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RECENT STUDIES CONDUCTED AT LIEGE UNIVERSITY IN THE FIELD OF COMPOSITE CONSTRUCTION

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2 Introduction

The present informal document summarises recent or ongoing studies conducted at Liège University in the field of steel-concrete composite constructions; they are summarised in a Table presented at the following page.

Only the main results are presented herein. For all the addressed topics, references to documents, reports and articles where details are available will be made; a copy of these documents can be obtained by contacting us.

At the end of each paragraph, a section dedicated to some perspectives is given. § 9.2 summarises the main topics in which investigations could be conducted in the future; these perspectives are also reported in the Table here after.

The activities related to the response of composite structures subjected to seismic or fire actions (mainly covered in Liège by Prof. Plumier and Prof. Franssen respectively) are not reported here as they are covered by other more "transversal" technical committees of ECCS (TC3 for the fire and TC13 for the seismic action).

The "composite joints" topic is included here even if a technical committee relative to the behaviour of joints exists within ECCS (TC10); indeed, in the past, all the developments relative to composite joints were discussed and accepted within TC11 and not within TC10. Of course, it is up to the present TC11 members to decide whether this topic has or not to be considered within TC11.

Finally, the robustness of composite structures and in particular, the robustness of the structures subjected to the exceptional event "loss of a column further" is also considered herein as this topic is not yet covered by any other technical committee. Again, it is up to the TC11 members to decide whether this topic has to be discussed within TC11.

Topics		Recent developments in Liège	Ongoing activities in Liège	Perspectives
Cross-sections		-	-	Classification of sections with account of the concrete properties (§ 3.4 & § 9.2)
Structural elements	*Slabs*	-	-	-
	Beams	Stability of beams during the erection phase (§ 6)	PhD thesis of Rodrigues (§ 6)	Effect of partial interaction on the beam deformation at ELS (§ 9.2)
	Columns	-	ATTEL project (§ 7) HSS-SERF project (§ 5.6)	-
	Joints	Study of the behaviour of joints subjected to M and to combined M-N (§ 5)	HSS-SERF project (§ 5.6)	Behaviour of joints subjected to combined M-N-V (§ 9.2) Prediction of the joint ductility (§ 5.6 & § 9.2)
Structures	*Static*	Behaviour of composite sway frames (§ 3)	-	Global analysis of composite frames (§ 9.2)
	Seismic	TC 13	TC13	TC13
	Fire	TC3	TC3	TC3
	Robustness	Loss of a column in a composite building (§ 4)	ROBUSTFIRE project (§ 4.5)	Dynamic effects associated to a column loss (§ 9.2)

3 Behaviour of composite sway frames

3.1 Context

Most steel-concrete composite structures are laterally restrained by efficient bracing systems, such as concrete cores. This practice does not favour the use of composite structures. Indeed, once concrete construction companies are involved into major parts of a building, the reason for using composite structures for subsequent parts is often questionable.

As an alternative, moment resisting frames, without bracing systems, offer a flexible solution to the user of the buildings, especially for the internal arrangements and the exploitation of the buildings. When sufficient stiffness and strength with regard to lateral forces are achieved, such frames offer a structural solution, which can resist lateral loads. In seismic regions, properly designed moment resisting frames are the best choice regarding the available ductility and the capacity to dissipate energy. This is stated in Eurocode 8 devoted to earthquake engineering in which high values of the behaviour factor are recommended.

Obviously, the construction of tall buildings and large industrial halls without wind bracing systems is susceptible to make global instability a relevant failure mode; this is not yet well covered by Eurocode 4 and other references which mainly deal with composite construction under static loading. Indeed, as far as the European codes are concerned, Eurocode 4 contains design procedures for non-sway composite buildings and gives design rules for composite slabs, beams, columns and joints; composite sway frames are allowed in Eurocode 4 but no information on how to design them are given.

That is the reason why two European research projects on composite sway frames, in which Liège University was involved, have been conducted recently:

- the first one involving seven institutions and titled “Applicability of composite structures to sway frames” (Contract N° 7210-PR-250) was funded by the European Coal and Steel Community (ECSC) for three years (from July 2000 to June 2003);
- the second one was an Ecoleader project (from July 2000 to June 2003) titled “3-D full scale seismic testing of a steel-concrete composite building at ELSA” involving six institutions.

The objective of these projects was to provide background information on the behaviour of such frames under static and seismic loads and to provide simplified design rules as a result of experimental, numerical and analytical investigations.

Concerning the experimental investigations, two main experimental tests were planned within these projects:

- a 3-D composite building under dynamic loading tested in Ispra (Italy), called the “Ispra” structure;
- a 2-D composite frame under static loading tested in Bochum (Germany), called the “Bochum” structure.

Beside these tests, cyclic and static tests on isolated joints were also performed in different European laboratories (involving the Argenco Laboratory of liege University) so as to get the actual behaviour of the constitutive structural joints of these two structures; indeed, the behavioural responses of the joints are known to significantly influence the global behaviour of structures.

Concerning the analytical and numerical investigations performed at Liège University, they were divided in three parts:

- validation and development of an analytical procedure to predict the response of composite joints subjected to hogging and sagging bending moments (this aspect will be addressed in § 5);
- numerical analyses of existing structures further to the validation of the numerical tools used for the numerical investigations;
- development of design guidance with proposals of simple analytical methods to design composite sway structures.

The conducted investigations are detailed in the PhD thesis of Demonceau [1]. Within this paragraph, only the followed strategy will be briefly presented in § 3.2 and then, in § 3.3, an innovative simplified analytical method able to predict the ultimate load factor of steel and composite sway frames will be introduced.

3.2 Objectives and research steps

Composite sway structures are prone to global instability phenomena and to second-order effects; the latter have to be predicted carefully because they may govern the design. These second-order effects are amplified by an additional source of deformability with regards to steel sway structures: the concrete cracking. Indeed, this effect, which is specific to concrete and composite constructions, tends to increase the lateral deflection of the frame, amplifies consequently the second-order effects and reduces the ultimate resistance of the frames. In other words, for a same number of hinges formed at a given load level in a steel frame and in a composite frame, larger sway displacements are reported in the composite one.

The objective of this work was to investigate the effects of these phenomena on the behaviour of composite sway frames, to highlight the particularities of their behaviour and to propose simplified analytical procedures for the design of such frames; these objectives have been achieved by means of numerical and analytical studies on 2-D plane frames extracted from realistic or actual buildings. The research steps were the followings:

- The behavioural response of the joints is known to significantly influence the global behaviour of sway structures. Accordingly, investigations devoted to the study of the behaviour of composite joints were conducted through experimental and analytical investigations. This point is addressed in § 5.
- Then, the finite element software used for the prediction of the composite sway frame responses has been validated through a benchmark study and through comparison with experimental test results performed in two European laboratories.
- Numerical and analytical investigations performed with the so-validated tool have been realized on frames extracted from actual buildings in order to clearly highlight the particular behaviour of composite sway frames.
- Afterwards, according to the results obtained through these investigations, a new design method for the design of steel and composite sway building frames has been developed and

validated through parametrical studies. The developed method is founded on the Ayrton-Perry formulation which is the method recommended in the Eurocodes to deal with the instability phenomena and permits to compute the ultimate load factor of a sway frame and the associated collapse mode.

The followed strategy is illustrated in Figure 1.

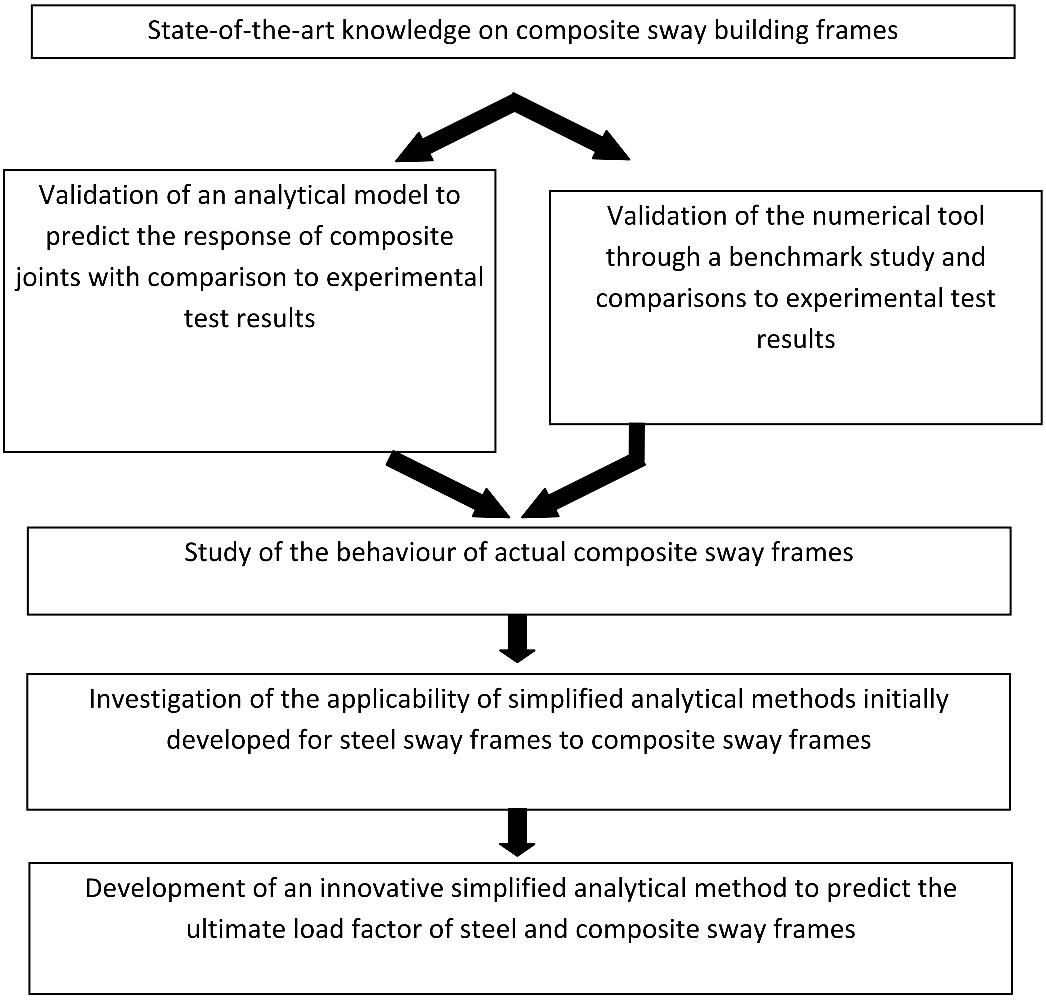


Figure 1. Followed strategy

In the following paragraph, only the developed simplified analytical method will be introduced.

3.3 Developed simplified analytical method

3.3.1 Introduction

In previous investigations [2], the applicability of the "Amplified sway moment method" and the "Merchant-Rankine approach" (respectively based on elastic and plastic design philosophies) to composite sway structures was investigated.

For the "amplified sway moment method", it was demonstrated that:

- A good accuracy of this method is obtained when applied to sway composite structures.

- The simplified method proposed in Eurocode 3 for the computation of λ_{cr} permits to obtain an accurate prediction of the latter which, consequently, do not significantly affect the accuracy of the amplified sway moment method.

For the "Merchant-Rankine approach", it was shown that:

- The conclusions concerning the accuracy of this method which were drawn for steel sway structures still appropriate for composite sway structures:
  - o safe for beam plastic mechanisms;
  - o adequate for combined plastic mechanisms;
  - o unsafe for panel plastic mechanisms.
- The nature of the plastic mechanism considered in the Merchant-Rankine does not always correspond to the one occurring at failure of the frame (computed through a non-linear analysis); this phenomenon is due to the second-order effects which differently influence the yielding of the structure according to the nature of the considered plastic mechanism [2].

According the lack of accuracy and sometimes the unsafe character of the "Merchant-Rankine approach", a new simplified analytical method able to predict the ultimate load factor of a steel or composite sway frame was developed and validated.

