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subjected to Exceptional or Accidental Actions
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NUMERICAL, ANALYTICAL AND EXPERIMENTAL INVESTIGATIONS ON THE RESPONSE OF STEEL AND COMPOSITE BUILDINGS FURTHER THE LOSS OF A COLUMN

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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 exceptional events. According to Eurocodes and some different other national design codes, the structural integrity of civil engineering constructions should be ensured through appropriate measures but, in most of the cases, no precise practical guidelines on how to achieve this goal are provided. A 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 ensuring 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 has been performed at Liège University. Present paper describes analytical, numerical and experimental investigations 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 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, a 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 ensuring 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 and/or axial forces.

The partners involved in this previously mentioned project are:

- Stuttgart University, Germany;
- Liège University, Belgium;

- ArcelorMittal Long Carbon Europe R&D, Luxembourg;
- Feldmann + Weynand Ingenieure, Germany and;
- Trento University, Italy.

The present article which summarizes works performed at Liège University 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 are described in Section 3 and;
- finally, in Section 4, the substructure test is presented together with the main results.

2 EXCEPTIONAL EVENTS COVERED AND STRATEGY ADOPTED

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 is 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 could 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 have been shared among the partners:

- Trento University is in charge of “event 6” (earthquake);
- Long Carbon R&D 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);
- F+W 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; one of the objectives is to validate the numerical FEM tool.

Finally, through parametrical studies carried out numerically for the selected events, robustness requirements are aimed to be derived.

In Figure 1, the strategy adopted at Liège University is summarized.

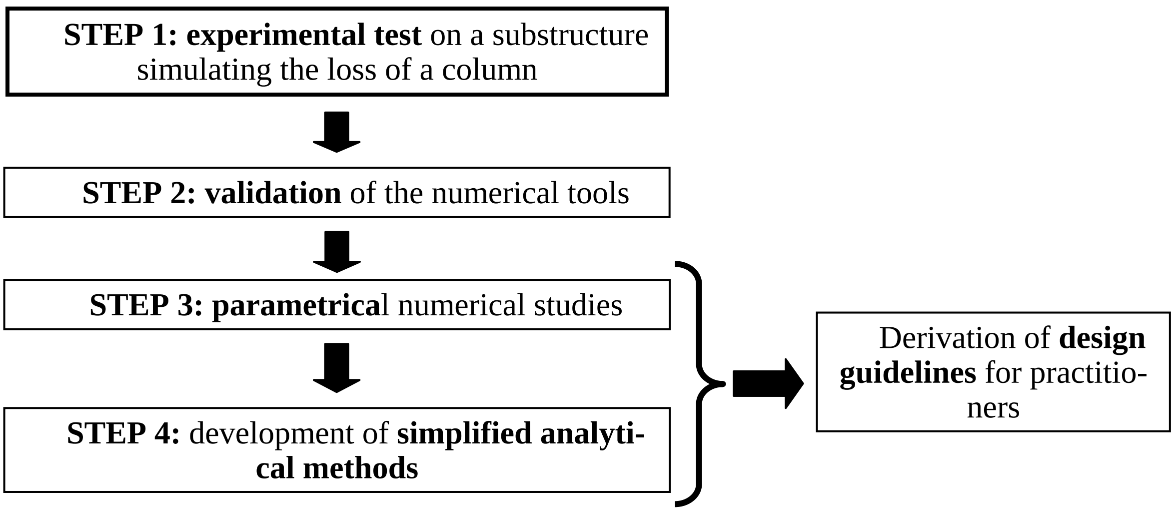


Figure 1: Global strategy followed in Liège.

In the next section, first analytical and numerical investigations performed at Liège University on “Event 1” (as a contribution to STEP 3 and 4 – see Figure 1) are described.

3 LOSS OF A COLUMN IN A BUILDING – FIRST ANALYTICAL AND NUMERICAL INVESTIGATIONS

3.1 Introduction

As mentioned in the previous section, analytical and numerical investigations have been conducted on “Event 1” dealing with the loss of a column in office or residential building frames, as illustrated in Figure 2.

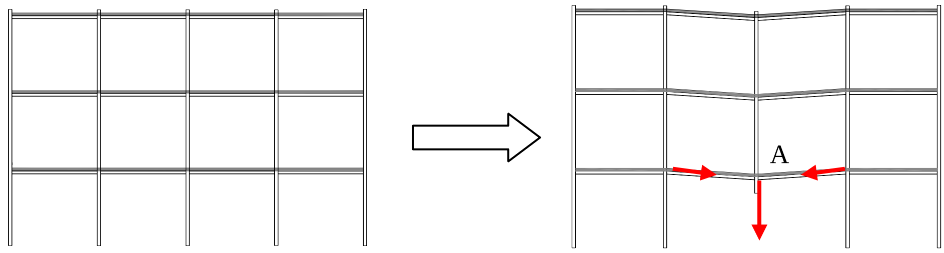


Figure 2: Loss of a column in a residential or office building frame.

At first, before the event, the beam-to-column joints and the beams are mainly subjected to bending moments and shear forces. When the column looses its carrying capacity (because it is impacted, for instance), catenary action develops in the beams (as illustrated in Figure 3); 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 beam-to-column joints (or in the beam) are qualitatively illustrated in Figure 4.

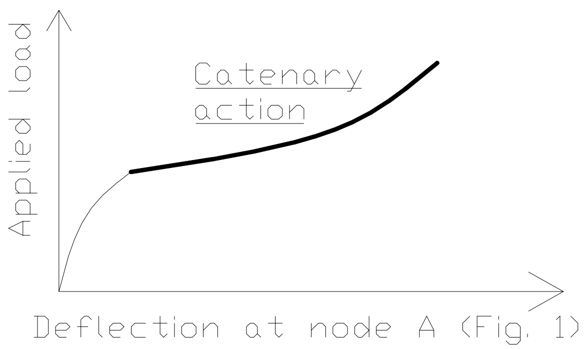


Figure 3: Development of a catenary action in the structure – “applied load/beam deflection” curve.

