# Recent investigations on the behaviour of buildings after the loss of a column

J.F. Demonceau, H.N.N. Luu & J.P. Jaspart
*M&S Department, Liège University, Liège, Belgium*

ABSTRACT: Recent events such as natural catastrophes or terrorism attacks have highlighted the necessity to ensure the structural integrity of buildings under exceptional events. According to Eurocodes and some different other national design codes, the structural integrity of civil engineering structures should be ensured through appropriate measures but, in most of the cases, no precise practical guidelines on how to achieve this goal are provided. An European RFCS project entitled "Robust structures by joint ductility" has been set up in 2004, for three years, with the aim to provide requirements and practical guidelines allowing to ensure the structural integrity of steel and composite structures under exceptional events through an appropriate robustness. In particular, one substructure test simulating the loss of a column in a composite building will be performed at Liège University. The present paper described first analytical and numerical studies carried out at Liège University as part of this European project.

## 1 INTRODUCTION

A structure should be designed to behave properly under service loads (at SLS) and to resist design factored loads (at ULS).

The type and the intensity of the loads to be considered in the design process may depend on different factors such as:

- the intended use of the structure: type of variable loads…
- the location (region, altitude, …): wind action, snow, level of seismic risk…
- and even the risk of accidental loading: explosion, impact, flood…

In practice, these individual loads are combined so as to finally derive the relevant load combination cases.

In this process, the risk of an exceptional (and therefore totally unexpected) event leading to other accidental loads than those already taken into consideration in the design process in itself is not at all covered. This is a quite critical situation in which the structural integrity should be ensured, i.e. the global structure should remain globally stable even if one part of it is destroyed by the exceptional event (explosion, impact, fire as a consequence of an earthquake, …).

In conclusion, the structural integrity will be required when the structure is subjected to exceptional loads not explicitly considered in the definition of the design loads and load combination cases.

According to Eurocodes (prEN 1991-1-7, 2004, ENV 1991-2-7,1998) and some different other national design codes (BS 5950-1:2000, 2001, UFC 4-023-03, 2005), the structural integrity of civil engineering structures should be ensured through appropriate measures but, in most of the cases, no precise practical guidelines on how to achieve this goal are provided. Even basic requirements to fulfil are generally not clearly expressed.

Different strategies may therefore be contemplated:

- Integrate all possible exceptional loads in the design process in itself; for sure this will lead to non-economic structures and, by definition, the probability to predict all the possible exceptional events, the intensity of the resulting actions and the part of the structure which would be affected is seen to be "exceptionally" low.
- Derive requirements that a structure should fulfil in addition to those directly resulting from the normal design process and which would provide a certain *robustness* to the structure, i.e. an ability to resist locally the exceptional loads and ensure a structural integrity to the structure, at least for the time needed to safe lives and protect the direct environment. Obviously the objective could never be to resist to any exceptional event, whatever the intensity of the resultant actions and the importance of the structural part directly affected.

In the spirit of the second strategy, an European RFCS project entitled "Robust structures by joint ductility – RFS-CR-04046" has been set up in 2004, for three years, with the aim to provide requirements and practical guidelines allowing to ensure the structural integrity of steel and composite structures under exceptional events through an appropriate robustness.

The robustness is required from the structural system not directly affected by the exceptional event (to avoid the local destruction of the structural element where the event occurs being often not possible). In this process, the ability to redistribute plastically extra forces resulting from the exceptional event is of high importance. This requires from all the structural elements and from the constitutive joints a high degree of plastic deformability under combined bending, shear, … or axial forces.

The partners involved in this project are:

– Stuttgart University, Germany;
– Liège University, Belgium;
– ProfilArbed-Research (PARE) from the Arcelor Group, Luxembourg;
– PSP-Prof. Sedlaceck & Partner - Technologien, Germany and;
– Trento University, Italy.

The present article presents the first developments performed at Liège University as part of this European project and is organized as follows:

– Section 2 presents the different exceptional events covered within the project and the adopted strategy;
– then, first numerical and analytical developments performed at Liège University are described in Section 3 and;
– finally, in Section 4, the substructure test to be performed at Liège University is presented.

## 2 COVERED EXCEPTIONAL EVENTS AND ADOPTED STRATEGY

As a general procedure to derive robustness requirements, different structural systems subjected to exceptional events are analytically and numerically investigated within the previously mentioned project in order to see how steel and composite structures work when part of the structure is destroyed as well as how and how far redistribution takes place.

