truss system at the roof level, *i.e.*, 'roof-truss', connected to all column ends of the last story. This additional structure enhances the robustness and redundancy of the building, making available more alternate load paths and providing a wider and more effective redistribution. The proposed strategy has several advantages: *1)* the intervention produces a very little increase of mass, and the lateral stiffness for the horizontal actions is unaffected: this turns in a nearly unchanged seismic behaviour of the structure; *2)* the roof localization concentrates most of the work at the rooftop level, meaning no significant interruption of the ordinary functions of the building; *3)* such a system, placed at the last floor of the building, can result as an effective intervention for different column removal scenarios.

The objective of the study is to provide the original structure with a wider redistribution capability by providing an alternative load path, and the introduction of the 'roof-truss' is an effective strategy in order to achieve this objective. However, its strength and stiffness must be properly designed as they affect the force redistribution. In Figure 5 is depicted the truss system considered. The number and spans of the bays is the same of the original case study frame and the removal has been assumed in the central column. The stiffness *K* has been taken as the force *F* above the removal divided by the corresponding vertical displacement *δ*. Two main simplifications are introduced in this part of the study: *1)* the diagonals are assumed to be only tension effective; *2)* the members of the frame are much bigger than the diagonals and based on this, all the deformability has been concentrated in diagonal members, modelling the other ones as rigid. In order to develop a design procedure for this retrofit strategy, the influence of the two main parameters has been investigated, respectively the area of diagonals *A* and the height of truss *H*.

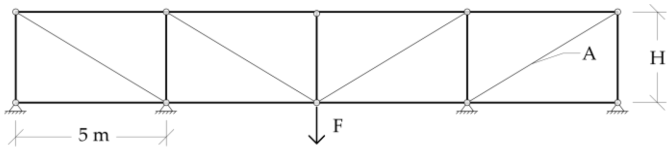


Figure 5 Roof-truss model for a central column removal scenario.

The parametric analysis has been carried out on the isolated truss model in order to assess the influence of *A* and *H* on the yielding force *F* and stiffness *K*. The results are reported in Figure 6. Moreover, the above simplifications and the study of the isolated 'roof-truss' allowed the development of an analytical formulation to provide some insights for the design as reported in the following equations:

$$F(A,H) = 2\,A\,f_y\,\sin\left(\tan^{-1}\frac{H}{L}\right) \tag{4}$$

$$K(A,H) = \frac{2\,A\,E}{H}\left[\sin\left(\tan^{-1}\frac{H}{L}\right)\right]^3 \tag{5}$$

The results show a linear trend that relate the stiffness and the diagonals' area (Figure 6(a)), while the relationship with the height is more than linear (Figure 6(b)). With this regard, an initial range of height values can be detected as responsible of a stiffness increase. Beyond a limit value, it starts to decrease, meaning lower effectiveness of the higher truss system. Commercial areas and height have been chosen for the truss model investigated, identifying the range of height values which in the present case provide a higher stiffness. A further parametric analysis has been conducted, investigating the redistribution capability of the truss subjected to a rising downward force *F* above the removal. The 'roof-truss' base reactions have been monitored and the results are reported in Figure 7.

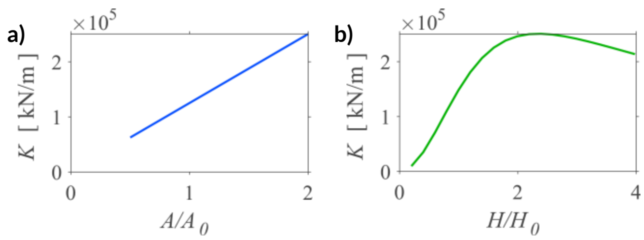


Figure 6 Roof-truss parametric analyses outcomes.

Figure 7 shows that a low height of the 'roof-truss' leads to a poor redistribution capability, since the most stressed supports are those adjacent to the collapsing column. For higher values of height, it shows a better redistribution performance, as the external supports are more involved in the redistribution mechanism, unburdening the adjacent-to-removal ones which goes even in tension in some cases.

