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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_j 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.

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

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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 δ_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 δ_j

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

$$\delta_j = \frac{A - A^{'}}{A}, 0 \leq \delta_j \leq 1 \tag{1}$$

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:

$$if\ \delta_j = 0 \Rightarrow No\ damage \tag{2}$$

$$if\ \delta_j \in (0, 1) \Rightarrow Partial - Damaged\ State \tag{3}$$

$$if\ \delta_j = 1 \Rightarrow Fully - Damaged\ State \tag{4}$$

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

$$\sigma A = \sigma^{'} A^{'} \tag{5}$$

where σ is the stress of the pristine element, $\sigma^{'}$ the effective stress of the damaged element and therefore:

$$\sigma^{'} = \frac{\sigma}{1 - \delta_j} \tag{6}$$

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

$$\{\varepsilon\} = \left\{E^{-1}\right\}\{\sigma\}^{'} \tag{7}$$

$$\{\varepsilon\} = \left\{E^{-1}\right\}^{'}\{\sigma\} \tag{8}$$

where E is the pristine Young's modulus, $E^{'}$ is the effective Young's modulus and therefore from (5), (6) and (7) we have:

$$\{E\}^{'} = \{E\}(1 - \delta_j) \tag{9}$$

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:

$$DS_f(k) : f \in \{1, 2, ..., n\}\ and\ k \in \{1, 2, ..., 11\} \tag{10}$$

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 $\delta_j^{A,f} = 1$ (full damage, meaning column removal) for all the elements of the corner column A of floor f and $\delta_j^{B,f} = 0$ (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):

$DS_f(1).\delta_j^{A,f} = 1; \quad \delta_j^{B,f} = 0$
$DS_f(2).\delta_j^{A,f} = 0.9; \quad \delta_j^{B,f} = 0.1$
$DS_f(3).\delta_j^{A,f} = 0.8; \quad \delta_j^{B,f} = 0.2$
$DS_f(4).\delta_j^{A,f} = 0.7; \quad \delta_j^{B,f} = 0.3$
$DS_f(5).\delta_j^{A,f} = 0.6; \quad \delta_j^{B,f} = 0.4$
$DS_f(6).\delta_j^{A,f} = 0.5; \quad \delta_j^{B,f} = 0.5$
$DS_f(7).\delta_j^{A,f} = 0.4; \quad \delta_j^{B,f} = 0.6$
$DS_f(8).\delta_j^{A,f} = 0.3; \quad \delta_j^{B,f} = 0.7$
$DS_f(9).\delta_j^{A,f} = 0.2; \quad \delta_j^{B,f} = 0.8$
$DS_f(10).\delta_j^{A,f} = 0.1; \quad \delta_j^{B,f} = 0.9$
$DS_f(11).\delta_j^{A,f} = 0; \quad \delta_j^{B,f} = 1$