Exceptional events have been selected; many could be contemplated, but few preliminary ones have been considered as reference cases to be studied first:

1 loss of a column in an office or residential building frame;
2 loss of a beam in an office or residential building frame;
3 loss of a column in an industrial portal frame;
4 loss of a bracing in an industrial portal frame;
5 loss of a bracing in a car park;
6 unexpected earthquake;
7 unexpected fire.

For the five first cases, finite element (FEM) numerical simulations are carried out so as to understand how the structure and its constitutive elements behave and how the redistribution of forces takes place in the unaffected part of the frame. In this process, a special attention will be devoted to the study of the loading sequence inside the joints. As a result of these FEM numerical simulations and associated parametrical studies, simplified behavioural models should be developed and validated; these ones should progressively lead to analytical models, from which requirements to be satisfied by the structural system and by the joints could be derived.

Progressively, other exceptional situations should be investigated in the same way and related design requirements should be derived.

Possibly similarities between different exceptional events and their corresponding failure modes will be identified and more general requirements are so expected to be formulated.

For the six and seventh here-above listed events, the work consists in expressing requirements that structures which have not been explicitly designed for fire and/or seismic actions should fulfil so as to possess a certain amount of robustness against such unexpected extreme situations. In different countries, “good practice” detailing recommendations and conceptual design guidelines exist (for instance for so-called “non-engineered structures”) and the work should therefore consist in gathering and analysing this available material and present it into an adequate format.

Within the previously mentioned European project, the analytical and numerical investigations has been shared among the partners:

– Trento University is in charge of “event 6” (earthquake);
– PARE covers “event 7” (fire);
– Stuttgart University studies “event 5” (loss of a bracing in a car park);
– Liège University focuses on “events 1 and 3” (loss of a column in office or residential composite building frames and in industrial steel structures)
– PSP contributes to the knowledge on “events 1 and 3” by studying 3-D aspects as well as the loss of more than one column;

Liège University is in charge of coordinating the whole activity.

Also, one of these exceptional events, the loss of a column in a composite structure, is intended to be tested experimentally at Liège University, as part of the project; this will allow to validate the numerical tools used in the preliminary study.

Finally, parametrical studies will be carried out numerically for the selected events and robustness requirements will be derived.

In the next section, first analytical and numerical investigations performed at Liège University on

“Event 1” (loss of a column in an office or residential building frame) are described.

## 3 LOSS OF A COLUMN IN A BUILDING - ANALYTICAL AND NUMERICAL INVESTIGATIONS

### 3.1 *Introduction*

As mentioned in the previous section, first analytical and numerical investigations have been conducted at Liège University on “Event 1” dealing with the loss of a column in office or residential building frame, as illustrated in Figure 1.

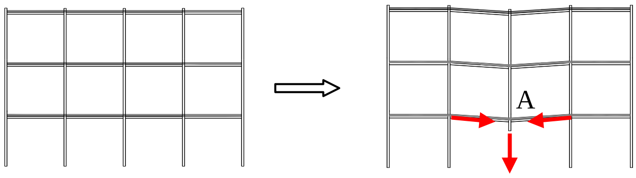


Figure 1. Loss of a column in a residential or office building frame

At first, before the event, the joint and the beams are mainly subjected to bending moments. When the column looses its carrying capacity, catenary action develops in the beams (as illustrated in Figure 2); axial forces increase (because of loads transferred by the column stub located just over the impacted one) until the joint or the beam reaches a full plastic state (under moment and axial forces). The beam takes large transverse displacements and axial forces increase further while bending moments decrease; this loading path and the evolution of the bending moment and axial force in the joint (or in the beam) are qualitatively illustrated in Figure 3.

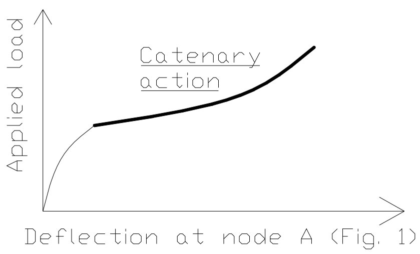


Figure 2. Development of the catenary action in the structure – illustration in an “applied load/beam deflection” curve.