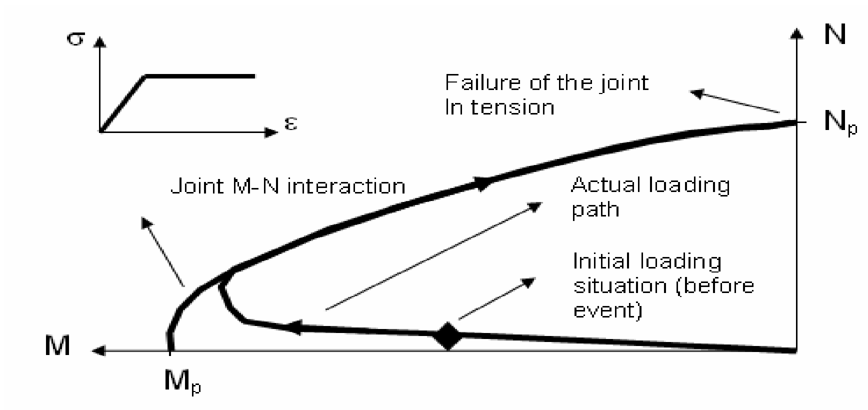


Figure 4: Actual loading in the joint or in the beam until failure.

At the end, the joints and the beam 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 (axial resistance) is not reached in the joints or in the beam, 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; so the joints are the "weak" elements when catenary actions develop.

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 (as for some seismic design procedures), 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 simpler one limited to the study of a single "two-beams" system (Figure 5), 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.

And as a result, it may be concluded that by the study of the equivalent "two-beams" system may be adequately substituted to the study of the whole frame.

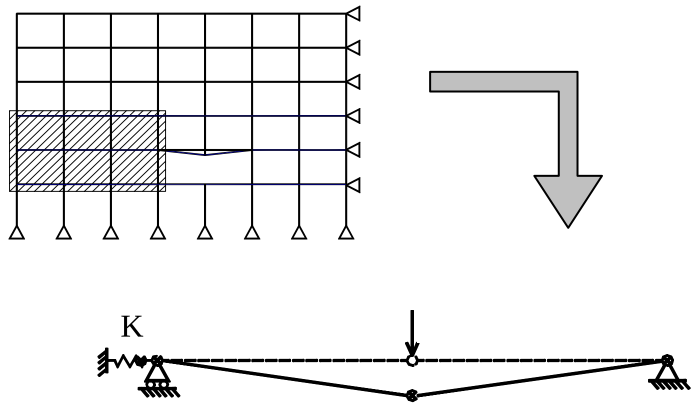


Figure 5: Global and local modelling of "event 1" (loss of a column).

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 6. The system is loaded by a uniformly distributed load; the total length of the system is equal to 4m.

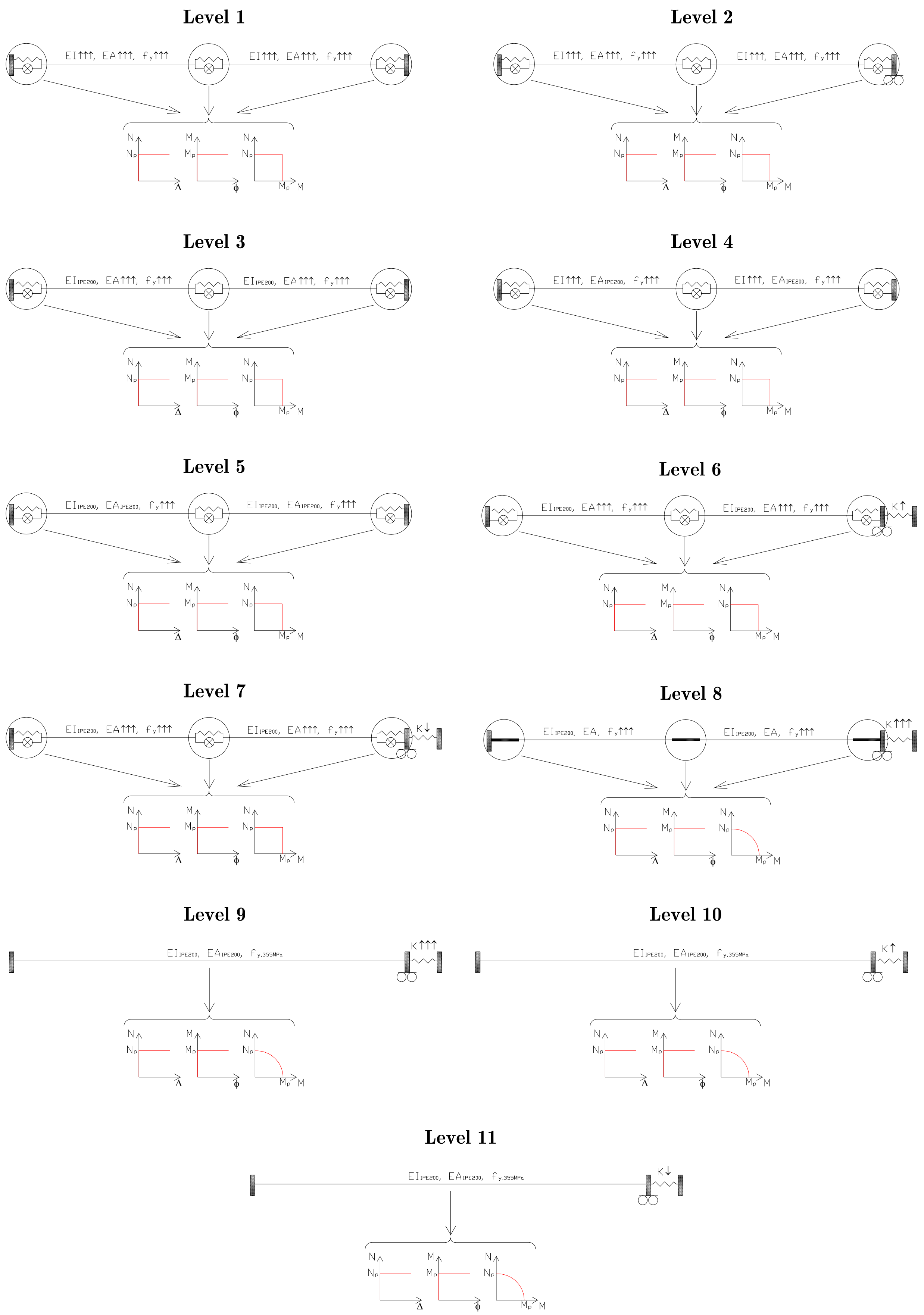


Figure 6: Investigated levels for the parametrical study of the subsystem

The numerical investigations are performed with the homemade finite element software FINELG developed at Liège University (ArGEnCo 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 7, 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_p.

