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# NUMERICAL INVESTIGATION OF RETROFIT MEASURES TO MITIGATE PROGRESSIVE COLLAPSE IN STEEL STRUCTURES

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**Abstract**: *Man-made hazards, such as fires, explosions, or impacts, may have severe social and economic consequences and, therefore, should be carefully considered during the design of new, as well as, during the retrofitting of existing structures. Among others, these events could induce the progressive collapse of structures, in which the localised failure spreads from the single affected structural component to other parts of the structure. It is important to highlight that most existing structures worldwide have been designed before the introduction of design rules against progressive collapse. Therefore, it is nowadays of paramount importance to identify effective retrofit measures to renovate existing structures and return safer buildings to the community, including explicit design considerations against progressive collapse. The present paper investigates the effectiveness of three different retrofit measures, namely roof-truss, bracing, and cable systems, conceived to increase the structural robustness and hence mitigate the progressive collapse risk in steel structures. A case study steel moment resisting frame (MRF) was studied by performing non-linear static analyses in OpenSees and investigating its response before and after retrofitting. The progressive collapse was simulated by considering central column loss scenarios, and the ability to prevent the spread of failures of the original and retrofitted structures was examined. The present study sheds some light on the effectiveness and limitations of the considered retrofit measures in improving the overall robustness of the frame. The results show that, after the column removal, the original configuration of the selected MRF fails due to column buckling. Therefore, only the roof-truss and bracings strategies effectively improve the frame's robustness and allow the creation of alternative load paths. Additionally, some critical aspects to be carefully considered in the design of the retrofit measures are indicated.*

## Introduction

Events such as fires, explosions, or impacts may induce the progressive collapse of structures. These events, though typically characterised by a relatively low probability of occurrence, may have significant social and economic consequences, and therefore, progressive collapse dedicated design is of utmost importance. An increasing understanding of the phenomenon has been achieved in the last few decades (Izzuddin *et al.* (2007), Vlassis *et al.* (2007), Demonceau and Jaspart (2010), El-Tawil *et al.* (2014), Dinu *et al.* (2016). Adam *et al.* (2018), Freddi *et al.* (2022)), and several design codes worldwide currently incorporate recommendations to protect structures from progressive collapse (EN 1991–1–7, UFC 4–023, GSA (2003)).

Disasters such as the collapse of the Ronan Point Building (London, 1968) (Pearson and Delatte, (2005)), the Murrah Federal Building (Oklahoma City, 1995) (Sozen *et al.* (1998)), and the World Trade Center (New York, 2001) (Bažant, and Verdure (2007)) caught the attention, amongst others, of researchers, which extensively investigated the topic. An increasing understanding of the structural response was achieved, but the need for further investigation, in particular in the context of retrofit of existing structures to resist progressive collapse, has been made evident again by the recent progressive partial collapse of the 12-storey Champlain Towers in 2021 in Miami (Kong and Smyl (2022)). This building was of recent construction (completed in 1981) but was designed before the introduction of detailed recommendations against progressive collapse. Such a situation is quite common worldwide, considering that, for instance, 85% of the European

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building stock was built before 2001 (European Commission (2020)), while the first code accounting for progressive collapse resistance was introduced in Europe only in 2006 (EN 1991–1–7). This highlights the significant need to define effective retrofit measures to enhance structural robustness.

Whilst a 'good' level of knowledge of the progressive collapse phenomenon has been achieved over the last few years, the scientific community mainly addressed the problem of progressive collapse design of new buildings, while a very limited number of research studies focused on mitigating progressive collapse in existing structures. Among others, some of the proposed solutions for steel MRFs aim at increasing the strength, stiffness, and ductility of beams and/or beam-column joints intervening with local retrofit measures. Galal and El-Sawy (2010) studied the effectiveness of beams' strength and/or stiffness enhancement by evaluating the influence on three performance indicators,*i.e.,* chord rotation, tie forces, and displacement ductility demand. Liu (2010) suggested strengthening simple joints by changing the partial-strength shear-resisting joints to the full-strength moment-resisting joints to favour the development of catenary action. Structural details exploiting the strength and ductility of duplex stainless steel pins were proposed Ghorbanzadeh *et al.* (2019)) to increase the tensile resistance and rotation capacity of normally pinned joints. Karns *et al.* (2007) showed that the addition of side plates to the joints in steel structures could furnish additional stiffness, strength, and ductility to the connections with beneficial effects in terms of robustness. Moreover, global interventions, such as the roof-truss system conceived by Freddi *et al.* (2022), were proved to be a viable strategy to mitigate progressive collapse with low invasiveness on the ordinary functions of the building, *i.e.,* low business interruption. Moreover, as proposed by Mirvalad (2013), solutions consisting of rooftop hanging systems compensate for the sudden reduction in vertical stiffness and strength with minimal effect on the seismic design. Cables may be used to provide additional paths for the development of catenary actions, as shown by Astaneh-Asl *et al.* (2001), Papavasileiou, and Pnevmatikos (2018) and Zhu *et al*. (2019). More in general, though not specifically designed as retrofit interventions, several protective measures or design details can effectively reduce the risk of progressive collapse, as extensively documented in Kiakojouri (2022).

