A new partial-distributed damage method for progressive collapse analysis of steel frames

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Abstract

The alternate load path method has so far dominated the field of progressive collapse of structures; in order to assess the resilience of structural systems, the concept of the removal of a key element is utilized as a means of damage introduction to the system. Recent studies have indicated that the complete column loss notion is unrealistic and unable to describe a real extreme loading event, e.g. a blast, that will introduce damage to more than one elements in its vicinity. This paper presents a new partial distributed damage method (PDDM) for steel moment frames, by utilizing powerful finite element computational tools that are able to capture loss of stability phenomena. Through the application of a damage index δ and the investigation of damage propagation, it is shown that the introduction of partial damage in the system can significantly modify the collapse mechanisms and overall affect the response of the structure.

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1. Introduction

Progressive collapse analysis aims at assessing the performance of structures under the occurrence of a wide range of triggering events that introduce localized damage in the system. These triggering events (usually identified as blast events/terrorist attacks, vehicle impact, fire, structural design or construction defects and so forth) are often disproportionate to the resulting structural consequences, as was the case of the Ronan Point Collapse of 1968. During the last decades there have been numerous publications that present many different aspects of appropriate methods for progressive collapse analysis and for assessing the capability of structures to withstand localized damage ([1–5]). In this environment, both [6] and [7] employ the Alternate Load Path Method (APM) which attempts to quantify the robustness of a structural system by introducing damage through the loss of a primary load-bearing element, i.e. column. Through the use of computational structural analysis tools, the method can investigate key element removal scenarios in order to assess the vulnerability of the structure.

Most commonly, the response of structures under damage scenarios is highly nonlinear. Therefore it is critical to perform an appropriate progressive collapse analysis using a powerful finite element code that includes material and geometric nonlinearities and thus is able to account for nonlinear loss of stability phenomena. In recent papers ([1],[8] and [9]), the importance of stability considerations under a material and geometric nonlinearity analysis configuration is highlighted, in order to correctly identify the collapse modes and the corresponding collapse loads. The most common collapse modes include firstly the yielding-type failure of beam elements above the removal initiated by extensive plastification and secondly the buckling of column elements adjacent to the column removal. Other modes of collapse could also include the shear failure of the connections of the beams to the columns [10], or even a system loss of stability failure which appears more often in tall and slender structures [11].

According to [12], the concept of complete loss of one structural element is described as unrealistic, mainly for two reasons. Firstly, even under an extreme local event it is very improbable that an element will fail completely throughout its whole length and secondly, if such an extreme event does happen there will be non-negligible damage to other elements (beams or columns) as well. Therefore, the definition of a damaged state for a structure cannot be limited just to the notional removal of one of the components of the structure but can also include the partial damage of adjacent components. This is for example the case of a blast event during which the components in the vicinity of the blast will be affected, each one of them undergoing different levels of damage [13–15]. For many blast cases, especially with large charges, the alternate load path method is simply not enough to properly model the damaging event and therefore a new method is needed. [15] demonstrates that for certain cases, the damage can be distributed even to four columns and for that reason the alternate load path method
would be highly unconservative. However, so far, the design codes for progressive collapse have only included the notional component removal without allowing for a more sophisticated method of analysis which could include multiple component damage scenarios. This limitation of the alternate load path method can be also emphasized by two very famous progressive collapse events. Firstly, the Alfred P. Murrah Building in Oklahoma which experienced damage at several perimeter columns after a blast event [22]. This building would experience a very similar collapse mode even if the notional column removal design procedure would have been applied, due to the fact that the distribution of important structural damage within the structural system was extensive affecting several components. Secondly, the World Trade Center in New York which suffered extensive damage in multiple components after the plane collision.

Therefore, although the notion of complete column removal can serve the purposes of simplicity in a design against progressive collapse, it is not the most accurate, realistic and potentially conservative method for progressive collapse analysis. Along these lines, it is considered very interesting to investigate the influence of introducing partial distributed damage to different columns of the structural system, as opposed to the APM notion of one full column removal. The concept of partial damage of structural elements has been introduced in [16], by examining different cases of single and multiple partial losses of columns, aiming at the investigation of a more distributed damage scenario. However, this study was limited to a short steel frame, for which the stability considerations are generally not critical.

This paper presents a new method for progressive collapse analysis introducing partial distributed damage scenarios. The current state-of-the-art approach of one complete column removal scenario is compared to new partial distributed damage scenarios of multiple adjacent columns. The locality of the damaging event is maintained and the introduction of damage is applied to adjacent columns only. A damage index \( \delta_j \) is utilized to parametrically attribute different extent of local damage to the columns, where the upper bound is full local damage and the lower bound is intact condition. The method is applied on a 2D 15-floor steel frame and through the discussion of results, it is shown that the introduction of partial damage to the structural system not only leads to lower and more critical collapse loads but also changes the observed collapse mechanisms, alternating between yielding-type and stability-type collapse modes.