The solution proposed here is to develop a procedure based on three formulas, one for each type of plastic mechanisms which could appear in the studied frame (i.e. beam, panel and combined plastic mechanisms):

- Formula1($\lambda_{p,beam}$, λ_{cr}) → $\lambda_{u,beam}$;
- Formula2($\lambda_{p,panel}$, λ_{cr}) → $\lambda_{u,panel}$;
- Formula3($\lambda_{p,combined}$, λ_{cr}) → $\lambda_{u,combined}$.

Through these formulas, three predicted ultimate load factors are computed; the smallest one is then considered as the ultimate load factor of the studied frame: $\lambda_u = \min(\lambda_{u,beam}, \lambda_{u,panel}, \lambda_{u,combined})$.

These new formulas could have been developed from the Merchant-Rankine one; in fact, the actual Merchant-Rankine formula could be used as "Formula3" as it was demonstrated in [2] and in previous studies on steel sway frames that this formula gives satisfactory results for frames with the first-order rigid-plastic mechanism associated to a combined one. Nevertheless, it is chosen to develop these formulas from the Ayrton-Perry formulation (see Table 1), which is already used in the Eurocodes to treat the member instability phenomena (plane buckling, lateral buckling and lateral torsional buckling); this proposal is in agreement with the recommendation of the last draft of Eurocode 3 where it is stated that such formulation should be used to verify "*the resistance to lateral and lateral torsional buckling for structural components such as single members (built-up or not, uniform or not, with complex support conditions or not) or plane frames or subframes composed of such members which are subject to compression and/or mono-axial bending in the plane...*" (§ 6.3.4 (1) of Eurocode 3). A great advantage is that the Ayrton-Perry formulation implicitly permits to respect the limit conditions which are:

- when λ_{cr} is very high, no instability phenomena will appear and the failure occur through the appearance of a plastic mechanism ($\lambda_u \rightarrow \lambda_p$);

- when λ_p is very high, no yielding appears in the frame and the failure occurs through an instability phenomenon ($\lambda_u \rightarrow \lambda_{cr}$).

Table 1. From the Ayrton-Perry formulation (for column buckling) to the formulas to be included in the new simplified analytical design method

Ayrton-Perry formulation – Eurocode 3 Erreur ! Source du renvoi introuvable.	**Formulas included in the new simplified design method for sway frames**
$N_{b,Rd} = \frac{\chi \alpha_{ult,op}}{\gamma_{M1}}$ $\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{op}^{\,2}}}$ $\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}}$ $\phi = 0,5\left[1 + \alpha(\bar{\lambda}_{op} - \bar{\lambda}_0) + \bar{\lambda}_{op}^{\,2}\right]$	$\lambda_u = \chi \lambda_p$ $\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{op}^{\,2}}}$ $\bar{\lambda}_{op} = \sqrt{\frac{\lambda_p}{\lambda_{cr}}}$ $\phi = 0,5\left[1 + \mu(\bar{\lambda}_{op} - \bar{\lambda}_0) + \bar{\lambda}_{op}^{\,2}\right]$

Within this formulation, χ is called the reduction factor and $\bar{\lambda}_{op}$ the non-dimensional relative slenderness. The parameter $\bar{\lambda}_0$ represents the length of the plateau in a $\bar{\lambda}_{0p}$ - χ graph when χ equal to 1 (see Figure 2), i.e. the length on which the ultimate resistance is assumed to be equal to the plastic resistance and, accordingly, where the influence of the second-order effects is neglected; as no strain hardening and cladding effects were considered within the performed studies, the plateau length is taken equal to 0 as in the Merchant-Rankine approach.

So, to develop this new method, only the parameter μ has to be determined. The parameter μ is used to implicitly take into account of the second order effects within the developed procedure. In fact this parameter influences the shape of the curve presented in Figure 2; the highest μ is, the smallest the reduction factor χ is and, accordingly, the smallest the predicted λ_u is.

Three values of the parameter μ have to be calibrated, one for each plastic mechanism (i.e. μ_{beam} for the beam plastic mechanism, $\mu_{combined}$ for the combined plastic mechanism and μ_{panel} for the panel plastic mechanism), as each type of plastic mechanisms are influence differently by the second order effects. These values have been calibrated through parametrical studies. At the end of this calibration, it is intended to obtain a higher value of μ for the panel plastic mechanism than the one for the combined plastic mechanism and the latter higher than the one for the beam plastic mechanism ($\rightarrow \mu_{panel} > \mu_{combined} > \mu_{beam}$). Indeed, it was shown that the influence of the second order effects is more important for the panel plastic mechanism than for the combined one and is not significant for the beam plastic mechanism.

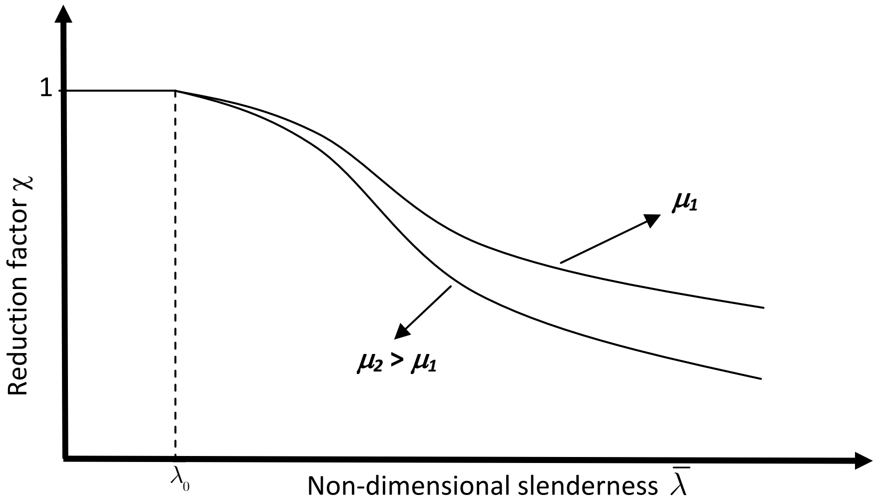


Figure 2. Example of “Ayrton-Perry” curves

As the same problems of accuracy are met with the Merchant-Rankine approach for steel and composite sway frames, the proposed method was developed for steel sway frame and for composite sway frames. Within the present document, we will only focus on the composite sway frames.

3.3.2 Parametrical study on composite sway frames

Within the parametrical study, three types of 2-D “academic” simple frames have been investigated (Figure 3); in total, 199 frames have been studied. Different types of structural elements are met within the investigated frames as described here below:

- Two types of composite beam configurations bent around their major axis:
  - upper hot-rolled profile flange fully connected to a concrete slab or;
  - upper hot-rolled profile flange fully connected to a composite slab.
- Two types of columns bent around their major axis:
  - steel hot-rolled profile ones or;
  - partially encased composite ones.
- The beam-to-column composite joints are rigid or semi-rigid ones and full-strength or partial-strength ones; the column bases are assumed to be rigid and fully resistant. The beam-to-column joints are assumed to have a sufficient ductility to develop plastic hinges and to allow a plastic analysis.

The steel material and the joint behaviour are modelled through an elastic-perfectly plastic bilinear law. For the concrete material, a parabolic behaviour law with account of tension stiffening is used. For the material properties, the characteristic values are used.

Through preliminary investigations performed before the parametrical study, an important phenomenon has been highlighted: even if the cross sections of a composite sway frame can be considered as Class 1 cross sections according to the recommendations given in Eurocode 4, it has been observed that the ductility of the concrete is sometimes not sufficient to allow the development of a full plastic mechanism within the frame. Indeed, the ultimate deformation allowed in the concrete in compression, i.e. ε_{cu} = 0,35% (defined according to Eurocode 2) was reached before the formation of the plastic mechanism. This phenomenon leads to a limitation of the ultimate loads that the frame could sustain. As this problem can be considered as a problem of cross section classification, which is not under investigation, this phenomenon has been bypassed assuming a sufficient ductility of the concrete to allow the development of the plastic mechanism. In future investigations, new criteria for the composite cross section classification, taking into account of the maximum deformation capacity of the concrete, should be developed.

The properties of the frames have been defined so as to cover the three types of plastic mechanisms, i.e. beam, combined and panel plastic mechanisms (obtained through first-order rigid-plastic analyses) with each type of structures and to obtain different types of collapse modes (plastic mechanisms or instability) through the full non-linear analyses. The parameters which are modified within these frames are:

- the type of structural elements (as mentioned previously);
- the height of the columns;
- the properties of the joints;
- the beam and column cross sections and;
- the applied loads.

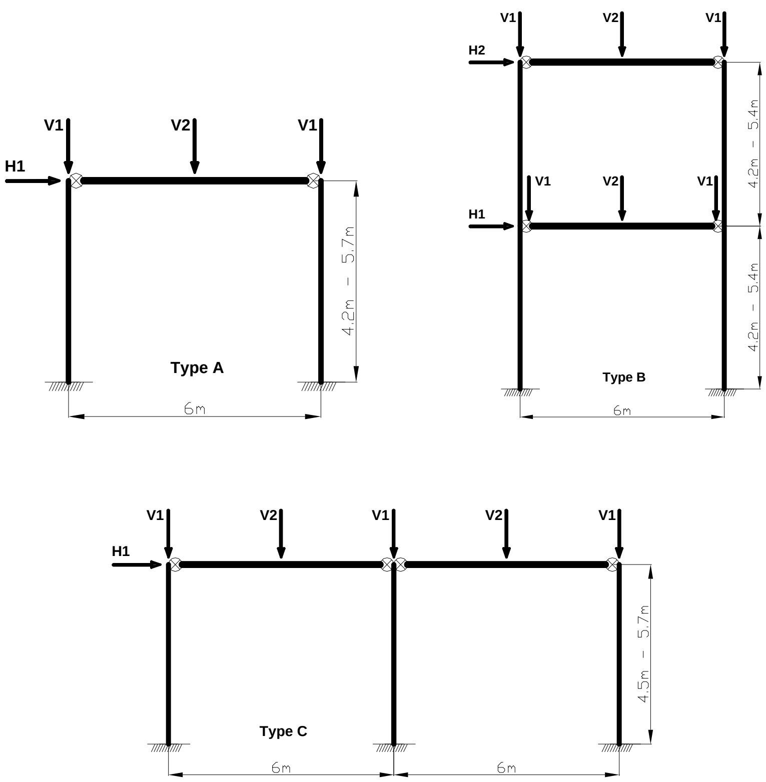


Figure 3. Structure types A, B and C

The types of analyses which have been performed are:

- Critical elastic analyses (λ_{cr});
- First-order rigid-plastic analyses (computation of the three plastic load factors, i.e. $\lambda_{p,beam}$, $\lambda_{p,combined}$ and $\lambda_{p,panel}$);
- Full non-linear analyses (λ_u).

For the computation of λ_{cr} and λ_u, the homemade FEM software FINELG is used. As recommended in Eurocode 4, an initial deformation is introduced in the computation. The formula to be used to estimate a value for the initial out-of-plumb ϕ is the same than the one proposed in Eurocode 3. Also, the shape of the initial deformation introduced in the computations is proportional to the first global instability mode obtained through the critical elastic analysis.

For the computation of the plastic load factors, a software based on an Excel sheet has been developed and validated through comparisons to numerical results. The *M-N* interaction in the columns for the computation of the plastic load factors is taken into account.

The results obtained with the new method and with the Merchant-Rankine approach are compared to the numerical results obtained through the performed non-linear analyses considered as the "reference" ones. In the analytical methods, the values of λ_{cr} which are used are the numerical ones obtained through FINELG.

The investigated frames were defined so as to cover the different types of collapse modes (obtained through non-linear analyses) and to cover a wide range of λ_p/λ_{cr} values (from 0,05 to 0,31), λ_p being the minimum value of the three plastic load factors $\lambda_{p,beam}$, $\lambda_{p,combined}$ and $\lambda_{p,panel}$.

