Université de Liège

Faculté des Sciences Appliquées

Applicability of composite structures to sway frames

Travail de fin d'études présenté par

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en vue de l'obtention du Diplôme d'Etudes Approfondies en Sciences Appliquées

Année académique 2003-2004

Acknowledgements

Professor J.P. Jaspart, promoter of this work, is gratefully acknowledged for having supervised this work and for his helpful advices.

I am also grateful to my colleagues and to the European partners of the ECSC project "Applicability of composite structures to sway frames" who have participated, in a way or another, to the elaboration of the present work.

Finally, I would like to thank my family and particularly my wife for their support.

Jean-François Demonceau

Table of contents

Abstract

In the last years, the construction of taller composite buildings and larger composite industrial halls without wind bracing systems is susceptible to make global instability a relevant failure mode; this one is not yet covered by Eurocode 4, which mainly deals 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 only and gives design rules for composite slabs, beams, columns and joints.

That is why a research project on global instability of composite sway frames has been funded by the European Community for Steel and Coal (ECSC) for three years. The objective of this project was to provide background information on the behaviour of such frames under static and seismic loads and to provide simplified design rules by means of experimental and analytical investigations.

The present work reflects parts of the investigations carried out by Liège University, as partner in the above-mentioned European project. The objective of this work is to analyse the behaviour of sway composite frames under static loading through numerical analyses and to investigate the applicability of simplified analytical methods initially developed for steel sway frames to composite ones.

To achieve this goal, a benchmark study is first performed so as to demonstrate the validity the homemade finite element software FINELG for the numerical simulation of composite structures. Secondly, different numerical analyses on actual composite sway buildings submitted to horizontal and vertical static loads are realised with the so-validated FINELG software so as to predict their "actual" response and to highlight their particular behavioural aspects. Finally, the applicability to composite sway structures of two simplified analytical methods, initially developed for steel ones, is investigated; these two methods are the "Amplified sway moment method" and "Merchant-Rankine" approaches, respectively based on elastic and plastic design philosophies. In this part, an alternative method based on the "Merchant-Rankine" approach formulas is also introduced for the analytical prediction of the ultimate load factor of non-proportionally loaded sway frames.

Notations

- A_{eq} : equivalent steel cross section of a composite member
- b_{eff} : effective width of a composite beam
- E : Young modulus
- E_t : strain hardening stiffness of the steel material
- f_{ck} : characteristic elastic strength of the concrete material
- f_y : elastic strength of the steel material
- G : sum of the self-weight and the permanent loads applied on a frame
- h : height of a storey
- H : total height of a building or a frame
- I_b : moment of inertia of a beam cross section
- I_e : equivalent moment of inertia of a composite member
- L : length of a beam
- L_0 : distance measured between consecutive points of contraflexure in the bending moment diagram of a beam for the determination of the composite beam effective width
- M_e : elastic resistance moment of a member cross section or a joint
- M_{pl} : plastic resistance moment of a member cross section
- $M_{pl,Rd,+}$: design plastic moment resistance of a composite beam under sagging moment
- $M_{pl,Rd,-}$: designplastic moment resistance of a composite beam under hogging moment
- M_{Rd} : design moment resistance of a joint
- M_{Sd} : design moment acting at a joint
- n_c : number of full height columns per plane – parameter used for the computation of the initial out-of-plumb ϕ_{ini}
- n_s : number of storeys – parameter used for the computation of the initial out-of-plumb ϕ_{ini}
- $N_{pl,Rd}$: design plastic resistance of a composite column to gravity load without account of the buckling phenomenon

- $N_{pl,Rd,buckling}$: design resistance of a composite column to gravity load with account of the buckling phenomenon
- P_{Rd} : design resistant load of a shear stud
- Q : sum of the variable loads applied on a frame
- $S_{j,ini}$: initial stiffness of a joint
- V_{cr} : elastic critical load producing a global sway instability.
- V_n : actual value of the shear forces introduced at a joint level in the column web panel
- $V_{pl,Rd}$: design plastic resistance to shear forces of a member
- V_{Rd} : design shear resistance of a joint or of a structural member
- V_{sd} : total design vertical load applied to a structure
- y_g : position of the gravity centre of a cross section according to the lower fibre of the latter
- α : factor applied to the concrete limit strength so as to take account of the long term loading effects
- δ : deflection of a member relative to the chord line connecting its ends
- Δ : horizontal top displacement of a structure
- ϕ : rotation associated to the connecting parts of a joint
- ϕ_{Cd} : design rotation capacity of a joint
- ϕ_{ini} : rotation at the column bases associated to the initial global frame imperfection
- γ : relative rotation between the beam and column axes associated to the column web panel in shear component of a joint
- λ_{cr} : critical load factor – load factor at which global sway instability occurs in a structure (obtained through a critical elastic analysis)
- $\lambda_{cr,cracked}$: critical load factor computed assuming that the concrete is cracked in the support region (hogging moment zone)
- $\lambda_{cr,uncracked}$: critical load factor computed assuming that the concrete is uncracked in the support region (hogging moment zone)
- λ_e : elastic load factor – load factor at which a first plastic hinge develops in a structure

- λ_p : plastic load factor – load factor at which a plastic mechanism develops in a structure (obtained through a first-order rigid-plastic analysis)
- $\lambda_{p,beam}$: plastic load factor associated to a beam mechanism
- $\lambda_{p,combined}$: plastic load factor associated to a combined mechanism
- $\lambda_{p,panel}$: plastic load factor associated to a panel mechanism
- λ_{sd} : design load factor
- λ_u : ultimate load factor – load at which the failure of a structure is reached (obtained through a non-linear analysis)
- θ : joint rotation
- Ψ_{braced} : lateral flexibility of a structure with a bracing system
- $\Psi_{unbraced}$: lateral flexibility of a structure without a bracing system

Chapter 1 : Introduction

1.1 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 arrangement 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 which mainly deals 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 only and gives design rules for composite slabs, beams, columns and joints.

That is why a research project on global instability of composite sway frames has been funded by the European Community for Steel and Coal (ECSC) for three years ("Applicability of composite structures to sway frames" – Contract N° 7210-PR-250). The objective of this project 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 in the project:
    - a 3-D composite building under dynamic loading tested in Ispra (Italy);
    - a 2-D composite frame under static loading tested in Bochum (Germany).

  Beside these tests, cyclic and static tests on isolated joint were also performed in different European laboratories so as to get the actual behaviour of the constitutive joints of these two structures; the behavioural responses of the joints are known to significantly influence the global behaviour of the structure.

- Concerning the analytical investigations, they were divided in two parts:

- numerical analyses of existing structures further to a preliminary benchmark study aimed at validating the numerical tools used for the numerical investigations;
- development of design guidance with proposals of simple analytical methods to design composite sway structures.

The present thesis reflects the analytical investigations carried out at Ulg as partner of the above-mentioned European project. The objectives of the work and the research steps followed in the present study are briefly described in the following paragraph.

1.2 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 respectively, larger sway displacements are reported in the composite one.

The objectives of the present thesis is 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 will be achieved by means of numerical and analytical studies on plane frames extracted from actual buildings. The research steps are the following:

- *Chapter 2* first gives a general overview of the available knowledge on composite structures and the design methods for sway structural systems. So, details about the sway effects in steel building frames and the design methods available in Eurocode 3 for steel sway frames are first given; then, design rules for composite elements, as given by Eurocode 4, are described.

- After that, *Chapter 3* describes in details the five composite buildings which have been considered in the numerical and analytical investigations. These buildings have been either built in Europe or tested in European laboratories.

- The following chapter presents different software which have been developed in Liège for the evaluation of the geometrical and mechanical properties of the composite structural members; these software are used when required in the next chapters.

- *Chapter 5* presents and analyses the numerical investigations performed on frames extracted from the actual buildings presented in *Chapter 3*. The assumptions made to model numerically the studied frames are first introduced. Then, a benchmark study aimed at validating the finite element software used for the numerical investigations is described and its main results are detailed. Finally, the results of the numerical investigations obtained by means of the so-validated finite element software are presented and analysed to highlight their particular behavioural aspects.

- Then, the applicability to composite sway frames of two simplified analytical methods commonly used for steel frames is contemplated in *Chapter 6*: the "amplified sway moment method" and the "Merchant-Rankine approach". Indeed, these methods initially developed for steel sway frames cannot be applied to composite sway frames in a straightforward way as the latter present an additional deformation source which is the concrete cracking. The results obtained through these methods are compared to the ones obtained through non-linear analyses presented in the previous chapter.
- Finally, general conclusions are presented in *Chapter 7* with a summary of the main results and the perspectives for future studies which should be presented in our future PhD thesis.

Chapter 2 : State-of-the-art knowledge on composite sway building frames

2.1 Introduction

As said in *§ 1.1*, Eurocode 4 limits its scope to "non-sway" composite buildings under static loading, giving rules to analyse and to check elements like composite slabs, beams, columns and joints. No rules are given for the analysis and the verification of "sway" composite building frames; Eurocode 4 only recommends to use "appropriate" design rules for such frames.

Few information about the behaviour of composite sway frames are available until now; first investigations in this field have been carried out in the last years. In particular, the applicability of the wind-moment method (see *§ 2.2.6.6*) to unbraced composite frames was first examined in a Ph.D thesis *[1]* submitted at Nottingham University; the use of this method for composite sway building frames will be discussed later on in *Chapter 6*. Two diploma works (*[2]* and *[3]*) with the objective of investigating the behaviour of sway composite structures were also submitted at Liège University; part of the obtained results will be presented later on in *Chapter 5*.

This chapter introduces the different concepts available in the actual codes, which will be the starting points for the developments presented in this work:

- *§ 2.2* first details the design methods proposed in Eurocode 3 for the analysis and the check of steel sway building frames;
- then, *§ 2.3* presents the design rules proposed in Eurocode 4 for the design of the composite building constitutive elements.

2.2 Sway effects in steel building frames

2.2.1 Introduction

Sway frames are characterised by significant lateral displacements. The latter can generate a global instability phenomenon under the gravity loads, as a result of second-order effects called "*P-Δ* effects" (see *§ 2.2.4.2*) which can significantly influence the behaviour of the frame. Furthermore, under increasing external loading, the apparition of plastic hinges in the frame decreases progressively its lateral stiffness; this has a detrimental influence on the maximum vertical loads leading to a global frame instability.

Eurocode 3, dedicated to steel buildings, is the only structural code providing indications on how to deal with the instability and, in particular, with the global frame analysis methods. Global frame analysis aims at determining the distribution of the internal forces and of the corresponding deformations in a structure subjected to a specified loading. It requires the adoption of adequate models which incorporate assumptions on the behaviour of the structure and in particular of its constitutive members and joints. And, these assumptions may be different if the structural building is "sway" or a "non-sway".

In the section, the procedure proposed in Eurocode 3 for the analysis and the verification of a steel sway frame is described; the applicability of the latter to sway composite frames is investigated in *Chapter 5* and *Chapter 6*. The paragraphs are organised as follows:

- First, the idealisation of actual buildings is described in *§ 2.2.2*.
- Secondly, a general description of the criteria for the member cross section and the frame classification is given in *§ 2.2.3* and *2.2.4* respectively.
- The possible global frame analyses are then briefly described in *§ 2.2.5* together with the different criteria which govern their choice; the different verifications to perform according to the selected analysis method are also presented.
- Finally, simplified analytical methods developed for steel sway frames are introduced in *§ 2.2.6*; these methods allow the designer to proceed to a rather simple structural design (not requiring a high capacity software to take account of the sway effects and the non-linearities).