These parametric analyses allowed to earn confidence on the performance obtainable with different height and area values. For the case study selected, a height of 4 m was deemed suitable for the objectives established. For diagonal members, steel bars with diameter of 85 mm have been chosen, while European profile HEM 260 have been selected for the orthogonal members.

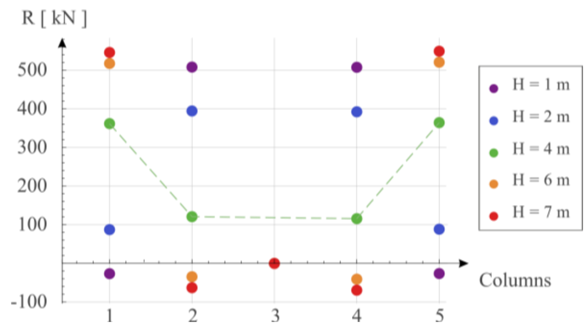


Figure 7 Roof-truss parametric analyses on the height values available.

5 Progressive collapse simulations

5.1 Original structure

The analysis on the original structure was performed and the results are shown in Figure 8, where the load factor *λ* of the structure and the WR of the most stressed columns are reported versus the vertical displacement above the removal. In the graph the two phases of the analysis, as previously described, can be recognized by the different stiffnesses. In the first phase, the presence of the column where the removal scenario is simulated, explains the stiff branch. As can be observed by Figure 8, the load factor *λ* cannot reach the unitary value before the most stressed member exhibits failure. This mean that the removal event could not get to the conclusion. For the case analysed, the beams resulted all safe while the adjacent-to-removal column underwent weak-axe flexural buckling, as showed from the horizontal displacements in *z* direction of the columns' middle nodes in Figure 9. The beam check is reported in Figure 10 where, Figure 10(a) shows the comparison between the demand and the yielding capacity of the beams in bending, while Figure 10(b) shows the comparison with respect to the rotation capacity. It can be observed that the beams rotation demand is far from reaching the maximum rotation capacity in all the columns and this highlights the strong proneness of the frame to column-type failure.

The analysis of the original frame highlighted the need of retrofit. The internal axial force in the columns shows a significant overloading of the columns adjacent from the removal, while the columns that are farther from the one that is collapsing are not significantly involved. The objectives of the design in this case should aim at achieving a wider load redistribution, in order to bridge the load from the removal location avoiding the overloading of few adjacent members. The capacity of distributing the load among a higher number of columns is related to the 'roof-truss' stiffness and this aspect was investigated and reported in the follow.

5.2 Retrofitted structure

The retrofitted structure has been analysed, its performances assessed and the results are shown in Figure 11. It can be observed that the load factor *λ* reaches the value of 1 before the failure of the other components monitored by the WR. This shows how the column removal could be completed and the redistribution achieved was sufficient for the objectives established. The intervention made available an alternative load path where the loads 'climb' back the columns above the removal by tension forces, reaching the 'roof-truss', which allows the redistribution among the other columns. It reduces the WR, and the horizontal displacements of columns' middle nodes, of the columns adjacent to the removal as shown in Figure 11 and Figure 12. As the vertical stiffness of the 'roof-truss' increases, less load is transferred trough the beams at each floor, as shown in Figure 13, and this explains the lower demand values on these components, both in terms of bending moments and rotations.

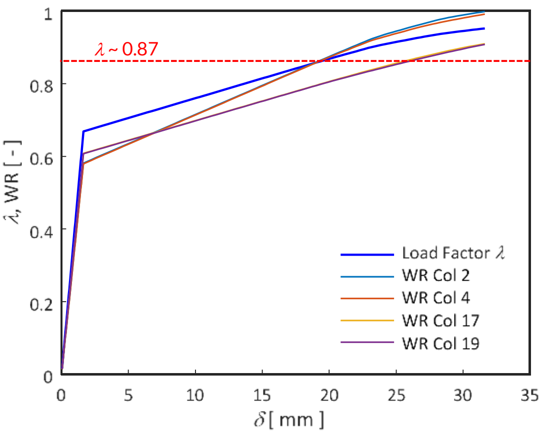


Figure 8 Original structure. Results of removal analysis.

