

# Robustness of steel structures subjected to a column loss scenario

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**ABSTRACT:** Recent events such as natural catastrophes or terrorism attacks have highlighted the necessity to ensure the structural integrity of buildings under an exceptional event. According to the modern design codes, the structural integrity of civil engineering structures should be ensured through appropriate measures but, in most cases, no precise practical guidelines on how to achieve this goal are provided. The Eurocode dealing with this topic is Eurocode 1, Part 1-7. For the above mentioned reason, this Eurocode is presently under revision with the objective of providing improved general recommendations to ensure an appropriate structural robustness in case exceptional events. In this framework, the robustness of steel and composite steel-concrete building frames is investigated at the University of Liege for many years following the so-called “alternative load path method”. The present paper gives a global overview of the ongoing researches in the field of robustness at the University of Liège.

## 1 INTRODUCTION

Recent events such as natural catastrophes or terrorism attacks have highlighted the necessity to ensure the structural integrity of buildings under an exceptional event. According to the Eurocodes and some different other national design codes, the structural integrity of civil engineering structures should be ensured through appropriate measures but, in most cases, no precise practical guidelines on how to achieve this goal are provided.

The Eurocode dealing with the structural integrity of structures is Eurocode 1, Part 1-7. For the above mentioned reason, this Eurocode is presently under revision with the objective of providing improved general recommendations to ensure an appropriate structural robustness in case of an exceptional event, knowing that the detailed application rules related to specific structural solutions would be reflected in the associated specific Eurocodes, i.e. in Eurocode 2 for concrete structures, in Eurocode 3 for steel structures...

In this framework, the robustness of steel and composite steel-concrete building frames is investigated at the University of Liege for many years following the so-called “alternative load path method” (Demonceau, 2008, Demonceau & Jaspart, 2010, Demonceau et al, 2011, Comeliau et al, 2012, Huvelle et al, 2013, Huvelle et al, 2015, Demonceau et al, 2017) with the final objective to propose design requirements useful for practitioners to mitigate the

risk of progressive collapse considering the conventional scenario “loss of a column” further to an unspecified event.

The present paper gives a global overview of the ongoing researches in the field of robustness at the University of Liège and of the adopted strategy aiming at deriving design requirements to be proposed for future implementation in the Eurocodes. In the following sections, the main outcome of recent researches with references to recently published papers will be reflected.

Firstly, the investigations conducted to study the quasi-static response of 2D steel and steel-concrete composite frames subjected to column loss scenarios are introduced in Section 2. Secondly, based on the knowledge gained from the studies presented in Section 2, the behaviour of 3D structures has been investigated and, in Section 3, it is demonstrated how the developments achieved for 2D frames can be easily extended to predict the behaviour of 3D structures. Finally, the dynamic effects which may be associated to a column loss are considered in Section 4. In particular, it is explained how the dynamic response of steel and steel-concrete structures can be derived from the quasi-static response.

## 2 BEHAVIOUR OF 2D FRAMES

When a frame suffers a column loss, two parts can be identified in the structure: the directly affected

part and the indirectly affected (Figure 1). The directly affected part contains all the beams, columns and beam-to-column joints located above the lost column. The rest of the structure (i.e. the lateral parts and the storeys under the lost column) is defined as the indirectly affected part.