Remark: these paragraphs are largely inspired by the following lecture notes prepared within a European project in which ULg has been deeply involved: "Structural Steelwork Eurocodes – Development of a Trans-national Approach (SSEDTA)" (*[4]*, *[5]* & *[6]*).

2.2.2 Frame idealisation

Global analysis of frames is conducted on a model based on many assumptions including those for the structural model, the geometric behaviour of the structure and of its members and the behaviour of the sections and of the joints.

A 3-D structure is composed of "members" linked together through joints and has to respect resistance, stability and serviceability conditions; each element has their own characteristics and has also to respect the same conditions. Members are classified according to their cross section properties (see *§ 2.2.3*) or the kind of loading they sustain. They are termed as beams if bending predominates, as columns (compression members or tension members) if axial load predominates and as beam-columns if significant amounts of both bending and axial load are present. A brief description of each type of members is given here below:

- In a general way, **beams** are designed to carry loads which produce in-plane bending (bi-axial bending is rare in such members). Because unavoidable initial imperfections in the beam geometry and unintentional small eccentricity of the loads on the beam occur, some torsion will be always present in the case of in-plane bending. Under increasing loads, the out-of-plane deformations can become magnified to such an extent that the usefulness limit of the beam may be reached. Failure is then said to occur due to lateral-torsional buckling. For most typical frames this type of failure can usually be avoided by providing adequate lateral restraint to the compression flange (restraint provided in the plane of the floor or by the floor slab itself for instance). Beams are made up of plate elements, which may sometimes be sensitive to local buckling. Local buckling in combination with lateral-torsional buckling may be the cause of failure of some steel beams in a structure.

- **Columns** are designed to carry axial tension or compression forces. When a column is subjected to axial compression, it may be classified by length. Whilst a short compression member fails by crushing or squashing, a long or slender compression member fails by buckling instability. The squash load occurs where there is full plastification in compression throughout the cross section of the member. The failure load of any compression member that fails by buckling depends on its slenderness and, as a consequence, the buckling load of the member can be significantly lower than its squash load. The design resistance of tension members is based on yield in the gross section and/or on rupture in the net section. Special attention has to be paid to where ductile behaviour is required.
- Members subject to both significant bending and axial compression are called **beam-columns**. Such members are typically the vertical members of a frame structure. Members subject to both significant bending and axial tension can be included in this category. Strictly speaking, most members are beam-columns; beams represent the limiting case where the axial forces can be disregarded and columns are the limiting case where bending moments are not significant.

The links between the different members are the **joints**. Historically, it has been common practice to assume joints to be either rigid or pinned. Whilst it is still usual today, it is now possible to model joint behaviour more precisely and to introduce their actual behaviour in the analysis (see *§ 2.3.5*). The main forces that a joint between two members must transmit are the shear forces and, when the joint is not pinned, the in-plane bending moment.

As an alternative to analysing the main structure as a one three-dimensional framework, it may be analysed as two series of independent plane frames running in two horizontal directions approximately at right angles to each other (*Figure 2-1*) provided each such plane frame has sufficient out-of-plane restraint to ensure its lateral stability.

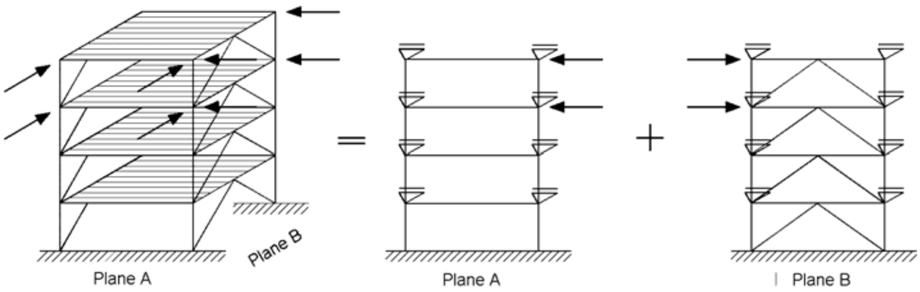


Figure 2-1: from a 3-D framework to 2-D frames

2.2.3 Member cross section classification

Member sections, be they rolled or welded, may be considered as an assembly of individual plate elements, some of which are internal (e.g. the webs of open beams or the flanges of boxes) and others are outstand (e.g. the flanges of open sections and the legs of angles). As the plate elements in structural sections are relatively thin compared with their width, when loaded in compression (as a result of axial loads applied to the whole section and/or from bending) they may buckle locally. The disposition of any plate element within the cross section to buckle may limit the axial load carrying capacity, or the bending resistance of the section, by preventing the attainment of yield. Avoidance of premature failure arising from the effects of local buckling may be achieved by limiting the width-to-thickness ratio for individual elements within the cross section. This is the basis of the section classification approach.

Eurocode 3 (and also Eurocode 4) defines four classes of cross section. The class into which a particular cross section falls depends upon the slenderness of each element (defined by a width-to-thickness ratio) and the compressive stress distribution i.e. uniform or linear. The classes are defined in terms of performance requirements for resistance of bending moments:

- **Class 1** cross sections are those which can form a plastic hinge with the required rotational capacity for plastic analysis.
- **Class 2** cross sections are those which, although able to develop a plastic moment (M_{pl}), have limited rotational capacity and are therefore unsuitable for structures designed by plastic analysis.
- **Class 3** cross sections are those in which the calculated stress in the extreme compression fibre can reach yield but local buckling prevents the development of the plastic moment resistance.
- **Class 4** cross sections are those in which local buckling limits the moment resistance (or compression resistance for axially loaded members); the elastic moment (M_e) cannot develop in such sections. Explicit allowance for the effects of local buckling is necessary.

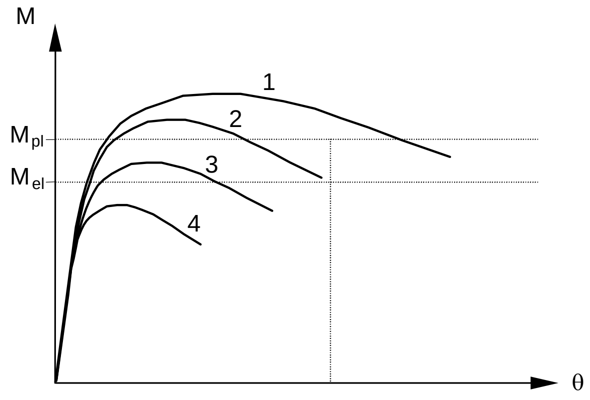


Figure 2-2: shape of the moment-rotation curve according the class of the section

So, the choice between an elastic or a plastic analysis will be mainly govern by the class of the cross sections of the structural members (see *§ 2.2.5*).

2.2.4 Frame classification

2.2.4.1 Braced and unbraced

At a preliminary design stage, a decision usually has to be made as to whether the structure is to have a braced or unbraced classification. This determines how the vertical and horizontal load effects (including those due to frame imperfections) are to be considered in the analysis.

When bracing is provided it is normally used to prevent, or at least to restrict, sway in multi-storey frames. Common bracing systems are trusses or shear walls (*Figure 2-3*).

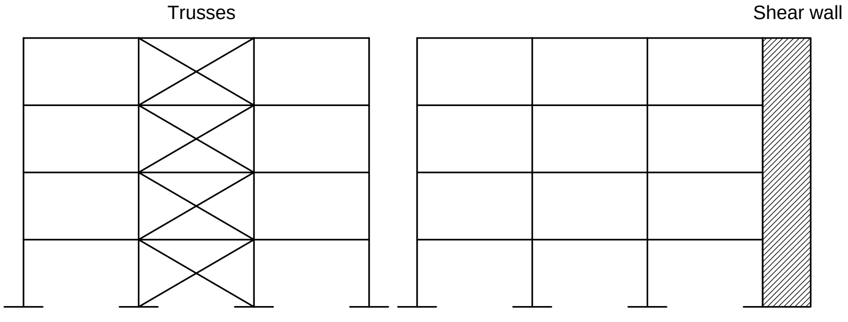


Figure 2-3: common bracing systems

For a frame to be classified as a **braced frame**, it must possess a bracing system which is adequately stiff.

Indeed, the existence of a bracing system in a structure does not guarantee that the frame structure is to be classified as braced. Only when the bracing system reduces the horizontal displacements by at least 80% can the frame be classified as braced:

- if no bracing system is provided: the frame is **unbraced**;

- if a bracing system is provided: the frame is **braced** when $\Psi_{braced} \leq 0{,}2\ \Psi_{unbraced}$ where Ψ_{braced} is the lateral flexibility of the structure with the bracing system and $\Psi_{unbraced}$ is the lateral flexibility of the structure without the bracing system.

When it is justified to classify the frame as **braced**, it is possible to analyse the frame and the bracing system separately as follows :

- The frame without the bracing system can be treated as fully supported laterally and as having to resist the action of the vertical loads only.
- The bracing system resists all the horizontal loads applied to the frames it braces, any vertical loads applied to the bracing system and the effects of the initial sway imperfections from the frames it braces and from the bracing system itself.

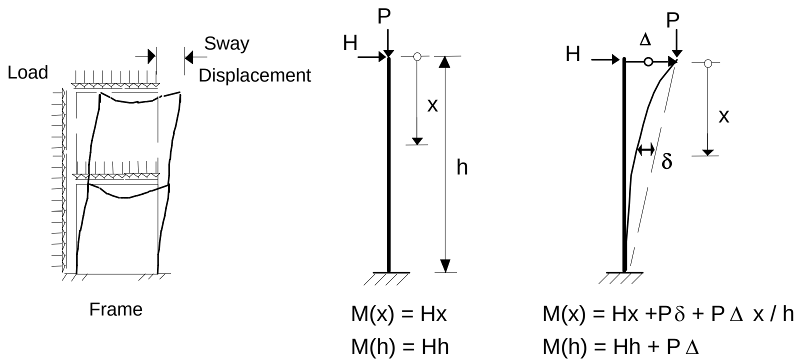
For frames without a bracing system and also for frames with a bracing system but which is not sufficiently stiff to allow classification of the frame as braced, the structure is classified as **unbraced**. In all case of unbraced frames, a single structural system, consisting of the frame and of the bracing when present, shall be analysed for both the vertical and horizontal loads acting together as well as for the effects of imperfections.

2.2.4.2 Sway and non-sway

- **Second-order effects**

Prior to the definition of the sway – non-sway classification, it is important to define the underlying concept of "second-order effects".

The second order theory consists in expressing the equilibrium between the internal and external forces of the structure in the deformed shape; in opposite, a first-order theory expresses the equilibrium in the undeformed shape. As an example, *Figure 2-4* shows that an additional level arm for the vertical loads appears with account for the deformation of the structure; so, additional bending moments develop in the structure (called "second-order bending moments").



where h is the height from the column base to the inflexion point
Δ is the sway relative to the column base of the infexion point

Figure 2-4: first-order and second-order moments in a beam-column

These global second-order moments are commonly referred to as the ***P-Δ* effects**. In addition a local second-order moment, commonly referred to as the ***P-δ* effects**, arises in the axially loaded member due to the deflections (δ) relative to the chord line connecting the member ends (*Figure 2-4*). These second-order effects are initiated by the global frame imperfections and the local member imperfections which are present in all structures (*Figure 2-5*).