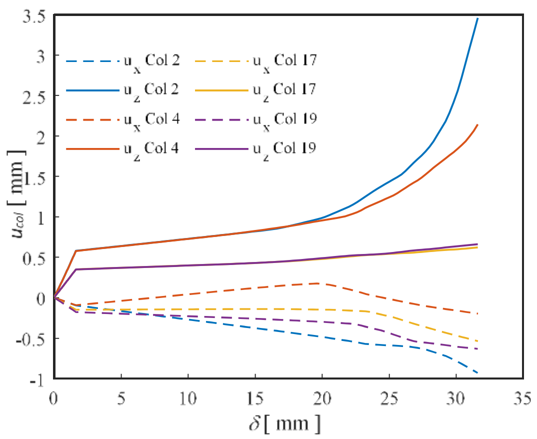
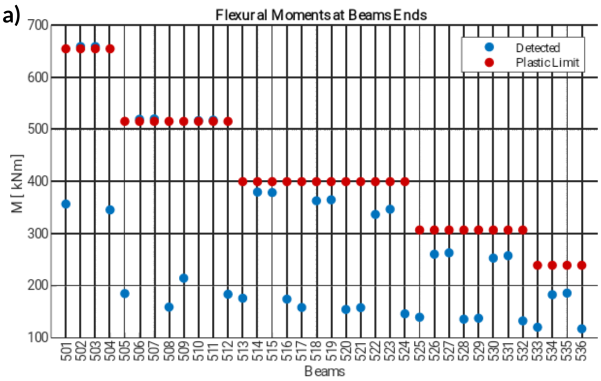


Figure 9 Original structure. Middle nodes displacement of columns

a)



b)

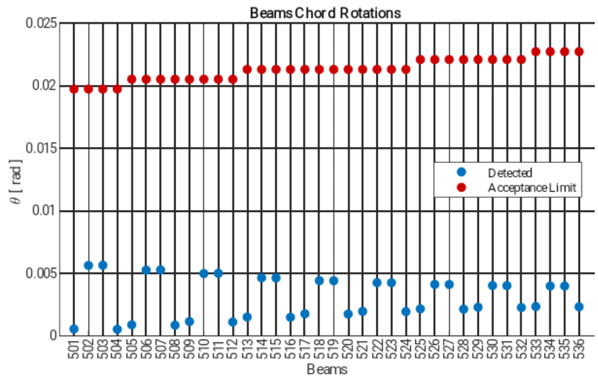


Figure 10 Original structure. Moment and rotations of all beams of original structure versus the acceptance criteria from UFC [22].