The results obtained for the different levels are summarised in Figure 8.

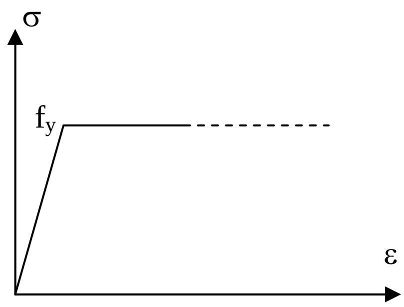


Figure 7. Infinite ductility assumption for steel material.

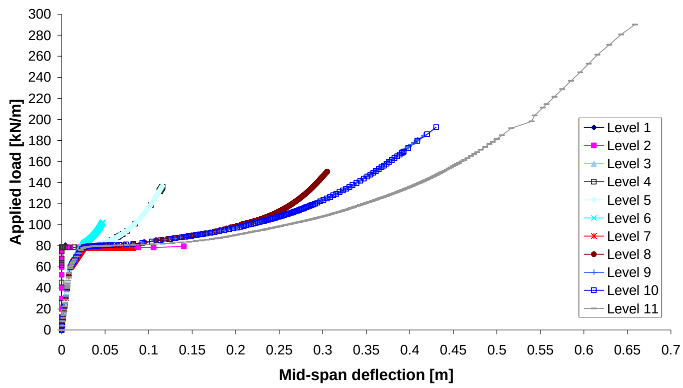


Figure 8. Obtained results for the different investigated levels – "applied load/mid-span beam deflection" curves.

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, 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, 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. Also, as previously predicted (see Section 3.1), 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. 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” sub-system;
- the derivation of design requirements in terms of resistance and ductility;
- the validation of the use of a “two-beams” sub-system.

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 third author of the present paper and validated through few hundreds of numerical simulations. In his 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 (Luu, 2008).

4 DESCRIPTION OF THE EXPERIMENTAL TEST ON A SUBSTRUCTURE (DEMONCEAU ET AL, 2006c)

4.1 Introduction

Within the RFCS European project, a test on a substructure simulating the loss of a column in a composite building has been performed in Liège. 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 was 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).

Then, the loading path followed for the test is described in Section 4.4 and, finally, the main test results are presented in Section 4.5 where the main phenomena observed during the test are also described.

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 9.

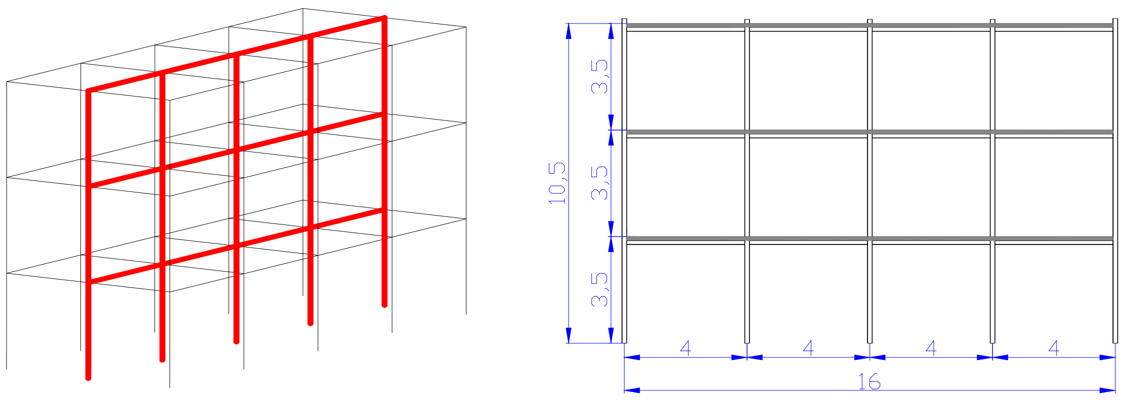


Figure 9: General layout of the reference composite building.

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 10.

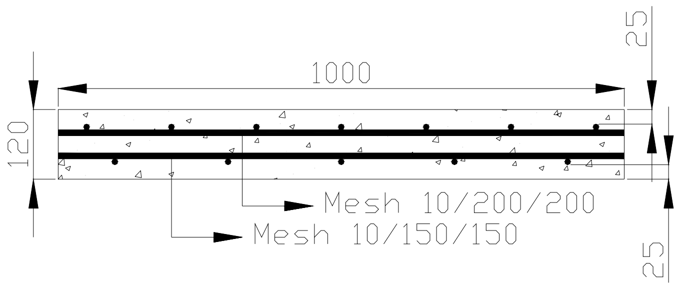


Figure 10: Slab properties

The composite beam cross-section is seen in Figure 11. A S355 IPE140 profile is used and a full shear connection is assumed between the steel profile and the concrete slab.

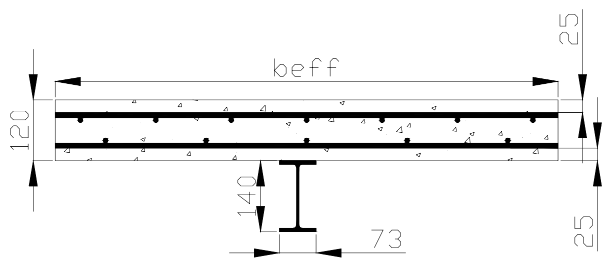


Figure 11: Composite beam cross-section.

→ The columns are steel ones (S355 HEA160).

→ Partial-strength and semi-rigid joints are considered (Figure 12 and Figure 13). The properties of these joints allow them to exhibit a ductile behaviour (with account of possible overstrength effects).

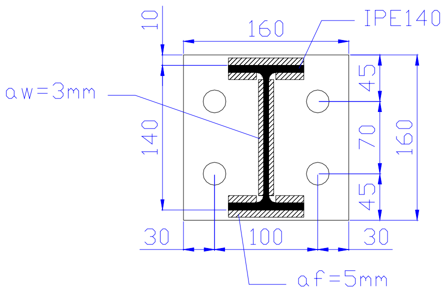


Figure 12: Dimensions of the end-plates.