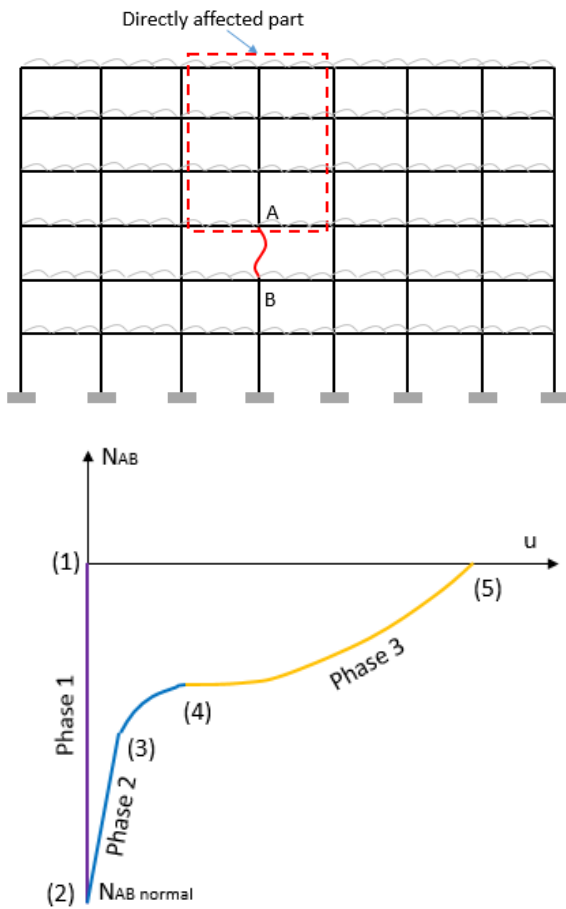


Figure 1. Behaviour of a 2D frame further to a column loss (Huvelle et al, 2015)

For a frame losing one of its columns (column AB in Figure 1), the evolution of the compression force  $N_{AB}$  in the lost member vs. its vertical shortening ( $u$ ) can be divided in 3 phases as illustrated in Figure 1.

During Phase 1 (from (1) to (2) in Figure 1), the column is “normally” loaded (i.e. the column supports the loads coming from the upper storeys) and its compression load is defined as equal to  $N_{AB \text{ normal}}$ .

Phase 2 (from (2) to (4) in Figure 1) begins when the column loss is initiated. During Phase 2, a plastic mechanism develops progressively in the directly affected part. Each change of slope in the curve during this phase corresponds to the development of a new hinge in the directly affected part, until a complete plastic mechanism is reached at point (4) on Figure 1.

Phase 3 (from (4) to (5) in Figure 1) starts when this plastic mechanism is formed: the vertical displacement at the top of the column increases significantly since there is no more “first order” rigidity in the structure. As a result of these large displacements, catenary actions are developing in the beams

of the directly affected part, giving a “second-order” stiffness to the structure. The role of the indirectly affected part during Phase 3 is to provide a lateral anchorage to these catenary actions: the stiffer the indirectly affected part is, the more catenary action will develop in the directly affected part. In the extreme situation where the indirectly affected part has no lateral stiffness, then catenary action will not develop and there will be no Phase 3.

Point (5) is reached with the vanishing of the axial force in the column. On the basis of the displacement associated to point (5), one can estimate the deformation capacities required in the plastic hinges and evaluate the forces in all the structural elements; accordingly, it is possible to check if the structure is sufficiently robust to reach this new state of equilibrium.

The prediction of the response of the frame during Phase 1 and Phase 2 can be obtained easily while the prediction the response during Phase 3 is much more complicated as significant deformations and displacements inducing second order effects are observed during this phase. Accordingly, the research conducted in Liege has mainly been dedicated to this phase.

In particular, a fully analytical model has been developed to predict the response of a steel or composite frame during Phase 3 (Huvelle et al, 2013, Huvelle et al, 2015, Demonceau et al, 2017). The proposed analytical model is founded on the definition of a substructure able to reflect the behaviour of the whole frame (Figure 2) and allows easily predicting the response of a 2D frame further to a column loss using the following assumptions:

- a progressive (static) column loss is assumed;
- the hinges can develop in the beam cross-sections or in the beam-to-column joints;
- all columns are made of an unique cross-section type; and it is the same for the beams;
- column bases are assumed to be perfectly fixed;
- only the loss of internal columns (i.e. columns which are not at the corners) is considered;
- no yielding develops in the indirectly affected part (i.e. its behaviour is assumed to be infinitely elastic).

The analytical model developed to predict the response of the so-defined substructure is detailed in (Huvelle et al, 2015). In this model, the behaviour of the yielded zones at the extremities of the beams subjected to combined bending moments and axial loads is reflected through a multi-layer spring model; this model allows reflecting the behaviour of this yielded zone if plasticity is developing in the beam or in the beam-to-column joints (if no group effects are developing between the bolt rows in case of bolted connections and if no stress interactions are considered at the level of the components).