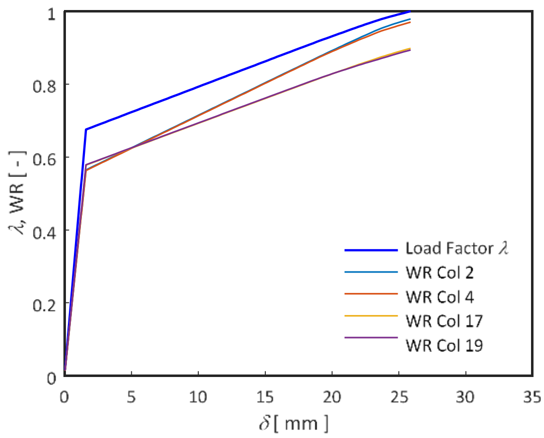


Figure 11 Retrofitted structure. Results of removal analysis

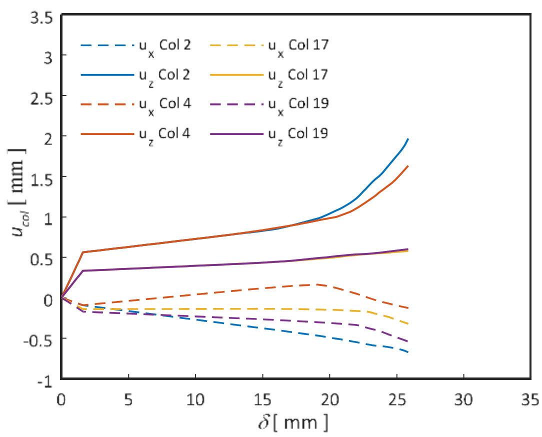
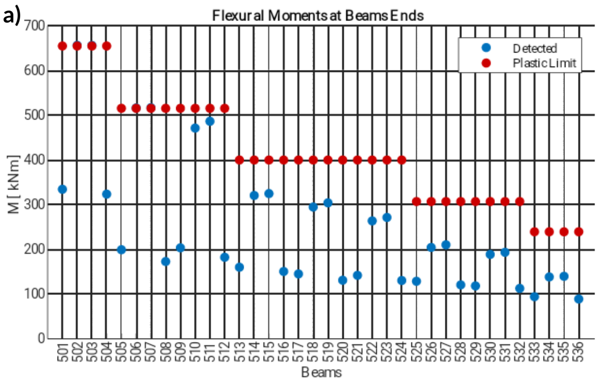


Figure 12 Retrofitted structure. Middle nodes displacement of columns.

a)



b)

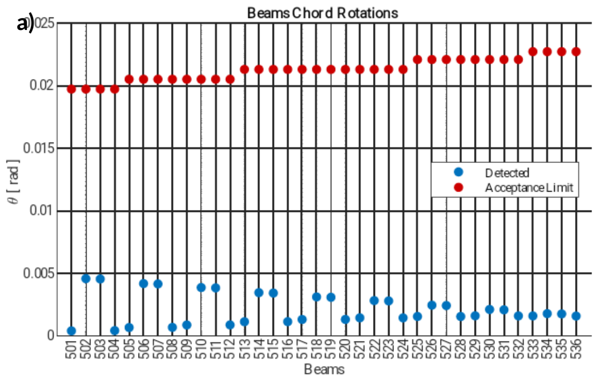


Figure 13 Retrofitted structure. Moment and rotations of all beams of retrofitted structure versus the acceptance criteria from UFC [22].

b)

However, the presented application of the proposed retrofit solution enlightened some critical aspects. Focusing on the relative participation of the columns at the same story there is a strong influence of the 'roof-truss' stiffness as can be observed in Figure 14. The strength of the columns and/or of the column splice connection, in tension may be limited and this aspect needs to be carefully considered while designing the stiffness of the 'roof-truss'. Too high stiffness of the 'roof-truss' may induce high tension forces in the columns and hence the intervention may require local strengthening. Similarly, the buckling resistance of the columns adjacent to the column removal has been identified as the most vulnerable mechanism. In order to limit the axial forces in compression in the columns, the stiffness of the 'roof-truss' should be high enough to allow a load redistribution involving also the elements that are farther from the column removal. This highlight the need for a careful calibration of the 'roof-truss' stiffness and strength. Figure 14 shows the comparison of the WR in the columns considering two 'roof-trusses': *1)* with height equal to 4 m; *2)* an infinitely rigid one. It can be observed that while the infinitely rigid 'roof-truss' allows a higher involvement of the columns farther from the removal in compression (Figure 14(b)), it induces higher forces of the column above the removal in tension (Figure 14(a)).