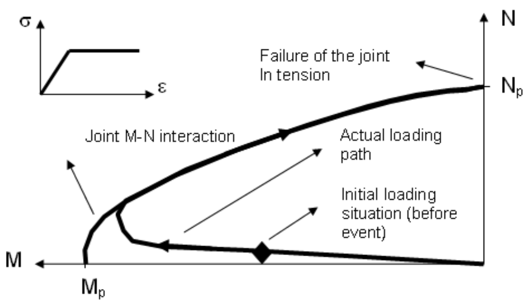


Figure 3. Actual loading in the joint or in the beam until failure.

At the end, the joint and the beams work mainly in tension. If the transverse forces applied to the beam (loads acting on the beam itself and loads from the upper storeys) is such that the value of $N_p$ is not reached in the joint or in the beams, the system has a sufficient robustness to face the event; if not, a lack of robustness has to be contemplated.

The scope of the previously mentioned project is to reach robustness through joint ductility. So, the frames under consideration possess partial-strength and semi-rigid beam-to-column joints; the “weak” elements when catenary action develop are then the joints.

From the previous observation, requirements on the required joint tensile resistance may be derived; but it should not be forgotten that the joint will only be able to develop an adequate resistance all along the loading sequence if the ductility of the joint is sufficient to avoid a premature brittle failure inside the joint (welds, bolts, rebars in case of composite joints, …). That is why the requirements have to be expressed in terms of resistance and ductility, and not only, as it is the case in the few presently available design recommendations (e.g. BS 5950-1:2000, 2001 in UK), in terms of resistance.

The intention in Liège is to substitute the complex problem of the loss of a column in a frame by a far more simple one limited to the study of a single “two-beams” system (Fig. 4), by referring to the definition of a *K* restraining coefficient.

The K spring simulates the restraint offered by the undamaged part of the frame to the development of very high transverse displacements at mid-span of the two-beams system when the column is impacted. Through this structural restraint K, a catenary action may develop in the system.

In order to validate this simplification, the following steps have to be crossed:

– proceed to the numerical simulation of the full non-linear response of the impacted frame;
– proceed to the numerical simulation of the full non-linear response of the “two-beams” simplified system;
– compare the good agreement between the numerical responses got respectively for the full frame and for the “two-beams” system.

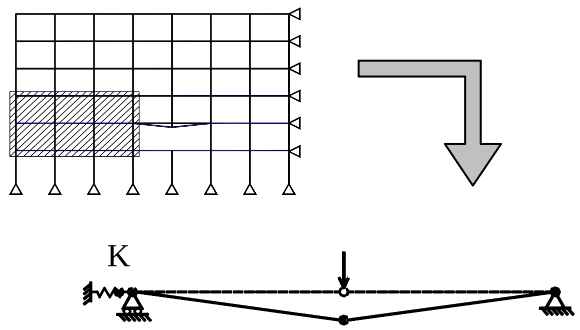


Figure 4. Global and local modelling of “event 1“ (loss of a column).

The final objective is to develop an analytical model, the use of which could allow the derivation of design requirements for robust structures, in case of the loss of a column.

Practically speaking, the influence of each of the main parameters on the response of the impacted system is studied and conclusions are drawn so as to see whether and how, at the end, these parameters have to be further contemplated.

In a first step, in order to understand how the various parameters influence the response of the "two-beams" system, an eleven-level parametrical study has been carried out. The latter is presented in the next section.

### 3.2 *Parametrical study of the subsystem*

So as to identify the parameters influencing the response of the subsystem under the considered exceptional event, an eleven-level parametrical numerical investigation has been performed on the subsystem previously defined.

The main parameters considered are the following ones:

- The beam response: the stiffness of the beams in bending (*EI*) and under axial force (*EA*) are varied, as well as the yield strength $f_y$ of the constitutive material; a high value of EI allows to simulate "rigid" beams, while the adoption of high values of $f_y$ enables to simulate a fully elastic response of the beam elements.
- The K restraint: the importance of the membrane effects in the beam increases with the *K* values, while the beam transverse displacements at failure decrease. For high values of *K*, high tying forces are obtained at beam ends, while demand in terms of rotational capacity is requested at beam ends when large displacements occur in the beam, i.e. for low values of *K*.
- The resistance properties of the beam end sections: in this preliminary study, no connection is assumed to act at beam ends; so possible plastic hinges develop in the beam itself for an axial force equal to $N_p$ (tension resistance of the beam), for a bending moment equal to $M_p$ (bending resistance of the beam cross-section) or under a combination of moment and axial forces. In the parametrical study, no interaction between axial forces and bending moments is first contemplated; then a non-linear interaction resistance curve characterizing the beam cross-section is considered.