2. Partial distributed damage method for progressive collapse (PDDM)

2.1. Damage index \( \delta_j \)

Based on the classical definition of damage ([17]), we introduce damage via Kachanov indexes \( \delta_j \) which define the damage degree, satisfying:

\[
\delta_j = \frac{A - A'}{A}, \quad 0 \leq \delta_j \leq 1
\]

(1)

where \( A \) the overall area of the element \( j \) and \( A' \) the effective resisting area. The lower bound \( \delta_j = 0 \) corresponds to the intact state condition (no damage), the upper bound \( \delta_j = 1 \) corresponds to the fully damaged state, while any other value of the damage index corresponds to the partial-damage state. An element is considered removed if the full damage condition \( \delta_j = 1 \) holds for all its elements. Essentially:

\[
\text{if } \delta_j = 0 \Rightarrow \text{No damage}
\]

(2)

\[
\text{if } \delta_j \in (0, 1) \Rightarrow \text{Partial – Damaged State}
\]

(3)

\[
\text{if } \delta_j = 1 \Rightarrow \text{Fully – Damaged State}
\]

(4)

The introduction of damage in an element has an effect on the stress in the element:

\[
\sigma A = \sigma' A'
\]

(5)

where \( \sigma \) is the stress of the pristine element, \( \sigma' \) the effective stress of the damaged element and therefore:

\[
\sigma' = \frac{\sigma}{1 - \delta_j}
\]

(6)

Based on the hypothesis of strain equivalence, we can write for the undamaged and damaged state respectively:

\[
\{\varepsilon\} = \{E^{-1}\}\{\sigma\}
\]

(7)

\[
\{\varepsilon\} = \{E^{-1}\}\{\sigma'\}
\]

(8)

where \( E \) is the pristine Young’s modulus, \( E' \) is the effective Young’s modulus and therefore from (5), (6) and (7) we have:

\[
\{E\}' = \{E\}(1 - \delta_j)
\]

(9)

2.2. Partial distributed damage scenarios

A set of vertical push-down static analyses are performed in order to investigate the effect of partial damage distribution on the response of the structure. Let us assign a Damage Scenario vector:

\[
DS_f(k) : f \in \{1, 2, ..., n\} \text{ and } k \in \{1, 2, ..., 11\}
\]

(10)

where \( f \) are the different building floors and \( k \) the different damage scenarios. The floors of the building are \( n \) and there are \( 11 \) different damage scenarios utilized to introduce damage in the columns of the building. These damage scenarios include 2 complete column removal scenarios \( DS_f(1) \) and \( DS_f(11) \) and a set of ten partial distributed damage scenarios \( DS_f(2) - DS_f(10) \), for which damage is introduced to two adjacent columns. This configuration represents a more realistic localized damaging event that affects two rather than one structural elements, e.g. a blast event close to two corner columns of a frame.

For example, for a typical steel frame such as the one in Fig. 1, the first damage scenario includes the complete loss of the corner column \( A \) of number \( f \)/floor. This scenario corresponds to damage indices \( \delta_{ij} = 1 \) (full damage, meaning column removal) for all the elements of the corner column \( A \) of floor \( f \) and \( \delta_{ij} = 0 \) (no damage, meaning intact column) for all the elements of the adjacent column \( B \) of floor \( f \). The 11 damage scenarios, along with the initial \( DS_f(1) \) are listed below (the following numbering will be used as reference nomenclature from now on):

- \( DS_f(1), \delta_{ij}^A = 1; \delta_{ij}^B = 0 \)
- \( DS_f(2), \delta_{ij}^A = 0.9; \delta_{ij}^B = 0.1 \)
- \( DS_f(3), \delta_{ij}^A = 0.8; \delta_{ij}^B = 0.2 \)
- \( DS_f(4), \delta_{ij}^A = 0.7; \delta_{ij}^B = 0.3 \)
- \( DS_f(5), \delta_{ij}^A = 0.6; \delta_{ij}^B = 0.4 \)
- \( DS_f(6), \delta_{ij}^A = 0.5; \delta_{ij}^B = 0.5 \)
- \( DS_f(7), \delta_{ij}^A = 0.4; \delta_{ij}^B = 0.6 \)
- \( DS_f(8), \delta_{ij}^A = 0.3; \delta_{ij}^B = 0.7 \)
- \( DS_f(9), \delta_{ij}^A = 0.2; \delta_{ij}^B = 0.8 \)
- \( DS_f(10), \delta_{ij}^A = 0.1; \delta_{ij}^B = 0.9 \)
- \( DS_f(11), \delta_{ij}^A = 0; \delta_{ij}^B = 1 \)
3. Procedure for the propagation of failure and progressive collapse analysis

The last damage scenario refers to the full column removal of column B of floor f and the intact state condition of column A of floor f. This set of analyses is performed in turn for all floors of the building, f = (1,2,...,n). Hence, the total number of progressive collapse scenarios examined is the 11 damage scenarios multiplied by the total number of floors n, i.e. 11 x n. Moreover, it can be easily inferred from the partial damage scenarios selection that the total level of damage introduced in the structure is equivalent to one full column removal, since in every case the following relation is satisfied:

\[ \delta^A_f + \delta^B_f = 1 \]  \hspace{1cm} (11)

The reasoning behind this notion is to achieve consistency in comparing the results of full column removal and the alternate load path method to the results from the partially distributed damage scenarios.

