GLOBAL STRUCTURAL BEHAVIOUR OF A BUILDING FRAME FURTHER TO ITS PARTIAL DESTRUCTION BY COLUMN LOSS

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INTRODUCTION

In 1969, a catastrophic event occurred in UK at the Ronal Point residential building where the progressive collapse of the building, initiated by a localized gas explosion, was observed. Further to this event, some codes and standards, and in particular the British Standards, impose to the designer to ensure the structural integrity of building in order to avoid the progressive collapse of the latter further to an exceptional action.

In fact, the progressive collapse of a building is the result of a change in the structural system and of the load path associated to the loss of one or more main structural members. In other words, the existing load path is suddenly modified and the loads supported by the building are reported to the foundation through an alternative load path. Accordingly, the remaining structural members are overloaded by additional loads; the latter have to be able to support these loads and to possess a sufficient ductility.

In the last decade, researches on this topic were initiated at the Argenco Department of Liège University *[1], [2], [3], [4]*. The present paper reflects part of these activities; in particular, some investigations dedicated to the investigation of the behaviour of a frame further to a column loss are presented. In another article presented by J.F Demonceau, H.N.N. Luu and J.P Jaspart within the present conference *[3]* is devoted to the investigation of the behaviour of the frame when significant membranar forces developed within the structure. In particular, in this paper, a simplified substructure which is able to simulate the behaviour of a structure further to a column loss is defined. The present paper presents the analytical methods requested to define the parameters influencing the response of the simplified substructure.

1 GENERAL CONCEPT

As mentioned in *[3]*, a structure is losing a column can be divided in two main parts, as illustrated in *Fig. 1.a.*:

- the directly affected part which represents the part of the building which is directly affected by the loss of the column, i.e. the beams, the columns and the beam-to-column joints which are just above the loss column and;
- the indirectly affected part which represents the part of the building which is affected by the loads developing within the directly affected part and which influences the development of these loads.

Also, as shown in *Fig. 1.b.*, the curve representing the evolution of the normal load N_{lo} in the loss column AB (see *Fig. 1.a*) according to the vertical displacement Δ_a at point A can be divided in three phases:

- From point (1) to (2) (Phase 1), the design loads are progressively applied, i.e the "conventional" loading is applied to the structure; so, N_{lo} progressively decreases (N_{lo} becomes negative as the column "AB" is subjected to compression) while Δ_A can be assumed to be equal to 0 during this phase (in reality, there is a small vertical displacement at point A associated to the compression of the columns below point "A"). It is assumed that no yielding appears in the investigated frame during this phase, i.e. the frame remains fully elastic.
- From point (2) to (5), the column is progressively removed. Indeed, from point (2), the compression in column "AB" N_{lo} is decreasing until reaching a value equal to 0 at point (5) where the column can be considered as fully destroyed. So, in this zone, the absolute value of

N_{lo} is progressively decreasing while the value of Δ_A is increasing. This part of the graph is divided in two phases as represented in *Fig. 2*:

- From point (2) to (4) (Phase 2): during this phase, the directly affected part passes from a fully elastic behaviour (from point (2) to (3)) to a plastic mechanism. At point (3), first plastic hinges are appearing in the directly affected part.
- From point (4) to (5) (Phase 3): during this phase, high deformations of the directly affected part are observed and second order effects play an important role. In particular, significant catenary actions are developing in the bottom beams of the directly affected part.