The validity of the analytical model has been checked by comparing the predictions resulting from

its application to numerical results and experimental results (Demonceau & Jaspart, 2010). Through these comparisons, it has been demonstrated that the proposed analytical model is able to predict accurately the response of a steel or a steel-concrete composite frame during Phase 3.

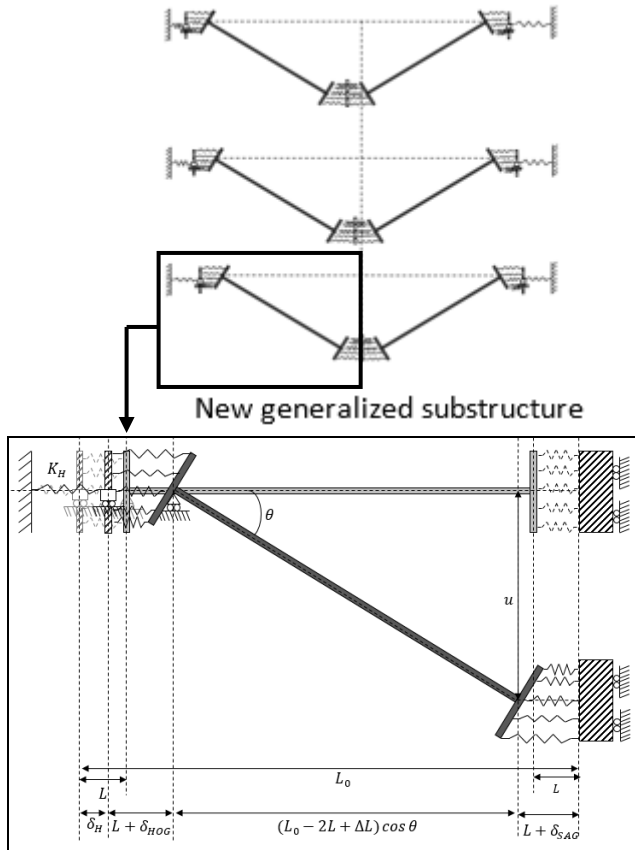


Figure 2. Substructure model (Huvelle et al, 2015)

As previously mentioned, the proposed model is presently limited to specific joint configurations for which the behaviour of the joints is not affected by group effects and/or by stress interactions at the level of the components. However, these phenomena can have a significant influence on the resistance of joints subjected to M-N and so, more globally, on the response of the frames further to a column loss.

A further improvement should consist in substituting to the multi-layer spring model illustrated in Figure 2 a more refined one able to (i) catch the group effects, (ii) take into account of the stress interactions at the level of the components and (iii) cover the successive loading conditions during the column loss (pure bending, bending and axial forces...). Such a spring model is under development at Liège University. Very promising results have already been obtained; they should be published in a near future.

These works on joints are in the continuity of past investigations led in Liège since two decades. In particular, the component method which is nowadays recommended in the Eurocodes to characterise the structural properties of joints in bending has been extended to predict the M-N resistance curve of steel and composite joints. Indeed, the component method

was first adapted in (Cerfontaine, 2003) to predict the behaviour of steel joints subjected to M-N and then, was adapted in (Demonceau, 2008, Jaspart, 2008) to predict the behaviour of composite joints subjected to M-N. Finally, the proposed model was also adapted to predict the behaviour of joints subjected to M-N at elevated temperatures (Demonceau, 2013, Haremza, 2016), loading situation which may occur, for instance, when the column loss is associated to a localised fire. The validity of these developments has been checked through comparisons to experimental results and the very good accuracy of the proposed model is clearly demonstrated in the previously mentioned references.

### 3 BEHAVIOUR OF 3D STRUCTURES

Based on the knowledge gain from the studies on 2D structures, investigations on 3D steel and steel-concrete composite structures have been conducted to investigate the possibility of extending the analytical 2D model presented in the previous section.