The three values of μ, i.e. μ_{beam}, $\mu_{combined}$ and μ_{panel}, calibrated so as to minimize the difference between the predicted values of λ_u through the new method and the ones numerically predicted are the following:

- $\mu_{beam} = 0{,}02;$
- $\mu_{combined} = 0{,}42 \text{ and};$
- $\mu_{panel} = 0{,}7.$

The comparison between the predicted values of λ_u obtained through the analytical methods (the new one and the Merchant-Rankine approach) and the numerical simulations is given in Figure 4 and Figure 5 for all the frames.

From Figure 4, it can be observed that the new method gives more accurate results than the Merchant-Rankine approach; indeed, the points obtained through the new method are closer to the line "AB" than the ones obtained with the Merchant-Rankine approach. Also, more points on the "unsafe side" of the graph (i.e. below the line AB) are present with the Merchant-Rankine approach than with the new method; indeed, 81 cases (i.e. 40,7 % of the investigated frames) are unsafe through the Merchant-Rankine approach for only 15 (i.e. 7,5 % of the investigated frames) cases through the new method.

From Figure 5, it can be observed that the number of frames for which the differences on the value of λ_u is between 0 % and 10 % is equal to 167 with the new method (i.e. 83,9 % of the frames) and to 51 with the Merchant-Rankine approach (i.e. 25,6 % of the frames) which confirms the better accuracy of the proposed method.

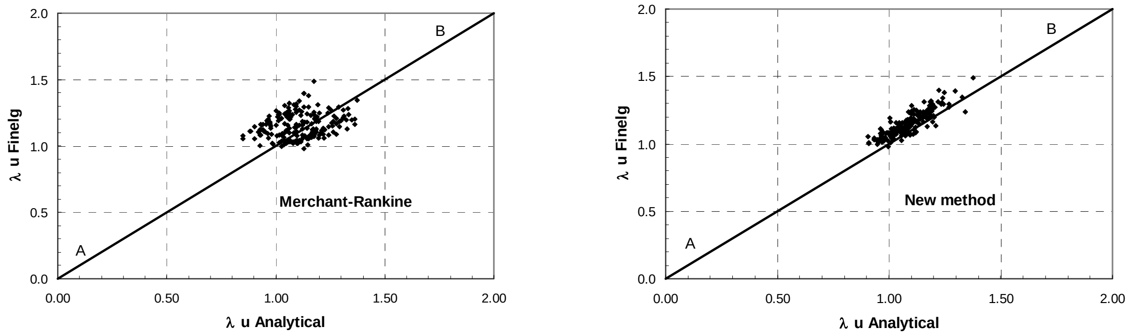


Figure 4. Comparison between the analytical and the numerical results for the prediction of λ_u (all the investigated composite frames)

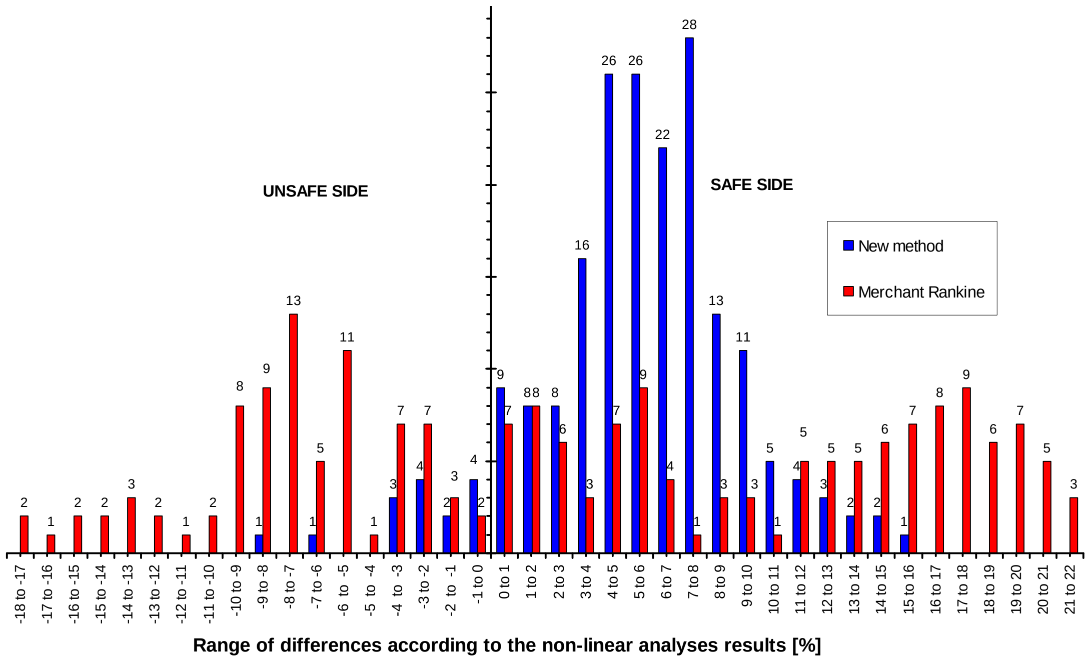


Figure 5. Evaluation of the accuracy of the analytical methods (all the investigated composite frames)

Also, amongst the investigated composite frames, there are cases (38 in total, i.e. 19,1 % of the investigated composite frames) where the collapse mode associated to λ_u do not correspond to the one associated to the minimum value of λ_p. It is interesting to underline that, with the new method, the type of plastic mechanism associated to the minimum value of λ_u corresponds to the one appearing through the fully non-linear numerical analysis for 99,5 % of the investigated frames.

Also, it is recommended to apply the Merchant-Rankine method to structures with a λ_p/λ_{cr} between 0,1 and 0,25. If only the frames respecting this condition are considered (which is the case for 150 of the investigated composite structures), the results presented in Figure 6 and in Figure 7 are obtained. From the latter, the previous observations are still valid; it can be observed that:

- Only 13 unsafe situations (i.e. 8,7 % of the considered frames) are obtained with the new method for 57 (i.e. 38 % of the considered frames) with the Merchant-Rankine approach.
- The number of frames for which the differences on the value of λ_u is between 0 % and 10 % is now equal to 131 with the new method (i.e. 87,3 % of the considered frames) and to 40 with the Merchant-Rankine approach (i.e. 26,7 % of the considered frames) which confirms the better accuracy of the proposed method.

- The type of plastic mechanism associated to the minimum value of λ_u obtained with the proposed new method corresponds to the one appearing through the fully non-linear numerical analysis for 100 % of the investigated frames.

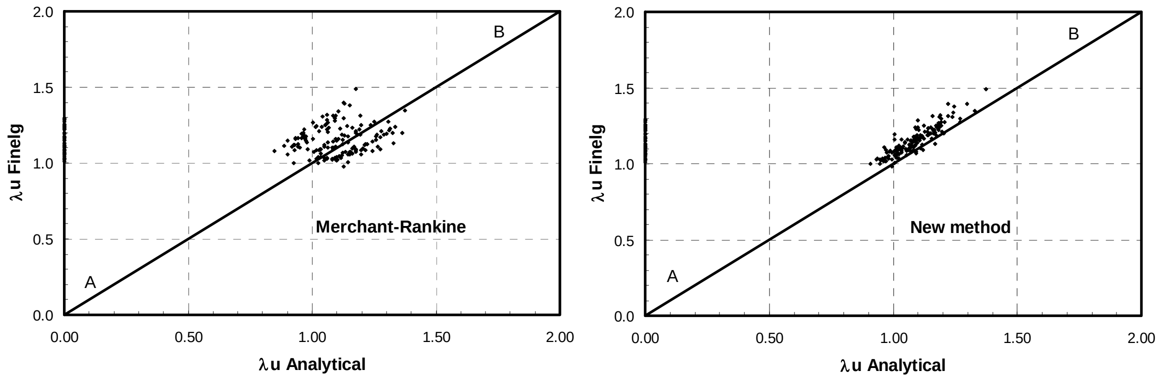


Figure 6. Comparison between the analytical and the numerical results for the prediction of λ_u ($\lambda_p/\lambda_{cr} \in [0,1 ; 0,25]$)

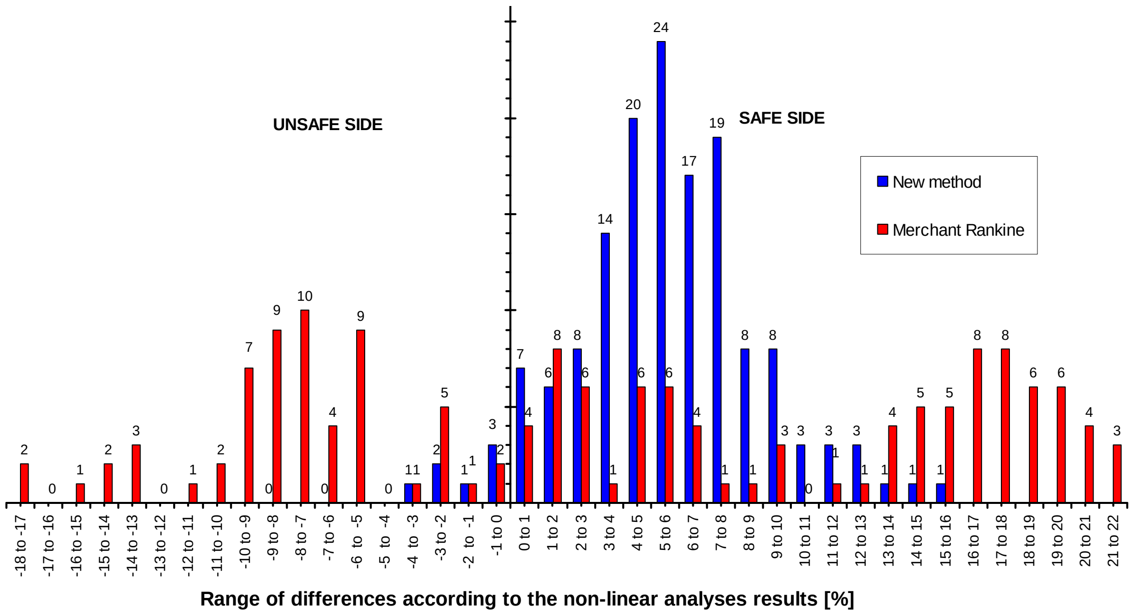


Figure 7. Evaluation of the accuracy of the analytical methods ($\lambda_p/\lambda_{cr} \in [0,1 ; 0,25]$)

Finally, in a recent diploma work [3], it was demonstrated that the developed method remains accurate even if the critical load factor λ_{cr} is computed through simplified analytical methods and in particular, through the method proposed in Eurocode 3 for steel frames.

3.4 Perspectives

- When performing the parametrical study on the composite sway frames to develop the simplified analytical method based on the Ayrton-Perry formulation, it was mentioned that problems were met with the deformation capacity of the concrete. Indeed, even if the cross sections of a composite sway frame can be considered as Class 1 cross sections according to the recommendations given in Eurocode 4, it has been highlighted, through preliminary investigations that the ductility of the concrete, as defined in Eurocode 2, is sometimes not sufficient to allow the development of a full plastic mechanism within the frame.

More investigations on this topic should be performed. An idea would be to introduce an additional criterion relative to the ductility of the concrete when considering the classification of composite cross sections.

- The validation of the analytical method developed to predict the response of steel and composite sway frames have been performed through comparisons with numerical results obtained for "academic" structures. It would be interesting to apply the developed method to actual buildings to see if the very good accuracy of the method is kept for such buildings.

4 Robustness of structures

4.1 Introduction

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. A European RFCS project called "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 event through an appropriate robustness.

The investigations performed at Liège University, as part of this European project, are mainly dedicated to the exceptional event "Loss of a column in a steel or steel-concrete composite building frame"; the main objective is to develop a simplified analytical procedure to predict the frame response further to a column loss. The development of this simplified procedure is detailed in two complementary PhD theses: the thesis of Demonceau *[1]* and the thesis of H.N.N. Luu [8]. The present paragraph describes experimental and analytical studies carried out in Liège on this topic. In particular, a simplified analytical method allowing the prediction of the frame response with account of the membrane effects is described.

Also, an ongoing RFCS project called ROBUSTFIRE, coordinated by Liège University, will be briefly introduced.