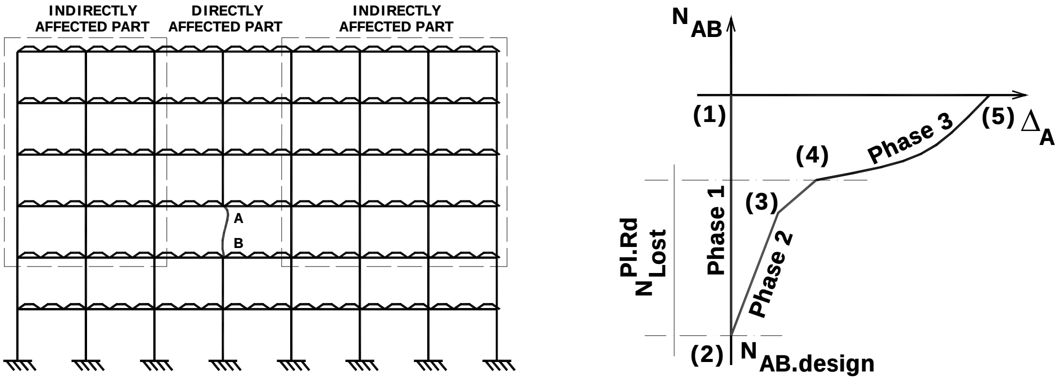

Fig. 1. a. Representation of a frame losing a column; *b.* Evolution of the axial load in the loss column AB

In *[3]*, it is shown that it is possible to predict the behaviour of a structure during Phase 3 with a simplified substructure modelling presented in *Fig. 2*. To define this simplified substructure, some parameters have to be defined; in particular, the characteristics of the horizontal spring K and F_{Rd}, simulating the behaviour of the indirectly affected part (see *Fig. 1.a.*) subjected to the membranar forces have to be determined. The analytical models developed in *[2]* to predict the latter are presented in the following sections

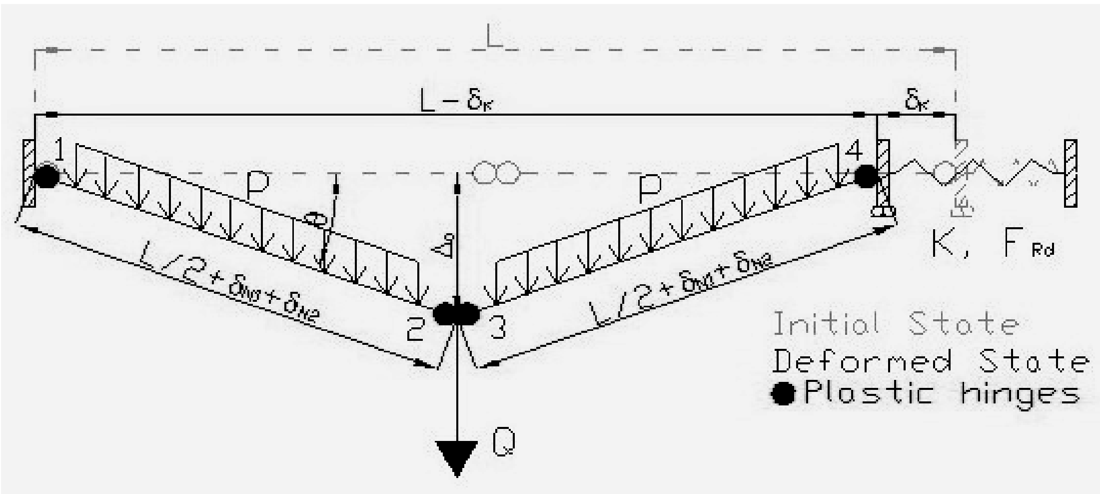

Fig. 2. Simplified substructure simulating the behaviour of the frame during Phase 3

2 EVOLUTION OF THE LOADS AND DISPLACEMENTS WITHIN THE INDIRECTLY AFFECTED PART

Within the present section, the members in the indirectly affected part which are placed within the rectangle in the *Fig. 3.a.*, i.e. the columns at the same storey of the loss column, are under consideration; indeed, the latter represent the columns which have to support the most important additional loads during the column loss.