In this context, the present paper assesses the effectiveness of three different retrofit strategies for a steel structure subjected to progressive collapse after the removal of the central column. In detail, after ascertaining with numerical simulation the lack of robustness and the need for retrofitting in the original structure, a roof-truss, bracing, and cable systems were independently considered as possible retrofit measures. The results of the analyses show that roof-truss and bracing systems guarantee the attainment of the target load with a residual capacity of 8% and 1%, respectively. Conversely, it is shown that cables, which could be effective in improving beam mechanisms, are ineffective for this particular structure, as the progressive collapse mechanism results in being governed by the column buckling. The paper is organised as follows: Section 2 describes the case study structure and the numerical model; Section 3 presents the numerical procedure for the progressive collapse analysis and the results of the analysis for the original structure; Section 4 reports the design and the results of the retrofitted structure; finally, Section 5 provides some conclusive remarks.

## Case study and numerical model

A 9-storey moment resisting frame (MRF), characterised by inter-story heights of 3 m and a total height of 27 m, was selected for case study purposes. This MRF was already investigated in previous research works focusing on progressive collapse, and detailed information can be found in Gerasimidis *et al.* (2012) and Freddi *et al.* (2022). The frame was seismically designed for a peak ground acceleration equal to 0.16 g and complying with EN 1991–1–1, EN, EN 1993–1–1, and EN 1998–1. In the analysed direction, the building has 4 bays with a 5 m span, while in the perpendicular direction, the bay span is equal to 7 m. Steel sections are oriented with the major axis within the frame plane, and rigid, full-strength welded beam-column joints were considered. The employed steel sections are summarised in Table 1. S235 steel grade was used, with nominal yield strength $f_y$ = 235 MPa, Young's modulus E = 210000 MPa, and Poisson ratio v = 0.3.

Loads consist of the Dead Load (DL), equal to 5.00 kN/m$^2$ applied on all floors and the loads owing to the self-weight of beam and columns applied directly on the structural elements; the Live Load (LL), equal to 2.00 kN/m$^2$ applied on all floors except for the roof level; and the Snow Load (SL) applied only to the roof and equal to 0.69 kN/m$^2$ based on Eurocode guidelines (EN 1991–

1–1) for the Greek climate region in Zone III, 200 m of altitude and standard conditions. The progressive collapse resistance of the frame is assessed by considering the following load combination, according to the UFC 4–023.

$$q_d = 1.2\mathrm{DL} + 0.5\mathrm{LL} + 0.0\mathrm{SL} \quad (1)$$

Thirty-three concentrated masses at each story were considered to properly simulate the dynamic behaviour. Preliminary analyses were performed to determine the appropriate masses discretisation along the beams, and it was found that a finer discretisation did not produce significant variations in the results. A Rayleigh damping with damping ratio ξ equal to 5% was employed.

A 3D numerical model of the case study structure was developed in OpenSees (Mazzoni *et al.* (2009)), considering both in-plane and out-of-plane local imperfections and global equivalent imperfections according to EN 1993–1–1. As depicted in Figure 1, sinusoidal local imperfections were applied along all the column lines alternating the direction at every storey to maximise the unfavourable effects. Conservatively, the possible positive contribution of the slab to the progressive collapse resistance was neglected. A corotational formulation was employed to accurately account for geometrical non-linearities and possible large displacements. Columns were modelled with a distributed plasticity approach considering their torsional stiffness and their elastic shear stiffness included through the 'Section Aggregator' command. Conversely, a lumped plasticity approach was used to model beams. Beams were modelled as elastic, and the 'Parallel Plastic Hinge' (PPH) model proposed by Lee *et al.* (2009) was used to model the non-linear response at beam ends. The PPH model was chosen as it allows accounting for the bending moment and axial force interaction which is essential to simulate the catenary effects typically observed in progressive collapse scenarios. This model was validated against previous experimental tests on column removal scenarios to increase confidence in the numerical simulations. The panel zones of beam-column joints were modelled through the "Scissor Model" (Castro *et al.* (2005)) which accounts for the deformability of both the column web panel and flanges.

| Storey | Columns | Beams |
|---|---|---|
| 1 | HE400B | IPE550 |
| 2, 3 | HE400B | IPE500 |
| 4, 5, 6 | HE280B | IPE450 |
| 7, 8 | HE220B | IPE400 |
| 9 | HE220B | IPE360 |

*Table 1. Case study structure: columns and beams sections (adapted from Gerasimidis et al. (2012))*

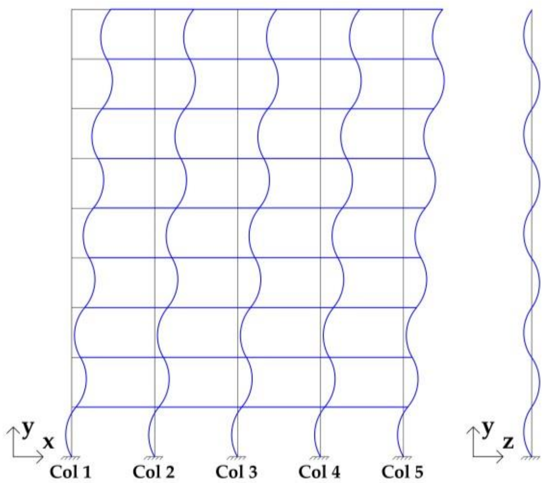



Figure 1. Simulated geometric imperfections in the case study moment resisting frame (MRF)

## Progressive collapse analysis of the original structure

The column removal of the central column at the 1st storey is the progressive collapse scenario considered in this study. The UFC 4–023 suggests investigating the redistribution capacity of a structure subjected to an element removal scenario by using non-linear static analyses, in which relevant loads are amplified by an increase factor, namely the Dynamic Increase Factor (DIF), accounting for the dynamic effects. Accordingly, numerical simulations were performed in this paper with a three-step static procedure, as depicted in Figure 2.