4.2 General concepts

The loss of a column can be associated to different types of exceptional actions: explosion, impact of a vehicle,… Under some of these exceptional actions, dynamic effects may play an important role; within the performed studies, it is assumed that the action associated to the column loss does not induce significant dynamic effects. So, the performed investigations are based on static approaches.

When a structure is losing a column, the latter can be divided in two main parts, as illustrated in Figure 8:

- the directly affected part which represents the part of the building directly affected by the loss of the column, i.e. the beams, the columns and the beam-to-column joints which are just above the loss column and;

- the indirectly affected part which represents the part of the building which is affected by the loads developing within the directly affected part and which influences the development of these loads.

If a cut in the structure is realised at the top of the loss column (see Figure 8), different internal loads in the vertical direction are identified:

- the shear loads V_1 and V_2 at the extremities closed to the loss column;
- the axial load N_{up} in the column just above the loss column and;
- the axial load N_{lo} in the loss column.

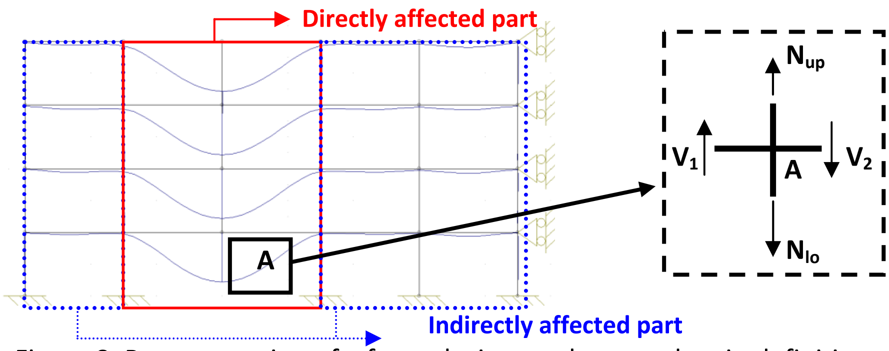


Figure 8. Representation of a frame losing a column and main definitions

In Figure 9, the curve representing the evolution of the normal load N_{lo} in the loss column (see Figure 8) according to the vertical displacement Δ_a is illustrated:

- From point (1) to (2) (Phase 1), the design loads are progressively applied, i.e. the "conventional" loading is applied to the structure; so, N_{lo} progressively decreases (N_{lo} becomes negative as the column "AB" is subjected to compression) while Δ_A can be assumed to be equal to 0 during this phase (in reality, there is a small vertical displacement at point A associated to the compression of the columns below point "A"). It is assumed that no yielding appears in the investigated frame during this phase, i.e. the frame remains fully elastic.
- From point (2) to (5), the column is progressively removed. Indeed, from point (2), the compression in column "AB" N_{lo} is decreasing until reaching a value equal to 0 at point (5) where the column can be considered as fully destroyed. So, in this zone, the absolute value of N_{lo} is progressively decreasing while the value of Δ_A is increasing. This part of the graph is divided in two phases as represented in Figure 9:
  - From point (2) to (4) (Phase 2): during this phase, the directly affected part passes from a fully elastic behaviour (from point (2) to (3)) to a plastic mechanism. At point (3), first plastic hinges are appearing in the directly affected part.
  - From point (4) to (5) (Phase 3): during this phase, high deformations of the directly affected part are observed and second order effects play an important role. In particular, significant catenary actions are developing in the bottom beams of the directly affected part.

It is only possible to pass from point (1) to (5) if:

- the loads which are reported from the directly affected part to the indirectly affected part do not induce the collapse of elements in the latter (for instance, buckling of the columns or formation of a global plastic mechanism in the indirectly affected part);
- if the different structural elements have a sufficient ductility to reach the vertical displacement corresponding to point (5).

It is also possible that the complete removal of the column is reached (i.e. N_{lo} = 0) before reaching Phase 3.

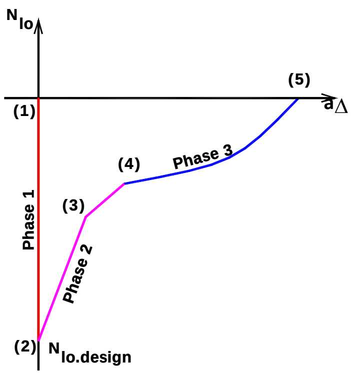


Fig. 2. Evolution of N_{lo} according to the vertical displacement at the top of the loss column

The objective of the work developed in Liège was to be able to predict, through simplified analytical models, the response of the structure during the loss of the column in order to be able to derive the requested rotation capacities of the structural elements and the loads present within the directly affected part and reported on the indirectly affected part in order to check the resistance of the latter.

The investigation of the response of the frame during Phase 1 and 2 is the topic of the thesis of Luu [8] while the response during Phase 3 is the topic of the thesis of Demonceau [1]. The adopted strategy to study Phase 3 is presented in Figure 10:

- Step 1: an experimental test is carried out in Liège on a substructure with the aim to simulate the loss of a column in a composite building frame.
- Step 2: analytical and numerical FEM tools are validated through comparisons with the experimental results
- Step 3: parametric studies based on the use of the models validated at step 2 are carried out; the objective is to identify the parameters influencing the frame response during Phase 3.
- Step 4: a simplified analytical method is developed with due account of the parameters identified at step 3 and validated through comparisons with the experimental test results of step1.

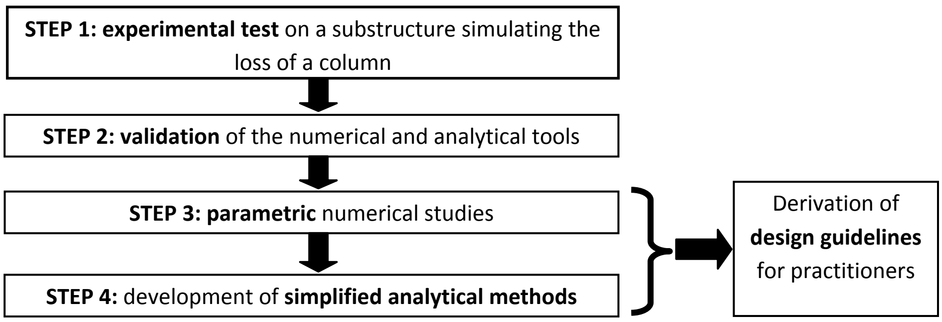


Figure 10. Strategy followed at Liège University to investigate Phase 3

The present paragraph is mainly dedicated to the investigations performed within steps 1 and 4.

4.3 Experimental test on a substructure

As previously mentioned, a test on a composite substructure has been performed in Liège to simulate the loss of a column. The main objective of the test is to observe the development of catenary actions within a frame and the effect of these actions on the behaviour of the semi-rigid and partial-strength composite beam-to-column joints. Indeed these joints are initially designed and loaded in bending, but have progressively to sustain tensile loads as a result of the development of membrane tying forces in the beams.

To define the substructure properties, an "actual" composite building has been first designed according to Eurocode 4, so under "normal" loading conditions. 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; it has been chosen so as to respect the dimensions of the testing floor in the laboratory but also to exhibit a similar behaviour than the one in the actual frame. The tested substructure is presented in Figure 11. As illustrated, horizontal jacks were placed at each end of the specimen so as to simulate the lateral restraints brought by the indirectly affected part of actual building when catenary actions develop.

A specific loading history is followed during the test. First, the vertical jack at the middle is locked and permanent loads are applied on the concrete slab with steel plates and concrete blocks ("normal" loading situation). Then, the vertical jack is unlocked and large displacements develop progressively at point A (Figure 11) until the force in the jack vanishes (free spanning of 8 m). Finally, a downward concentrated vertical load is applied to the system above the impacted column and is then progressively increased until collapse. The "vertical load vs. vertical displacement at point A" curve is reported in Figure 12.

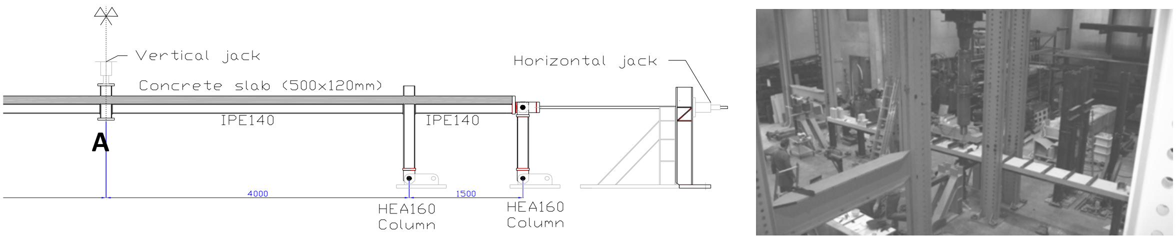


Figure 11. Tested substructure

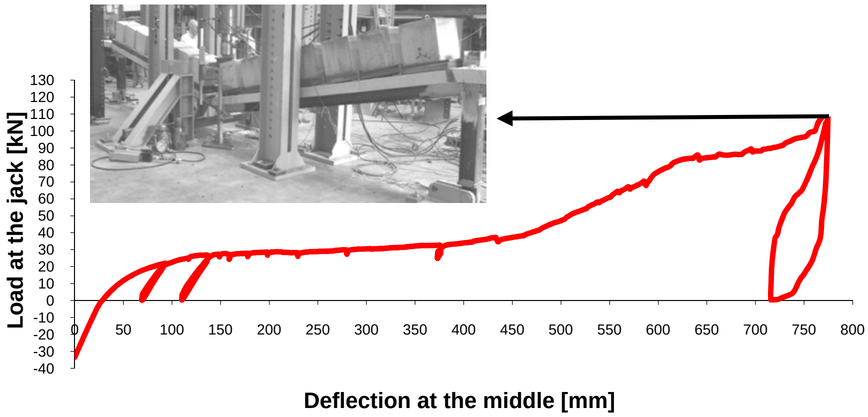


Figure 12. “Vertical load at the jack vs. vertical displacement at point A” curve

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 realised 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.

4.4 Development of an analytical tool to predict the response of a structure with account of the membrane effects

Through numerical investigations [1], it was shown that it is possible to extract a simplified substructure (see Figure 13) able to reproduce the global response of a frame further to a column loss. The objective with the analytical procedure is to predict the behaviour of the substructure in the post-plastic domain, i.e. after the formation of the beam plastic mechanism in the substructure; accordingly, the analytical model is based on a rigid-plastic analysis. 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 in the developed procedure are presented in Figure 13:

- p is the (constant) uniformly distributed load applied on the storey modelled by the simplified substructure and the concentrated load Q simulates the column loss (= $-(N_{lo} - N_{up})$) (see Figure 8) with N_{up} constant and equal to N_{up} at point 4 (see Figure 9) what is demonstrated in [8]);
- 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 13) 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 (see Figure 8). In [8], analytical procedures have been defined to predict these properties.

The results obtained with the so-developed analytical procedure are compared to the substructure test results in Figure 14. 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 [1].

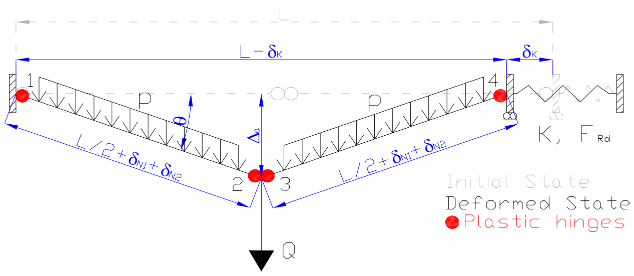


Figure 13. Substructure to be investigated

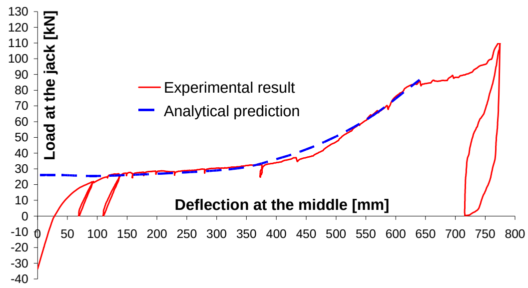


Figure 14. Comparison analytical prediction vs. experimental results

4.5 ROBUSTFIRE project

A RFCS called ROBUSTFIRE (Robustness of car parks against localised fire) is actually in progress. The project started in July 2008 and will end in June 2011.