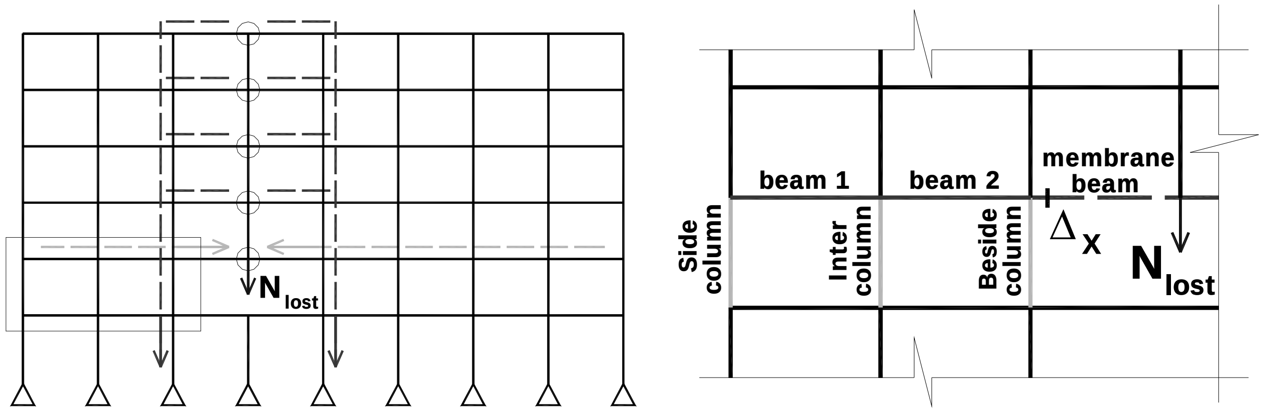

Fig. 3. a. The frame and identification of the investigated zone   *b.* Member names and positions

Fig. 4 illustrates the evolution of internal forces within the columns under consideration on the left side of the storey where the column loss occurs.
During the first phase, the loads applied to the frame mainly induce axial compression loads within the columns. Only the external column has to support significant bending moments. The axial forces in the internal columns are more or less equal to twice the value appearing in the external column. The bending moment and axial force values are respectively called (N_{design}, M_{design}) at the end of Phase 1. During this Phase, it can be reasonably assumed that the horizontal displacement at the top of the considered columns is equal to 0 ($\Delta_X \approx 0$).

$$\begin{aligned} &\boldsymbol{N} = 0 \nearrow \boldsymbol{N}_{design}; \\ &\boldsymbol{M} = 0 \nearrow \boldsymbol{M}_{design}; \\ &\Delta_X \approx 0. \end{aligned} \tag{2}$$

with

M_{design} and N_{design} are the designed bending moment and axial force within the columns at the end of Phase 1.

Δ_X is the horizontal displacement of at the top of the considered columns.

During Phase 2, the column is progressively removed. The column loss induces an increase of the bending moments at the beam extremities within the directly affected part. Within the indirectly affected part, the columns which are mainly affected by the column loss are the columns each side of the loss column; in particular, an increase of the applied bending moments and axial compression load is observed. The other columns are not significantly affected as illustrated in *Fig. 4*. At the end of Phase 2, the internal forces within the columns can be calculated as the sum of the design value at the end of Phase 1 plus the value associated to the column loss at the end of Phase 2 (i.e. at point (4) in *Fig. 1.b.*) ($M_{Elastic}^{Max}$). Also, during this Phase, the variation of the internal loads within the considered columns is proportional to the variation of the load within the loss column, i.e. to the axial load which is loss N_{lost} within the loss column, as noted in Formula (3).
During Phase 2, it is assumed that the horizontal displacement at the top point of the considered columns is not significant and, accordingly, can be neglected.

$$\begin{aligned} &\boldsymbol{N} = \boldsymbol{N}_{design} \nearrow \boldsymbol{N}_{Elastic}^{Max} = \boldsymbol{N}_{design} + \Delta \boldsymbol{N}_2; \\ &\Delta \boldsymbol{N}_2 = \boldsymbol{n}_1 \boldsymbol{N}_{lost}^{Pl.Rd} \\ &\boldsymbol{M} = \boldsymbol{M}_{design} \nearrow \boldsymbol{M}_{Elastic}^{Max} = \boldsymbol{M}_{design} + \alpha \Delta \boldsymbol{N}_2; \\ &\Delta_X \approx 0. \end{aligned} \tag{3}$$

with

$M_{Elastic}^{Max}$ and $N_{Elastic}^{Max}$	are the maximum internal force values within the considered columns at the end of Phase 2
α	is the coefficient linking the bending moment and the axial load within the considered columns during Phase 2 ($\Delta \boldsymbol{N}_2$)
n_1	is the coefficient linking $\Delta \boldsymbol{N}_2$ and $\boldsymbol{N}_{lost}^{Pl.Rd}$