3.3. Progressive collapse capacity

The progressive collapse capacity for every damage scenario is then defined as the maximum between \( CL_f(k) \) and \( CL_{A,B}(k) \):

\[ CL_f(k) = \max \{ CL_f(k), CL_{A,B}(k) \} \]  \hspace{1cm} (12)

Let us clarify the definition of the final collapse load and the propagation of failure:

- If the first analysis identifies a column buckling at a collapse load \( CL_f(k) \), while the second analysis leads to another buckling failure at a higher collapse load \( CL_{A,B}(k) \), then the set of two analyses form a damage propagation phenomenon that is finalized at the higher load of the second analysis. Essentially, the two buckling failures are initiated one after the other (the second one requires an even higher load to occur).
- If the first analysis identifies a column buckling at a collapse load \( CL_f(k) \), while the second analysis leads to another buckling failure at a lower collapse load \( CL_{A,B}(k) \), then the second buckling failure is immediate to the first one, since the first loading level is high enough to cause both buckling failures simultaneously.
- If the first analysis identifies a column buckling at a collapse load \( CL_f(k) \), while the second analysis leads to a yielding-type beam failure at a higher collapse load \( CL_{A,B}(k) \), then the set of two analyses form a damage propagation phenomenon that is finalized at the higher load of the second analysis.
- If the first analysis identifies a column buckling at a collapse load \( CL_f(k) \), while the second analysis leads to a yielding-type beam failure at a lower collapse load \( CL_{A,B}(k) \), then the second yielding-type failure is immediate to the first one, since the first loading level is high enough to cause the second failure simultaneously.
If the first analysis identifies a yielding-type beam failure, the progressive collapse phenomenon is initiated and the progressive collapse capacity is defined as \( CL_f(k) \).

It must be mentioned here that the method of subsequent column removals after a member has failed is generally accepted by the research community in the progressive collapse field. Although a more sophisticated approach would be to examine the post-buckling behavior of the failed elements, it is not within the scope of the current research work to account for that kind of phenomena. The propagation of damage procedure for the assessment of the final progressive collapse capacity is schematically shown in Fig. 2.

4. Numerical application

4.1. Description of the finite element model

The 15-floor steel frame used for the analyses is shown in Fig. 1. It is a typical steel moment resisting frame designed according to the Eurocodes ([19,20]), with floor height equal to 3m and bay width equal to 5m. The tributary load areas are calculated using a selected distance between consecutive frames of 7m, which is a typical value for steel frames. Indicatively, column sections vary from HEB650 to HEB200, while beam sections range from IPE450 to IPE550. Full section selection, load combinations and design considerations of the frame can be found in [18]. The structural steel components follow the non-linear material characteristics of structural steel S235, described by an elastic–plastic material model with bilinear stress–strain behavior. The yield stress of the material is 235 MPa and the ultimate stress is 360 MPa at strain 23%, with isotropic strain hardening.

The frame is simulated using the commercial FEM code ABAQUS Simulia ([21]), using beam B22 elements to simulate both beams and columns. The mesh representing the finite element model is studied to be sufficiently fine in the areas of interest to accurately capture the structural behavior of the system. Apart from the elements used to model the connections, each column comprises 5 elements while each beam comprises 10 elements, leading to a total number of 1262 elements and 5956 nodes for each model. The rigid body constraint offered in ABAQUS is utilized to model all beam-to-column connections, by constraining the relative motion of regional elements around the connections. All base nodes of first floor columns are considered pinned.

The proposed partial distributed damage method is applied on the 15-story frame for all the aforementioned damage scenarios, leading to a total number of 165 progressive collapse scenarios. Each damage scenario from \( DS_f(2) \) to \( DS_f(10) \) includes 2 consecutive analyses, except damage scenarios \( DS_f(1) \) and \( DS_f(11) \) that are described by a single static push-down analysis. All analyses are set to account for both material

Fig. 2. Damage propagation procedure for the assessment of the final progressive collapse capacity, according to the new partial distributed damage method (PDDM).
and geometric nonlinearities which is shown to be the only appropriate method to detect loss-of-stability phenomena and thus to identify the correct collapse mechanisms ([8], [9]).

4.2. Analysis results

4.2.1. Damage scenarios for the first floor

Damage Scenario DS1(1), (δ_B,1 = 1; δ_B,1 = 0). The first analysis involves the removal of the corner column of the first floor, A1. Fig. 3a and b include the horizontal (lateral) displacement at mid-height of column B1 and the axial force of column B1, with respect to the vertical load applied on the beams of the structure, while Fig. 3c presents the deformed shape of the structure. All graphs illustrate the buckling failure of column B1, while the beams exhibit little plastification and mostly remain within the elastic range. Both the deformed shape and the displacement plot show a rapid increase in the horizontal displacement after the application of the vertical load of 56.4 kN/m, while the axial force increases linearly until the column reaches its axial capacity for the same loading level. After this point it is clear that the member cannot undertake any additional axial force, therefore the collapse load is \( CL_{1}(1) = 56.4 \text{kN/m} \).

An additional analysis is not required in this case, since the failure of the second column certainly leads to the collapse of the structure. The kind of buckling occurring in this case is nonlinear inelastic buckling, since the axial force cannot exceed the yield capacity of \( A_{f} \), where \( A \) is the cross sectional area and \( f_{y} \) is the material yield stress. In addition, the Euler buckling capacity of column B1 is much higher than its yield capacity, meaning that this column with the prescribed geometry and section properties is prone to nonlinear inelastic buckling rather than linear elastic Euler buckling (which is the case for slender columns). Detailed description of different buckling failure modes can be found in [9]. It must be mentioned at this point that the critical failure mode of all columns examined for this structure is nonlinear inelastic buckling, as opposed to Euler buckling of the weak or strong axis.