In a first step, parametrical studies on braced and unbraced steel structures made of steel members (without accounting for any contribution from the slab) have been conducted (Kulik, 2014, Ghimire, 2015). Though this work, it has been demonstrated that (i) the behaviour of 3D structures is similar to the one observed for 2D structures and (ii) the behaviour of the 3D structures can be obtained by summing the contribution from the two 2D frames intersecting at the level of the lost column as illustrated in (Figure 3).

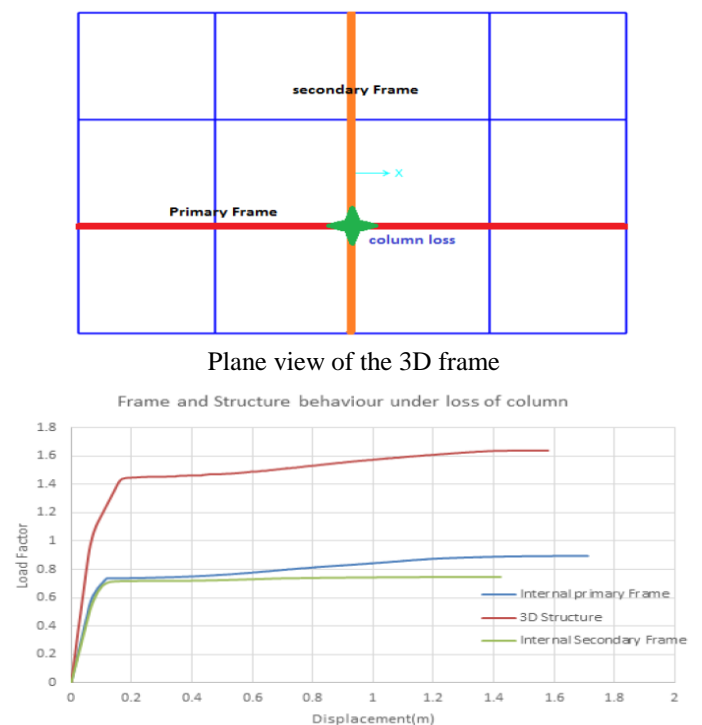


Figure 3. Illustration of the response of a 3D braced structure further to a column loss (Ghimire, 2016)

Accordingly, through this observation, it can also be concluded that the analytical model presented in the previous section can be easily used to predict the response of a 3D structure made of steel (or steel-concrete) columns and of steel-concrete composite beams (through the concept of effective width for the concrete slab, as a simplified way to account for the latter).

However this way of accounting for the presence of the concrete slab in a composite 3D structure is seen to be conservative and that is why further experimental (University of Stuttgart, 2016) and numerically (Mollica, 2015) studies on this specific aspect have been performed.

Through these investigations, it has been possible (i) to demonstrate the ability of different numerical tools to simulate the behaviour of composite 3D structures in case of column loss scenario and (ii) to characterise the behaviour of composite floors (concrete or composite slabs connected to the upper flanges of a steel grid) through parametrical numerical studies performed with the so-validated numerical tools. In particular, it has been demonstrated that:

- The behavior of the slab is mainly governing the behavior of the composite slab in case of column loss scenario as membrane forces are developing in the slab as soon as small displacement are appearing while the level of deformation reached after the column loss is not sufficient to activate membrane forces in the composite beams;
- The global response a composite floor can be obtained by summing the contribution from the slab analyzed in isolation and the contribution from the composite beams analyzed in isolation as illustrated in Figure 4 where it can be observed that the curve reflecting the response of the composite floor can be obtained by summing the curve reflecting the behavior of the slab simulated alone, i.e. without taking into consideration the steel beams connected to the slab though their upper flange and the curve reflecting the behavior of the composite beams simulated by only considering the part of the slab directly above the upper flange and collaborating with the steel profile (effective width).

So it appears that the behaviour of a composite floor can be analytically predicted as soon as analytical models are available to predict the behaviour of the composite beams and of the slab in isolation.