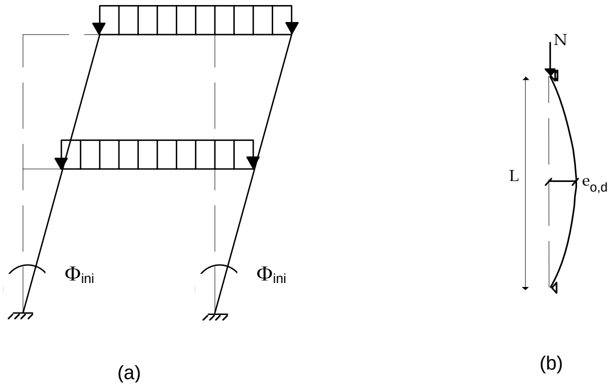


Figure 2-5: global frame initial imperfection (a) and local member initial imperfections (b)

- **Sway – non-sway classification**

The term **non-sway frame** is applicable when the frame response to in-plane horizontal forces is sufficiently stiff for it to be acceptable to neglect any additional forces or moments arising from horizontal displacements of its nodes. The global second-order effects (i.e. the P- Δ sway effects) may be neglected for a non-sway frame. When the global second-order effects are not negligible, the frame is said to be a **sway frame**.

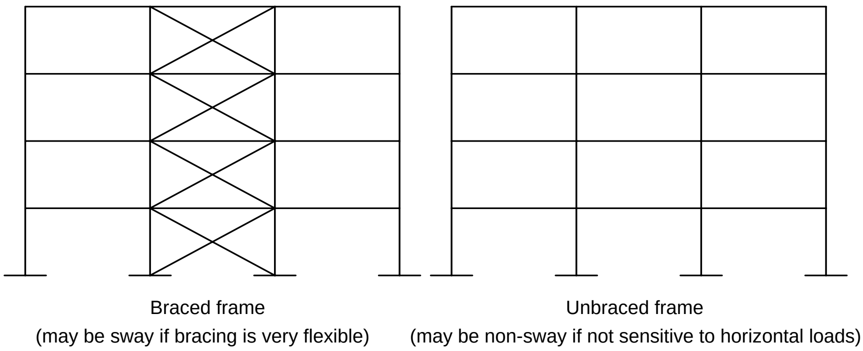


Figure 2-6: braced and unbraced frame

Normally a frame with bracing is likely to be classified as **non-sway**, while an unbraced frame is likely to be classified as **sway**. However, it is important to note that it is theoretically possible for an unbraced frame to be classified as non-sway (this is often the case of one storey portal frame buildings) while a frame with bracing may be classified as sway (possible for multi-storey buildings) (*Figure 2-6*).

When a frame is classified as **non-sway**, a first-order analysis may always be used; when a frame is classified as **sway**, a second-order analysis shall be used.

The classification of a frame structure as sway or non-sway is based on the value of the ratio of the design value of the total vertical load V_{Sd} applied to the structure to its elastic critical value V_{cr} producing sway instability (failure in the sway mode).

Obviously, the closer that the applied load is to the critical load, the greater is the risk of instability and the greater are the global second-order effects on the structure (the *P-Δ* effects).

The classification rule is as follows:

- if $\frac{V_{sd}}{V_{cr}} \le 0.1$ (2.1), the structure is classified as **non-sway**;
- if $\frac{V_{sd}}{V_{cr}} > 0.1$ (2.2), the structure is classified as **sway**.

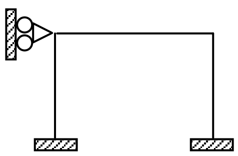
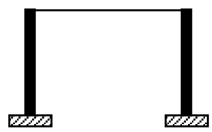
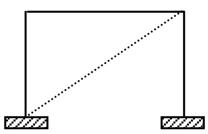
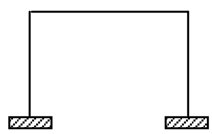
This rule can also be expressed in the following way:

- if $\lambda_{cr} = \frac{V_{cr}}{V_{Sd}} \ge 10$ (2.3), the structure is classified as **non-sway**;
- if $\lambda_{cr} = \frac{V_{cr}}{V_{Sd}} < 10$ (2.4), the structure is classified as **sway**.

2.2.4.3 Summary

The different possibilities of frame classification can be summarized by *Table 2-1* where all the combinations between braced – unbraced and sway – non-sway are presented.

Table 2-1: possibilities of frame classification

	Braced	**Unbraced**
Non-sway		
Sway		

The choice between a first-order or a second-order theory for the global structural analysis will mainly be governed by the frame classification (see *§ 2.2.5*).

2.2.5 Types of structural analyses and associated verifications

2.2.5.1 Introduction

The previous paragraphs have introduced the different classifications for the member cross sections and for the frames; these classifications will mainly influence the selection of the global structural analysis to perform on the frame.

On one hand, the class of the member cross section influence the choice between a plastic or an elastic analysis: plastic analyses can only be applied to structures which present member cross sections of class 1 where plastic hinges take place; if it is not the case, only the realization of an elastic analysis is authorized.

On the other hand, the classification of the frame will influence the choice between a first-order and a second-order theory. The different possibilities are presented in *Table 2-2*.

Table 2-2: influence of the frame classification on the choice between a first or second order theory

	Braced	**Unbraced**
Non-sway	First order theory or Second order theory	First order theory or Second order theory
Sway	Second order theory	Second order theory

One aspect which has also to be considered in this choice is that the design checks to be carried out after the analysis depend on the sophistication of the analysis "tool" used (see *Figure 2-7*): the number of design checks decreases with the increase of the analysis sophistication (from first-order elastic analysis to "full" non-linear analysis).

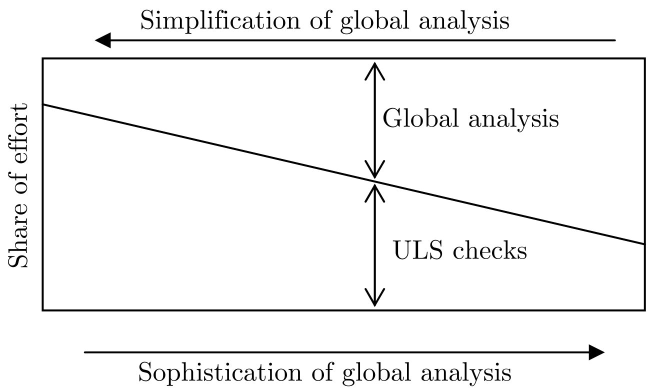


Figure 2-7: balance between global analysis and ultimate limit states (ULS) checks

The present paragraph presents the different type of structural analysis which can be applied to frames. For each type, the different ultimate limit states (ULS) checks to perform are described. The serviceability limit states (SLS) have also to be verified; the different limitations recommended in Eurocode 3 (and in Eurocode 4) are presented in *Table 2-3* where L is the length of a beam, H the total height of the structure and h the height of a storey. A last paragraph (*§ 2.2.5.8*) compares the results obtained through the different analyses described in this paragraph through a qualitative illustration in *Figure 2-8*.

Table 2-3: deflection limitations recommended in Eurocode 3 and 4

Verification	Limitation
Beam deflection	$L/300$
Top sway displacement	$H/500$
Sway displacement for each storey	$h//300$

Remark: for all the presented analyses, the out-of-plane stabilities have to be checked. This point will not be reminded in the following paragraphs.

2.2.5.2 First-order elastic analysis

Linear elastic analysis implies an indefinite linear response of sections and joints. This analysis is applicable to non-sway buildings. It can be used for sway frames under certain conditions and provided that appropriate corrections are made to allow for the second-order effects when necessary (see *§ 2.2.6*); this is the only restriction to this type of analysis. The main advantage of such analysis is that the superposition principle is applicable.

As elastic analysis is used, it would seem appropriate to consider the attainment of the yield stress in the extreme fibres of the most loaded section as the design condition for a member; however, in usual cases, it is generally accepted that an elastic analysis can be safely used to determine the load corresponding to when the first plastic event occurs (if the class of the member cross section where the plastic event occurs permits it).

However, this assumes that the structure (in case of sway frames) and its members remain stable. So, it is important to check the stability of the structure and its members (buckling, lateral-torsional buckling) to be sure that an instability phenomenon does not decrease the design resistance value.

2.2.5.3 Critical elastic analysis

This analysis is based on the same assumption as the previous one (sections and joints indefinitely linear elastic) and is achieved so as to derive the elastic critical load $\boldsymbol{V_{cr}}$ (or the elastic critical load factor $\boldsymbol{\lambda_{cr}}$) that corresponds to the first mode of global instability. According to Eurocode 3 *[7]*, this value is used through the evaluation of the V_{Sd}/V_{cr} (or $\lambda_{Sd}/\lambda_{cr}$) ratio - V_{Sd} being the design vertical applied load - to determine whether a frame is laterally rigid or, in contrast, prone to sway (see *§ 2.2.4.2*).

2.2.5.4 Second-order elastic analysis

In this type of analysis, the indefinitely linear-elastic response of sections and joints is still applied. The distribution of the internal forces is now computed on the basis of a second-order theory (i.e. equilibrium equations express in the deformed structure). This type of analysis is applicable to all structures.

As for the first-order elastic analysis, the failure is assumed to be reached when the extreme fibres of the most loaded section yields or when the first plastic hinge takes place in the structure (if the member cross section class permits it).

The resistance of the sections and of the joints has to be checked. As the *P-Δ* effects (*§ 2.2.4.2*) are considered in a second-order analysis, it is not needed to check the global stability of the structure. If the *P-δ*(*§ 2.2.4.2*) effects are also included in the analysis, neither have the local stability to be checked; if it is not the case, it has to be verified but by computing the buckling length with the assumption of a non-sway structure, as the sway effects are included in the analysis.

2.2.5.5 First-order rigid-plastic analysis

In such analysis, the elastic deformations are ignored. This type of analysis is especially appropriate for non-sway frames while its use for sway frames is limited to specific cases; the latter is only applicable to structures which fulfil some conditions given in EC3 (steel properties, cross section class,...). This calculation results in the first-order rigid-plastic load factor $\boldsymbol{\lambda_p}$; the latter is often called the first-order "limit" load factor. It can be obtained easily by hand-calculation, or by using appropriate software. The first-order rigid-plastic load factor is required, for instance, to apply the simplified design method known as the "Merchant-Rankine approach" (see *§ 2.2.6.4*).

Adequate design requires that the value of the load multiplier λ_p be at least unity. As the first-order plastic method does not make allowance for any buckling phenomena as well at a local as at a global level, these checks shall be carried out with due allowance being made for the presence of plastic hinges.

2.2.5.6 Second-order rigid-plastic analysis

This analysis may be used in all cases for which a plastic analysis is allowed. This analysis differs from the previous one by the fact that equilibrium equations are now expressed with reference to the deformed frame configuration.

A second-order rigid-plastic analysis gives an indication on how second-order effects develop once the first-order rigid-plastic mechanism is formed and how much they affect the post-limit resistance. Because second-order effects are without significant influence on the plastic beam mechanisms, the second-order rigid-plastic response curve will not diverge notably from the one obtained from a first-order rigid-plastic analysis. In contrast, for panel and combined beam-panel plastic mechanisms, the larger the sway displacement, the more the second-order rigid-plastic load factor is reduced when gravity loads increase.

Concerning the stability of the studied structure and its constitutive elements, the remarks introduced for the first-order rigid-plastic analysis (in the previous paragraph) are also valid for the second-order rigid-plastic analysis except that it is not needed here to check the global stability of the structure as the P-Δ effects are involved in the analysis.

2.2.5.7 Non-linear analysis

A non-linear analysis is applicable to all cases. In this type of analysis, all the geometrical and material non-linearities are considered: realistic material stress-strain curves, semi-rigid response of the joints and second-order effects induced by frame and element geometrical imperfections. The initial deformation of the buildings is introduced in the analysis. Such an analysis enables an accurate estimation of the actual ultimate load factor $\boldsymbol{\lambda_u}$. The stability of the structure and the constitutive elements doesn't need to be checked, as P-Δ and P–δ effects are involved in the non-linear analysis.