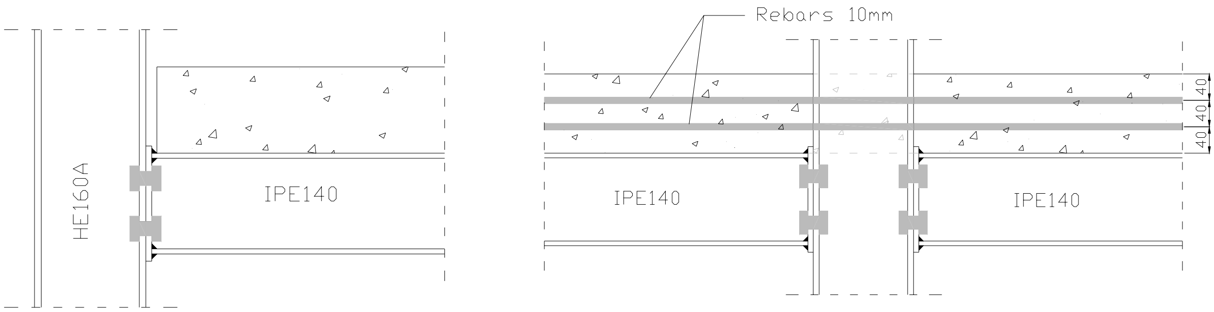


Figure 13: External steel joints and internal composite joints.

4.3 From the reference composite 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 to the dimensions of the testing slab but also exhibit a similar behaviour than the one which would be observed in 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 14.

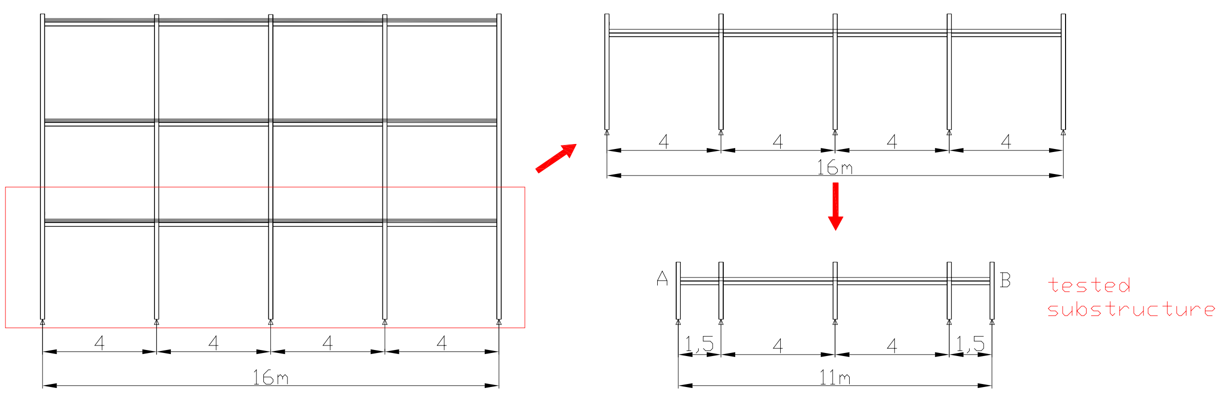


Figure 14: From the actual frame to the tested substructure

The width of the concrete slab is equal to 500mm. 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, 500mm corresponds to the value of the effective width of the concrete slab (under hogging moments) in the actual building, according to Eurocode 4.

The 10mm rebars used in the actual frame (see Section 4.2) are here substituted by 8mm 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 15); 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 the University of Stuttgart (Kuhlmann et al, 2004) and its efficiency has been demonstrated.

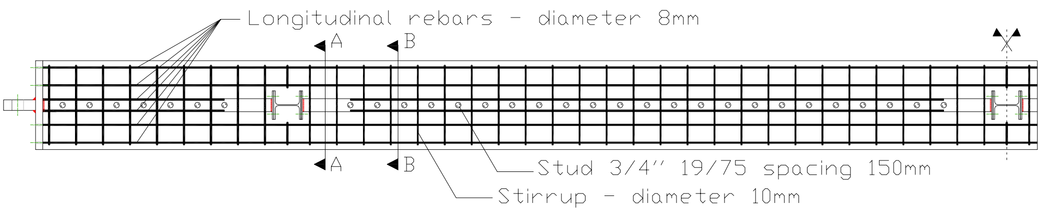
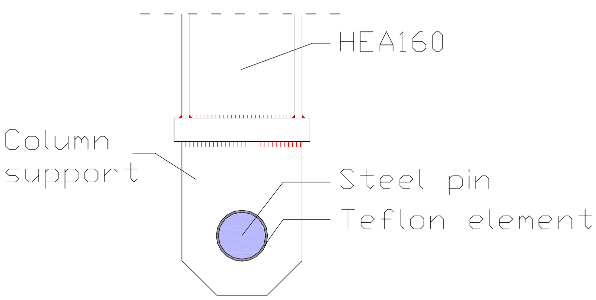


Figure 15: Reinforcement and studs layouts.

Column bases are assumed to be pinned (Figure 16). 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 12 and 13). Only the external beams are simply connected to the external columns (as shown in Figure 17) so as to limit the number of parameters which could influence the response of the internal beams during the test.



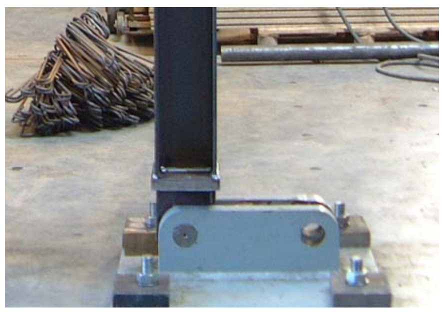


Figure 16: Actual hinges at the column bases.