For the composite beams, the previously introduced analytical model can be used. For the slab, it is recommended in (University of Stuttgart, 2016) to use the Bailey's model (Bailey, 2002). In (University of Stuttgart, 2016), first research works have been conducted to demonstrate the ability of the Bailey's model to predict the behaviour of a slab in case of column loss scenario with promising results. Further investigations are still required (i) to confirm these first results and (ii) to demonstrate that the sum of

the previously mentioned analytically predicted responses provides an accurate prediction of the global behaviour of composite floors. At the University of Liège, it is intended to perform such further investigations.

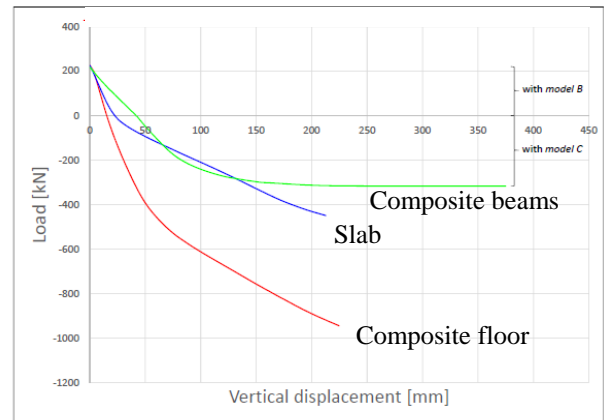


Figure 4. Numerical prediction of the behaviour of a composite floor (Mollica, 2015)

#### 4 DYNAMIC EFFECTS

The dynamic response of building frames further to a column loss has been investigated at the University of Liege in the last few years (Colomer, 2017, Comeliau, 2010, Comeliau, 2012).

To understand the behaviour of such frames, a basic substructure was first extracted and deeply studied for sake of simplicity. Indeed, frames losing a column proved to show many similarities in behaviour with the above-considered simple substructure (Demonceau, 2008). This one, illustrated in Figure 5, is a double-beam system with a horizontal spring placed at one of the extremities of the beam and reflecting the lateral restraint provided by the rest of the structure from which the studied beam is extracted (i.e. by the indirectly affected part – Figure 1). The load  $P$  simulates the column loss (statically or dynamically applied).

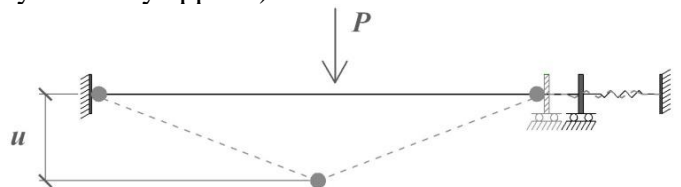


Figure 5. Investigated double-beam substructure (Demonceau, 2017)

As far as dynamic loadings are concerned, the applied force  $P_{\text{dyn}}(t)$  is considered to linearly increase during a rise time  $t_r$  and then to remain constant at a maximum value equal to  $P$  (Figure 6). If values of  $P$  higher than  $P_{\text{pl}}$  are considered ( $P_{\text{pl}}$  being the value of the static force causing the formation of the beam plastic mechanism), two main types of response have been observed through parametrical numerical studies, depending on the loading conditions defined

by  $P$  and  $t_r$ . These two types of response can be explained through comparisons between the dynamic curve  $u_d(t)$  expressing the time evolution of the displacement induced by the application of  $P_{dyn}(t)$  and the static curve  $u_s(t)$  providing at each particular time  $t^*$  the static displacement  $u_{stat}(P^*)$  associated to the value of the force  $P^*=P_{dyn}(t^*)$  (Figure 7 and Figure 8). For type 1 response (Figure 7), it can be seen that the maximum dynamic displacement which is reached is higher than the static displacement which would have been observed while for type 2 response (Figure 8), it can be observed that the maximum final displacement obtained through a dynamic analysis is similar than the one which would have been reached through a quasi-static removal of the column. The shape of the curves reported in Figure 7 and Figure 8 are described and justified in details in (Demonceau, 2017).