The eleven considered levels are illustrated in Figure 5. The system is loaded by a uniformly distributed load; the total length of the system is equal to 4m.

The numerical investigations are performed with the homemade finite element software FINELG developed at Liège University (M&S Department) and at Greisch design office (Liège, Belgium).

Full 2-D non-linear analyses are performed, with due account of geometrical and material non-linearities. The numerical technique implemented in FINELG enables to follow the behaviour of a structure under increasing external loading up to collapse or instability, and even beyond.

The scope of the presented study is to investigate the influence of different parameters on the development of the catenary action in the subsystem. So, in order to not to restrict the development of the catenary action in the numerical modelling, the plastic strain limitations have been deactivated in the software, as illustrated in Figure 6, i.e. it is assumed that the different members of the two-beams system have an infinite ductility. In conclusion, the collapse of the subsystem is assumed to be achieved when the axial forces in the system reach the axial resistance $N_{pl}$.

The results obtained for the different levels are summarised in Figure 7.

From this parametrical study, interesting conclusions may be drawn:

- The development of the catenary action depends on the relative values of the axial beam stiffness *EA/L* and the stiffness of the spring *K*. In practical situations, it has been shown that the influence of the axial beam stiffness can be neglected. Additional parametrical investigations have also been performed to confirm this observation (Demonceau et al., 2006).
- The influence of the bending stiffness *EI/L* on the development of the catenary action may be neglected. This has also been confirmed through additional parametrical studies (Demonceau et al., 2006).
- The maximum applied load which can be reached, for the loading path described in Figure 3, depends of the value of *K*. It increases with decreasing values of *K*. The needs in terms of ductility increase also when the *K* value is decreasing.

These numerical analyses only represent the first step of the works carried out at Liège University. As already said, the next steps to be reached are:

- the development of analytical formulations so as to predict the response of the "two-beams" subsystem;
- the derivation of design requirements in terms of resistance and ductility;
- the validation of the use of a "two-beams" subsystem.