During Phase 3, i.e. when a plastic mechanism is formed within the directly affected part, significant membranar forces developed within the directly affected part beams and, accordingly, significant additional loads have to be supported by the indirectly affected part, what influences the value of the internal loads within the considered columns. In fact, the additional compression within the columns is linearly proportional to the membranar forces developing in the directly affected part as detailed in Formula (4). Also, the bending moments within these columns evaluate according to the applied horizontal forces associated to the membranar forces. During this phase, significant horizontal displacements at the top of the considered columns are observed.

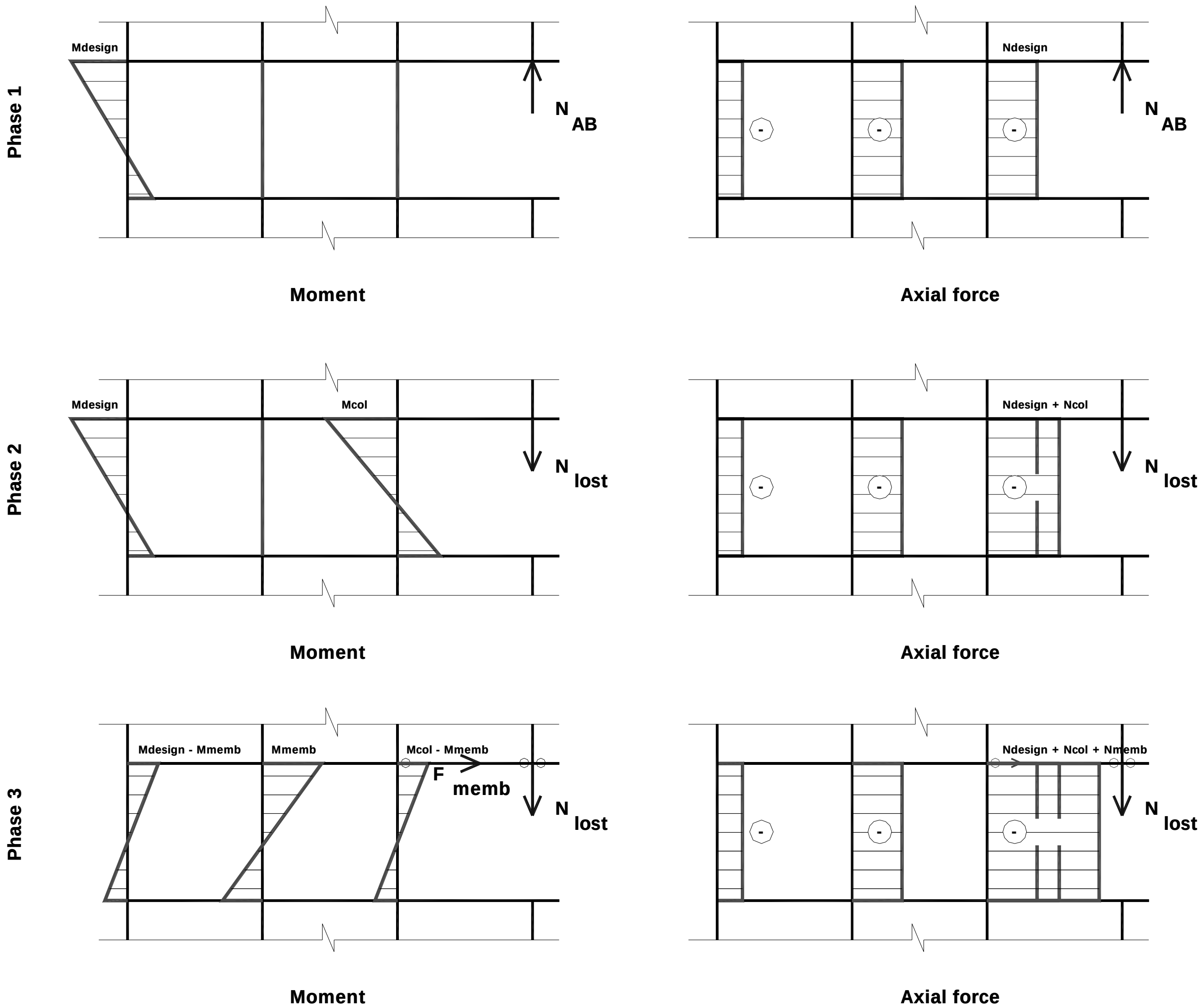

Fig. 4. Evolution of the bending moment ***M*** and axial force ***N*** within the columns of the indirectly affected part at the collapsed floor level

$$\begin{aligned} &\boldsymbol{N} = \boldsymbol{N}_{Elastic}^{Max} \nearrow \boldsymbol{N}_{Faillure} = \boldsymbol{N}_{Elastic}^{Max} + \Delta \boldsymbol{N}_3; \\ &\Delta \boldsymbol{N}_3 = \boldsymbol{n}_2 \boldsymbol{F} \\ &\boldsymbol{M} = \boldsymbol{M}_{design} \searrow 0 \searrow \boldsymbol{M}_{Faillure}; \quad \boldsymbol{M} = \boldsymbol{M}_{design} + \boldsymbol{M}_{Elastic}^{Max} - \beta \boldsymbol{F} \\ &\Delta_X \neq 0. \end{aligned} \tag{4}$$

with