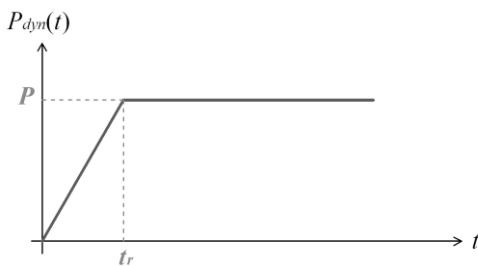


Figure 6. Dynamic loading applied to the substructure (Demonceau, 2017)

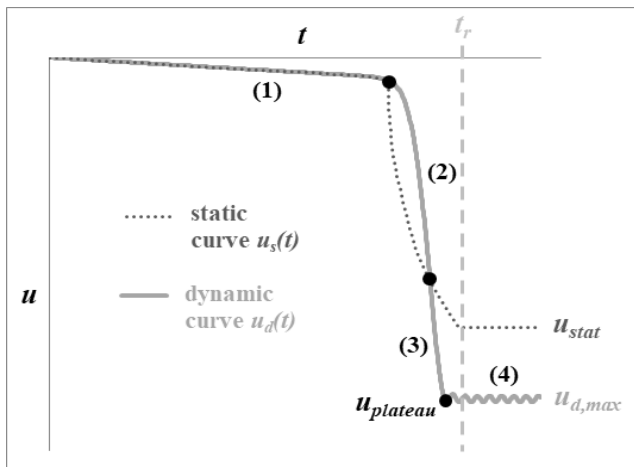


Figure 7. Example of a type 1 response

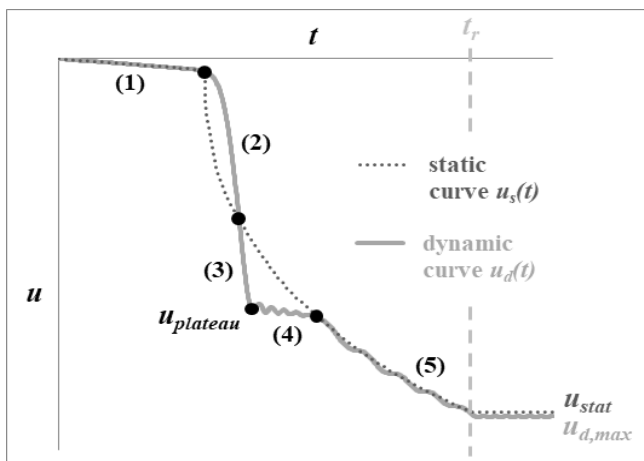


Figure 8. Example of a type 2 response

Based on these observations, a simplified model was developed for the prediction of the substructure dynamic response (Comelieau, 2012). This model is based on energy considerations and consists in writing that the work done by the external load is equal to the sum of the kinetic energy and the deformation energy. For this simple system, the deformation energy could be expressed as a function of the vertical displacement at mid-length, considering the work related to the plastic hinges rotation and the lateral spring elongation. The plastic interaction between the bending moment and the tension force in the plastic hinges has been taken into account as well as the elongation of the beams. With the developed analytical model, it is possible to predict the displacement vs. time response of the simplified model knowing its quasi-static response (which can be predicted using the analytical model previously presented in this paper). This analytical model has been validated through comparison between numerical and analytical predictions. Illustrations of such comparisons for type 1 and type 2 responses are given in Figure 9.

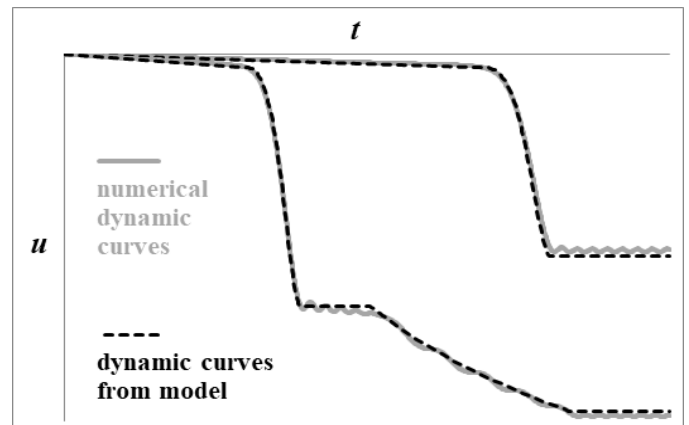


Figure 9. Validation of the substructure dynamic model (Demonceau, 2017)

Then, in (Demonceau, 2017), it is demonstrated how the developed model can be easily extended to the prediction of the response of a full frame subjected to the dynamic loss of a column. In particular, it is shown that the displacement vs. time response of the frame is predicted with a very good accuracy when compared to numerical results.