2.2.5.8 Overview of the considered frame analyses

In *Figure 2-8*, the results of the different analyses described in this paragraph are qualitatively illustrated. This figure shows how the sway displacement Δ influences the value of the load factor λ got from the several types of analysis.

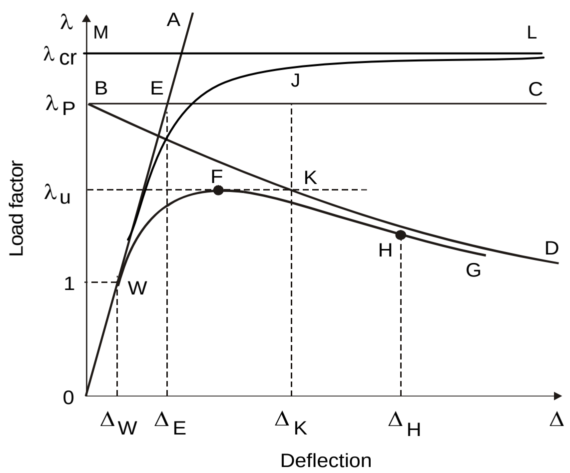


Figure 2-8: graphical representation of the results obtained through the different structural analyses

The line "OA" corresponds to a purely elastic first-order analysis. The result of the elastic critical analysis is given by the horizontal line "ML", the ordinate of which corresponds to the elastic critical load factor λ_{cr}. Curve "OJL" corresponds to the second-order elastic analysis; this curve approaches asymptotically the horizontal line "ML" corresponding to the elastic critical analysis result. The first-order rigid-plastic analysis is represented by the curve "OBC"; when the first-order rigid-plastic load factor λ_p is reached (in "B"), the failure develops under constant load ("BC" line). The behaviour got from the second-order rigid-plastic analysis is represented by the "OBD" curve: when the rigid-plastic load factor λ_p is reached (in "B"), its value decreases with an increasing transverse displacement ("BD" curve). The "OFG" curve results from a non-linear analysis; it is likely to reflect the "actual" frame behaviour. The ultimate load factor λ_u corresponds to the peak ordinate of the load-displacement curve (in “F”). If, at λ_u, the failure of the structure is due to the formation of a full plastic mechanism, the “actual” behavioural curve “OFG” obtained through the non-linear analysis and the line “OBD” relative to the second-order rigid-plastic analysis join at point “F”, in this particular case, point “F” should correspond to point “K” in *Figure 2-8*. If a global frame instability occurs before the development of a plastic mechanism, the “actual” curve remains below the second-order rigid-plastic one “OBD” and point “F” differs from point “K”. This situation is the one illustrated in *Figure 2-8*.

2.2.6 Simplified analytical methods for steel sway frames

2.2.6.1 Introduction

Several simplified analytical methods for frame analysis and design exist and some of them are proposed in Eurocode 3 *[7]*. These methods allow the designer to proceed to structure design without high capacity software which could take account of the sway effects and the non-linearities. Some of them are briefly described in this paragraph.

These methods assume that the materials and the joints have linear or rigid-plastic behaviours and are based on physical and empirical approaches of the problem; the proportion between the two types of approaches depends of the chosen method.

They permit to derive a design load resistance which allows to verify the ULS; nevertheless, the SLS should also be verified for the studied structure. This point will not be recalled in the following paragraphs.

2.2.6.2 Amplified sway moment method

This simplified analytical method is proposed in Eurocode 3 *[7]*. In this method, first-order linear elastic analyses are first carried out; then, the resulting internal forces are amplified by a “sway factor” so as to ascertain for second-order sway effects. Finally, the design load resistance of the frame is derived by computing the load at which a first plastic hinge develops in the frame (→ the elastic load factor $\boldsymbol{\lambda_e}$ is derived).

The steps to be crossed when applying this elastic design procedure are as follows:

- A first-order elastic analysis is performed on the frame fitted with horizontal supports at the floor levels (*Figure 2-9.A*); it results in a distribution of bending moments in the frame and reactions at the horizontal supports.
- Then, a second first-order elastic analysis is conducted on the initial frame subjected to the sole horizontal reactions obtained in the first step (*Figure 2-9.B*); the resulting bending moments are the so-called “sway moments”.

- Approximate values of the "actual" second order moments result from the summing up of the moments obtained respectively in the two frame analyses, after having amplified the sole sway moments by means of the sway factor:

$$\frac{1}{1-\frac{V_{sd}}{V_{cr}}} \tag{2.5}$$

  where V_{sd} is the design vertical applied load and V_{cr} is the lowest elastic critical load associated to a global sway instability.
- The maximum elastic resistance of the frame is reached as soon as a first plastic hinge forms in the frame.

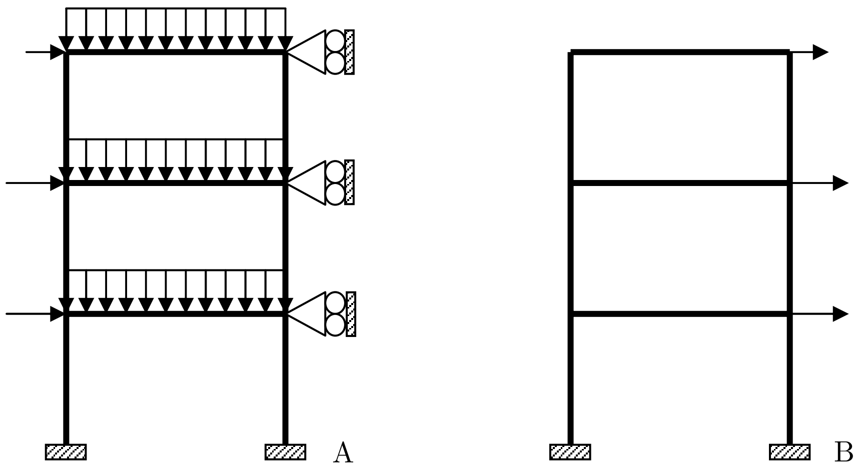


Figure 2-9: static schemes used for the amplified sway moment method

Above design procedure is rather simple, as it only requires first-order elastic analyses. Also the principle of superposition remains applicable, what is especially useful when having to combine several individual loading cases. According to Eurocode 3 *[7]*, the amplified sway moment method is restricted to structures characterized by V_{sd}/V_{cr} ranging from 0.1 to 0.25.

The global stability check is involved in the method. For the in-plane local buckling check of the members, the buckling length for the non-sway mode is used in conjunction with the amplified moments and forces. Out-of-plane stability has also to be checked.

2.2.6.3 Sway-mode buckling length method

The Sway-mode buckling length method is another indirect method to allow for second-order sway effects when using a first-order elastic analysis and is close to the previous method. It may be adopted for structures for which the sway sensitivity is unknown.

The internal forces (moments, shear and axial forces) are computed on the basis of a first-order analysis. The sway moments in beams and joints are then amplified by a nominal factor of 1,2 and afterwards added to the remainder of the moments (those not due to sway).

The so-obtained amplified forces are used for the design checks of joints and member cross sections and member in-plane and out-of-plane stability. The global stability check is included in the method; when checking in-plane local buckling of the members, the sway mode buckling length must be used for member design, which is a main difference according to the amplified sway moment method. Out-of-plane stability must also be checked.

This method often gives too safe results, which explains the fact that this method is rarely used.

2.2.6.4 Merchant-Rankine approach

The origin of this method is described in *[8]*. The Merchant-Rankine method is a second-order elasto-plastic approach, which was developed for bare steel frames; it allows to assess the ultimate load factor through a formula that takes account of interactions between plasticity (λ_p) and instability (λ_{cr}) in a simplified and empirical way. A direct comparison with the ultimate load factor $\boldsymbol{\lambda_u}$ got through a non-linear analysis may be achieved. The Merchant-Rankine basic formula ("MR") writes:

$$\frac{1}{\lambda_u} = \frac{1}{\lambda_{cr}} + \frac{1}{\lambda_p} \qquad (2.6)$$

or:

$$\lambda_u = \frac{\lambda_p}{1 + (\lambda_p / \lambda_{cr})} \ngtr \lambda_p \qquad (2.7)$$

Should the frame be very stiff against sway displacements, then λ_{cr} is much larger than λ_p with the result of a low λ_p/λ_{cr} ratio: a minor influence of the geometrical second-order effects is expectable and the ultimate load is therefore close to the first-order rigid-plastic load. In contrast, a flexible sway frame is characterised by a large value of the λ_p/λ_{cr} ratio. It shall collapse according to a nearly elastic buckling mode at a loading magnitude, which approaches the elastic bifurcation load.

Strain hardening tends to raise plastic hinge moment resistances above the values calculated from the yield strength. Therefore most practical frames with only a few storeys in height attain a failure load at least equal to the theoretical rigid-plastic resistance. When the ratio λ_{cr}/λ_p is commonly greater than 10, the effects of material strain hardening more than compensate those of changes in geometry. Sometimes, additional stiffness due to cladding is sufficient to compensate such changes.

To allow, in a general treatment for the minimum beneficial effects to be expected from both strain hardening and cladding, Wood suggested a slightly modified Merchant-Rankine formula ("MMR"):

$$\lambda_u = \frac{\lambda_p}{0.9 + (\lambda_p / \lambda_{cr})} \ngtr \lambda_p \qquad (2.8)$$

in the range $\lambda_{cr}/\lambda_p \geq 4$. He recommended not to use it in practice when $\lambda_{cr}/\lambda_p < 4$ but to carry a non-linear analysis in this range.

When $\lambda_p/\lambda_{cr} \leq 0.1$, λ_u is limited to λ_p, what means that the frame can be designed according to the simple first-order rigid-plastic theory. A clear and direct relationship may be established between this criterion and the one, which enables, according to Eurocode 3 *[7]*, to classify steel frames as sway ($V_{Sd}/V_{cr} > 0{,}1$) or rigid ($V_{Sd}/V_{cr} \leq 0{,}1$). Similarly, the limitation of the field of application of the amplified sway moment method to V_{sd}/V_{cr} values lower than 0,25 is seen to be strongly related to the here-above expressed $\lambda_{cr}/\lambda_p \geq 4$ range of application of the modified Merchant-Rankine formula.

The use of *Formula (2.8)* is commonly restricted to frames in buildings, in which:

1. the frame is braced perpendicular to its own plane;
2. the average bay width in the plane of the frame is not less than the greatest storey height;
3. the frame does not exceed 10 storeys in height;
4. the sway at each storey, due to non-factored wind loading, does not exceed 1/300 of the storey height;
5. $\lambda_{cr}/\lambda_p \geq 4$.

From complementary studies carried out at Liège University *[8]*, the "MMR" approach is seen to exhibit a different degree of accuracy according to the type of first-order rigid-plastic failure mode which characterises the frame under consideration:

- safe for beam plastic mechanisms;
- adequate for combined plastic mechanisms;
- unsafe for panel plastic mechanisms.

As a result, the application of the "MMR" approach to structures exhibiting a first-order panel plastic mechanism should therefore be prohibited.

In *[8]*, the scope of the "MMR" formula is extended to structures with semi-rigid and/or partial-strength joints.

The global stability check is involved in the "MR" and "MMR" methods but the local stability of the members must be checked.