Damage Scenario DS1(2), (δ_A,1 = 0.9; δ_B,1 = 0.1). The next analysis performed is the partial damage distribution scenario with \( \delta_{A,1} = 0.9 \) and \( \delta_{B,1} = 0.1 \). This analysis leads to the buckling failure of column A1 at the vertical load of \( CL_{1}(2) = 20.7 \text{kN/m} \) (illustrated by the dashed line in the plot of the axial force in Fig. 4a). After the removal of column A1, the damage propagation phenomenon ends with the buckling failure of B1 at the collapse load of \( CL_{A,1}(2) = 51.0 \text{kN/m} \) (Fig. 4b), which defines the progressive collapse capacity for this damage scenario \( CL_{1}(2) = \max \{ CL_{A,1}(2), CL_{B,1}(2) \} = 51.0 \text{kN/m} \). This collapse load is lower than the \( CL_{1}(1) = 56.4 \text{kN/m} \) collapse load of the full column A1 removal scenario. A first important finding is the decrease of the capacity when applying a partial distributed damage scenario that includes damage distribution very close to the initial single column removal assumption; only 10% of the damage is ‘transferred’ from column A1 to B1 and the collapse load is yet decreased also almost by 10%.

Damage Scenarios DS1(3) - DS1(5) and DS1(11). The same trend is observed when the other partial distributed damage scenarios are investigated, reaching the collapse loads of \( CL_{1}(3) = 45.4 \text{kN/m} \) and \( CL_{1}(4) = 51.5 \text{kN/m} \) for the cases of \( \delta_{B,1} = 0.8; \delta_{B,1} = 0.2 \) and \( \delta_{B,1} = 0.7; \delta_{B,1} = 0.3 \), respectively. Interestingly, the collapse load for both these cases is lower than \( CL_{1}(1) = 56.4 \text{kN/m} \), which is the collapse load of the complete A1 column removal scenario.

However, for the DS1(5) (\( \delta_{A,1} = 0.6 \) and \( \delta_{B,1} = 0.4 \)) damage scenario, the collapse mode of the structure is altered. Instead of the buckling failure of A1 (which has the highest level of damage), column B1 is the one that actually buckles first at \( CL_{B,1}(5) = 61.3 \text{kN/m} \). The collapse load of the second analysis which results in the buckling of A1 is now \( CL_{A,1}(5) = 36.8 \text{kN/m} \), which is lower than the initial 61.3kN/m. Therefore, the loading level of 61.3kN/m is large enough to cause the buckling failure of both columns A1 and B1 and the final collapse load for this case is thus defined as \( CL_{1}(5) = 61.3 \text{kN/m} \).

At this particular point, it is interesting to examine the complete loss of column B1 which is described by DS1(11). Fig. 5c depicts the deformed shape of the structure, which shows the buckled shape of column C1. When plotting the axial force of that column with respect to the vertical applied load (Fig. 5b), the axial capacity is reached at \( CL_{1}(11) = 69.0 \text{kN/m} \), where at the same load the column mid-height horizontal displacement begins to increase rapidly (Fig. 5a). The \( CL_{1}(5) = 61.3 \text{kN/m} \) collapse load is higher than \( CL_{1}(1) = 56.4 \text{kN/m} \) but lower than \( CL_{1}(11) = 69 \text{kN/m} \). Moreover, since the first element that buckles in DS1(5) is B1, it is considered more reasonable to compare the

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**Fig. 3.** Damage scenario DS1(1): Full column A1 removal of the first floor. (a) Horizontal (lateral) displacement \( \delta \) of the mid-height node of the adjacent-to-the-removal column B1, (b) axial force with respect to the vertical load and (c) deformed shape.
DS(5) case with the results of DS(11) than the results of DS(1), even if the damage level initially introduced to A1 is higher than B1.

Damage scenarios DS(3) and DS(4) lead to lower collapse loads than the corresponding complete column removal scenarios (either DS(1) or DS(11)).

Damage Scenarios DS(6) - DS(10). Similarly to DS(5), all partial damage scenarios DS(6) - DS(10) are compared to the B1 full column removal scenario, DS(11). The first analysis of DS(6) ($\delta_{i,j}^{1,1} = 0.5$ and $\delta_{i,j}^{2,1} = 0.5$) leads to the buckling failure of column B1 at $CL_{1}(6) = 52.9\text{kN/m}$. The results of the second analysis show buckling of A1 at $CL_{2,1}(6) = 45.2\text{kN/m}$. Therefore, the collapse load is the highest of the two, i.e. $CL_{1}(6) = CL_{1,1}(6) = 52.9\text{kN/m}$, which is remarkably lower (23.3%) than $CL_{1}(11) = 69.0\text{kN/m}$.

Furthermore, the scenarios DS(7) and DS(8) show similar behavior, for which in the first analysis column B1 fails and the second analysis reveals the buckling failure of column A1 at $CL_{1}(7) = CL_{1,1}(7) = 55.0\text{kN/m}$ and $CL_{1}(8) = CL_{1,1}(8) = 62.7\text{kN/m}$. Lastly, DS(9) and DS(10) lead to a final collapse load of $CL_{1}(9) = 66.9\text{kN/m}$ and $CL_{1}(10) = 67.6\text{kN/m}$, respectively, due to the inelastic buckling failure of column C1, resembling the collapse mode of the DS(11).

The major finding of this analysis is the significant decrease of the collapse load in almost all partial distributed damage scenarios, showing that the notion of complete column removal is a less conservative approach than the proposed partial distributed damage method.
(PDDM). In addition, there is a noticeable change in the observed collapse mechanisms. The final buckling failure can occur either at columns A1, B1 or C1 interchangeably, depending on the specific partial damage distribution applied in the model.