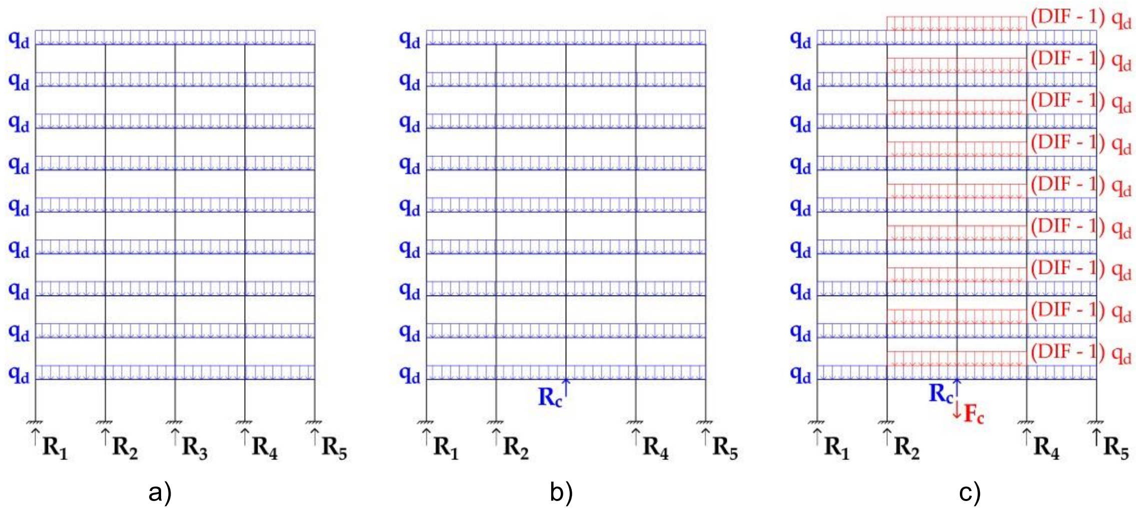



*Figure 2. Analysis procedure for the progressive collapse analysis: (a) Gravity analysis of undamaged structure, (b) State restoring analysis of the damaged structure, (c)Removal analysis*

In the first step (Figure 2(a)), a static gravity analysis of the undamaged structure is performed to evaluate the vertical reaction at the column which is meant to be removed. This reaction is applied monotonically as an upward force together with the gravity loads in step 2 (Figure 2(b)), in which the damaged structure, *i.e.,* with the column removed, is investigated. This step of the procedure reproduces the state reached at the end of step 1 in a numerical model without the damaged column, and is the starting point for step 3. In the latter (Figure 2(c)), an additional downward force $F_c$, with the same application point and magnitude but opposite direction to $R_c$, is applied in order to simulate the column removal. In addition, the loads in the adjacent beams above the removal are amplified with the DIF.

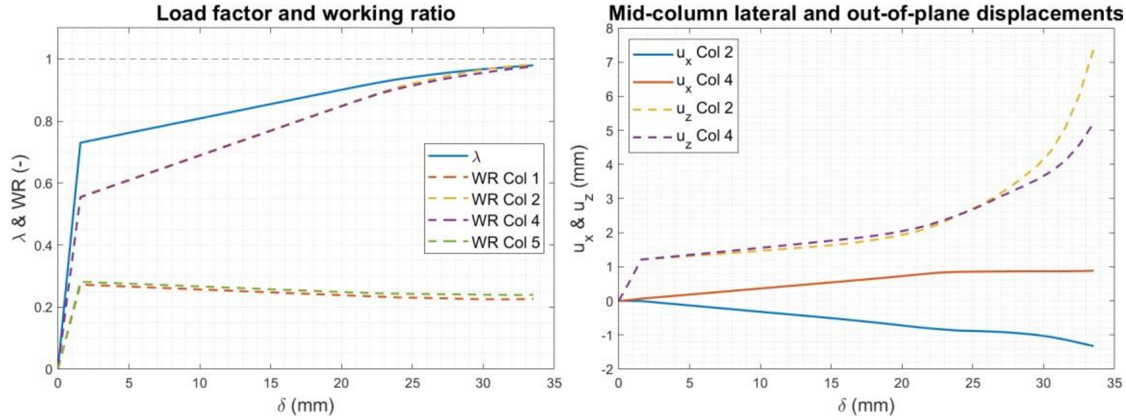
In the present study, the value of the DIF was determined according to UFC 4–023 as a function of the target structural response level and expected ductility demand of beam elements. Considering welded unreinforced flanges connections, which ensure a plastic rotation angle of $\theta_{pra}$ = 0.0284 –0.0004h where h is the beam depth, a DIF value of 1.24 was obtained.

The application of the loads was monitored by the load factor λ, defined as follows:

$$\lambda = \frac{\sum_{i=1}^{n} R_i}{Q_{tg}} \qquad (2)$$

where $\sum_{i=1}^{n} R_i$ is the sum of the vertical base reaction forces of the frame, and $Q_{tg}$ is the load target the structure is supposed to bear in the specified situation. When λ =1, all the loads were applied. Moreover, the Work Ratio coefficient (WR), defined as the ratio between the axial force N and the value that would cause failure, *i.e.,* yielding or buckling, was evaluated to monitor the performance of the most stressed columns.