In this project, a general philosophy for the design of robust structures against exceptional events will be developed and practical design guidelines for its application to car parks under localised fire will be derived. To achieve this goal, experience will be gained from previous or ongoing RFCS projects related to various individual aspects (temperature distribution, joint behaviour …) and further experimental, numerical and analytical developments will be achieved.

In particular, the behaviour of car parks further to a localized fire developing around a column and leading to the loss of the latter will be investigated in details. The contribution of the development of membrane effects within the composite slab will be taken into account. The behaviour of joints subjected to fire and to combined bending and axial loads will also be studied analytically, numerically and experimentally.

4.6 Perspectives

- As mentioned previously, the analytical method proposed to predict the response of the simplified substructure simulating the response of a frame further to the loss of a column with account of the membrane forces is founded on a static approach.
  It will be requested to improve the developed method in order to take into account of the dynamic effects which can be associated to a column loss. A first diploma work on this topic

[9] has recently been presented in Liège but the latter was dedicated to steel structures only. So, further investigations are still requested to investigate the response of composite structures further to a column loss with account of the dynamic effects.

- The presented studies have been performed on 2D frame only. Accordingly, the influence of the 3D response of the frame and, in particular, of the slab (which can be a composite one) is not taken into account. However, significant membrane forces can develop within the slab, what can improve the behaviour of the structure further to a column loss. Further investigations are also requested on this point. As mentioned in § 4.5, this aspect should be investigated within the ongoing project ROBUSTFIRE.
- At the end of the day, the development of simplified models, useful for practitioners in design office will be requested. The idea would be to develop a general concept which could be applied to a structure, whatever the used material is and then, to come with design recommendations which take into account of the specificity of the constitutive material(s) of the structure.

5 Behaviour of composite joints

5.1 Introduction

The behavioural response of the joints is known to significantly influence the global behaviour of structures. This paragraph summarizes recent investigations conducted in Liège on this topic.

5.2 Behaviour of single-sided composite joints subjected to “hogging” moments

For three years (2000-2003), in the framework of a European research project funded by the European Community for Steel and Coal (ECSC – Report EUR 21913 EN) in which Liège University was deeply involved, intensive experimental, numerical and theoretical investigations have been carried out on the behaviour of composite sway buildings under static and seismic loading.

As part of the above European project, Liège University carried out experimental and analytical studies with the objective to investigate the behaviour of a single-sided composite joint configuration (see Figure 15) under static loading (under hogging moment).

A particular failure mode occurred during the experimental tests, which had not yet been detected previously; it is described in details in [1]. During the tests, first cracks in the concrete slab appeared i) transversally, close to connection and ii) longitudinally, just behind the column, as shown in the following figures. The transversal cracks result from the tension forces in the longitudinal rebars while the longitudinal ones are due to shear forces. The tests was stopped with the collapse of the concrete behind the column has illustrated by the hatched part of the slab in Figure 15.

In fact, this failure mode is not taken into account in the component method proposed in EC4 for the characterisation of composite beam-to-column joints. Indeed, a minimum amount of rebars to be placed behind the column is only proposed to avoid the concrete crushing against the column; however, this recommendation seems not to be satisfactory as the latter was respected for the tested specimen.

So, an analytical formula covering this particular phenomenon and based on a theoretical model has been developed.

In EC4, the resistance of "slab rebars in tension" component is defined as $F_{Rd,13} = \frac{A_s\, f_{sk}}{\gamma_s}$ where A_s is the total area of the longitudinal slab rebars in tension with a diameter higher than 6 mm, f_{sk} is the yield strength of the rebars in tension and γ_s is the safety coefficient for the rebars (equal to 1,15 according to EC4). The formula which is here suggested to include the identified collapse mode is $F_{Rd,13} = \min\left[\frac{A_s\, f_{sk}}{\gamma_s}; \frac{2\, A_{s,2}\, \left(f_{sk}/2\right)}{\gamma_s}\right]$ where $A_{s,2}$ is the total area of the transverse slab rebars behind the column and the factor "2" in front of $A_{s,2}$ is justified by the presence of two sections of failure (one at each side of the column).

This new formula is validated through comparisons with the experimental results (see graph in Figure 16).

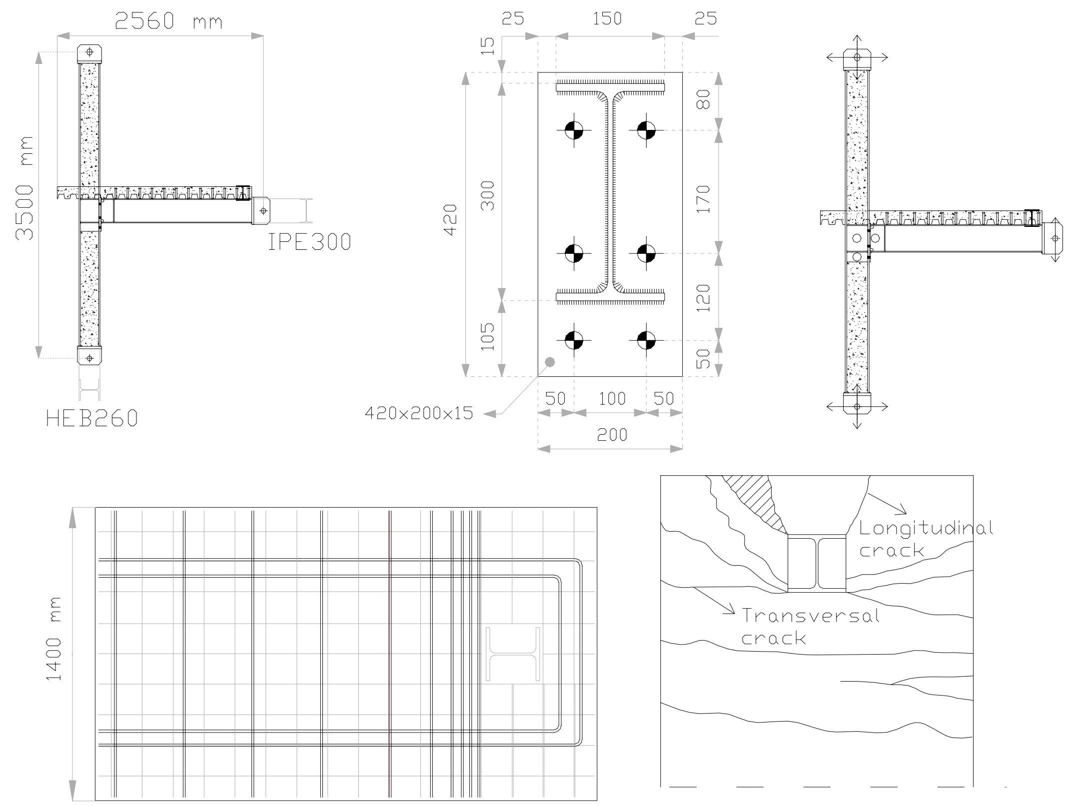


Figure 15. Tested joint configuration and cracks in the concrete slab at the end of the test

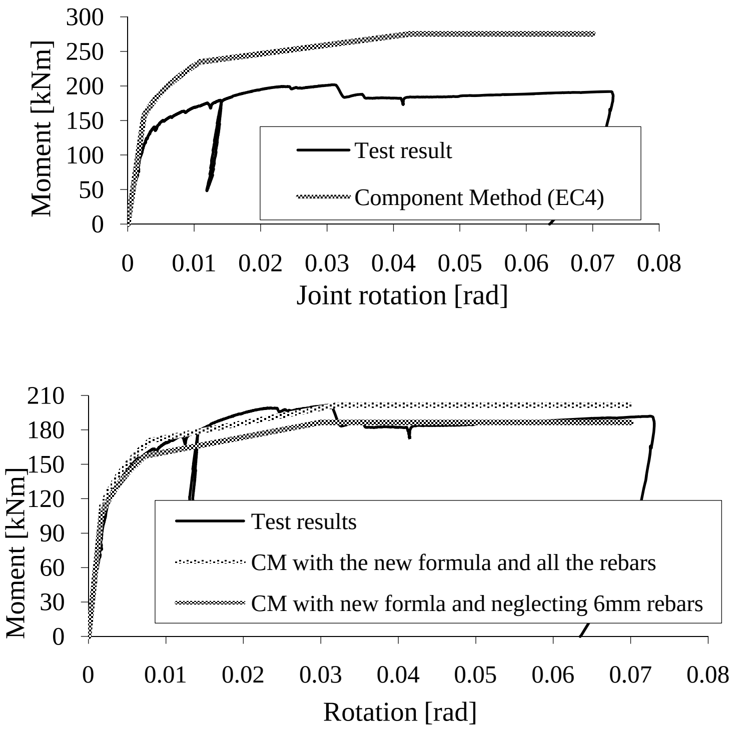


Figure 16. Comparison of analytical predictions versus test results

Another solution would be to adapt the value of the minimum amount of rebars to be placed behind the column. According to the developed formula, the section of rebars $A_{s,2}$ behind the column should be at least higher than A_s instead of $A_s/2$ as actually recommended.

5.3 Behaviour of joints subjected to "sagging" moments

Within the Eurocodes, the analytical procedure recommended for the design of joints is the "component method". This one does not allow the prediction of the behaviour of composite joints subjected to sagging bending moments. Indeed, no method is available to characterise one of the active joint components, the "concrete slab in compression" (see Figure 17).

Figure 17. Concrete slab in compression in the vicinity of the column (test performed at the University of Trento)

In recent researches, methods to characterise this component, in terms of *resistance*, have been proposed; they aimed at defining an effective rectangular cross-section of concrete contributing to the joint resistance. The procedure which is described in this section combines two methods proposed respectively by Ferrario [17] and by Liew [18]. The combination of these two methods permits to reflect in a more appropriate way how the concrete resists to the applied load in the vicinity of the joint.

Also, a formula for the characterisation of this component in terms of *stiffness* is proposed in [1].

These methods are first described and then the validation of the latter is illustrated through comparisons with experimental test results.

5.3.1 Proposed analytical method

In the PhD thesis of Ferrario [17], a formula is proposed to compute the width of concrete $b_{eff,conn}$ which has to be considered for the "concrete slab in compression" joint component:

$$b_{eff,conn} = b_c + 0,7h_c \leq b_{eff}$$

where b_c is the width of the column flange, h_c the height of the column cross-section and b_{eff} the effective width of the concrete/composite slab to be considered in the vicinity of the joint (given in Eurocode 4); b_c represents the contribution of the concrete directly in contact with the column flange while $0,7h_c$, the contribution of the concrete rods developing in the "strut-and-tie" model illustrated in Figure 18.

In the paper of Liew et al [18], the width of the concrete is taken as equal to the width of the column flange ($b_{eff,conn} = b_c$) and the development of the concrete rods in compression in the "strut-and-tie" model is neglected.

The definition of the width given by Ferrario in [17] is used in the present study as it reflects in a more appropriate way, according to the observations made during experimental tests ([1] and [19]), the mechanism actually developing in the concrete slab.