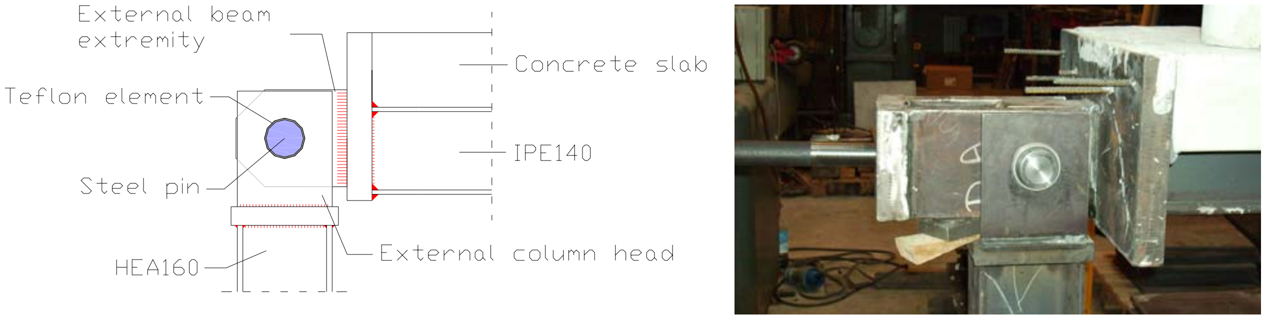


Figure 17: Actual hinges at the external beam-to-column joints.

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) (see Section 3.1), 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 14) 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 18); this should facilitate the application of the loads and the measurements during the test.

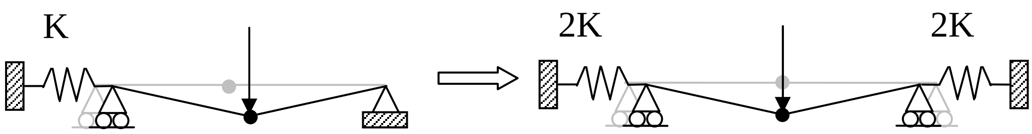


Figure 18: From the unsymmetrical actual behaviour to the symmetrical test behaviour.

In practice, the restraints will be brought by two horizontal calibrated jacks (Figure 19); the restraint will be assumed to be elastic until the end of the test.

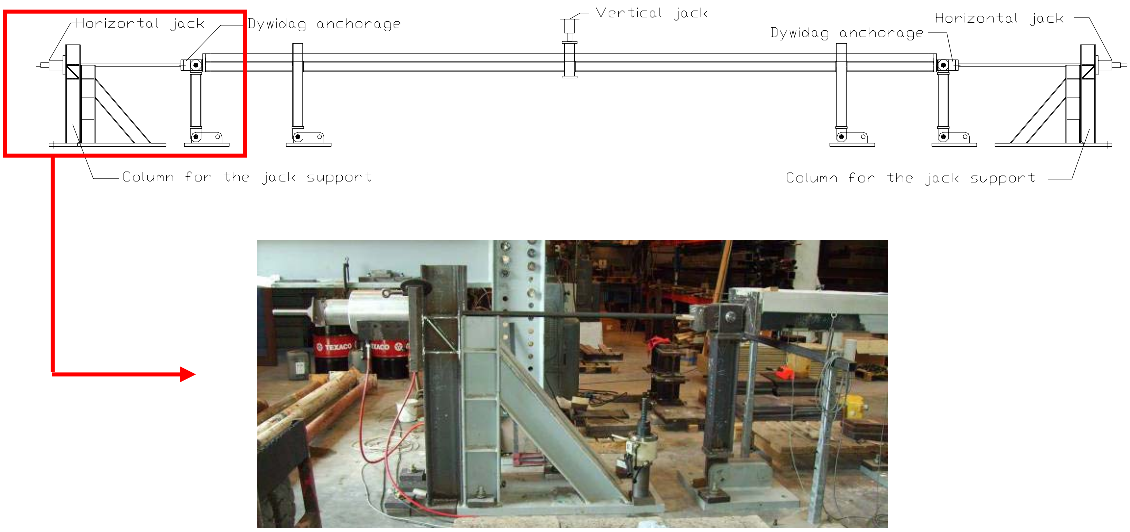


Figure 19: Configuration of the substructure test to be performed at Liège University

4.4 Loading path

The load path during the test is as follows:

- The substructure is first preloaded with an uniformly distributed load on the internal beams to simulate the reaction of the concrete slab on the main frame in the actual building (see Figure 9); during this preloading, two locked jack are placed at the middle of the substructure to simulate the presence of the column, as illustrated in Figure 20.
- In a second step, the support brought by the jacks is progressively removed by unlocking the jacks; when the latter are removed, the free deflection of the system is observed. The further procedure is to apply a vertical force with a jack on the column thus further deformation will occur (see Figure 21).

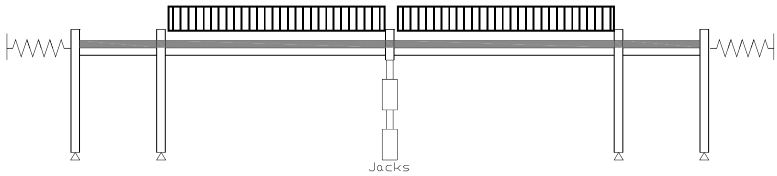


Figure 20: Column at the middle simulated by two locked jacks

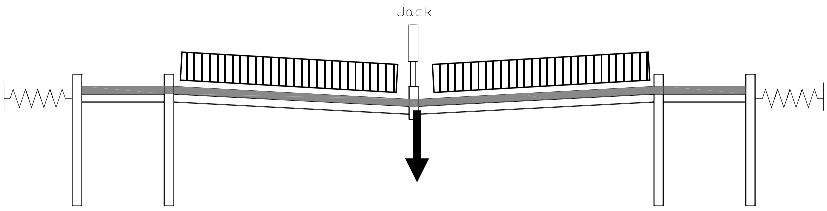


Figure 21: Application of a vertical load with a vertical jack until collapse

4.5 Experimental test results

As explained before, an uniformly distributed load is first applied on the substructure (by means of steel plates and concrete blocks). The vertical reaction which is associated to the uniformly distributed load and to the self-weight of the substructure is equal to 33,5kN as illustrated in Figure 22 (value of the load at point “O”) presenting the evolution of the vertical displacement at the middle of the structure according to the vertical load. After the application of the uniform load, the jacks at the middle are unlocked and progressively removed. The system is completely released when a deflection of 29mm is reached. At this stage, first cracks at the vicinity of the external joints are observed and first steel yielding is seen in the column web panel of the internal composite joint. This first step of the test is illustrated by part “OA” of the curve presented in Figure 22; from the latter, it can be seen that the structure still be in the elastic range when “A” is reached.