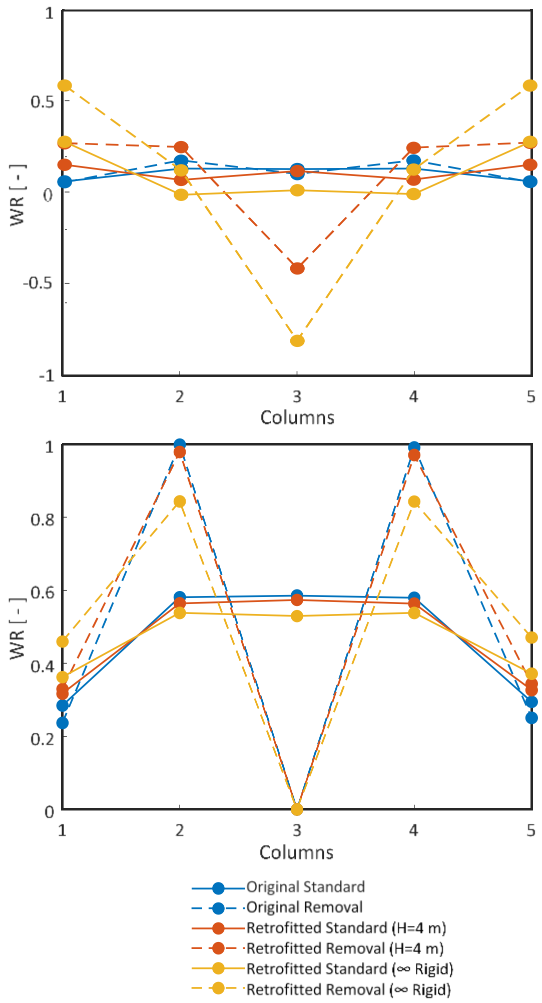


Figure 14 Work ratios for (a) last story columns and (b) ground story columns.

Figure 15 offers a different representation of the variability of the forces in the columns. The enhanced robustness effects can be read from a more uniformly distributed axial load in the columns in compression. On the other hand, as previously observed, an increasing vertical stiffness of the 'roof-truss' entails higher values of tension in the columns above the removal. It is clear as these members was not designed in these load combinations and hence a failure condition could be reached in these members due to this abnormal working scenario.

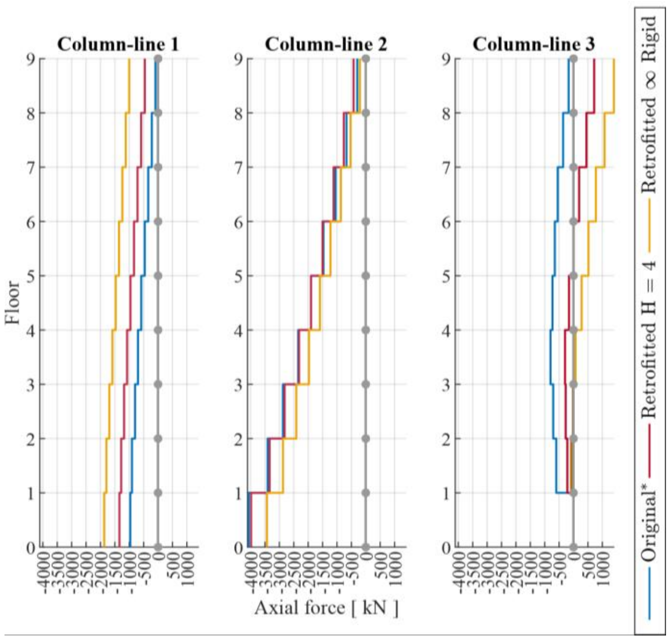


Figure 15 Columns axial force distribution.

6 Conclusions

The present work investigates the performance and the design of a retrofit solution to increase the robustness of a steel moment resisting frames. A case study structure is selected and modelled in OpenSees, including both mechanical and geometrical non-linearities. Non-linear static analyses have been carried out on the frame, simulating a column loss scenario to investigate the subsequent load redistribution capacity. The simulations showed that the case study was unable to redistribute the load and hence retrofitting was required. Among others, a truss system was added at the rooftop level of the building allowing the definition of an alternative load path. The analyses outcomes showed how the proposed retrofit method allows the increase of the robustness of the case study structure and the definition of critical remarks on the checks required when this retrofit system is employed. Amongst others, the analyses performed led to the following noticeable considerations: 1) the analyses on the original structure highlighted the need of a wider redistribution. The far from removal columns showed very low participation in the alternate load path while the adjacent ones were found to be overloaded; 2) the stiffness of the 'roof-truss' must be calibrated in order to provide the redistribution capacity and, at the same time to control the tension forces induced in columns above the removal. This could potentially turn in a critical aspect of the retrofit design. Future work will focus on the study of the influence of the change in terms of dynamic amplification before and after the retrofit which could play a significant role. Moreover, three-dimensional 'roof-trusses' and possible optimised configurations will be investigated.

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