### 4.2.2. Partial damage distribution at 14th floor

**Damage Scenarios DS_{14}(1), (\delta^{A, 14} = 1; \delta^{B, 14} = 0) and DS_{14}(11), (\delta^{A, 14} = 0; \delta^{B, 14} = 1).**

The next discussion of results will present the partial damage distribution study at a higher floor, where the dominant failure is the extensive yielding of beams instead of the buckling collapse mode of a column. The reference point to examine and compare all the partial distributed damage scenarios will be the two column removal analyses of A14 and B14, corresponding to DS_{14}(1) and DS_{14}(11) respectively.

For the scenario DS_{14}(1), the beam edges at floors 14 and 15 reach the plastic bending moment limit at the vertical load of \( CL_{14}(1) = 62.5 kN/m \) (Fig. 6b). At the same load, the vertical displacement of the node above the removal has nearly reached the value of 10 cm, while after this point it continues to grow at a higher rate (Fig. 6a). Finally, Fig. 6c displays the deformed shape of the structure which obviously indicates the yielding-type collapse mechanism initiated by the excessive deformation and plastification of the beams above the removal.

The analysis of column B14 being removed from the model leads to very similar results. As depicted in Fig. 7, plastic hinges are formed at all beams edges of both bays AB and BC, for a collapse load of \( CL_{14}(11) = 75.5 kN/m \).

**Damage Scenario DS_{14}(2), (\delta^{A, 14} = 0.9; \delta^{B, 14} = 0.1).** Following the same configuration, the next analysis involves the application of 90% damage level for column A14, while column B14 is damaged only 10%. For this scenario, the inelastic buckling of column A14 occurs at a relatively low level of external load. After the removal of this buckled element, the next analysis leads to the yielding-type collapse mechanism of beams at the same vertical load of \( CL_{14}(2) = CL_{14}(2) = 62 kN/m \). The collapse load in this case is practically the same as the DS_{14}(1) full A14 column removal scenario.

**Damage Scenario DS_{14}(3), (\delta^{A, 14} = 0.8; \delta^{B, 14} = 0.2).** The next analysis involves the application of 80% damage level for column A14, while column B14 is damaged by 20%. For this scenario, the inelastic buckling of column A14 occurs at a higher collapse load than DS_{14}(2) and the second analysis reveals again the yielding-type collapse of the beams above the damaged area, after the removal of the buckled elements. The collapse load in this case is governed by the buckling of column A14 and it is higher than the collapse loads of DS_{14}(1) and DS_{14}(2).

**Damage Scenarios DS_{14}(3) - DS_{14}(6).** The next three partial damage scenarios show a significant change in the overall behavior of the structure. Interestingly, when applying 70%, 60% or 50% damage level for column A14 (equivalently 30%, 40% or 50% damage level for column B14) the results of all three analyses show the buckling failure of column C4 at the 4th floor for practically the same vertical load of \( CL_{14}(1) = CL_{14}(5) = CL_{14}(6) = 89.3 kN/m \). Fig. 8a depicts the plot of the axial forces of both columns C4 and A14 for Analysis 1 of damage scenario DS_{14}(4), where it is obvious that column C4 reaches its axial capacity before any failure occurs at A14. When removing the buckled column C4, all three damage scenarios lead to the buckling failure of column B4 at a lower loading level of \( CL_{14}(14) = CL_{14}(5) = CL_{14}(6) = 64.1 kN/m \) (Fig. 8b for DS_{14}(4)). Consequently, the vertical load of 89.3 kN/m is large enough to cause the buckling failure of both C4 and B4, therefore \( CL_{14}(4) = CL_{14}(5) = CL_{14}(6) = 89.3 kN/m \).

Although this load is higher than for the DS_{14}(1) full A14 column removal scenario (even higher than the DS_{14}(11) B14 removal scenario), the most important finding from this scenario is the tremendous change in the collapse mechanism; instead of the yielding-type collapse mechanism of the beams, the structure fails due to loss of stability of columns at a much lower floor. The assessment of the progressive collapse vulnerability of this structure is highly affected by the specific partial damage distribution scenario applied to the members of the frame. Essentially, the yielding-type of progressive collapse is avoided in the vicinity of the introduction of damage when the damage is distributed almost evenly in two columns. The damaged columns are able to prevent the yielding type of collapse and therefore the collapse is triggered in a completely different area of the frame.

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**Fig. 6.** Damage scenario DS_{14}(1): Full column A1 removal of the 14th floor. (a) Vertical displacement \( \delta \) of the node above the removal, (b) bending moment at the beam edges of the 14th and 15th floor with respect to the the vertical load and (c) deformed shape.
Damage Scenarios DS14(7) - DS14(10). Lastly, the damage propagation for DS14(7) - DS14(10) can be described by the buckling failure of $B_{14}$ and then, after the removal of $B_{14}$, by the yielding failure of the beams at the 14th and 15th floor. Except $DS_{14}(7)$, the collapse load of which is $CL_{14}(7) = 79.1\text{kN/m}$, all the other collapse loads are slightly lower than the $CL_{14}(11) = 75.5\text{kN/m}$ collapse load of the full $B_{14}$ column removal scenario.

Overall, the response of the structure is only mildly affected under the event of partial distributed damage at a higher floor, where the dominant collapse mechanism is the beam yielding-type. However, three ($DS_{14}(4) - DS_{14}(6)$) out of nine ($DS_{14}(2) - DS_{14}(10)$) partial distributed damage scenarios reveal that the expected critical collapse mode of beam yielding can be replaced by the inelastic buckling failure of columns located at lower floors, rendering a parametric partial

**Fig. 7.** Damage scenario $DS_{14}(11)$: Full column $B_{1}$ removal of the 14th floor. (a) Vertical displacement $\delta$ of the node above the removal, (b) bending moment at the beam edges of the 14th and 15th floor with respect to the the vertical load and (c) deformed shape.