2.2.6.5 Simplified second-order plastic analysis

As an alternative to a second-order elastic-plastic analysis, Eurocode 3 *[7]* allows the use of rigid-plastic first-order analysis for particular types of sway frames. As for the indirect methods with first-order elastic analysis, the second-order sway effects are accounted for indirectly by multiplying moments (and associated forces) by a similar magnification factor. However in this case, all of the internal moments (and associated forces) are magnified (and not just those due to sway alone as it is done in the elastic analysis case – see *§ 2.2.6.2* and *2.2.6.3*). The limitation imposed on its use excludes the use of slender members for which member imperfections would have to be accounted for. The magnification factor is the same as for the first-order elastic analysis (*Formula (2.5)*).

M. Pecquet demonstrated in his diploma work *[3]* that this method is equivalent to the Merchant-Rankine approach if this method is used so as to predict the ultimate load factor λ_u; so, this method will not be presented with more details.

2.2.6.6 Wind moment method

The wind moment method is a British one (mainly used in North America and UK) which is fully empirical. It is closer to a pre-design method than an analytical one. It permits the pre-design of sway structures with semi-rigid joints.

The distinguishing factors of this method, that set it apart from other methods, is the assumptions that are used during the design stage *[1]*:

- under gravity loads, the connections are assumed to act as pins (*Figure 2-10.a*);
- under wind loads the connections are assumed to behave as rigid joints, with points of contraflexure occurring at mid-height of the columns and at mid-span of the beams (*Figure 2-10.b*).

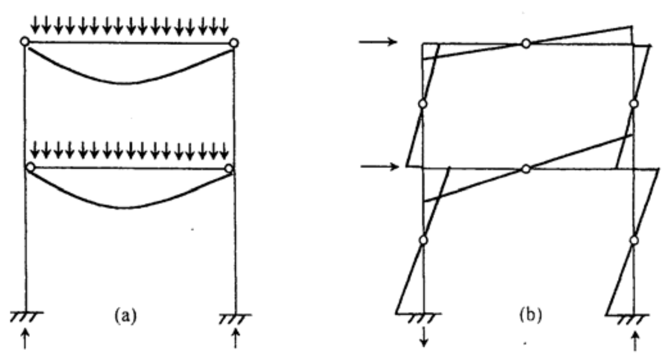


Figure 2-10: internal moments and forces according to the wind-moment method *[1]*

The first step in the design sequence is to design the beams under gravity loads. Then the frame is analysed under horizontal wind loads, with the assumption that the beam-to-column connections behave in a rigid manner. The internal forces and moments are then combined using the principle of superposition. The design for ULS is then completed by amending the initial section sizes and connection details, to withstand the combined effects.

P-Δ effects are accounted for by designing the columns using effective lengths that are greater than the true column lengths. The sway displacements are computed assuming the beam-to-column joints are rigid; a sway factor is used to account for their true behaviour.

The advantage of this method is its simplicity, as the frame is rendered statically determinate. Nevertheless, the horizontal beams tend to be overdesigned (as the joints at their extremities are assumed to be hinges) when vertical columns tend to be underdesigned (as the bending moments coming from the beams are neglected). The joints are also overdesigned as hogging moments coming from the vertical loads are neglected.

2.2.7 Conclusions

In *§ 2.2*, we have described in details the different steps to cross for the selection of a global structural analysis according to EC3 and the structural analyses available for the determination of the internal forces of a frame; the different verifications to perform which are associated to the chosen analysis have also been described. In addition, we have also introduced in *§ 2.2.6* simplified analytical methods available for steel sway frames.

Some of the global structural analyses presented in this paragraph will be applied to sway composite frames through the numerical investigations presented in *Chapter 5*. The applicability of some simplified analytical methods developed for steel sway frames to composite ones will be investigated in *Chapter 6*.

2.3 Composite structures

2.3.1 Introduction

Eurocode 4 *[15]* presents design rules for non-sway steel-concrete composite buildings and gives mainly rules to analyse and to check structural elements like beams, columns, slabs and joints. The present paragraph is devoted to the presentation of the main characteristics of the constitutive elements of a composite structure (*Figure 2-11*): composite slabs (*§ 2.3.2*), composite beams (*§ 2.3.3*), composite columns (*§ 2.3.4*) and composite joints (*§ 2.3.5*).

The objective of the paragraph is not to describe all the design procedures for these elements but to introduce the properties and rules which will be used in the following chapters.

This paragraph is inspired by the following documents:

- Lectures from SSEDTA (*[10]*, *[11]* and *[12]*);
- "Composite construction elements" of R. Maquoi and F. Cerfontaine *[13]*;
- Eurocode 4 (*[9]* and *[15]*);
- COST C1 document *[14]*.

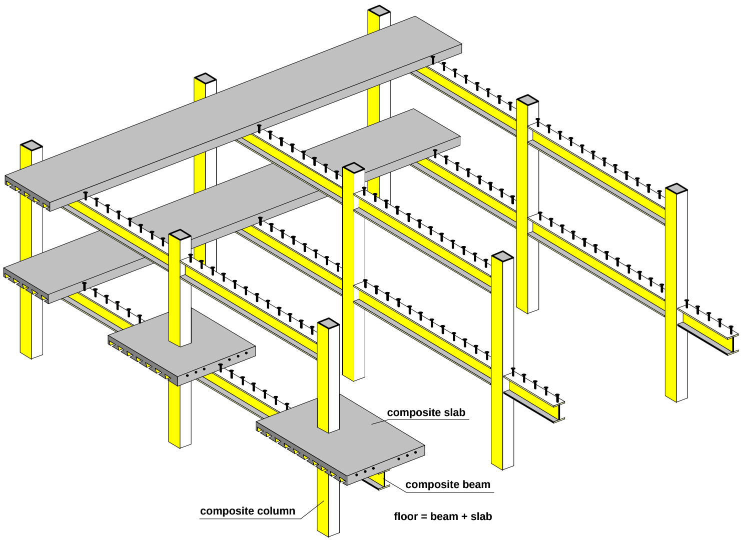


Figure 2-11: composite buildings with its constitutive elements

2.3.2 Composite slabs

All the structures studied in the thesis are structures with composite slabs (see *Chapter 3*): concrete slab with a collaborating profiled steel sheeting (*Figure 2-12*). The main advantages of this type of slab are:

- the profiled steel sheeting provides an excellent safe working platform at the working stage which speeds the construction process;
- the steel sheeting is a structural part of the slab as it can be considered as tensile reinforcement at the bottom of the hardened concrete slab.

In a composite slab, an interlock must be provide to ensure the interaction between the steel and the concrete elements. These interlocks can be of several types as:

- chemical interlock which is very brittle and unreliable (not consider in the computations);
- frictional interlock which is not able to transfer large shear forces;
- mechanical interlock by interlocking embossments of the steel decking;
- end anchorage like headed bolts, angle studs or end-deformations of the steel sheeting which brings a very concentrated load introduction at the ends.

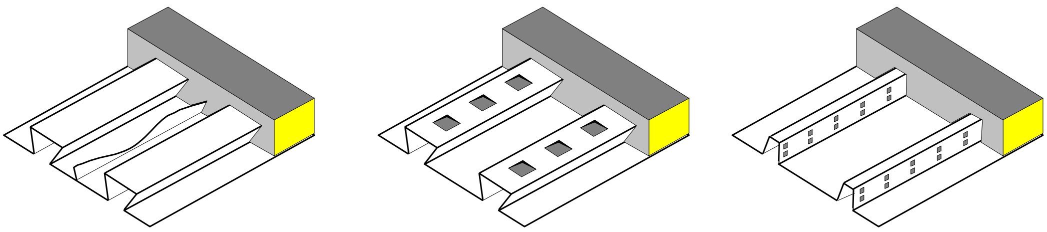


Figure 2-12: example of composite slabs

According to Eurocode 4 *[9]*, the verifications of composite slabs for the ULS include (when the concrete is hardened):

- the verification of the bending resistance under sagging and/or hogging moments at the critical cross sections;
- the verification to the longitudinal shear with or without end anchorage;
- the verification to the vertical shear;
- the verification to the punching shear.

The verifications for the SLS involve the check of the concrete cracking in hogging moment regions, the verification of the deflections and the verification for the vibrations.

2.3.3 Composite beams

A composite beam is composed of a steel beam connected to a concrete or composite slab through shear connectors which ensure the collaboration between the steel and the concrete elements. In the studied structures (see *Chapter 3*), all the composite beams are I steel profiles connected to composite slabs through stud shear connectors; in some cases, the I steel profiles can be partially-encased ones (*Figure 2-13*).

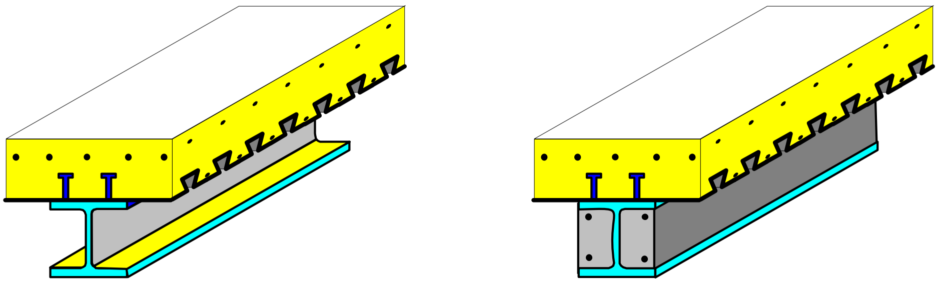


Figure 2-13: examples of composite beams (I steel profiles connected to composite slabs)

In sagging moment regions, compression stresses are introduced in the concrete slab through the connectors. The induced stress distribution in the slab is not uniform: it is higher close to the steel beam and decreases progressively away from the beam. This phenomenon is known as **shear lag**. In order to treat a composite floor as an assembly of independent tee-sections, the concept of an effective width $\boldsymbol{b_{eff}}$ of the slab is introduced (*Figure 2-14*); a width of slab is associated with each beam such that the normal flexural constraint calculated by Navier's assumption and applied to the composite section thus defined, would provide the same maximum constraint as that originating in the actual non-uniform distribution. The value of b_{eff} depends, in quite a complex manner, on the relation of the spacing $2b_i$ (see *Figure 2-14*) to the span L of the beam, on the type of load, on the type of supports to the beam, on the type of behaviour (elastic or plastic) and on other factors besides. That is why in the domain of building, most of the design codes are satisfied with simple safe formulae. Eurocode 4 proposes the following expression:

$$b_{eff} = b_{e1} + b_{e2} \tag{2.9}$$

with b_{ei} = min ($L_o/8$; b_i) where L_o is equal to the distance measured between consecutive points of contraflexure in the bending moments diagram.

For a beam on two supports, the length L_o is equal to the span L of the beam. For continuous beams, L_o is determined from *Figure 2-15*.

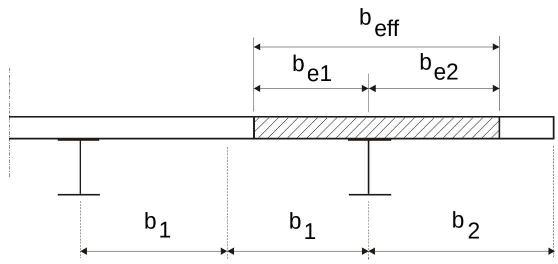


Figure 2-14: effective width of slab for beam

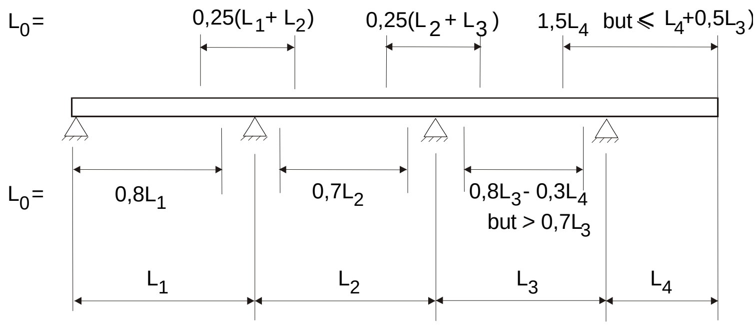


Figure 2-15: lengths L_0 for the determination of effective width

Important remark: in *Figure 2-15*, the case of a hogging bending moment at a beam extremity is not covered although such force can appear under gravity loads when the beam extremity is rigidly (or semi-rigidly) connected to a column. In the following chapters, L_0 will be assumed to be equal to "0.25 L" in this case, with L equal to the length of the adjacent span.