The results of the progressive collapse analysis are shown in Figure 3. The load factor λ and the WR of the columns at the first storey are plotted against the vertical displacement above the removal in Figure 3(a). The sudden change of stiffness identifies the transition from the state before removal, *i.e.,* step 2, to the behaviour of the structure during the column removal, *i.e.,* step 3. In this step, the load factor λ does not reach the unitary value before column 2 (see Figure 2) exhibits failure. The maximum WR value is a little lower than 1, Figure 3(a), but the evolution of the lateral and the out-of-plane displacements of the columns' middle nodes in Figure 3(b) demonstrates that column 2 fails due to buckling. Moreover, columns farther from the removal are not particularly stressed and exhibit a significative capacity reserve. Therefore, a possible intervention to enhance the structural behaviour against the central column loss might aim at a wider load redistribution, bridging a bigger portion of the load from the removal location to the farther elements.

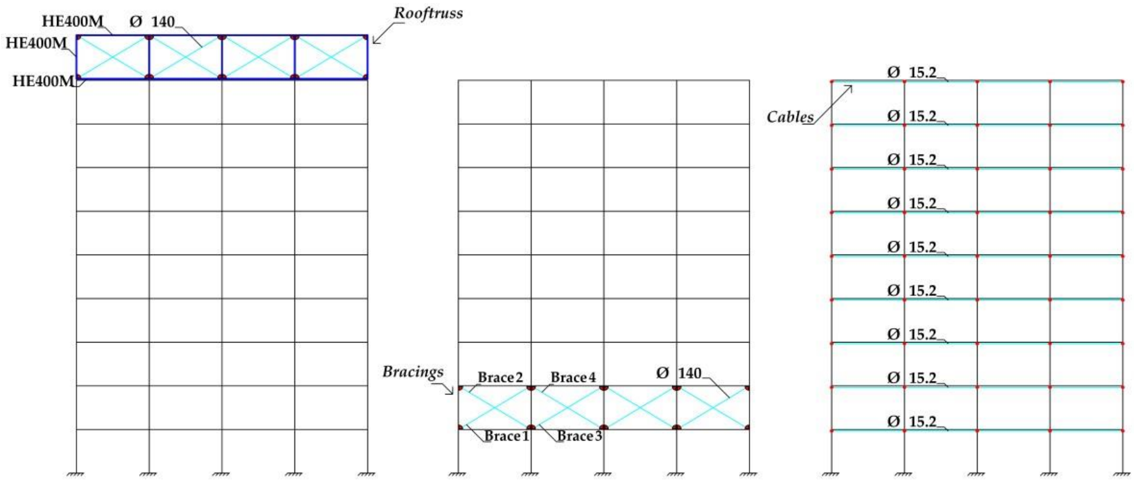



a) b)

*Figure 3. Results for the existing structure: (a) load factor (λ) and work ratio (WR) of columns at the 1st storey, (b) horizontal displacements of columns' middle nodes ($u_{Col}$).*

## Retrofitted structures

Three independent retrofit strategies, as depicted in Figure 4, were investigated to evaluate their effectiveness in improving the progressive collapse resistance of the structure. The same procedure performed for the original structure was adopted, and the results, together with a brief description of the retrofit measures, are presented hereafter. Advanced risk and cost-benefit analyses should be considered in the selection of the retrofit strategy but were not exploited in this preliminary work. Conversely, the three investigated solutions were selected among others to mitigate one of the most hazardous scenarios, *i.e.,* central base column removal, providing useful information on the structural behaviour. This information could be later complemented with other analyses to accomplish a more holistic selection of the retrofit solution.



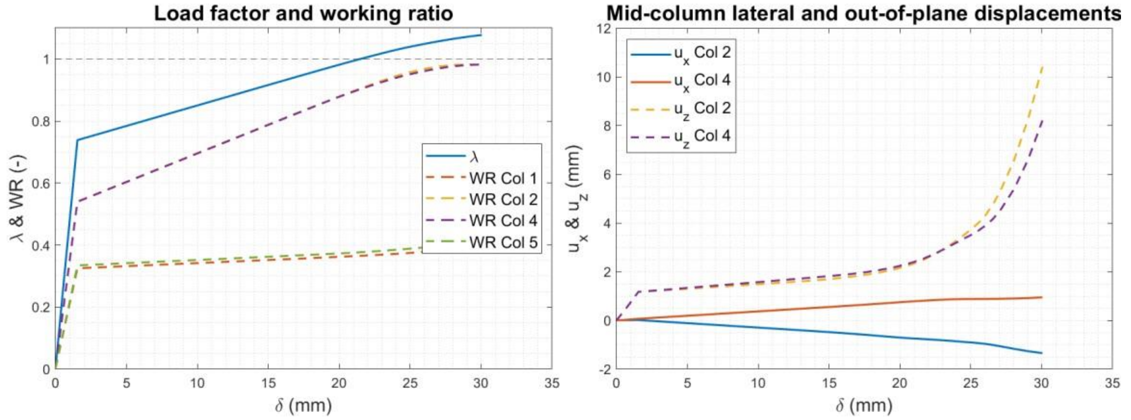

a) b) c)