**Fig. 8.** Damage scenario $DS_{14}(4)$: (a) Axial force of column $A_{14}$ and $C_{4}$ for Analysis I, (b) axial force of column $B_{4}$ for Analysis II with respect to the the vertical load and (c) collapse sequence.
damage distribution study critical for the correct evaluation of the structural integrity of a system in case of a progressive collapse scenario.

4.2.3. Synopsis of analysis results

Fig. 9 provides the synopsis of all collapse loads for all damage scenarios for the 15 floors of the frame, divided by the collapse load of the single column removal case. The upper part of the graph, Fig. 9a, illustrates the ratio \( CL_f(k)/CL_f(1) \), while the lower part, Fig. 9b, depicts the ratio \( CL_f(k)/CL_f(11) \). The complete column removal at gridlines A or B (DSf(1) or DSf(11), denoted by the red line) constitutes the reference alternate load path scenario to which all other scenarios are compared, as it represents the current state-of-the-art progressive collapse approach of the Alternate Path Method (APM). Both graphs show that many damage scenarios included in the proposed Partial Distributed Damage method reach lower collapse loads than DSf(1) and especially DSf(11), since they are located below the red line. This trend is less apparent for the upper floors of the structure, where the collapse loads are only mildly affected by the introduction of partial distributed damage, due to the yielding-type nature of collapse mechanisms observed. The reduction of the collapse load for many damage scenarios is remarkable though for the rest of the floors, where buckling failure is dominant, resulting in \( CL_f(k)/CL_f(11) \) ratios as low as 0.65 (up to 35% reduction in the collapse load).

Another interesting finding from Fig. 9 illustrated by the red indicators at the bottom of each graph is that for all floors except at the last two floors, the most critical scenario is DSf(3) (\( \delta_f^A = 0.8 \) and \( \delta_f^B = 0.2 \)), as it leads to the largest decrease in the collapse load. In contrast, for the 15th floor the most critical scenario in terms of lowest collapse load is the full column A15 (DS15(1)) removal. This finding, however, depends on the design characteristics of the analyzed frame and cannot generally be extended to other building configurations.

4.2.4. Discrepancy of progressive collapse capacity between the new partial distributed damage method and the alternate load path method

The most important finding of this work is that the capacity of the structure is much lower when the partial distributed damage method is considered in comparison to the simplistic notional column removal approach of the state-of-the-art APM. It is therefore considered highly unconservative for many cases to perform the alternate load path method for progressive collapse analysis of steel frames.
The discrepancy between the new method and the alternate load path method is highlighted in Table 1 which includes discrepancy measures for all the analyses conducted in this work. The first row of the table includes the collapse loads which correspond to the full column removal at A for all the different floors, while the last row of the table includes the collapse loads which correspond to the full column removal at B for all the different floors respectively. All these values are in kN/m and refer to vertical downward loading on the beams of the structure. The rest of the rows in the table include a measure of the discrepancy of the two methods which can be defined through the following simple equation:

\[
D_{f}(k) = \frac{CL_{f}(k) - CL_{f}(1)}{CL_{f}(1)} \quad \text{for } k = 2, 3, 4, 5
\]  
(13)

\[
d_{f}(k) = \min \left\{ \frac{CL_{f}(k) - CL_{f}(1)}{CL_{f}(1)}, \frac{CL_{f}(k) - CL_{f}(11)}{CL_{f}(11)} \right\} \quad \text{for } k = 6
\]  
(14)

\[
d_{f}(k) = \frac{CL_{f}(k) - CL_{f}(11)}{CL_{f}(11)} \quad \text{for } k = 7, 8, 9, 10
\]  
(15)

where \(d_{f}(k)\) is the discrepancy measure between the two methods. The scheme of Eqs. (13)–(15) is defined so that the results of the new method for \(k = 2, 3, 4, 5\) are compared to the full column removal at A and the results of the new method for \(k = 7, 8, 9, 10\) are compared to the full column removal at B. It is considered more appropriate to compare the full column removal scenario at A, to the partial distributed damage scenarios which include more damage at column A, than at column B. Similarly, the full column removal at B results are compared to the partial distributed damage scenarios which include more damage at column B, than at column A. For the symmetric case of \(k = 6\), the minimum of the two comparisons is adopted.

The red colored cells in Table 1 represent the cases for which the collapse load of the new method is lower (and more critical) than the collapse load of the alternate load path method. A first finding from the table is that from the 135 analyses with partial distributed damage, 92 of them lead to more critical collapse loads than the alternate load path method, showing therefore the importance of the results.

Secondly, the decrease in the collapse load in many cases is considered very high. For example for the first floor this decrease reaches \(D_{f}(6) = -23.3\%\) which is considered very high. Other outstanding cases are \(D_{f}(4) = -21.8\%\), \(D_{f}(7) = -20.3\%\), \(D_{f}(8) = -19.6\%\), \(D_{f}(9) = -19.5\%\), \(D_{f}(10) = -19.5\%\) and many others. This finding is considered the most important one of this work, since it shows that for these cases, the alternate load path method is highly unconservative and clearly overpredicts the progressive collapse capacity of the structure.