Without numerical tools, only two types of analyses are available according to Eurocode 4: a first-order rigid-plastic analysis (*§ 2.2.5.5*) or a first-order elastic analysis (*§ 2.2.5.2*). For the elastic analysis, the concrete cracking can be taken into account by two different ways:

- uncracked elastic analysis: a constant inertia is considered all along the composite beam to compute the internal forces (*Figure 2-16*); this inertia is computed by assuming that the concrete is uncracked. The so-obtained bending moment distribution are then redistributed to include the concrete cracking, the non-linear behaviour and all types of buckling (the percentages of redistribution are given in Eurocode 4 according to the class of the composite beam cross sections)
- cracked elastic analysis: in this analysis, a smaller inertia is adopted in the support regions (15 % each side of the support - *Figure 2-16*); the same inertia than for the uncracked elastic analysis is kept for the other regions. The so-obtained bending

moment distribution is then redistributed but without including a redistribution for the concrete cracking, as this factor is included in the performed analysis (the percentage of redistribution is smaller than for the uncracked analysis).

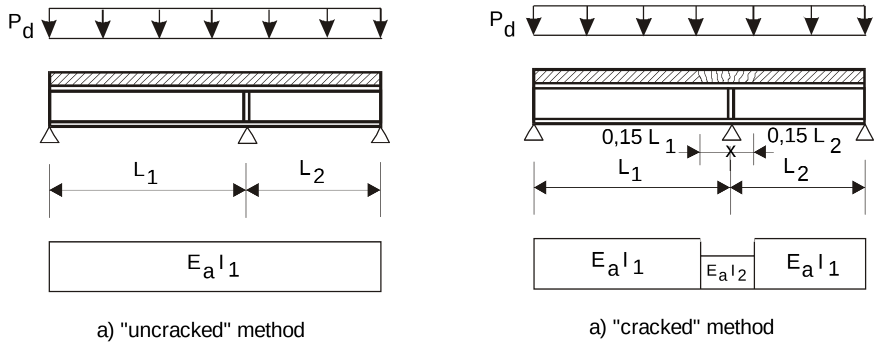


Figure 2-16: elastic analysis methods

According to Eurocode 4 *[9]*, the verifications of composite beams for ULS include (when the concrete is hardened):

- the verification of the bending resistance under sagging and/or hogging moments at the critical cross sections (elastic or plastic resistant moment);
- the verification of the vertical shear resistance (with account of the local shear buckling phenomenon if needed);
- the verification of the shear – bending moment interaction if needed;
- the verification to the lateral-torsional buckling;
- the verification of the shear connection between the steel profile and the concrete or composite slab (verification of the connectors and verification of the transversal reinforcement resistance).

The verifications for SLS must involve the verifications for the concrete cracking, for the beam deflections and for the vibrations.

2.3.4 Composite columns

Some examples of cross sections of composite columns are given in *Figure 2-17*. Only partially-encased composite columns (*Figure 2-17.b*) are used in the studied structures presented in *Chapter 3*.

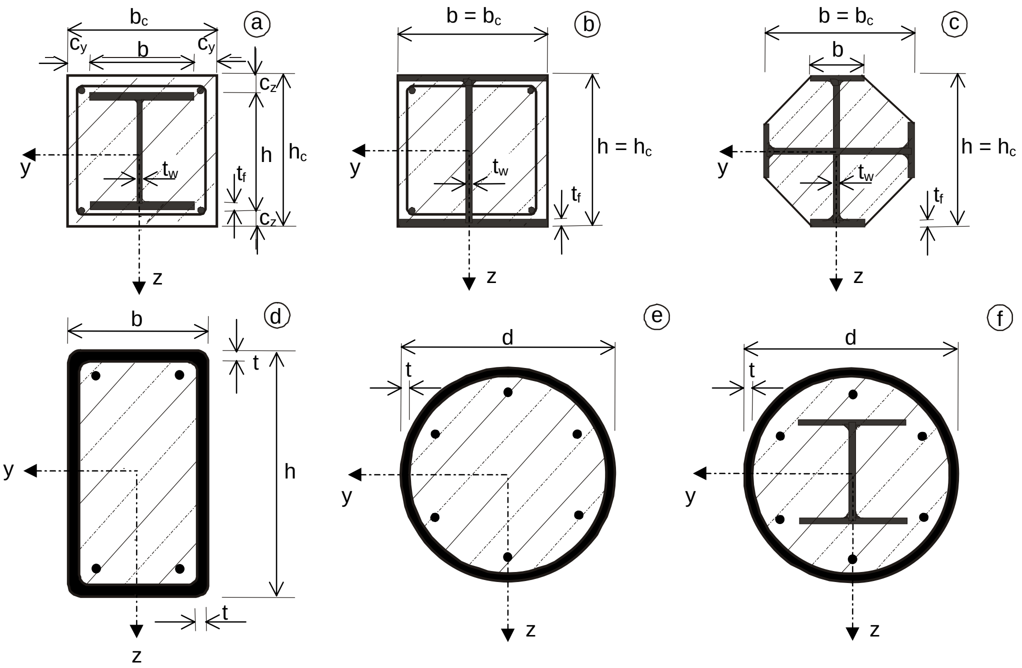


Figure 2-17: typical cross sections of composite columns

Eurocode 4 proposes two different methods to design these members:

- a general one which takes account of the second-order effects and the imperfections of these members;
- a simplified one which is based on the European buckling curves.

To apply the simplified method, the studied member must respect some conditions given in Eurocode 4.

The following verifications must be performed for the ULS:

- verification of the bending moment resistance if moments are applied;
- verification of the axial compression resistance with account of the buckling phenomenon;
- resistance of the shear resistance (with account of the local shear buckling phenomenon);
- verification of the interactions between the different types of internal forces which can occurs at the same time. For instance:
  - verification of the M-N interaction through the M-N interaction curve (*Figure 2-18*);
  - verification of the interaction between the shear forces and the bending moments;
  - ...
- verification of the shear connection and load introduction in the vicinity of the beam-to-column or column-to-column joints.

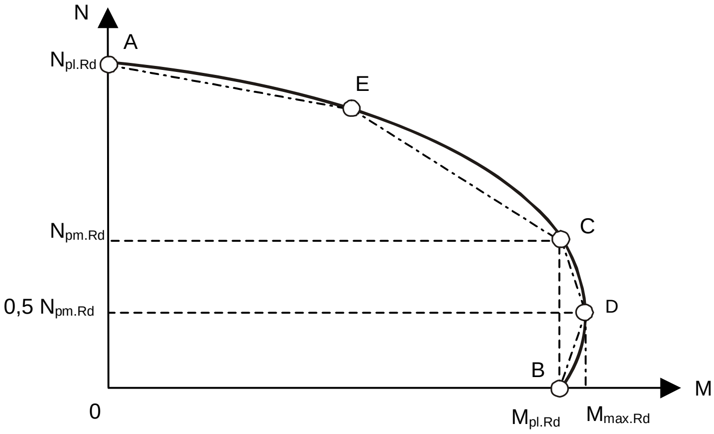


Figure 2-18: example of a M-N interaction curve

For the SLS, the lateral deflections of the columns must be checked.

2.3.5 Composite joints

2.3.5.1 Introduction

According to Eurocode 4 *[9]*, a composite joint is a joint between composite members where the concrete slab reinforcements contribute to the joint resistance and stiffness. So, the slab reinforcements must be continuous in the vicinity of the joint. Examples of composite joint configurations are given in *Figure 2-19*.

An important step when designing a frame consists in characterising the rotational response of the joints, i.e. to evaluate the mechanical properties in terms of stiffness, strength and ductility. The general procedure for the characterisation is presented in *§ 2.3.5.2*. Then, *§ 2.3.5.3* gives different criteria allowing the joint classification in accordance with their properties. Finally, *§ 2.3.5.4* presents different possibilities for the joint modelling and idealisation in accordance with the joint classification and the chosen global analysis.

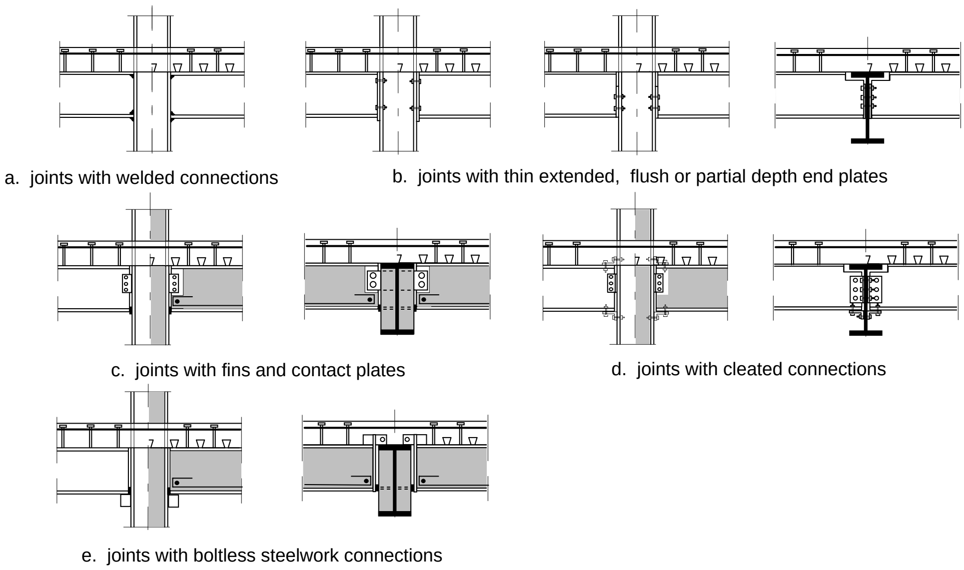


Figure 2-19: examples of composite joint configurations

2.3.5.2 Joint characterisation

The objective of the joint characterisation is to get their main properties (*Figure 2-20*):

- the initial stiffness $\boldsymbol{S_{j,ini}}$;
- the design moment resistance $\boldsymbol{M_{Rd}}$;
- the shear resistance $\boldsymbol{V_{Rd}}$;
- the design rotation capacity $\boldsymbol{\phi_{cd}}$.

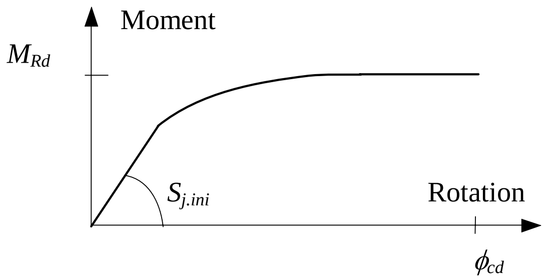


Figure 2-20: M-ϕ behaviour curve of a joint

Three main approaches may be followed for the characterisation:

- experimental;
- numerical;
- and analytical.