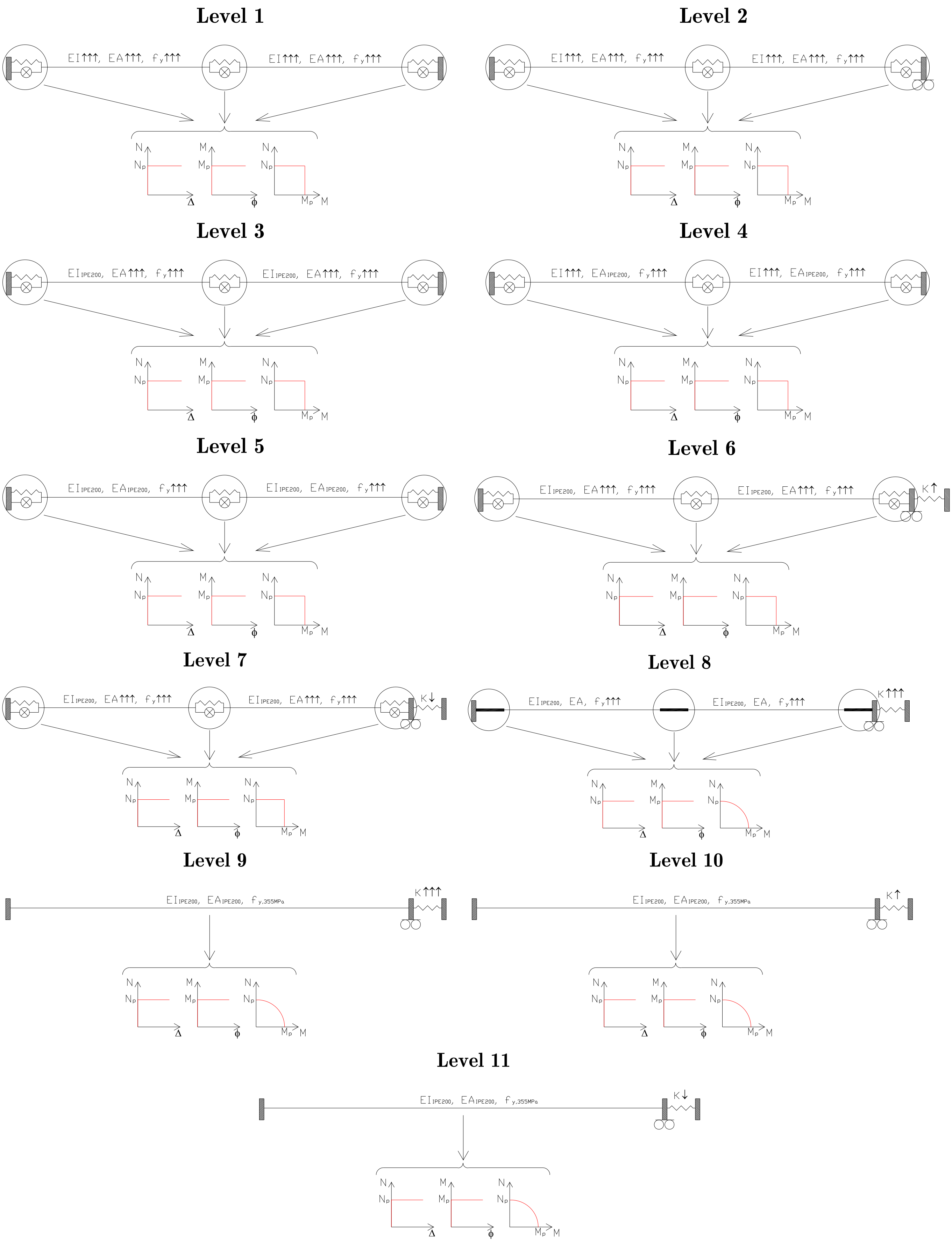


Figure 5. Investigated levels for the parametrical study of the subsystem

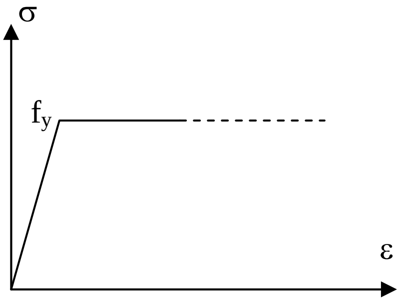


Figure 6. Infinite ductility assumption for steel material.

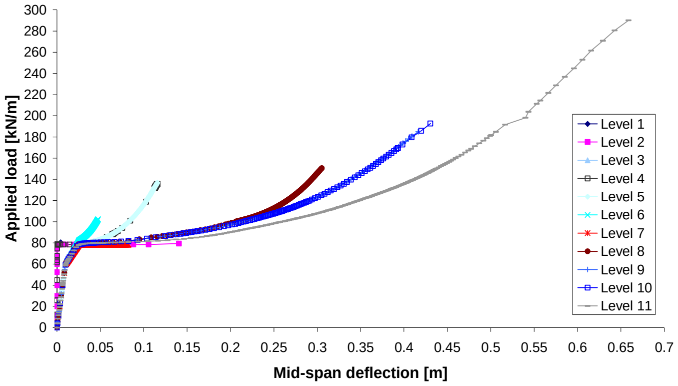


Figure 7. Obtained results for the different investigated levels – "applied load/mid-span beam deflection" curves.

The validation of the subsystem - through comparisons of its response with the one obtained by simulating the whole structure – requires the preliminary evaluation of the stiffness of the *K* restraint. This work has been carried out by the second author of the present paper and validated through few hundreds of numerical simulations. In this study, the position of the impacted column in the structure has been considered, as well as the braced/unbraced character of the structure. The analytical formulation of the K factor resulting from these investigations is intended to be published soon.

## 4 DESCRIPTION OF THE EXPERIMENTAL TEST ON A SUBSTRUCTURE

### 4.1 *Introduction*

Within the RFCS European project, a test on a substructure simulating the loss of a column in a composite building is planed to be performed at Liège University. The aim is to validate the numerical tools used for the parametrical investigations.