### Table 1

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<thead>
<tr>
<th>Floor</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
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<tbody>
<tr>
<td>(D_{f}(1))</td>
<td>56.4</td>
<td>56.4</td>
<td>54.6</td>
<td>53.1</td>
<td>55.9</td>
<td>55.8</td>
<td>53.6</td>
<td>57.7</td>
<td>62.0</td>
<td>59.7</td>
<td>66.5</td>
<td>65.3</td>
<td>61.8</td>
<td>62.5</td>
<td>45.0</td>
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<tr>
<td>(D_{f}(2))</td>
<td>-9.6%</td>
<td>-3.7%</td>
<td>2.0%</td>
<td>-9.7%</td>
<td>-7.0%</td>
<td>0.6%</td>
<td>-9.7%</td>
<td>-6.3%</td>
<td>-1.9%</td>
<td>-9.9%</td>
<td>-4.7%</td>
<td>5.8%</td>
<td>-1.4%</td>
<td>-0.8%</td>
<td>96.3%</td>
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<tr>
<td>(D_{f}(3))</td>
<td>-19.5%</td>
<td>-16.1%</td>
<td>-2.9%</td>
<td>-19.6%</td>
<td>-17.0%</td>
<td>-9.4%</td>
<td>-19.3%</td>
<td>-16.5%</td>
<td>-12.3%</td>
<td>-19.5%</td>
<td>-14.5%</td>
<td>-1.8%</td>
<td>-12.2%</td>
<td>20.5%</td>
<td>98.7%</td>
</tr>
<tr>
<td>(D_{f}(4))</td>
<td>-8.7%</td>
<td>-5.0%</td>
<td>8.4%</td>
<td>-10.2%</td>
<td>-7.2%</td>
<td>2.3%</td>
<td>-8.4%</td>
<td>-4.1%</td>
<td>2.9%</td>
<td>-3.6%</td>
<td>4.8%</td>
<td>38.9%</td>
<td>17.4%</td>
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<tr>
<td>(D_{f}(5))</td>
<td>8.7%</td>
<td>16.1%</td>
<td>30.2%</td>
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<td>13.0%</td>
<td>23.7%</td>
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<td>-11.8%</td>
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<tr>
<td>(D_{f}(7))</td>
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<td>-8.4%</td>
<td>-4.0%</td>
<td>-19.0%</td>
<td>-14.2%</td>
<td>-4.1%</td>
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<td>26.8%</td>
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<td>-1.2%</td>
<td>-1.7%</td>
<td>-2.0%</td>
<td>-1.4%</td>
<td>-1.3%</td>
<td>-1.5%</td>
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<td>26.8%</td>
</tr>
<tr>
<td>(D_{f}(9))</td>
<td>-3.0%</td>
<td>-1.0%</td>
<td>0.0%</td>
<td>-1.1%</td>
<td>0.1%</td>
<td>-1.1%</td>
<td>-1.0%</td>
<td>-0.7%</td>
<td>-0.8%</td>
<td>-1.0%</td>
<td>-0.7%</td>
<td>-0.8%</td>
<td>-2.1%</td>
<td>10.2%</td>
<td></td>
</tr>
<tr>
<td>(D_{f}(10))</td>
<td>-2.0%</td>
<td>1.0%</td>
<td>0.0%</td>
<td>-0.3%</td>
<td>0.0%</td>
<td>-0.3%</td>
<td>-0.5%</td>
<td>-0.4%</td>
<td>-0.4%</td>
<td>-0.3%</td>
<td>0.3%</td>
<td>-0.3%</td>
<td>-0.4%</td>
<td>-1.1%</td>
<td>-1.0%</td>
</tr>
<tr>
<td>(D_{f}(11))</td>
<td>69.0</td>
<td>66.9</td>
<td>66.4</td>
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<td>65.5</td>
<td>70.2</td>
<td>72.2</td>
<td>69.5</td>
<td>75.5</td>
<td>70.4</td>
</tr>
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</table>

4.2.5. Discrepancy of progressive collapse mechanisms between the new partial distributed damage method and the alternate load path method

The second important finding of this paper is the change in the observed collapse mechanisms in case of partial damage distribution. Fig. 10 demonstrates the damage propagation of all the damage scenarios from 1st to 5th floor performed in this paper, along with some selected damage scenario deformed shapes for 1st and 3rd floor. Each bar represents the collapse load of one analysis (20 analyses per floor, 300 analyses in total for all 15 floors). The sequence of analyses is shown by consecutive bars, with the red bars illustrating the analyses that yield the higher collapse load and are thus considered the dominant analyses. On top of each bar, the failing element at the end of each analysis is depicted rather than the removed element in the beginning of each analysis, to avoid confusion. The yielding-type collapse mechanisms are denoted by the letter Y (yielding).

The results show the alteration of the failure elements in case of partial distributed damage, as opposed to full column removal scenarios \(D_{f}(1)\) or \(D_{f}(11)\). In case of buckling modes of collapse, the element that buckles is not always the adjacent column or the column that the corresponding full removal scenario is indicating. For example, the current state-of-the-art Alternate Load Path Method (damage scenario \(D_{f}(11)\)) predicts the buckling failure of column C1 after the complete column B1 removal. However, according to the proposed Partial Distributed Damage Method, the deformed shape of Analysis II when considering damage scenario \(D_{f}(3)\) reveals the buckling failure of column B1 (after removing the previously buckled column A1 in Analysis I). What is also remarkable is the reduction of the collapse load according to PDDM which in this case is 34%. Similarly, at the 3rd floor, the
state-of-the-art APM predicts buckling of column C4 (damage scenario $DS_3(1)$), whereas the finally buckling element according to the proposed PDDM (damage scenario $DS_3(3)$) is column B3. Therefore, the position of the buckling element highly depends on the damage scenario examined in each case, so an integrated progressive collapse vulnerability analysis must take into account all damage scenarios and thus apply any design or retrofit technique so as series of vulnerable elements within the structural system.