*Figure 4. Retrofitted structures: (a) Roof-truss, (b) bracing, and (c) cable systems.*

*Roof-truss*

The first retrofit intervention exploits the redistribution capability of a truss system connected to the column's ends at the roof level, namely roof-truss. The considered roof-truss is 3 m high and consists of HE400M vertical and horizontal members, and circular diagonals with a diameter ϕ=140 mm. S355 steel grade is used with nominal yield strength $f_y$= 355 MPa and Young's modulus E= 210000 MPa. The roof-truss system was modelled with 'elasticBeamColumn' elements for the vertical and horizontal members since, as it has been checked a posteriori, they behaved elastically. Diagonals were modelled with a distributed plasticity approach and implementing initial imperfections according to EN 1993–1–1. Fully rigid connections were considered between vertical and horizontal members, and rigid elements were employed to account for the extensions of the sections in the connection zones. Internal hinges were employed to connect the diagonal elements to the rest of the roof-truss.

Figure 5 shows that the roof-truss system effectively redistributes the vertical loads allowing for achieving and surpassing the required bearing capacity. Indeed, collapse occurs owing to the buckling of column 2 when $\lambda$ reaches the value of 1.08. This additional load is transmitted by the roof-truss to the farther column, which in fact, reaches higher WRs at the structural failure. Besides, $\lambda = 1$ is attained with a reduced displacement of $\delta$ = 21 mm and quite before the columns' lateral and out-of-plane displacements suggest column buckling (see Figure 5b).

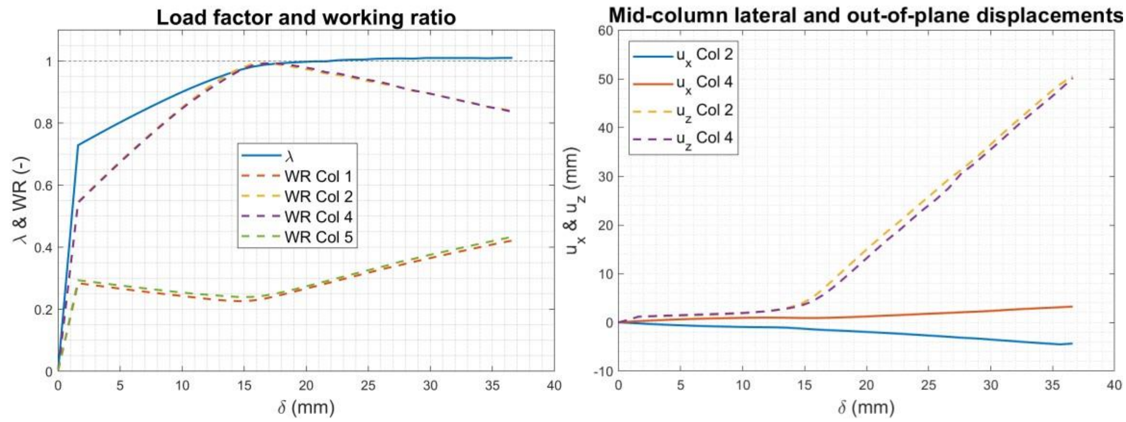



a) b)

*Figure 5. Results for the structure retrofitted with the roof-truss: (a) load factor (λ) and work ratio (WR) of columns at the 1st storey, (b) horizontal displacements of columns' middle nodes ($u_{Col}$).*

*Bracing system*

The second retrofit measure adopted is a bracing system consisting of diagonal members made of S355 steel (yield strength $f_y$= 355 MPa and Young's modulus E= 210000 MPa), with a diameter of $\phi$=120 mm. To account for buckling, these members were modelled with a distributed plasticity approach and considering initial imperfections according to EN 1993–1–1. Internal hinges were used to connect the diagonal's ends with the rigid diagonal elements simulating the dimensions of the steel sections in the beam-column connection zones.

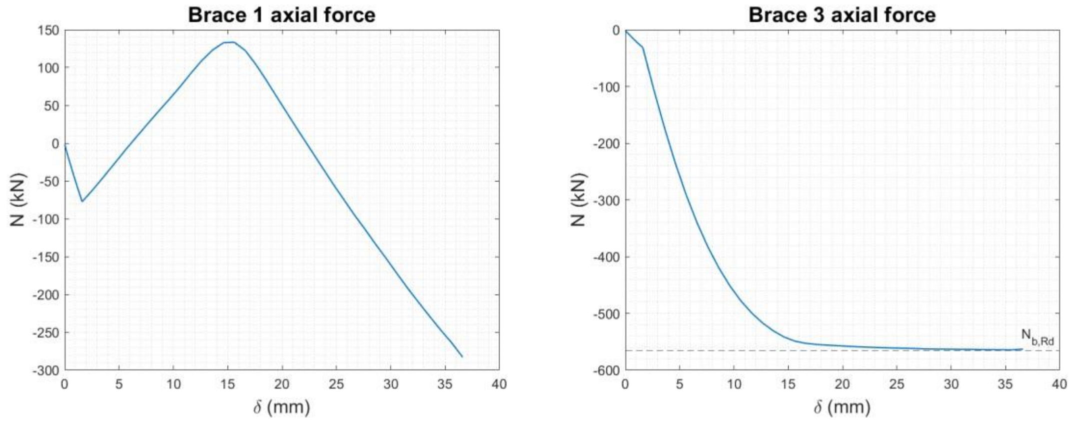



a) b)