The only practical option for the designer is the analytical approach. Although the ENV for Eurocode 4: Part 1.1 *[15]* describes what is meant by a composite connection, no design rules are given; this contrasts with Eurocode 3, which provides detailed rules for the common types of moment-resisting steel joints. Recently, revised provisions have been published in an amendment to the ENV for Eurocode 4 *[9]*. Chapter 8 provides model clauses for the design of composite joints. These are compatible with the revised rules for steel joints.

The general analytical procedure proposed in Eurocode 3 and 4 (*[16]* and *[9]* respectively) is the **component method**. It applies to any type of steel or composite joints, whatever the geometrical configuration, the type of loading (axial force and/or bending moment, ...) and the type of member sections.

The component method considers any joint as a set of **individual basic components**. For the particular joint shown in *Figure 2-19.b* (composite joint configuration with a flush end-plate connection subjected to hogging bending), the relevant components are the following:

Compression zones:

- column web in compression;
- beam flange and web in compression;

Tension zones:

- column web in tension;
- column flange in bending;

- bolts in tension;
- end-plate in bending;
- beam web in tension;
- slab rebars in tension;

Shear zone:

- column web panel in shear.

Each of these basic components possesses its own strength and stiffness either in tension or in compression or in shear. The column web is subject to coincident compression, tension and shear. This coexistence of several components within the same joint element can obviously lead to stress interactions that are likely to decrease the resistance of the individual basic components.

The application of the component method requires the following steps:

a) **identification** of the active components in the joint being considered;

b) **evaluation of the stiffness and/or resistance characteristics** for each individual basic component (specific characteristics - initial stiffness, design resistance, ... - or the whole deformability curve);

c) **assembly** of all the constituent components and evaluation of the stiffness and/or resistance characteristics of the whole joint (specific characteristics - initial stiffness, design resistance, ... - or the whole deformability curve).

In *Figure 2-21*, the principles of the component method are illustrated in the specific case of a beam-to-column steel joint with a welded connection.

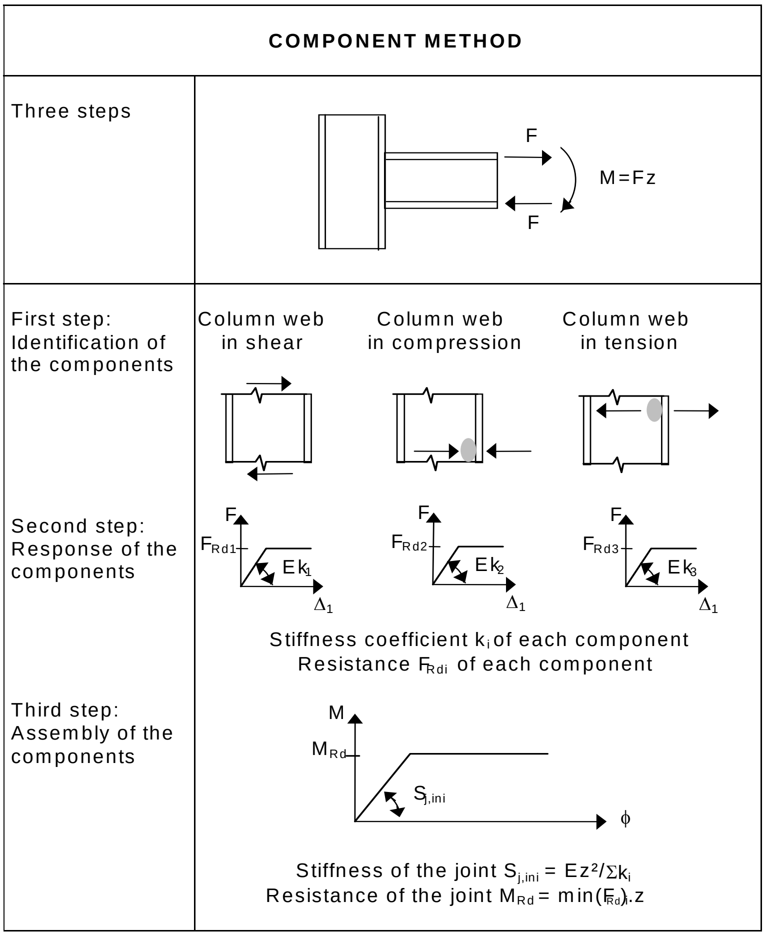


Figure 2-21: application of the component method to a welded joint

The assembly procedure consists in deriving the mechanical properties of the whole joint from those of all the individual constituent components. This requires a preliminary distribution of the forces acting on the joint into internal forces acting on the components in a way that satisfies equilibrium. In Eurocode 3 and 4 (*[16]* and *[9]*), the analytical assembly procedures are described for the evaluation of the initial stiffness $S_{j,ini}$ and the design moment resistance M_{Rd} of steel and composite joints respectively.

The application of the component method requires a sufficient knowledge of the behaviour of the basic components. Those covered by Eurocode 3 for steel joints are listed in *Table 2-4* (components 1 to 12); those covered by Eurocode 4 for composite joints are the same than for steel joints with two additional components also presented in *Table 2-4* (components 13 and 14). The combination of these components allows one to cover a wide range of joint configurations, which should be sufficient to satisfy the needs of practitioners as far as steel and composite beam-to-column joints and beam splices in bending are concerned.

Table 2-4: components covered by Eurocode 3 and Eurocode 4

N°	Component
1	Column web panel in shear
2	Column web in compression
3	Beam flange and web in compression
4	Column flange in bending
5	Column web in tension
6	End-plate in bending
7	Beam web in tension
8	Flange cleat in bending
9	Bolts in tension
10	Bolts in shear
11	Bolts in bearing (on beam flange, column flange, end-plate or cleat)
12	Plate in tension or compression
13	Reinforcement in tension
14	Contact plate in compression

2.3.5.3 Joint classification

The joints can be classified according to three different criteria: their stiffness, their resistance and their ductility. The classification of the joint depends of the stiffness and resistance properties (I_b and $M_{pl,Rd}$ respectively) of the closest beam cross section.

The classification criterion according to the stiffness properties is the following (E = young modulus of the beam material and L = span of the beam):

- if $S_{j,ini} > 25\dfrac{EI_b}{L}$, the joint is classified as **rigid** → there is no relative rotations between the connected members;
- if $S_{j,ini} < 0.5\dfrac{EI_b}{L}$, the joint is classified as **nominally pinned** → a free rotation is assumed at the joint level;
- in all the other cases, the joint is classified as **semi-rigid**.

The classification criterion according to the resistance properties is the following:

- if $M_{Rd} \geq M_{pl,Rd}$, i.e. the joint resistance is higher than the beam one, the joint is classified as **full strength**;
- if $M_{Rd} < 0.25M_{pl,Rd}$, it is assumed that the joint transmits no moments and the latter is classified as **nominally pinned**;
- in all the other cases, the joint is classified as **partial strength**.

The classification according to the ductility of the joint is associated to the rotation capacity of the joint which depends of its collapse mode. Plastic hinges must form in ductile joints; so, the realisation of a plastic analysis is possible if the studied structure is composed of ductile joints (if plastic hinges must form at these places). The joints which are not ductile collapse through a brittle failure or through a local instability phenomenon.

2.3.5.4 Joint modelling and idealisation

In a strong axis beam-to-column joint configuration, two main sources of deformability are identified (see *Figure 2-22*):

- the deformation of the connection associated to the deformation of the connection elements (end-plate, angles, bolts,...), to that of the column flange and to the load-introduction deformability of the column web;
- the shear deformation of the column web associated mostly to the pair of forces F_b carried over by the beam(s) and acting on the column web at the level of the joint.

These components are illustrated in *Figure 2-22* for the particular case of a joint between a column and a one-sided beam ("single-sided beam-to-column joint configuration" according to Eurocode 3). The flexural deformability of the connection elements is concentrated at the end of the beam (see *Figure 2-22.a*). The associated behaviour is expressed in the format of a moment-rotation $M_b - \phi$ curve.

The deformation of the ABCD column web panel is divided into:

a) The load-introduction deformability which consists in the local transverse deformation of the column web in both tension and compression zones of the joint (respectively a lengthening and a shortening) and which results in a relative rotation ϕ between the beam and column axes; this rotation concentrates mainly along edge BC (see *Figure 2-22.b*) and provides also a moment-rotation deformability curve $M_b - \phi$.

b) The shear effect – due to shear force V_n – which results in a relative rotation γ between the beam and column axes (see *Figure 2-22.c*); this rotation makes it possible to establish a second deformability curve $V_n - \gamma$.

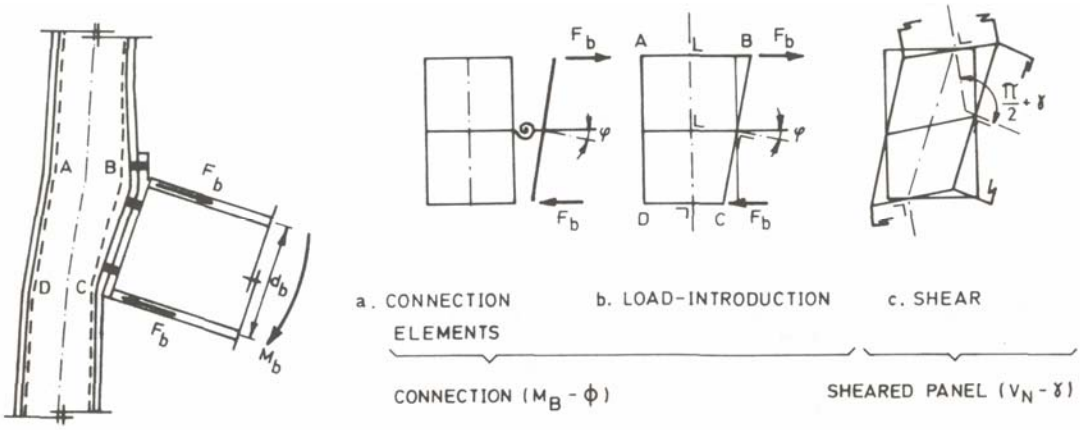


Figure 2-22: deformation of a single-sided beam-to-column joint configuration & main components of joint deformability

It is very important to stress that the deformability of the connection (connection elements + load-introduction) is only due to the forces carried over by the flanges of the beam (equivalent to the beam moment M_b), while the shear in a column web is the result of the combined action of these equal but opposite forces and of the shear forces in the column at the level of the beam flanges. In fact, the actual value of the shear force $\boldsymbol{V_n}$ may be obtained from the equilibrium equations of the web panel *[17]*; it is given by the following formula (see *Figure 2-23*):

$$V_n = \frac{M_{b1} + M_{b2}}{d_b} - \frac{V_{c1} + V_{c2}}{2} \tag{2.10}$$

The difference between the loading of the connection and that of the column web in a specified joint requires, at a theoretical point of view, that account be taken separately of both deformability sources when designing a building frame (see *Figure 2-23.a*). However, doing so is only practicable when the frame is analysed by means of a sophisticated computer program allowing for the separate modelling of both deformability sources. In all other cases, the actual behaviour of the joints must be simplified by concentrating the whole deformability at the beam end or at the intersection of the beam and column axes as represented in *Figure 2-23.b. Reference [18]* gives guidelines on how to "concentrate" the joint deformability in an accurate and safe manner for design practice and *Reference [19]* justifies the "transfer" of the rotational spring from its actual position at the connection level (see *Figure 2-23.a*) to the intersection of beam and column axes (see *Figure 2-23.b*).