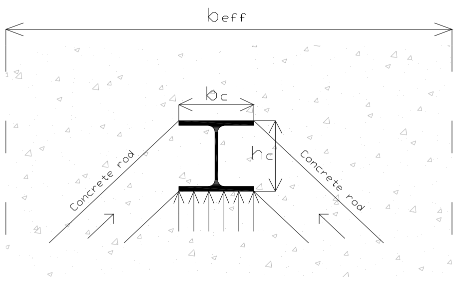


Figure 18. Plane view of the slab in the vicinity of the joint - development of concrete rods in compression under sagging moment

Another difference between the two approaches is the definition of the height of concrete to be considered and, accordingly, the position of the centre of compression within the joint. In [17], the centre of compression is assumed to be at mid-height of the concrete slab while in [18], the following procedure is given to compute the position of this point:

- the characterisation of the components in tension is performed according to the rules recommended in the Eurocodes;

- then, the height of the concrete/composite slab contributing to the joint behaviour is computed by expressing the equilibrium of the load developing in the concrete/composite slab in compression with the components in tension or in shear and by assuming a rectangular stress distribution in the concrete (defined as equal to 0,85 f_{ck}/γ_c in Eurocode 4). For instance, in the example illustrated in Figure 19, the concrete height to be considered is equal to:

$$z = \frac{F_{Rd,1} + F_{Rd,2} + F_{Rd,3}}{b_{eff,conn}.(0,85.f_{ck}/\gamma_c)} \leq h_{concrete}$$

  where $h_{concrete}$ is the total height of the concrete slab (in case of a composite slab, $h_{concrete}$ is equal to the concrete above the steel ribs);
- finally, the characterisation of the joint is performed assuming that the centre of compression is located at mid-height of the contributing part of the concrete slab (z).

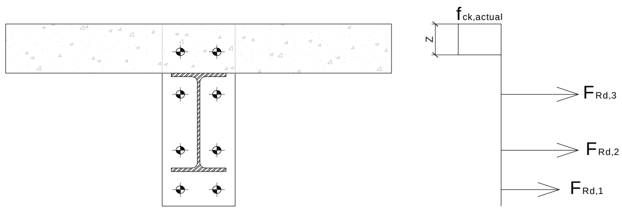


Figure 19. Height of the concrete to be considered in the characterisation of the new component

The Liew approach is here preferred as it more closely represents the reality observed during experimental tests [1].

Finally, the resistance of the "concrete slab in compression" component is computed as follows:

$$F_{Rd,csc} = b_{eff,conn}.z.(0,85.f_{ck}/\gamma_c)$$

The two previously mentioned references deal with the resistance of the "concrete slab in compression" component, but no formulae are proposed as far stiffness is concerned; this property is however required to predict the initial stiffness of the joint (and later on to derive the moment-rotation curve).

In [29], a formula is proposed to predict the stiffness of a concrete block bearing against a rigid plate. In the "concrete slab in compression" component, the steel column encased in the concrete slab may in fact be considered as a rigid plate; the formula proposed in [29] may therefore be directly extended by expressing the stiffness coefficient of the studied component as:

$$k_{csc} = \frac{E_c.\sqrt{b_{eff,conn}.z}}{1,275.E_a}$$

where E_c is the secant Young modulus for concrete, E_a, the elastic Young modulus for steel and k_{csc}, the stiffness of the component "concrete slab in compression" to be integrated into the component method.

5.3.2 Validation of the proposed analytical method

In [1], the analytical procedure described in the previous paragraph is validated through comparisons with results from experimental tests performed on composite joints in isolation. An example of such comparison is presented in Figure 20 where the analytical prediction is compared to results of experimental tests conducted at Trento University [19] on external composite joints (see Figure 21) within a European RFCS project called PRECIOUS in which Liège University was involved.

In the computations, the actual material properties (without safety factors), determined through coupon tests for the steel materials and through cylinder compression tests for the concrete, are used. The resistant bending moment M_{Rd} and the initial stiffness $S_{j,ini}$ are computed in full agreement with the component method recommended in the Eurocodes while the ultimate moment M_u, the post-limit stiffness $S_{j,post\text{-}limit}$ and the rotation capacity ϕ_u are computed according to the method proposed in the PhD thesis of Jaspart [20] (which is in full agreement with the component method), as no methods are actually proposed in the codes to evaluate these properties.

In Figure 20, two experimental curves are reported. They differ by the slab configuration: composite slab TEST 2 and full concrete slab for TEST 3 (see Figure 21).

A very good agreement is observed between the analytical prediction and the experimental results. For TEST 2, a loss of resistance in the joint is observed at a rotation of 29 mrad. This dicontinuity is not reflected by the analytical prediction; in fact, this loss of resistance is associated to a lack of ductility of the concrete in the vicinity of the connection, phenomenon not yet covered by the proposed analytical procedure.

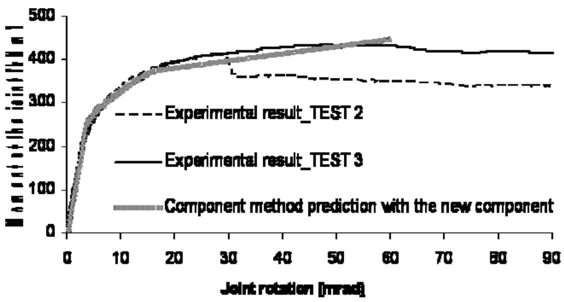


Figure 20. Comparisons analytical prediction vs. experimental results [1]

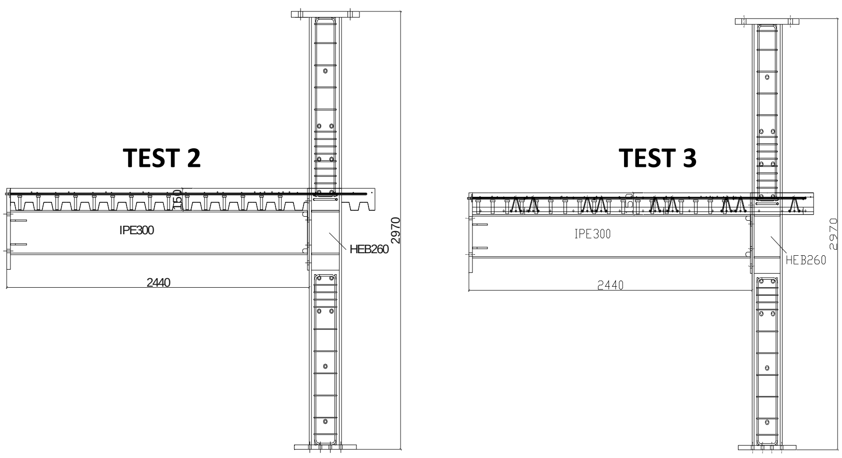


Figure 21. Tested joint configurations at Trento University [19]

5.4 Behaviour of joints subjected to combined bending moments and axial loads

As previously mentioned, recent investigations were conducted in Liège on the exceptional event "Loss of a column in a steel or composite building frame".

When a structure is subjected to such an exceptional event, significant vertical deflections appears within the structure, as illustrated in Figure 22, and membrane forces (i.e. axial loads) rapidly develop in the beams located just above the damaged or destroyed column. The joints are therefore subjected simultaneously to axial forces and bending moments.

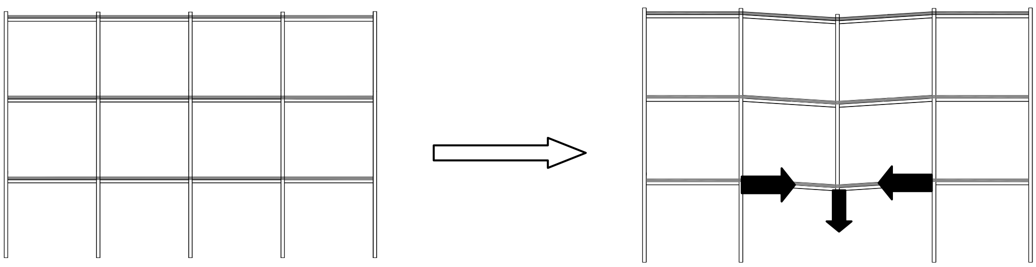


Figure 22. Loss of a column in a frame

The membrane forces have an influence on the rotational stiffness, the moment resistance and the rotation capacity of the joints acting at beam ends, and vice-versa; but, this "M-N interaction" is not presently covered by the Eurocodes where, as a direct consequence, the field of application is limited to joints in which the axial force N_{Ed} acting in the joint remains lower than 5 % of the axial design resistance of the connected beam cross section $N_{pl,Rd}$ (EN 1993): $\left| \frac{N_{Ed}}{N_{pl,Rd}} \right| \leq 0,05$.

Under this limit, it is assumed that the rotational response of the joints is not significantly affected by the axial loads. This limitation is fully arbitrary one and is not at all scientifically justified. It has also to be underlined that this criterion only depends of the applied axial load N_{Ed} and of the plastic resistance of the beam $N_{pl,Rd}$ (and not of the joint), what is quite surprising as far as the influence of the applied axial load on the joint response is of concern. If the above-mentioned criterion is not satisfied, the Eurocodes recommend to check the resistance by referring to "M-N" interaction diagram defined by the polygon linking the four points corresponding respectively to the hogging and sagging bending resistances in absence of axial forces and to the tension and compression axial resistances in absence of bending (see Figure 23).

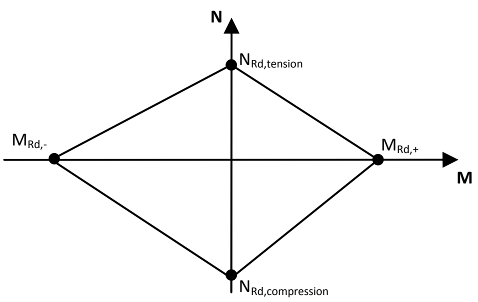
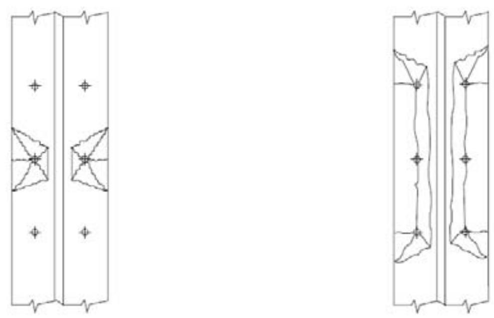


Figure 23. M-N resistant curve for a joint proposed in the Eurocodes

In a previous study [21], it has been shown that the proposed method is quite questionable. So, an improved design analytical procedure, based on the component method concept, has been developed in [21] by Cerfontaine to predict the response of ductile and non-ductile steel joints subjected to combined axial loads and bending moments.

The main difficulties that Cerfontaine met in developing this “M-N” design procedure relate to the three following aspects:

- In the component method, the resistance of the whole joint result from an assembling of the constitutive components. When the joints are subjected to combined bending moments and axial forces, the active components at each loading step are not obvious to define, as their number depends on the relative importance of the bending moment and axial load and on their respective signs.
- In the component approach, so-called “group effects” method had also to be carefully considered. These effects are likely to occur in plate components subjected to transverse bolt forces (i.e. mainly the components end-plate and column flange in bending – see Figure 24). Where a bolt force is applied, a yield plastic mechanism may develop in the plate component; if the bolt distances are high, separate yield lines will form in the plate component around the bolts (namely “individual bolt mechanism”), while a single yield plastic mechanism common to several bolts may develop when the distance between the latter decreases (namely “bolt group mechanism”). Group effects also affect the resistance of other components as the column web in tension and the beam web in tension. In the Eurocodes, group effects are duly considered, but only in the case of joints subjected to bending moments.



Individual bolt mechanism Bolt group mechanism

Figure 24. Example of possible plastic mechanism in a column flange

- Stress interactions may influence the behaviour of some components, particularly those belonging to the column: shear stresses in the web panel, longitudinal stresses due to axial and bending forces in the column flange and column web and transverse stresses due to load-introduction (column web in tension, column web in compression and column web in shear).

In [1], the Cerfontaine method has been extended to composite joints in which two main additional components are likely to be activated compared to steel ones: the slab rebars in tension and the concrete slab in compression. Thanks to the component approach, the extension has been easily achieved by just including the behaviour of the two additional components into the procedure.

The extended method has been validated through comparisons with experimental tests performed at Stuttgart University (see Figure 25 – [22]). The comparisons are presented in Figure 25. On the latter, it can be observed that two analytical curves are reported: one called “plastic resistance curve” which refer to the elastic resistance strengths of the materials and one called “ultimate resistance curve” which refer to the ultimate strengths of the materials.