*Figure 6. Results for the structure retrofitted with bracings at the 2nd storey: (a) load factor (λ) and work ratio (WR) of columns at the 1st storey, (b) horizontal displacements of column's middle nodes ($u_{Col}$).*

As shown in Figure 6a, the bracing system allows the structure to redistribute the load effectively but with almost no additional residual capacity. Interestingly, though the columns adjacent to the removed column suffer from buckling, the bracing system is able to carry the load and redistribute it to the external columns. Indeed, load redistribution is not activated until the axial force in the most stressed columns reaches the buckling resistance. After this point, the axial force gradually decreases in columns 2 and 4 and simultaneously increases in columns 1 and 5. Accordingly, Figure 6b shows that runaway deflections consequent to column buckling are impeded by the bracing system, which allows for large displacements in the columns adjacent to the removed column.

Failure occurs when the bracing system cannot withstand a further load increase owing to the buckling of the braces, as shown in Figure 7. The compressed brace between column lines 1 and 2, *i.e.,* brace 1, is compressed during the first loading phase, is unloaded, and goes in tension when the column is removed and is loaded in compression again after column 2 buckles, see Figure 7a. Buckling occurs in the compressed brace adjacent to the central column line, *i.e.,* brace 3, which allows for a limited loading increase after buckling of column 2, as shown in Figure 7b.




a) b)

*Figure 7. Braces axial force: (a) brace 1, (b) brace 3. (as indicated in Figure 4).*

*Cables*

The last retrofit measure studied is a cable system spanning along beams at all stories. Cables with a diameter $\phi$=15.2 mm and yield strength $f_y$= 1860 MPa were connected to the beam-column nodes truss elements allowing only for axial forces were used to model the cables. A pretension of 20% of the yield strength was applied to guarantee the activation of cables at lower vertical displacements.

Cables might be a good solution against progressive collapse by enhancing the catenary behaviour of the beams. The larger the vertical displacement above the removal, the higher the vertical load that can be transmitted through the cables to the beam-column joints and in turn to the columns. However, in this particular case, being the collapse governed by the exceedance of the buckling resistance in columns, the results show that cables do not significantly improve the structure's capability to resist progressive collapse. Hence, the evolution of the load factor $\lambda$ and the WR are not reported here as they are very similar to those presented in Figure 3a and Figure3b for the original structure. Nevertheless, the introduction of cables could be beneficial in other situations in which the collapse is related to the maximum beam rotations and/or to cases where failure occurs owing to the shear capacity of the beams in RC structures. Hence, the influence of cables will be further investigated in future studies, considering, for instance, RC structures or different structural typologies, *e.g.,* low-rise structures.

*Load redistribution*

Meaningful considerations can be provided by observing the capability of the retrofitted structures to redistribute loads. Figure 8 shows the axial forces in the 1st, 2nd, and 3rd column lines, reported for the original and retrofitted structures for the column loss scenario, at failure when $\lambda = 1$ was not reached, or at $\lambda = 1$. In the original structure, the columns on the 2nd line carry the biggest portion of the loads. The highest WRs are registered at the 1st, 4th, and 7th floor, and the buckling load is attained at the first floor before $\lambda = 1$ is attained.

The roof-truss solution allows for redistributing loads on the farther columns, and therefore, at $\lambda = 1$, lower axial forces are measured at the lower levels of column line 2. Axial forces significantly increase on the 3rd column line, but are still far from the buckling load. Nevertheless, some critical aspects should be emphasised as well. The load redistribution fostered by the roof-truss may induce tension forces at the higher storeys of column line 3. In general, columns and connections might not be designed to withstand significant tension forces, and therefore, the strength and stiffness of the roof-truss should be carefully designed to avoid excessive stresses. Indeed, a very stiff roof-truss may induce excessive tension forces and, consequently, local strengthening measures may be needed. Moreover, an adequate level of stiffness is required to allow the redistribution of a sufficient portion of the vertical load to the farther columns and avoid buckling in the columns adjacent to the column removal. Additional indications and studies on the calibration of the roof-truss can be found in Freddi *et al.* (2022).

For the structure retrofitted with the bracing system, load redistribution is shown at $\lambda = 1$, when the capacity of columns 2 and 4 is already reduced owing to buckling. Hence, a smaller part of the load from column line 2 is redistributed to column line 1, while the bigger part is transferred to the columns above the removal, *i.e.,* column line 3. Therefore, particular attention should be paid to avoid exceeding the buckling resistance of the columns above the removal, and therefore, as for the roof-truss, the stiffness and strength of the bracing system should be carefully calibrated. Finally, it should be observed that the roof-truss guarantees better performance, but a bracingsbased solution may be preferred since it might ensure a higher benefit-cost ratio. For instance, the roof-truss is effective for other scenarios, as the removal of a column at higher stories, but the proposed bracing system is less expensive and seems adequate to mitigate scenarios with a higher risk, *i.e.,* removal of a column at the first storey. Nevertheless, risks of different natures may threaten the structures. In a broader perspective, it should be carefully studied how retrofit measures for progressive collapse can affect the structural response during different accidental events. In particular, bracing systems influence the horizontal response of structures and may negatively affect their seismic behaviour.

As aforementioned, the introduction of cables is not effective for the present case study and therefore, the structure retrofitted with cables presents a load redistribution pattern that is very similar to the one of the original structure.