Then, a vertical load is progressively applied until collapse of the tested specimen. During this stage, two “unloading-reloading” are performed as illustrated in Figure 22. From point “A” to “B”, the substructure enters in the yielding stage to finally form a beam plastic mechanism at point “B” with formation of the plastic hinges at the joint level. During this stage, the cracks in the vicinity of the external composite joints are more pronounced and yielding of some steel components of the joints is observed (column web and beam flange in compression); also, for the internal composite joint, a detachment between the end-plate and the column flange at the bottom can be seen.

From point “B” to “C”, a plateau is observed. During this stage, the concrete cracks in the vicinity of the external composite joints continue to develop and yielding spreads in the steel components; one important phenomenon to be mentioned is the concrete splitting in the vicinity of the internal composite joint during this stage. At point “C”, membrane forces begin to develop as confirm by the shape of the curve “CD” in Figure 22.

When the point “D” is reached, the longitudinal rebars in the vicinity of the external composite joints are completely destroyed and the concrete at the internal joint is fully spited (Figure 23); at this moment, the joints work as steel ones. The yielding also spreads in the different steel components of the internal and external composite joints. At point “D”, a loss of stiffness is observed which is linked to the loss of the longitudinal rebars in the vicinity of the external joints; indeed, when these rebars are lost, the tensile stiffness of the external joints decreases, phenomenon which affects the development of the membrane forces.

At the end of the test (point “E”), a maximum vertical displacement of 775mm is reached for an applied vertical load of 114kN; the deformation of the specimen at this stage is presented in Figure 24. The maximum horizontal displacement at each side of the structure is equal to 45mm for a horizontal load of 147kN. The main components which have been activated within the joints are:

- For the external composite joints: yielding of the column web in compression, the beam flange and web in compression, the column flange in bending.
- For the internal composite joint: yielding of the column web in tension (Luders bands) associated to the membrane forces, column flange in bending, beam flange and web in tension.

The test was stopped with the apparition of cracks at the bottom weld between the IPE140 profile and the end-plate at the internal composite joint for a maximum rotation of 190mRad.