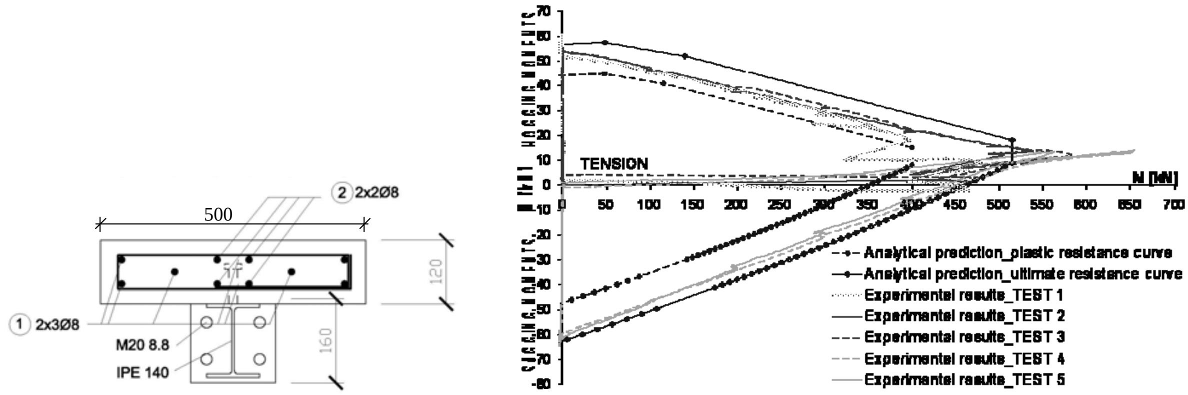


Figure 25. Tested joint configuration [10] and comparison of the resistance interaction curves

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 may be explained by the development of membrane forces in some joint components (for instance end-plate in bending), forces which are not taken into account in the analytical method [1].

5.5 Development of design dedicated software

5.5.1 General information and scope

A user-friendly software tool has been developed in order to make the application of the Eurocode 4 design rules for composite joints easier for the designer. The software is a special edition of the new version of the well known commercial software CoP (CoP stands for Connection Program). The software is developed by Feldmann + Weynand GmbH in cooperation with Liège University. The development has been supported by ArcelorMittal and this special edition is provided free of charge [23]. The ArcelorMittal edition of CoP includes also a so-called light version of the CoP steel modules. However this light version is rather limited in scope compared to the full version of CoP. For more information, reference is made to the CoP web site [24]. The following paragraphs give a short summary of the scope of the special edition and some screen shots are shown.

CoP is a standard Windows software for the design of joints in steel and steel-concrete composite building frames according to Eurocode 3 (EN 1993) and Eurocode 4 (EN 1994). The ArcelorMittal edition is an unprotected module of CoP which allows the user to design standard joints in composite constructions. A car park for example would be a typical application.

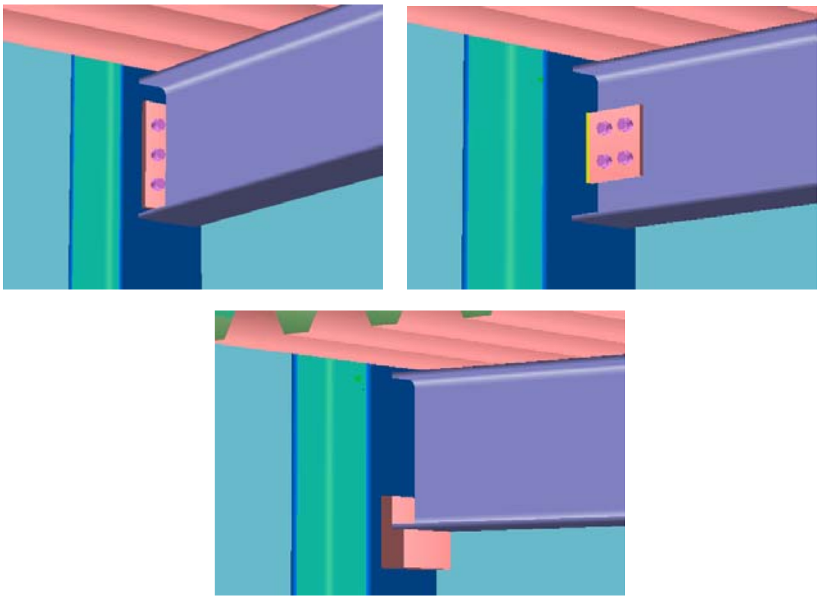


Figure 26. Example of connection types in the CoP ArcelorMittal edition

CoP considers various types of connections (such as bolted end-plate connection, double web cleats, header plates, fin plates) as well as various joint configurations (such as single sided beam-to-column joint configurations, double sided beam-to-column joint configurations, single sided beam-to-beam joint configurations, double sided beam-to-beam joint configurations). Figure 26 shows some examples of connection types for a beam-to-column joint configuration with a composite beam section.

The software consists of three main modules: (a) the user interface, (b) the calculation module and (c) the output processor. These main modules are described more in detail hereafter.

5.5.2 User interface

An easy-to-use and simple user interface is provided in order to input all necessary data to describe the geometry and the material properties of the joints, see Figure 27. The ArcelorMittal edition is available in English, French and German language.

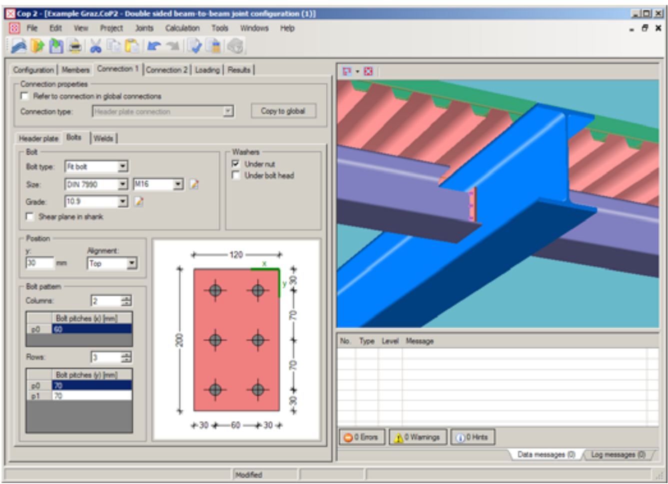


Figure 27. Main screen of CoP

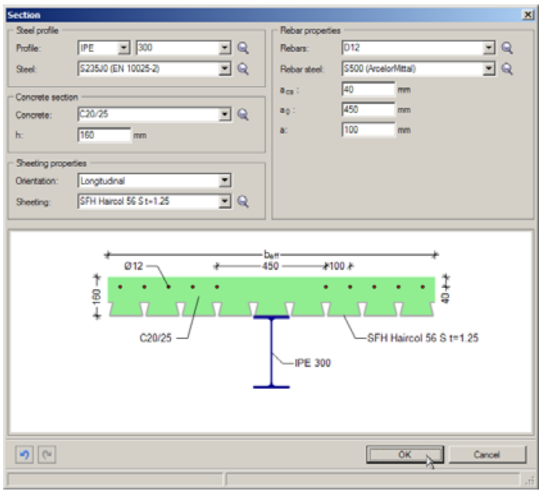


Figure 28. Input screen for composite beam sections

Individual joints may be specified by entering the member and connection data. A complete database containing profile and material characteristics is included in the software in order to facilitate the data input, see for example the data input screen for a composite beam section in Figure 28.

During the data input, a data check module is observing the consistency and validity of the data and it informs the user immediately about missing or wrong data. Furthermore, scaled 2D drawings and a 3D visualisation give the user an immediate feed-back about the current data input.

5.5.3 Calculation modules

The CoP calculation modules are designed to work with the component method. When the user runs the calculation module, either for the active joint or for all defined joints, the structural properties are calculated and a check of the resistance against the internal forces acting at the joint is made.

5.5.4 Output processor

Finally, CoP will generate a calculation note containing the data input, all results of the calculation of the joint properties and the design checks which are performed if internal loads (effects) are given. The language of the calculation note may be different from that of the user interface.

5.5.5 Availibility

The software is actually checked by Liège University, in collaboration with Feldmann + Weynand GmbH. The software should be made available for beginning of 2010.

5.6 HSS-SERF project

HSS-SERF (High Strength Steel in Seismic Resistant Building Frames) is a RFCS project coordinated by the University of Timisoara (Romania) and in which Liège University is involved. The project started in July 2009 and will end in June 2012.

The objectives of the project are the followings:

- To find reliable structural typologies (e.g. moment-resisting frames, concentrically braced frames, eccentrically braced frames) and connection detailing for dual-steel building frames, and to validate them by tests and advanced numerical simulations. In order to evaluate the possibility of using slim-floor solutions, in case of concentrically and eccentrically braced frames, both pinned and rigid beam-to-column connections will be considered. High strength steel will be used predominantly in non-dissipative structural components, while mild carbon steel will be used in dissipative components. To enhance stiffness and strength of columns (non-dissipative members), as well as fire resistance, composite steel-concrete sections will be used.

- To develop design criteria and performance based design methodology for dual-steel structures using high strength steel. Also, for the case of robustness based design under exceptional actions (i.e. beyond the intensity defined in design codes), criteria for assessment of ultimate building performance and prediction of the collapse mechanism will be proposed.
- To recommend relevant design parameters (i.e. behaviour factor q, overstrength factor W) to be implemented in further versions of EN 1998-1 in order to apply capacity design approach for dual steel framing typologies.
- To evaluate technical and economical benefit of dual-steel approach involving high strength steel.

As a contribution to this project, experimental tests on beam-to-composite column joints will be performed at Liège University and these joints will also be analytically and numerically investigated. In particular, joints with tubular composite columns with long bolts passing through the tube will be studied. Also, the possibility to use these long bolts to transfer shear loads between the steel tube and the concrete inside the tube will be investigated.

5.7 Perspectives

- Experimental and analytical investigations on the behaviour of beam-to-column joints have been presented herein. These investigations were mainly dedicated to the characterisation of the joints in terms of resistance and stiffness. An important joint property which has also to be considered is its ductility and in particular, its deformation capacity. Indeed, when performing plastic analysis or when looking the behaviour of a structure further to the loss of a column, a post-limit ductility can be requested at the joint level if the latter is partially resistant.

  However, no methods are proposed in the codes to predict this rotation capacity. Since some years, a step-by-step method is under development through diploma works at Liège University to predict the rotation capacity of joints subjected to bending moments. In future, the proposed method needs to be improved and further developments are requested; in particular, this method should be extended to predict the deformation capacity of joints subjected to combined bending moments and axial loads.
- Through the experimental tests performed on the beam-to-column joints, it has been put into sight that a loss of resistance associated to a lack of ductility of the concrete is observed in some cases. This phenomenon is not yet taken into account in the analytical methods to predict the response of the joints; additional investigations should be initiated to characterise this phenomenon and to propose analytical formulas to predict this type of collapse.

- Another perspective related to the behaviour of the joints concerns the prediction of the axial stiffness of the joint when moving on the M-N interaction resistance curve. What is actually available in the literature is a method to predict the axial stiffness of joints in the elastic range; what is requested here is the value of this stiffness in the post-elastic range, and in particular when the joints have already reached their plastic bending resistance in order to be able to apply the analytical models developed in the field of robustness. The same link has also to be characterised when the plastic hinges form in steel or composite beams. For this situation, methods to predict this property are not either available in the literature. So, investigations should also be performed to determine the axial stiffness of plastic hinges formed in steel or composite beams.

6 Verification of composite beams against lateral torsional buckling in the erection phase ([30] & [31])

In order to improve the fire resistance of their composite beams, the ArcelorMittal Long Carbon Steel Research Centre in Esch-sur-Alzette (Luxembourg) has developed a new type of partially encased composite beams. The basic idea consists in concreting successively, on site or in workshop, the spaces between the flanges of the profile, on both sides of the web. Then, under fire conditions, the steel profile is partially protected by the concrete, and the loss of resistance of the lower flange can be transferred partially to the lower longitudinal rebars. Of course, some stirrups are also necessary to ensure the integrity of the concrete. Few days after concreting, the beam may be put in place on site. Whenever shear connectors have been welded to the upper flange of the beam, then a composite action may take place with the reinforced concrete slab.