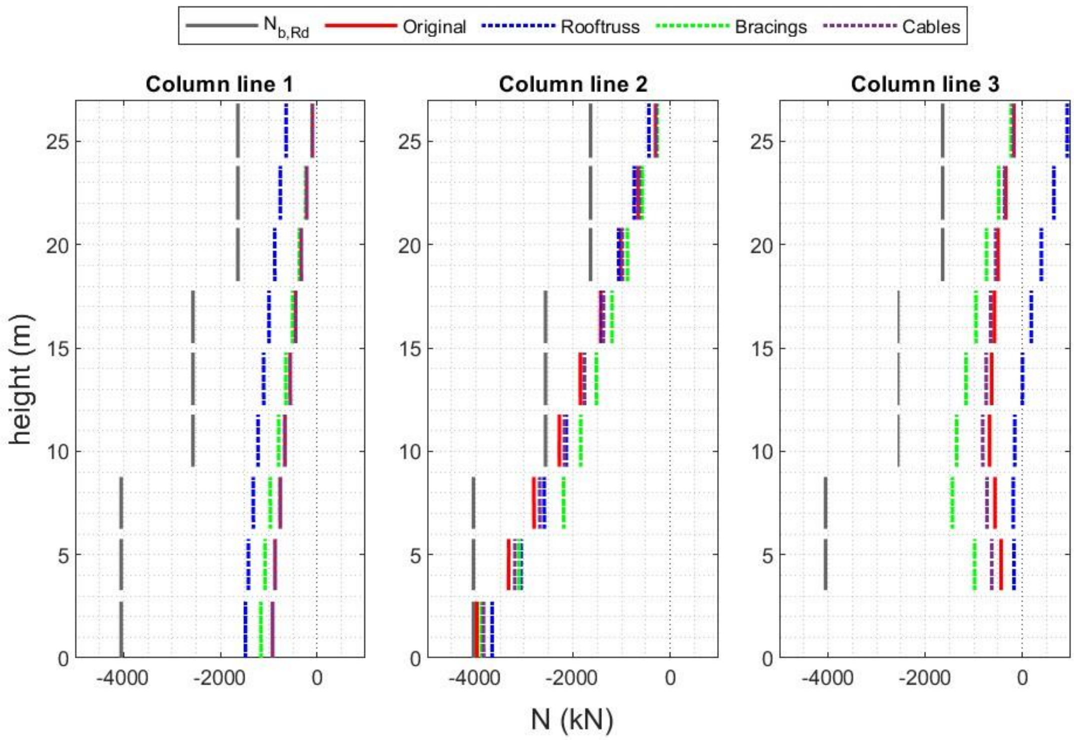



*Figure 8. Columns axial force distribution for the original and the retrofitted structures for λ = 1 or at failure.*

## Conclusions

The present paper investigates the progressive collapse resistance of a steel Moment Resisting Frame (MRF) considering a central column loss scenario and examines the effectiveness of three possible retrofit measures to enhance the structures' redistribution capacity. The study is based on numerical simulations performed on the original and retrofitted structures. Finite element models have been developed in OpenSees, considering both mechanical and geometrical nonlinearities to evaluate the structural response, including column buckling effects. Non-linear static analyses with the Alternate Path Method (APM) were exploited, accounting for dynamic effects after the removal of the column by increasing the relevant loads by a Dynamic Increase Factor (DIF), determined according to UFC 4–023. The original structure suffered from column buckling and exhibited structural failure before the target load could be applied. The structure was independently retrofitted, considering a roof-truss, bracing, and cable system. Only the structures retrofitted with roof-truss and bracing system met the required target, with an additional residual capacity of 8% and 1%, respectively. On the contrary, it was shown that cables do not provide any significant improvement in the ability to resist progressive collapse and are not suited for retrofitting the investigated structure. This is mainly due to the fact that, while the cables could be effective in improving the beams' ductility and rotation capacity hence promoting the formation of catenary actions, the original structure collapsed owing to column buckling, *i.e.,* a column-type mechanism. The roof-truss redistributed a significant part of the vertical loads from the central to the other columns, and tension forces were measured at the higher levels of the central column line. Conversely, the bracing system redistributed a smaller part of the loads to the perimetral column line, unloading occurred in the second column line starting from the second floor, and a significant load increase appeared in the central column line. Therefore, careful design of both the roof-truss and the bracing system is required in order to avoid excessive tension or compression forces above the removal as they may trigger column buckling or the failure of components, for instance, column splices that were not designed to withstand significant tension forces. Additional studies will be carried out on steel and reinforced concrete structures with different structural typologies and configurations to assess the effectiveness of the proposed retrofit measures in different situations.

## References

Adam, J.M.; Parisi, F.; Sagaseta, J.; Lu, X. (2018). Research and practice on progressive collapse and robustness of building structures in the 21st century. *Engineering Structures*; 173: 122–149.

Astaneh-Asl, A., Madsen E. A., Noble, C., Jung R. (2001). Use of catenary cables to prevent progressive collapse of buildings. Report number: UCB/CEE-STEEL-2001/02. *Univ of California at Berkeley*, US

Bažant, Z.P., Verdure, M. (2007). Mechanics of progressive collapse: Learning from World Trade Center and building demolitions. *Journal of Engineering Mechanics*, 133(3): 308319.DC

Castro, J.M., Elghazouli, A.Y, Izzuddin, B.A. (2005) Modelling of the panel zone in steel and composite moment frames. Engineering Structures, 27: 129–144.