To define the substructure properties, an "actual" composite building has been designed (Demonceau et al., 2006a) according to Eurocode 4 (NBN EN 1994-1-1, 2005), so under "normal" loading conditions (i.e. loads recommended in Eurocode 1 (EN 1991-1-1, 2002) for office buildings); the main properties of this building are briefly introduced in Section 4.2.

As it is not possible to test a full 2-D actual composite frame within the project, a substructure has been extracted from the actual frame described in Section 4.2; it has been chosen so as to respect the dimensions of the testing slab but also to exhibit a similar behaviour than the one in the actual frame (see Section 4.3.)

### 4.2 *Description of the reference composite building*

The building is composed of three main frames at a distance of 3m. Each frame has four bays (4m width each) and three storeys (3.5m height each); the general layout is given in Figure 8.

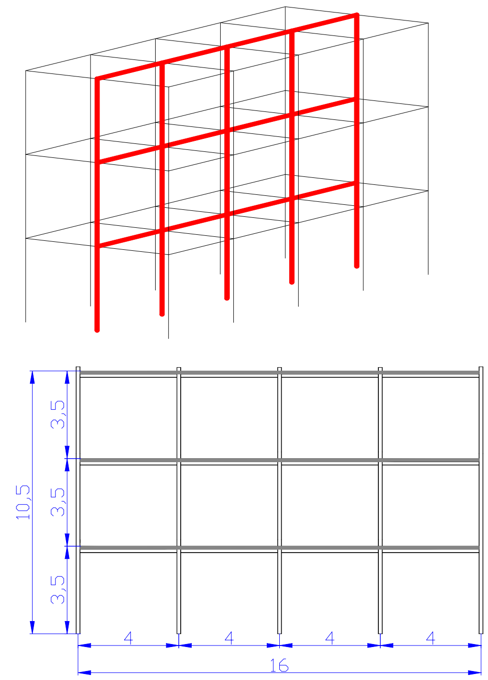


Figure 8. General layout of the reference composite building.

As previously said, the building has been designed according to Eurocode 4 and under normal loading conditions. Its structural characteristics are as follows:

– The slab is a reinforced concrete one (12cm thick and C25/30 concrete). The reinforcement is composed of two steel meshes: the upper one with 10mm rebars each 200mm and the lower one with 10mm rebars each 150mm. The steel grade for these rebars is S500C and the cover is equal to 25mm. The slab cross-section is shown in Figure 9.

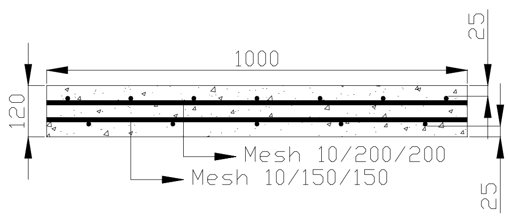


Figure 9. Slab properties

– The composite beams are seen in Figure 10. A S355 IPE140 profile is used and a full shear connection is assumed between the steel profile and the concrete slab.

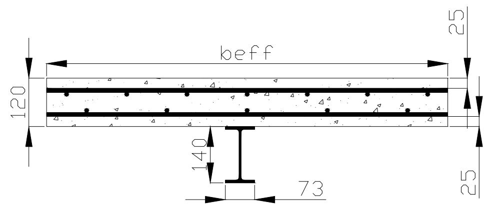


Figure 10. Composite beam cross-section.

- The columns are steel ones (S355 HEA160).
- Partial-strength and semi-rigid joints are considered (Figure 11 and Figure 12). The properties of these joints allow them to exhibit a ductile behaviour (with account of possible overstrength effects).

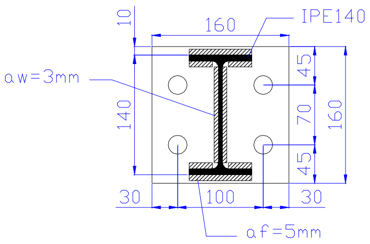


Figure 11. Dimensions of the endplates.