During the construction phase, instability problems may occur even if the beam is at that moment subjected to lower actions than in service conditions. There are two main reasons for that:

- First, the concrete of the slab is not resistant yet, and the strong stabilizing effect it provides during the final composite stage is not yet effective.
- Secondly, the concrete in the encasement, if available, is only 5 to 7 days old; as a consequence, the increase of torsional stiffness it provides to the beam is lower than after 28 days, making the beam more sensitive to lateral torsional buckling.

In reality, some restraints contribute actually to the lateral torsional buckling resistance, but are disregarded because of the difficulty in accounting for their beneficial effects.

It is worth mentioning that quite often *no verification at all* is performed in practice. On one hand, the design office generally does not care for erection conditions in the building construction, because the responsibility belongs to the constructor. On the other hand, the latter makes most often use of its practical expertise rather than to detailed stability calculations, with the consequence that the actual safety margin may be questionable.

In this context, it appeared useful to provide simple means for a realistic evaluation of the lateral torsional buckling resistance of partially encased beams against lateral torsional buckling during construction.

A method has been developed in Liège, in collaboration with ArcelorMittal, that intends at being a quite efficient and practical solution to this problem. This method has been included in a software called "ACP3 (previously called "LATORCON") and is freely distributed by ArcelorMittal; the aim was to propose a practical tool that could be used for a rapid verification of the beam in the erection phase.

Indeed, whenever a verification is performed for the *erection* phase, then the profile is normally known, because the design of the beam is most of the time only done for the *service* phase of the building. Then, the need for a rapid verification tool in the erection phase is obvious, in order to ensure the sufficient resistance of the beam under these particular load cases and restraints.

This software has been developed by F+W GmbH in Aachen, Germany (previously named PSP Technologien GmbH). It allows rapid and user-friendly calculations, both for serviceability and ultimate limit states for the construction phase.

Within this study, only the contribution of the steel sheet of the composite slab to the stability of the beam during the erection phase has been investigated. A PhD thesis prepared by M. Rodrigues is actually in progress at Liège University, in collaboration with and financed by the CTICM (Paris, France) in which the contribution of prefabricated slab on the stability of the beam during the erection phase will be investigated.

7 Behaviour of circular tubular composite columns with high strength steel

7.1 Introduction

A European research projects (RFCS) dealing with this topic and coordinated by Liège University is actually in progress; this project is entitled "Performance-based approaches for high strength tubular columns and connections under earthquake and fire loadings – acronym: ATTEL".

This project started in July 2008 and will end in December 2011. The objectives of this project are briefly summarised here below.

7.2 ATTEL project

The use of high strength steel (HSS) circular hollow sections (CHS) is still limited in the construction industry despite of their excellent structural and architectural properties and the fast development of end-preparation machines. Moreover, although EC3-1-12 extends its scope to steel grades up to S690/S700MC, limitations exist at the material, structural and design level. The ATTEL project aims at developing performance-based design approaches, where the capacity design – widely used in seismic engineering to avoid brittle failure and to ensure ductile behaviour – will be extended to HSS tubular CHS structures to prevent failure and collapse under both earthquake and fire loading.

In particular, the interest of using HSS tubular columns (steel or composite ones) in structure, from an economical point of view, is investigated. Solutions for column base connections and beam-to-column composite connections are also proposed and investigated analytically, numerically and experimentally (under static, pseudo-static and fire actions).

8 Education and continuing education

The present paragraph reflects activities that are developed in the field of education (i.e. students following the Civil Engineering lectures in Liège) and continuing education (i.e. practitioners working in companies, control or design offices).

8.1 Education

30 hours of lectures (16 hours of theory + 14 hours of practical exercises) on steel-concrete composite structures are given to the students in Civil Engineering at Liège University during the fourth year (first Master year) [32].

8.2 Continuing education

8.2.1 Books published by Infosteel

Infosteel is a Belgian organisation aimed at promoting the use of steel in construction and at ensuring the transfer of knowledge in this field.

Within the "transfer of knowledge" mission, one book (in French and in Flemish) was recently published by Infosteel with a design example of a steel-concrete composite structure (under static loading and fire design) [33].

A second book is actually under preparation [34]. The latter will deal with the design of building composite structures (static loading + fire) according to Eurocode 4 (NBN EN 1994) and should be available for end of 2009 in French and in Flemish. This book results from an agreement between different European organisations dedicated to the promotion of the use of steel in construction; a first draft of this book in Dutch was prepared by J.W.B. Stark and R.J. Stark from the Netherlands in and was then translated in French and in Flemish (with adaptations to the recommendations from the Belgian National Annexes) by a team of four persons, coordinated by Prof. Maquoi, in Belgium.

8.2.2 Continuing education lectures

A module of lectures, dealing with the design of building composite structures subjected to static loading and fire according to EN1994, has been prepared in French. These lectures are founded on exercises and the theory is directly explained through the exercises; with this approach, the participants to the formation can directly see the application of the theory to a "concrete" application. These lectures are given by two persons: one dealing with the theoretical aspects and one dealing with the exercises.

These lectures are regularly organised in Belgium by Infosteel and in France by "Ponts-Formation" from ENPC (Ecole National des Ponts et Charpentes – Paris). These lectures are given during three days; an example of program for these lectures is given here after. Some sessions are organised with the interested companies.

lundi 22 mars

9h00
Présentation de la session et tour de table

Généralités

9h30 M. MATHIEU - *Arcelor Mittal*
Mise en évidence des avantages de la construction mixte. Matériaux. Phasage de construction. Aspects technologiques et constructifs spécifiques. La construction mixte vis-à-vis de la résistance au feu.

Eléments de dimensionnement

14h00 M. JASPART
Points spécifiques relatifs à la classification des sections et à l'analyse globale des ossatures mixtes.
Résistance des sections mixtes.

15h15 M. JASPART
Résistance en section

16h00 M. MAQUOI
Connexion en construction mixte et son incidence sur la réponse structurale en termes de résistance et de déformation

17h45
Questions Réponses

mardi 23 mars
Exemples d'application

8h30 M. JASPART
Présentation du projet à l'étude

9h00 M. JASPART
M. MAQUOI
Dalle mixte
Points théorie / Application pratique

11h00 M. JASPART
M. MAQUOI
Poutre isostatique mixte
Points théorie / Application pratique

14h00 M. JASPART
M. MAQUOI
Poutre isostatique mixte (suite)

15h30 M. JASPART
M. MAQUOI
Poutre hyperstatique mixte
Points théorie / Application pratique

17h45
Questions - Réponses

mercredi 24 mars
Exemples d'application (suite)

8h30 M. JASPART
M. MAQUOI
Poteau mixte
Points théorie/Application pratique

11h30 M. CAJOT - *Arcelor Mittal*
Vérification au feu

14h00 M. CAJOT - *Arcelor Mittal*
Vérification au feu (suite)

16h00
Questions - réponses
Synthèse – Evaluation de la formation

Fin de la session à 16h45

Approche pédagogique :
Au terme d'exposés introductifs généraux, les éléments de cadrage nécessaires (points théorie) sont abordés au fur et à mesure du programme

9 Summary

9.1 Recent developments

- Behaviour of composite sway frames:
  - Identification of their particularities
  - Study of the applicability of simplified analytical methods already available for steel sway frames:
    - Amplified sway moment method → OK!
    - Merchant-Rankine method → same conclusions than when applied to steel sway frames
  - Development of an innovative simplified analytical method for the prediction of the ultimate load factor of a structure and its associated collapse mode
- Robustness of structures
  - Development of a general concept to predict the response of steel or composite frames further to a column loss
  - Development of simplified analytical models to predict the response of such frames further to a column loss – analytical models funded on static approaches and only the 2D response is considered
  - Through these models, it is possible to predict the request in terms of rotation capacity at the structural element level and the redistribution of the loads within the structure
- Behaviour of composite joints
  - Identification of a new collapse mode for single-sided composite joints + development of an analytical method to predict the latter
  - Development of an analytical method to predict the response of composite joints subjected to sagging moments, case not yet covered by the actual codes and standards

    - Validation of an analytical method to predict the response of composite joints subjected to combined bending moments and axial forces.
    - Contribution (technical support) to the development of a user-friendly software CoP to characterise the main mechanical properties of composite joints – this software will be soon freely available on the ArcelorMittal website.
- Education
    - Lectures given at Liège University to students following the lectures in Civil Engineering
    - Continuing education :
        - One book including a design example for a building composite structure has been recently published by Infosteel
        - One book on the "Design of composite structures according to the EN 1994" under preparation for Infosteel
        - Preparation of lectures for continuing education on the design of composite structures subjected to static and fire actions according to EN 1994 (regularly given in Belgium and in France)

9.2 Perspectives

1. When performing parametrical studies on composite sway frames to develop the simplified analytical method based on the Ayrton-Perry formulation, a problem linked to the deformation capacity of the concrete was identified.
   Indeed, even if the composite cross-sections of a sway frame can be considered as Class 1 according to the recommendations given in Eurocode 4, it has been highlighted through numerical investigations that the ductility of the concrete (if defined according to the properties suggested in Eurocode 2) is sometimes not sufficient to allow the development of a full plastic mechanism within the frame.
   More investigations on this topic should be performed. To solve this problem, two possibilities could be considered:
    - to revise the classification criteria for a composite cross-section, with account of the ductility of the concrete or;
    - to affect the plastic resistant moment of the composite cross-section by a reduction factor (to be defined) as it is done for composite cross-sections made of S420 or S460 steels.
2. When analysising of a beam to check the SLS, some conditions are given in Eurocode 4 allowing the designer to disregard the effect of a possible partial interaction at the connection level. If these conditions are not respected, the designer has to take this effect into account but no detailed indication is given. In a diploma work presented in Liège, the effect of a partial interaction on the beam deflection has been numerically investigated and it was demonstrated that this effect is significant, even if the conditions given in Eurocode 4 are respected [35]. This aspect should be investigated in details in order to propose a method (or to validate an existing one) accounting for the possible partial interaction.
3. In composite frames subjected to the exceptional event "loss of a column", when significant deflections occur within the structure, membrane effects develop within the beam and the joints, which were initially subjected to bending moments and shear loads; the joints are so progressively subjected to combined M-N-V forces. Through the development performed

recently in Liège, a method to predict the resistance of beam-to-column composite joints subjected to combined bending moments and axial loads has been proposed and validated. Until now, the effect of shear forces on the joint resistance has been neglected, what is reasonable for a structure subjected to a "conventional" loading; however, in a frame losing a column, the shear loads developing at the joint level become significant and to neglect V could lead to an overestimation of the joint resistance. Accordingly, it is requested to develop a method to predict the resistance of a joint when subjected to combined M-N-V.

4. Also, no method is available to predict the deformation capacity of joints when subjected to M, V and N forces, although this property is a key one when considering the behaviour of composite structures subjected to seismic loading or exceptional events. Investigations should take place in the future and analytical methods to predict this property should be developed.
5. When a joint is yielded, it is requested to know its post-limit stiffness (under bending moments and under axial loads) when "moving" on their M-N-V interaction surface to be able to predict the response of the structure when subjected to an exceptional event. This aspect should also be investigated.
6. In Eurocode 4, detailed rules are given for the analysis of structural elements (slab, beam or column) but few information is given for the analysis of a complete composite frame/structure, in particular for structures with semi-rigid joints. Investigations should be conducted on this topic in order to propose to the designer simplified rules to allow them to easily model and analyse a full composite structure. For instance, the problems to be addressed relate to the definition of beam effective widths, redistribution coefficients for the analysis,…
7. In the field of robustness, investigations have been conducted for the exceptional event "loss of a column" considering only the 2D behaviour of the structure and assuming that the dynamic effects which could be linked to the column loss can be neglected. Further investigations should be conducted to extend the models already available to account for the 3D behaviour of the structure and possible dynamic effects.

   Two levels of investigations should be considered:
   - The "element" level, i.e. the dynamic response of an element (with account of its boundary conditions) subjected to an exceptional action (for instance a column subjected to an impact or to an explosion) and;
   - The "structure" level, i.e. the dynamic response of a structure subjected to a specific column loss "speed".

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