Accordingly, a full analytical model able to predict the dynamic response of structures further to a column loss is available; knowing the geometrical and mechanical properties of the structural elements and the time  $t_r$  associated to the column loss (see Figure 6), it is possible to predict the maximum dynamic displacement  $u_{d,max}$  reached during the column loss. Knowing the maximum dynamic displacement, it is possible to assess the required ductility and resistance for the different structural elements.

Indeed, many finite element analyses (University of Stuttgart, 2016) demonstrated that the distribution of internal forces in a frame corresponding to a given value of the displacement  $u$  is the same no matter if this displacement is reached statically or dynamically (provided this displacement is reached for the first time - the unloadings and reloadings associated to the system oscillations are not considered).

Consequently, if the static response is known, the dynamic maximum displacement can be determined using the proposed model; and then the corresponding distribution of internal forces can be deduced from the static response. If the structure is able to statically reach the displacement  $u_{d,max}$ , then the dynamic robustness is ensured.

The main difficulty in the proposed procedure is to be able to predict the time  $t_f$  associated to the column loss, this value strongly depending of the nature of the exceptional event associated to the column loss. In the framework of the Robustimpact project (University of Stuttgart, 2016), the University of Liège and the RWTH of Aachen combined their efforts to solve it for the specific scenario “impact of a vehicle on a column” (Colomer, 2017). In particular, RWTH developed a model able to predict the local response of a column in a frame when subjected to an impact, taking into account the effect of the restraint and the inertia of the surrounding structure. Merging this model with the “Liège” model previously mentioned, it was possible to predict the response of a frame subjected to an impact (for different level of energy) through a fully analytical procedure (see Figure 10 - Colomer, 2017).

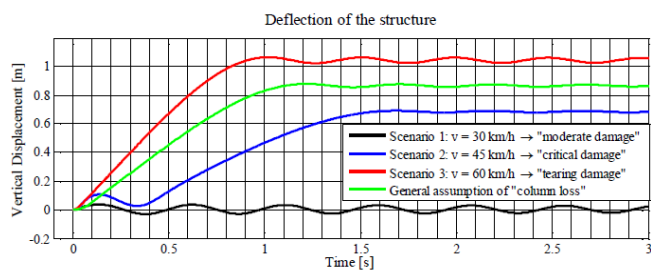


Figure 10. Dynamic response of a structure predicted with the proposed analytical model under different column damage scenarios (Colomer, 2017)

Last but not least, it has to be mentioned that in all the presented dynamic models, the possible strain rate effects have not yet been implemented. These effects are still under investigations at the University of Liège, in particular when these effects are developing at the level of the beam-to-column joints; this point is addressed in a second paper presented within the present conference.

## 5 CONCLUSIONS

Within the present paper, recent developments performed at the University of Liège in the field of robustness and, more precisely, for the specific scenario “loss of a column in a steel or steel-concrete composite building” have been briefly presented. The objective of the present paper was to highlight the followed global strategy, passing from the quasi-static behaviour of 2D frames to the dynamic behaviour of 3D structures, and the main outcomes of the conducted researches, referring to recent publications for the details.

In particular, it is demonstrated how analytical models have been developed with the objective of easily predicting the behaviour of structures for the considered scenario, the main interest of the analytical models being that all the structural properties and parameters affecting the response of the structures are identified and mastered. Having identified these structural properties, it will be possible in a next step to derive design requirements useful for practitioners and able to ensure an appropriate behaviour of a structure in case of a column loss scenario. These ones could then be possibly implemented one day in the Eurocodes. This constitutes the main perspective of the presented developments.

## 6 ACKNOWLEDGEMENT

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