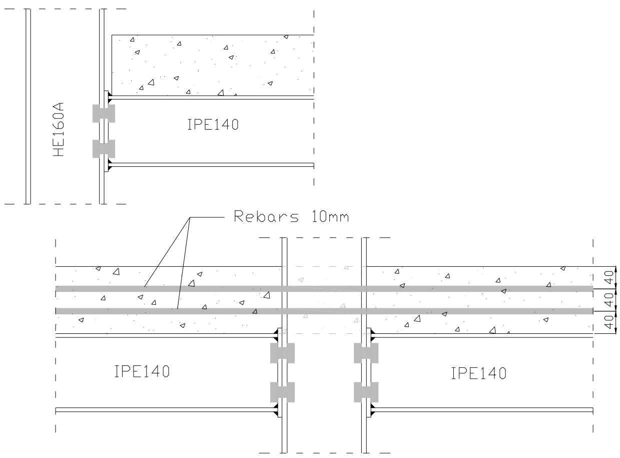


Figure 12. External and internal composite joints.

## 4.3 *From the actual building to the tested substructure*

Within the RFCS project, testing of the full reference composite frame may not be contemplated. So, a substructure has been extracted from the actual frame (Demonceau et al, 2006b). As previously mentioned, this substructure should conform with the dimensions of the testing slab but also to exhibit a similar behaviour than the one of the actual frame.

To achieve this goal, the bottom storey is isolated from the actual building, but the width of the external spans is then reduced, as illustrated in Figure 13.

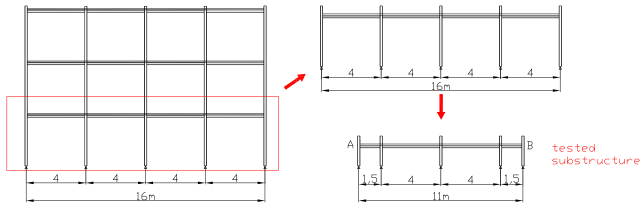


Figure 13. From the actual frame to the tested substructure

The width of the concrete slab is equal to 500 mm. It is fixed so as to ensure that, during the loading, the distribution of the stresses in the concrete is as uniform as possible; in fact, 500 mm corresponds to the value of the effective width of the concrete slab (under hogging moments) in the actual building, according to Eurocode 4.

The 10 mm rebars used in the actual frame (see Section 4.2) are here substituted by 8 mm ones; the objective is to increase the probability to develop a large number of small cracks in the slab, under hogging beam moments, instead of few big cracks and so to allow for more local ductility.

Besides that the distance between the first headed stud and the face of the column flange is larger than what is usually adopted and the amount of longitudinal reinforcement within this area is kept constant (see Figure 14); as a consequence, the slab is subjected to constant tension forces in this zone, what results in an especially high ductile behaviour. This specific detailing has been investigated at Stuttgart University (Kuhlmann et al, 2004) and its efficiency has been demonstrated.

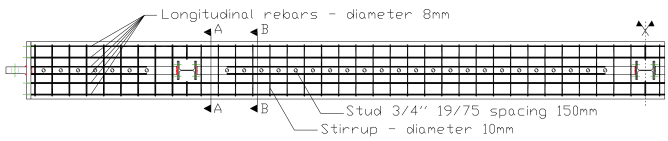


Figure 14. Reinforcement and studs layouts.

Column bases are assumed to be pinned (Figure 15). Teflon elements are used so as to limit the friction between the column steel supports and the pins during the loading.

The composite joints in the substructure are the same than in the actual building (Figures 11 and 12). Only the external beams are simply connected to the external columns (as shown in Figure 16) so as to limit the number of parameters which could influence the response of the internal beams during the test.

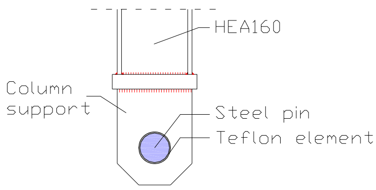


Figure 15. Actual hinges at the column supports.

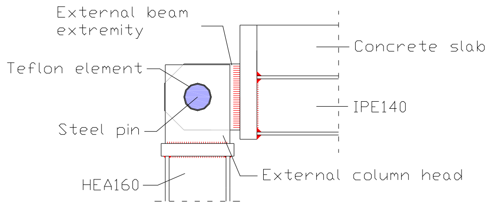

Figure 16. Actual hinges at the external beam-to-column joints.

As previously said, the response of the substructure should be as close as possible to the one of the reference frame. But by reducing the length of the external beam spans and placing hinges at the external joints, a key element is modified: the frame restraint (*K* factor), which strongly influences the catenary action.