Fig. 11 depicts the damage propagation of all damage scenarios from 6th to 10th floor, along with some selected damage scenario deformed shapes for 6th and 9th floor. In the same manner as the previous floors, the observed collapse mechanisms highly depend on specific damage scenarios investigated. For instance, at 6th floor, the state-of-the-art APM (damage scenario $DS_6(11)$) predicts buckling failure of column C7, while damage scenario $DS_6(3)$ predicts buckling failure of column B6 according to the proposed PDDM. Additionally, when observing the damage scenarios at 9th floor, the proposed PDDM for $DS_6(3)$ results in the buckling failure of column B9, in contrast to the state-of-the-art APM that results in the buckling failure of column B10.

Finally, Fig. 12 depicts the damage propagation of all damage scenarios for the last five floors of the building, where the dominant type of failure is beam yielding. The change in the collapse mechanisms is even more significant for those floors. For example, at the 12th floor, the current state-of-the-art method that includes full column B12 removal (damage scenario $DS_{12}(11)$) results in beam yielding failure of the beams spanning above the removal location; a failure type that is by nature ductile and thus allows for safety warning by appearance of significant plastic deformation. Conversely, when applying a more realistic damage distribution scenario such as $DS_{12}(5)$, the collapse mode that the proposed method predicts is tremendously different and includes loss-of-stability of a column at a much lower floor, even outside the damaged area, i.e. column B4. In the same environment, similar conclusions can be drawn when comparing the state-of-the-art method to the proposed method at floors 13th - 15th. As illustrated by the deformed shapes of the 14th floor, full column A14 removal ($DS_{14}(1)$) leads to yielding-type failure of the above beams, while damage scenario $DS_{14}(5)$ included in the proposed PDDM leads to buckling failure of C4 column, as previously discussed in Section 4.2.2.

Hence, it is clear that introducing a more widespread and realistic damage distribution that affects more than one column may not lead to the predicted progressive collapse of the flexural elements above the removal. This happens mainly due to the support that is provided by the partial damaged column; in the APM the removed column is absent and therefore there is a complete loss of support. This work shows that even when a small part of the column remains in the structure, it could prevent the yielding type of collapse. A parametric investigation about how damage is distributed into the system and how it affects the structural response, like the one proposed in the Partial Distributed Damage Method, is therefore crucial in order to detect the most vulnerable elements to loss-of-stability and apply the necessary precautions in their design.

5. Conclusions

Recent studies have questioned the validity of the simplistic current state-of-the-art concept of key element removal for the
performance of progressive collapse analysis. Although the alternate path method is simple enough for designers and practitioners to apply, it is far from being realistic. The reasoning behind this questioning is not only because the likelihood of occurrence of an extreme event that is able to cause the complete failure of only one element is extremely small, but also because in the case of an extreme event, the affected damage area will include more than one elements. This paper introduces a new method for progressive collapse analysis defining partial distribution damage scenarios.

The method is applied on a 15-floor steel frame showing that the column removal concept can be unconservative. The most important findings from the present work are the following:

- For the partial damage scenarios located at the lower floors of the frame (up to the 12th floor), where the dominant collapse mode is the column buckling failure, the collapse loads can be significantly less than the collapse loads of the corresponding full column removal scenarios. As a result, the application of the alternate path method is less conservative, since the structural behavior of the frame is seriously affected by the distribution of damage.
- For the upper floors of the frame (above 12th), where the dominant failure is the yielding-induced collapse mechanism due to beam failure, the partial distributed damage method (PDDM) leads to similar collapse loads with the complete column loss approach. Therefore, partial damage distribution only mildly affects the response of the structure in this case.
- The introduction of distributed damage in the system significantly changes the observed collapse modes of the structure. More specifically, for floors where the dominant failure is buckling, distributed damage can alternate the location of the failing element. For floors where the alternate path method leads to a yielding-type beam collapse mechanisms near the damaged area, some partial damage scenarios lead to the avoidance of the yielding-type collapse mode and the triggering failure is the buckling failure of elements located outside the damaged area, in much lower floors.
- For all cases examined except for the top two floors of the analyzed building, the partial damage scenario that leads to the lower collapse load is always $DS_f(3)$ ($\delta_j^{A,f} = 0.8$ and $\delta_j^{B,f} = 0.2$).

The conclusions of this work clearly show that the widely used and unrealistic column removal concept of the Alternate Path Method can be less conservative and predict collapse mechanisms and collapse loads which are not the most critical. For this reason, a reliable study to evaluate the progressive collapse capacity of a structure must include a partial damage distribution study as well, through the proposed partial distributed damage method (PDDM).

The next important task in order to extend the findings of this work is the application of the same analysis procedure to a 3D model. However, the collapse mechanisms and thus the main conclusions are expected to remain the same, since the columns examined herein are governed by inelastic nonlinear buckling (able to be captured in a 2D analysis configuration) rather than elastic Euler buckling of the weak axis that can only be detected by a 3D analysis configuration. Finally, another important task is to take into account the post-buckling behavior of the buckled elements and examine the extent to which the collapse loads and mechanisms will be affected.

![Fig. 11. Damage propagation and analyses sequence for all damage scenarios from 6th to 10th floor, with respect to the collapse load, and selected damage scenario deformed shapes for 6th and 9th floor.](image-url)
Acknowledgements

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References