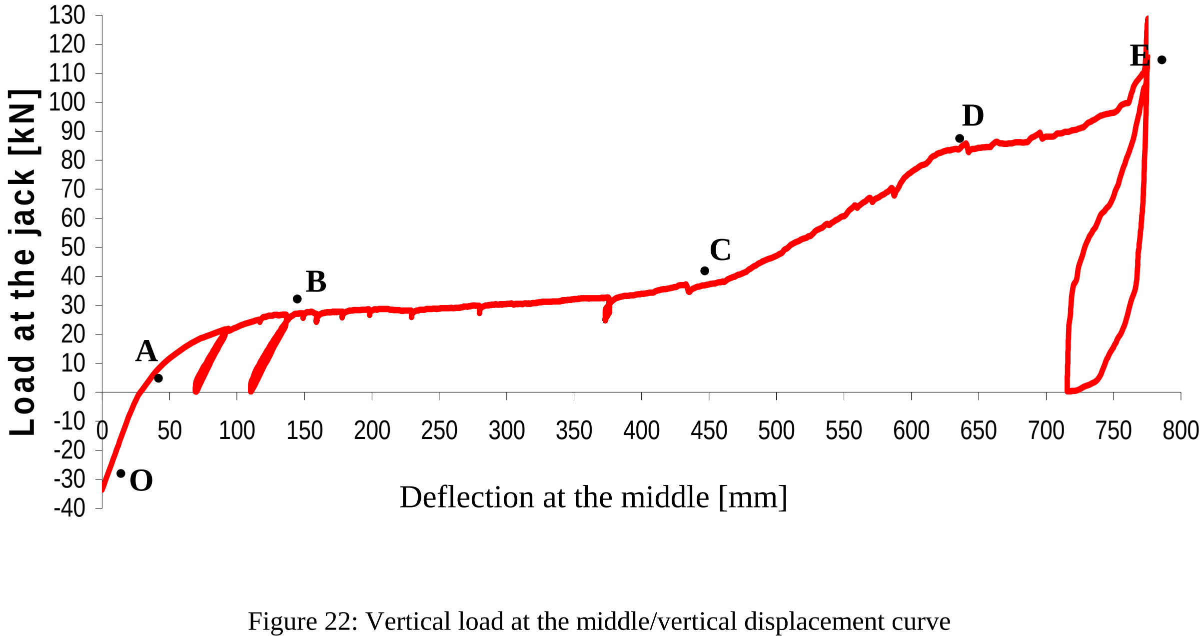


Figure 22: Vertical load at the middle/vertical displacement curve

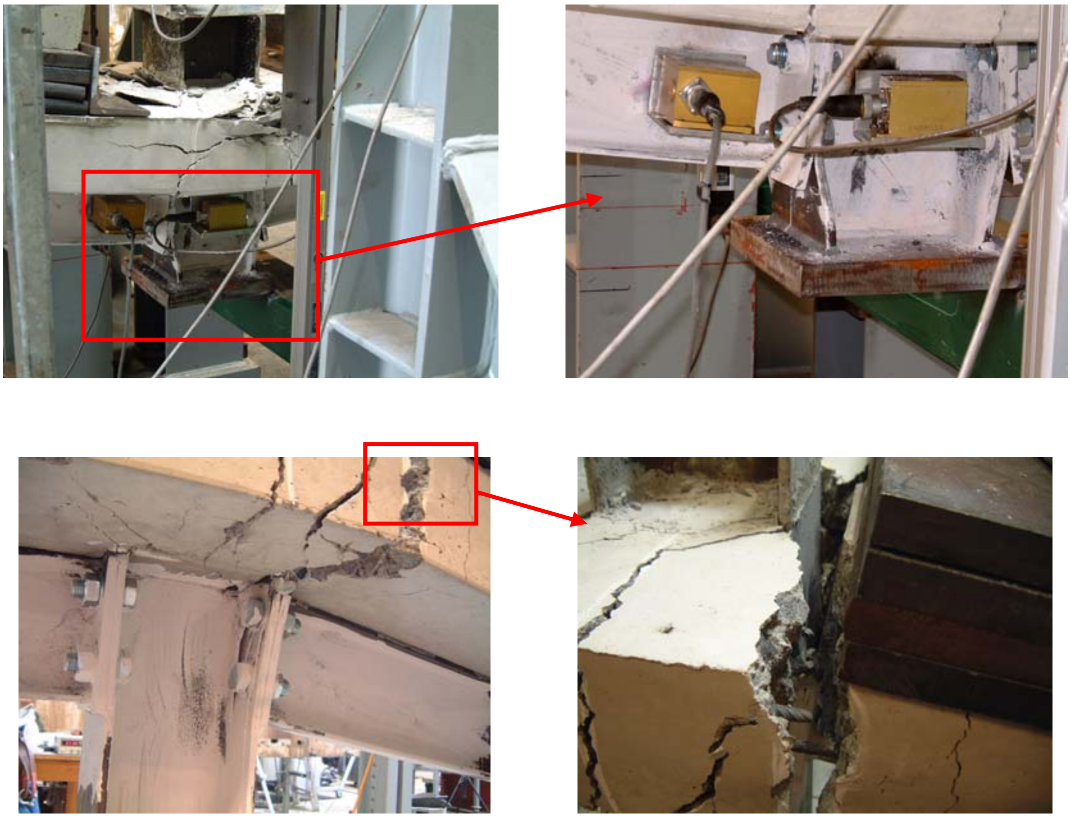


Figure 23: Internal and external composite joints at point “D” of Figure 22

Figure 24: Deformation of the specimen at the end of the test.

Besides that, experimental tests in isolation have been performed at Stuttgart University on the composite joints of the substructure, respectively under hogging or sagging bending moments and tensile axial forces; finally tests on joint components have been realized at Trento University. So as to be able to compare the results obtained in the laboratories, all the steel elements (profiles, plates and rebars) were provided by the same companies and came from the same rolling. A unique chain of consistent experimental results is so obtained.

5 DEVELOPMENT AND VALIDATION OF AN ANALYTICAL PROCEDURE TO PREDICT THE RESPONSE OF COMPOSITE JOINTS SUBJECTED TO COMBINED MOMENTS AND AXIAL LOADS

As previously mentioned, the structural joints during the exceptional event are subjected to combined bending moments and axial loads. In the PhD thesis of Cerfontaine (Cerfontaine, 2003), an analytical procedure is proposed to predict the response of steel joints subjected to such a loading. The proposed method is based on the component method which is the recommended method in the Eurocodes for the design of joints subjected to bending moments.

In (Demonceau, 2008), this method is extended to composite joints. The particularity of composite joint configurations is the fact that two main additional components are activated if compare to steel ones: the slab rebars in tension and the concrete slab in compression. As the analytical procedure presented in (Cerfontaine, 2003) is based on the component method concept, the latter is easily extended to composite joint configurations by including the behaviour of the two additional components into the procedure. However, the characterisation of the component "concrete slab in compression" is not yet available in the actual codes; accordingly, an analytical method to characterise this component in terms of resistance and stiffness is proposed and validated in (Demonceau, 2008).

The extended method is validated through comparisons to results coming from experimental tests performed at Stuttgart University on the tested substructure joint configuration (Kuhlman & al., 2007). These results are compared in Figure 25 where two analytical curves are reported:

- one called "plastic resistance curve" which is computed with the elastic resistance stresses of the materials and;
- one called "ultimate resistance curve" which is computed with the ultimate resistance stresses of the materials.

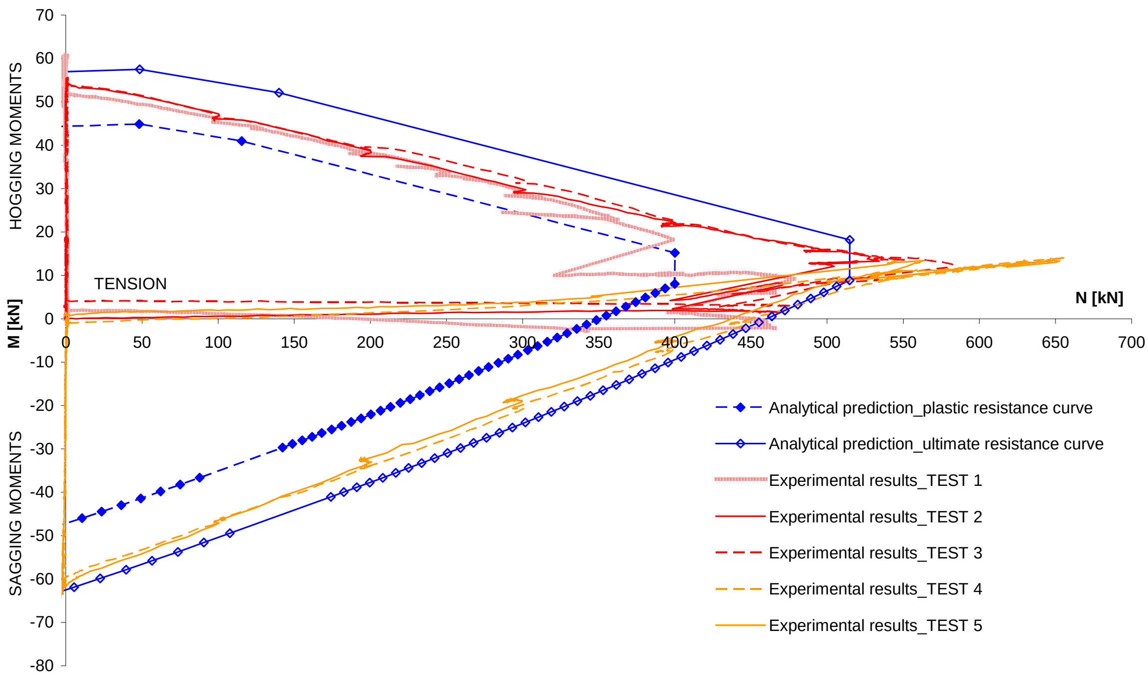


Figure 25: Comparison of the resistance interaction curves

According to Figure 25, the computed analytical curves are in very good agreement with the experimental results. Indeed, the experimental curves are between the plastic and ultimate analytical resistance curves what is in line with the loading sequence followed during the tests. The fact that the maximum tensile load reached during the experimental tests is higher than the one analytically predicted can be explained by membrane forces developing in some joint components in bending, forces which are not taken into account in the analytical method.