$M_{Failure}$ and $N_{Failure}$ are the maximum internal forces values associated to the collapse of the indirectly affected part.

$\boldsymbol{\beta}$ is the coefficient linking the bending moment and the horizontal load associated to the membranar forces.

$\boldsymbol{n_2}$ is the coefficient linking ΔN_3 and the membranar forces $\boldsymbol{F}$

3 SIMPLIFIED MODELS FOR THE PREDICTION OF K AND F_{Rd}

The analytical procedures to predict the indirectly affected part properties *K* and F_{Rd} are founded on a simplified modelling as illustrated in *Fig. 5* and are developed from the idea presented in *[5]*. Each beam is considered as tension member only; the columns are modelled through a beam element with springs (k_{1c}, k_{2c}) at their extremities as shown in *Fig. 5*. Within the columns, the loads are applied in three phases as explained in the previous section. From phase 2 to the end, the second order effects are taken into account within the developed procedures.

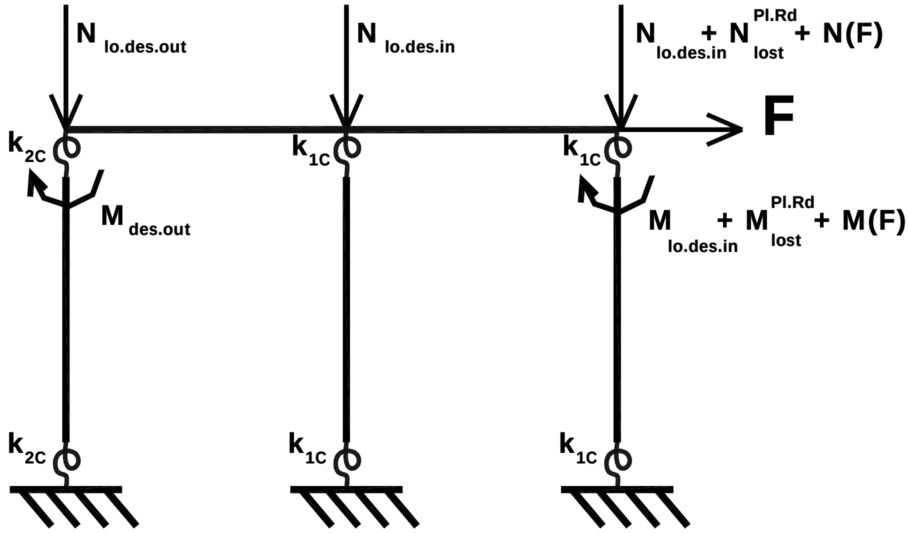

Fig. 5. The full part model included the transfer load in the phase 3

To predict the value of K, it is demonstrated in *[2]* that, in actual situation, the contribution of the axial stiffness of the beams within the prediction of K can be neglected. Accordingly, the value of $\boldsymbol{K}$ is assumed as equal to the sum of the contributions of the columns as shown in Formula (5), where "k" represent the stiffness of the column subjected to an horizontal load.