That is why lateral restraints are provided each side of the substructure (see point A and B in Figure 13) so as to simulate the actual frame restraints. Restraints are provided on both sides of the substructure in order to induce a symmetrical response of the substructure during the test (see Figure 17); this should facilitate the application of the loads and the measurements during the test.

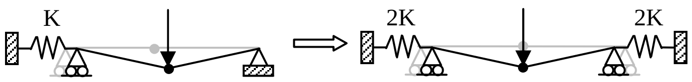

Figure 17. From the unsymmetrical actual behaviour to the symmetrical test behaviour.

In practice, the restraints will be brought by two horizontal calibrated jacks (Figure 18); the restraint will be assumed to be elastic until the end of the test.

## 5 CONCLUSIONS

In this paper, the global strategy defined at Liège University and adopted within the European project “Robust structures by joint ductility” is described for the study of the  behaviour of steel and composite structures under exceptional events.

In this project, Liège University covers in particular the problems related to the loss of a column in a residential or office steel or composite building.

First numerical investigations have been achieved and the main results have been presented; the objective was to study the influence of some key parameters on the structural response of the building. The experimental testing of a substructure is also planned in a near future; the specimen to be tested, the loading path to be followed and the objectives to reach are described.

The final objective of these works is validate analytical models, the use of which could allow the derivation of design requirements for robust structures, in the specific case of the loss of a column. These analytical models are still in development at Liège University.

Also, an experimental test on a composite substructure simulating the loss of an internal column is planned to be performed at Liège University. This test has been described in details. It is expected to perform this test in July 2006; first experimental results could be presented during the ICMS conference in Brasov, Romania, September 2006.

## 6 REFERENCES


BS 5950-1:2000. 2001. Structural use of steelwork in building – Part 1: Code of practice for design – Rolled and welded sections.

Demonceau J.F. 2006. Loss of a column in an office or residential building frame – Numerical investigation of the steel “two-beams” system. Internal report of the RFCS project “Robust structures by joint ductility - RFS-CR-04046”.

Demonceau J.F. & Jaspart J.P. 2006. Predesign of the substructure to be tested at the University of Liège – draft 4. Internal report of the RFCS project “Robust structures by joint ductility - RFS-CR-04046”.

Demonceau J.F. & Jaspart J.P. 2006b. From the “actual” composite building to the tested substructure – draft 1. Internal report of the RFCS project “Robust structures by joint ductility - RFS-CR-04046”.

EN 1991-1-1. 2002. Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings. April 2002.

ENV 1991-2-7. 1998. Eurocode 1: Basis of design and action on structures – Part 2-7 : Accidental actions due to impact and explosions. Final draft June 1998.

Kuhlmann U. & Schäfer M. 2004. Innovative verschieblichte Verbund-Rahmen mit teiltragfähigen Verbund-Knoten. Forschung für die Praxis P 505. Forschungsvereinigung Stahlanwendung e.V. im Stahl-Zentrum.

NBN EN 1994-1-1. 2005. Eurocode 4: Calcul des structures mixtes acier-béton – Partie 1-1: Règles générales et règles pour les bâtiments. February 2006.

prEN 1991-1-7. 2004. Eurocode 1 – Action on structures – Part 1-7: General actions – Accidental actions. Final project team draft (stage 34), 9 July 2004.

UFC 4-023-03. 2005. Unified Facilities Criteria (UFC) - Design of buildings to resist progressive collapse. Department of Defence, USA. 25 January 2005.


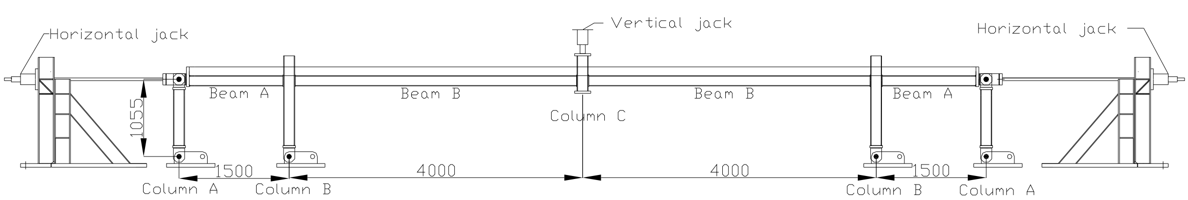

Figure 18. Configuration of the substructure test to be performed at Liège University