6 DEVELOPMENT AND VALIDATION OF AN ANALYTICAL PROCEDURE TO PREDICT THE RESPONSE OF THE TWO-BEAMS SYSTEM

As explained earlier, the two-beams system (see Figure 5) may adequately reproduce the global response of a frame further to a column loss.

The objective is to develop an analytical procedure for the prediction of the response of the substructure in the post-plastic domain, i.e. after the formation of the beam plastic mechanism; as a direct consequence, the here-proposed analytical model is based on a rigid-plastic approach. Also, as the deformations of the substructure are significant and influence its response, a second-order analysis is conducted.

The parameters to be taken into account are presented in Figure 26:

- p is the (constant) uniformly distributed load applied on the storey modelled by the simplified substructure and the concentrated load Q is the force acting in the upper column;
- L is the total initial length of the simplified substructure;
- Δ_Q is the vertical displacement at the concentrated load application point;
- δ_K is the deformation of the horizontal spring simulating the lateral restraint coming from the indirectly affected part;
- δ_{N1} and δ_{N2} are the plastic elongations at each plastic hinges;
- θ is the rotation at the plastic hinges at the beam extremities.

In addition, the axial and bending resistances at the plastic hinges N_{Rd1} and M_{Rd1} for the plastic hinges 1 and 4 and N_{Rd2} and M_{Rd2} for the plastic hinges 2 and 3 have also to be taken into account (it is assumed that the two plastic hinges 1 and 4 and the two plastic hinges 2 and 3 - see Figure 26 - have respectively the same resistance interaction curves).

In order to be able to predict the response of the simplified substructure, the parameters K and F_{Rd} have to be known; these parameters depend of the properties of the indirectly affected part. As already said, analytical procedures are proposed in (Luu, 2008) to predict these structural characteristics.

The results obtained with the so-developed analytical procedure are compared to the substructure test results in Figure 27. In this figure, it can be observed that a very good agreement is obtained between the analytical prediction and the experimental results, what validates the developed method. More details about the developed method are available in (Demonceau, 2008).

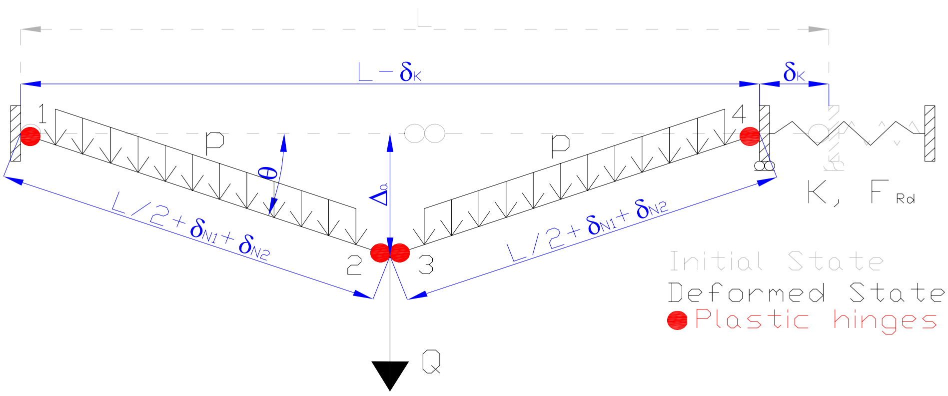


Figure 26: Substructure

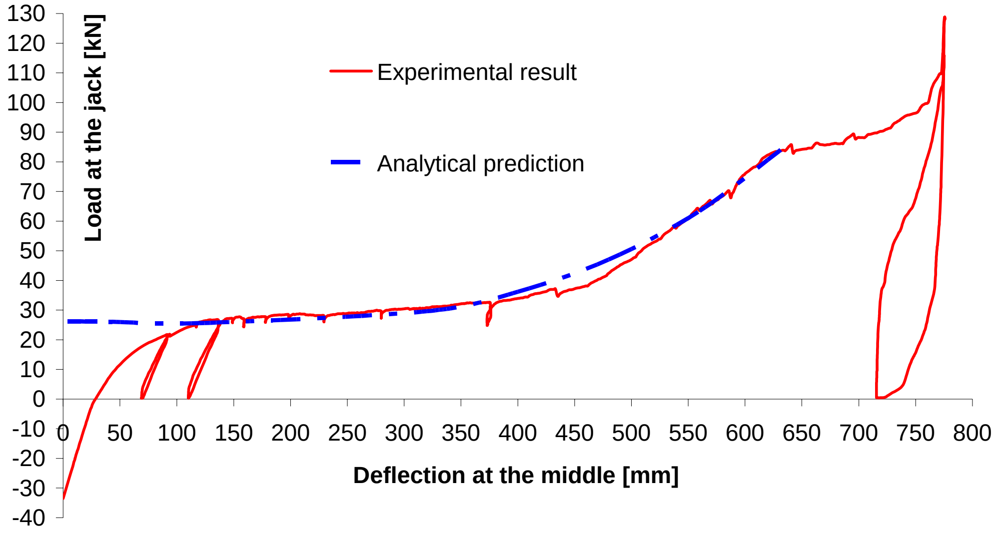


Figure 27: Comparison between analytical prediction and experimental test

7 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.

Also, an experimental test on a composite substructure simulating the loss of an internal column has been performed. This test has been described in details with the main obtained results. The performed test was successful and all the phenomena under investigations were registered. Indeed, the development of the catenary action in the system was observed and the registered curves confirmed the development of membrane forces in the beams. Also, the composite joints loaded by combined tensile forces and bending moments exhibited a ductile behaviour as expected.

Finally, analytical models aimed at predicting the full non-linear response of the structure further to the loss of a column have been developed and validated through comparisons with test results. The last step should now consist in deriving design requirements for robust structures, in the specific case of the loss of a column. This work is presently in progress in Liège.

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