$$\boldsymbol{K} \approx \boldsymbol{k}_{Beside} + \boldsymbol{k}_{Inter} + \boldsymbol{k}_{Side} \tag{5}$$

The resistance F_{Rd} is assumed to be reached when an ultimate limit state is reached in one of the columns included the indirectly affected part. The resistance of the indirectly affected part can be predicted through Formula (6) where the distribution of the internal loads according to the column stiffnesses is taken into account.

$$\boldsymbol{F}_{Rd} \approx \boldsymbol{F}_c^{Rd} \left(\frac{\boldsymbol{k}_{Side}}{\boldsymbol{k}_{Beside}} + \frac{\boldsymbol{k}_{Side}}{\boldsymbol{k}_{Inter}} + 1 \right) \tag{6}$$

4 VALIDATION

The validity of the proposed analytical procedures has been investigated in *[2]* through comparisons with numerical investigations. An example of such comparison for one of the investigated frame is presented here below.

The frame has 7 spans and 6 floors. The span length and the floor height are uniform. The column section are HE 360A ones and the beam sections are IPE400v ones. For this building, 12 positions of column loss were investigated in *[2]* as presented in the *Fig. 8*. One example of comparison is given in Fig. 9 where the analytical prediction obtained through the proposed procedures is compared to the response obtained through a full non linear numerical analysis.

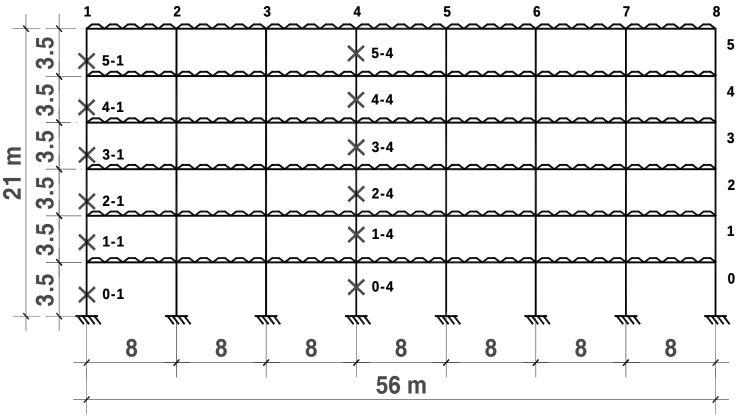

Fig. 8. Investigated steel frame

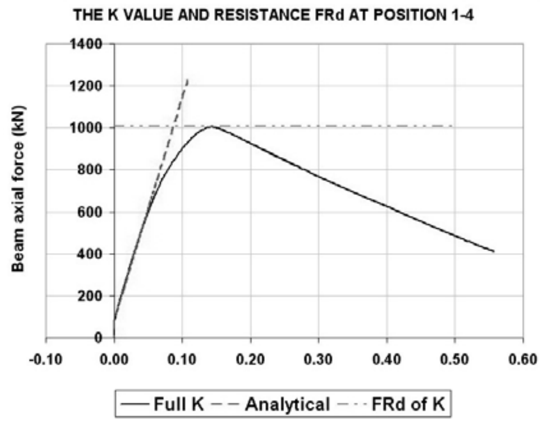

Fig. 9. Calculation of *K and* F_{Rd} at position 1-4

5 SUMMARY

Within the present paper, investigations dedicated to the exceptional event “loss of a column in a building frame” are presented. Part of the analytical methods developed in *[2]* to predict the values of the parameters influencing the development of the membranar forces within a structure are briefly described. The presented developments are contributions to a global concept under development at Liège University with the final aim to be able to predict, through simplified analytical methods, the response of a frame further to a column loss and the requested ductility within the structural members.

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