El-Tawil, S.; Li, H.; Kunnath, S. (2014). Computational Simulation of Gravity-Induced Progressive Collapse of Steel-Frame Buildings: Current Trends and Future Research Needs. *Journal of Structural Engineering* (United States), 140(8): A2513001: 1–12

Demonceau, J.F., Jaspart, J.P. (2010). Experimental test simulating a column loss in a composite frame. *Advanced Steel Construction*, 6(3): 891–913.

Dinu, F., Marginean, I., Dubina, D., Petran, I. (2016). Experimental testing and numerical analysis of 3D steel frame system under column loss. *Engineering Structures*; 113: 59–70

EN 1991–1–1:2004 *Eurocode 1: Actions on structures – Part 1–1: General actions - Densities, self-weight, imposed loads for buildings*.

EN 1991–1–7:2006. *Eurocode 1. Actions on structures – Part 1–7: General actions – Accidental actions*.

EN 1993–1–1:2005, *Eurocode 3: Design of steel structures – Part 1–1: General rules and rules for buildings*, Brussels.

EN 1998–1:2005, *Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic action and rules for buildings*.

UFC 4–023 Unified Facilities Criteria– *Design of Structures to Resist Progressive Collapse*.. July 2009 – Change 3, 1 November 2016, Arlington, Virginia.

European Commission. (2020). *A Renovation Wave for Europe - greening our buildings, creating jobs, improving lives*. Brussels, Belgium.

Freddi, F., Ciman, L., Tondini, N. (2022). Retrofit of existing steel structures against progressive collapse through roof-truss. Journal of Constructional Steel Research; 188: 107037.

GSA (General Services Administration). (2003). *Progressive collapse analysis and design guidelines for new federal office buildings and major modernisation projects.*, Washington Galal, K., El-Sawy, T. (2010). Effect of retrofit strategies on mitigating progressive collapse of steel frame structures. *Journal of Constructional Steel Research*, 66(4):520-531 Gerasimidis, S., Bisbos, C., Baniotopoulos, C. (2012). Vertical geometric irregularity assessment of steel frames on robustness and disproportionate collapse. *Journal of Constructional Steel Research*, 74:76-89.

Ghorbanzadeh, B., Bregoli, G., Vasdravellis, G., Karavasilis, T.L. (2019). Pilot experimental and numerical studies on a novel retrofit scheme for steel joints against progressive collapse. *Engineering Structures*; 200: 109667

Izzuddin, B.A., Vlassis, A.G., Elghazouli, A.Y., Nethercot, D.A. (2007). Progressive collapse of multi-storey buildings due to sudden column loss — Part I: Simplified assessment framework. *Engineering Structures,* 30: 1308–1318.

Karns J. E., Houghton D. L., Hall B. E. ,Kim J. (2007). Analytical verification of blast testing of steel frame moment connection assemblies, *Research Frontiers at Structures Congress 2007*, California, US

Kiakojouri F, De Biagi V., Chiaia B., Sheidaii M. R. (2022). Strengthening and retrofitting techniques to mitigate progressive collapse: A critical review and future research agenda, *Engineering Structures,* 262

Kong., X., Smyl., D. (2022) Investigation of the condominium building collapse in Surfside, Florida: A video feature tracking approach, Structures, 43.

Lee, C-H., Kim, S., Lee, K. (2009). Parallel axial-flexural hinge model for nonlinear dynamic progressive collapse analysis of welded steel moment frames. *Journal of structural engineering*, 136(2):165–173.

Liu, J.L. (2010). Preventing progressive collapse through strengthening beam-to-column connection, part 1: theoretical analysis. *Journal of Constructional Steel Research*; 66(2): 229–37.

Mazzoni, S., McKenna, F., Scott, M.H., Fenves, G.L. (2009). *Open system for earthquake engineering simulation user command-language manual*, OpenSees Version 2.0, University of California, Berkeley, CA

Mirvalad, S.J. (2013) *Robustness and Retrofit Strategies for Seismically-Designed Multistory Steel Frame Buildings Prone to Progressive Collapse*. Ph.D. Thesis, Concordia University, Montreal, Canada.

Papavasileiou, G., Pnevmatikos, N. (2018). Optimised retrofit of steel-concrete composite buildings against progressive collapse us-ing steel cables. 16th European Conference on Earthquake Engineering.

Pearson, C., Delatte, N. (2005). Ronan Point apartment tower collapse and its effect on building codes. *Journal of Performance of Constructed Facilities*, 19(2): 172-177.

Sozen, M.A., Thornton, C.H., Corley, W.G., Mlakar, P.F. (1998). The Oklahoma city bombing: Structure and mechanisms of the Murrah Building. *Journal of Performance of Constructed Facilities*, 12(3): 120-136.

Vlassis, A.G., Izzuddin, B.A., Elghazouli, A.Y., Nethercot, D.A. (2007). Progressive collapse of multi-storey buildings due to sudden column loss — Part II: Application. *Engineering Structures*, 30: 1424–1438

Zhu, Y.F., Chen, C.H., Keer, L,.M., Huang Y., Yao Y. (2019) Structural response and resilience of posttensioned steel frames under column loss. *J Construct Steel Res* 158:107–19