In Eurocode 3 and Eurocode 4, the simple solution resulting from the "concentration" is recommended and the values of the mechanical joint properties (stiffness, resistance, ductility) are computed accordingly. This last solution for the joint modelling will be used in the different numerical investigations performed in the following chapters (see *Chapter 5*).

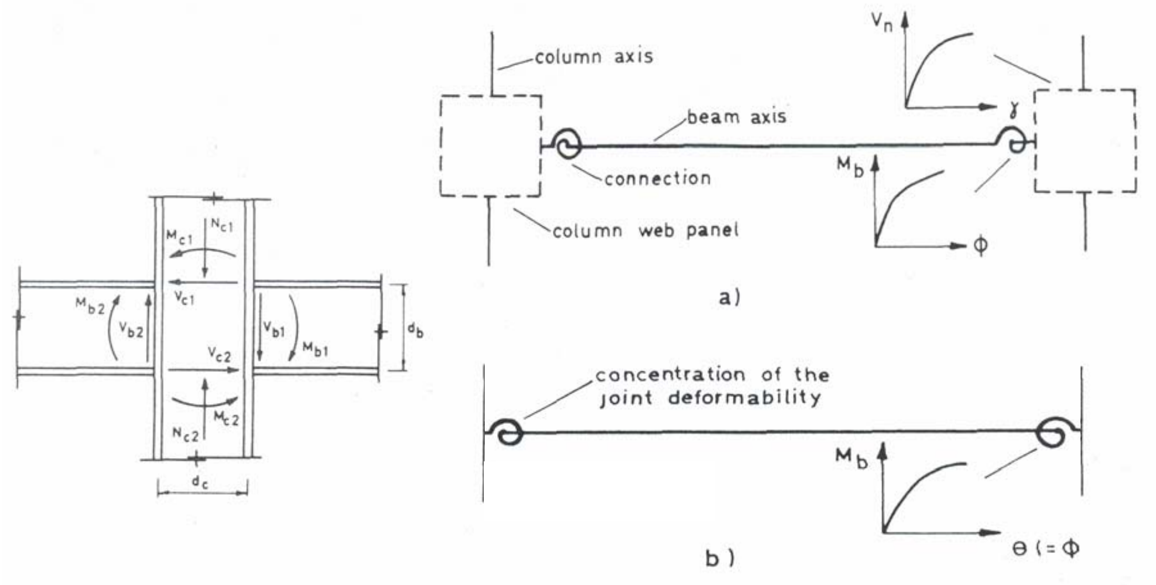


Figure 2-23: loading of a double-sided beam-to-column joint configuration & actual (a) and simplified (b) joint modelling

Depending on the software available for frame analysis, either the full non-linear shape of the joint characteristics or multi-linear simplifications can be assigned to the rotational springs (*Figure 2-24*). Further simplifications may also result from the nature of the method used for global analysis:

- if an elastic analysis is performed (see *§ 2.2.5.2*, *§ 2.2.5.3* and *§ 2.2.5.4*), only the properties of joint stiffness are important as the structure members and joints are assumed to be indefinitely elastic;
- if a rigid-plastic analysis is performed (see *§ 2.2.5.5* and *§ 2.2.5.6*), only the properties of resistance are important as the joints are assumed to be rigid until their yielding;
- if a full non-linear analysis is performed (see *§ 2.2.5.7*), both the stiffness and the resistance properties are important and must be involved in the analysis.

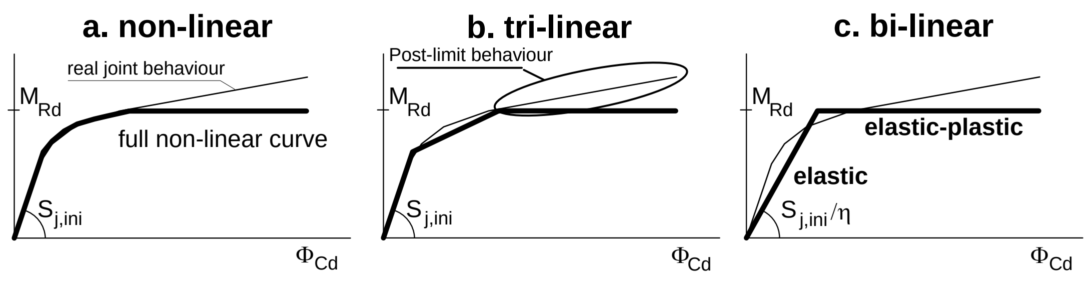


Figure 2-24: possibilities for curve idealisation

The most simple approximation is to represent the joint characteristic by a bi-linear curve (*Figure 2-24.c*). The joint stiffness is constant for all values of bending moment smaller than the design moment resistance. An appropriate stiffness can be calculated by dividing the initial joint stiffness $S_{j,ini}$ by a modification factor, denoted η. This depends on the type of steelwork connection and on the joint configuration. For example, $\eta = 2.0$ for a beam-to-column joint with a flush end-plate and 1.5 for a joint with a contact plate. For an indefinitely elastic analysis, the following assumptions are proposed in Eurocode 3 and 4:

- if $M_{Sd} \leq \frac{2}{3} M_{Rd}$, the joint stiffness is assumed to be equal to $S_{j,ini}$;
- if $M_{Sd} \leq M_{Rd}$, the joint stiffness is assumed to be equal to $S_{j,ini}/\eta$.

2.3.6 Conclusions

In *§ 2.3*, we have proceeded to a general introduction to the main characteristics of composite structures and their constitutive elements.

We have first given a description of the composite slabs with the design recommendations for such structural members. Then, we have presented the main properties of the composite beams; the important concept of effective width has been introduced in this part. The latter will be used in the following chapters. In the following paragraph, we have introduced some examples of composite column cross sections and the different verifications to perform for their design. Finally, we have proceeded to the presentation of the composite joint properties with the different steps to cross in order to introduce their semi-rigid and partial-strength behaviour in the frame analysis: the characterisation, the classification, the idealisation and the modelling.

2.4 Chapter conclusions

Until now, the knowledge about the behaviour of sway composite frames is weak; few investigations in this field have been performed. In this chapter, we have introduced the different concepts available in Eurocode 3 for the analysis of steel sway frames and the main rules given in Eurocode 4 for the design of the constitutive elements of non-sway composite frames. The concepts presented herein will be applied to composite sway buildings in *Chapter 5* and *Chapter 6*.

Chapter 3 : Studied Buildings

3.1 Introduction

Five actual buildings in which sway effects are likely to occur under static loading have been selected:

- the “Ispra” building;
- the “Bochum” building;
- the “UK” building;
- the “Eisenach” building;
- the “Luxembourg” building.

The difficulty in this task was to collect, for each building, enough data such as those on geometry, material properties and joint detailing; these ones strongly influence the global structural response. These structures are briefly described here below.

3.2 “Ispra” building

This 3-D full-scale building has been tested in Ispra (Italy) in seismic conditions in the field of the above-mentioned ECSC project. Tests on isolated joints have also been performed so as to get the actual properties of its constitutive structural joints; however, all the experimental results were not available when the numerical investigations were performed.

Two different configurations of this structure have been considered within the project: they aimed at resisting respectively static loading and seismic loading (*[20]* and *[21]*). Only the investigations performed on the first configuration are developed later on in *§ 5.4.2*, as a previous study *[3]* demonstrated that only this configuration can be qualified as sway under static loading with regards to Eurocode 3 criterion (see *Formulas (2.1)* and *(2.2)* in *§ 2.2.4.2*). This is the latter which is described herein.

The “Ispra” building is 12 m long and is composed of two-storey two-bay frames that are 12 m large, 7 m high and 3 m paced (*Figure 3-1*). These frames resist in-plane loading by frame action without exhibiting out-of-plane deformation because they are braced in the perpendicular direction.

The steel HEB200 columns are partially encased (*Figure 3-2.a*). The steel beams are made of IPE300 structural shapes; the upper flange of the profiles is connected to the composite slab by means of stud connectors. The slab reinforcement is constituted of a T6x6x150x150 mesh with two additional rebars with a diameter of 16 mm in the hogging moment zones; the covering of the rebars is equal to 30 mm. The collaborating hollow rib is an EGB210 one with a thickness of 1mm, from BROLLO (Italy) and the ribs are perpendicular to the steel beam axis; the total height of the slab is equal to 120 mm.

All the moment resistant joints develop a composite action and are classified as semi-rigid and partial-strength (see *§ 2.3.5.3*). The end-plate thickness is equal to 12 mm and the bolts are M20 10.9 ones (*Figure 3-2.b*). The column bases are ideally pinned. One of the constitutive frames is represented in *Figure 3-3*.

The steel grades for the constitutive elements are:

- S235 for the profiles and for the joint components;
- S500 for the rebars;
- S320 for the hollow rib.

The concrete class for the composite slab and the composite columns is C25/30.

In addition to the self-weight of the structure, a permanent load of 1,5 kN/m^2 and an imposed service load of 5 kN/m^2 are uniformly applied on both floors. The wind load are applied on the frame through concentrated loads at each floor level: 6.8 kN for the first level and 3.4 kN for the second one.

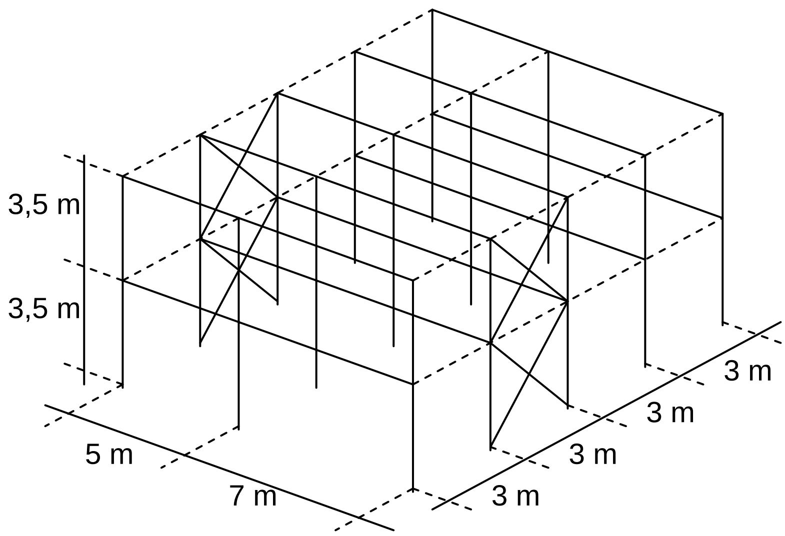


Figure 3-1: general layout of the 3-D "Ispra" building

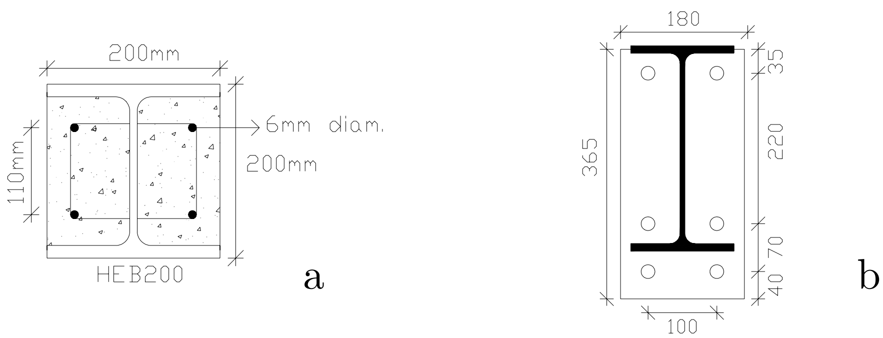


Figure 3-2: composite column and joint details

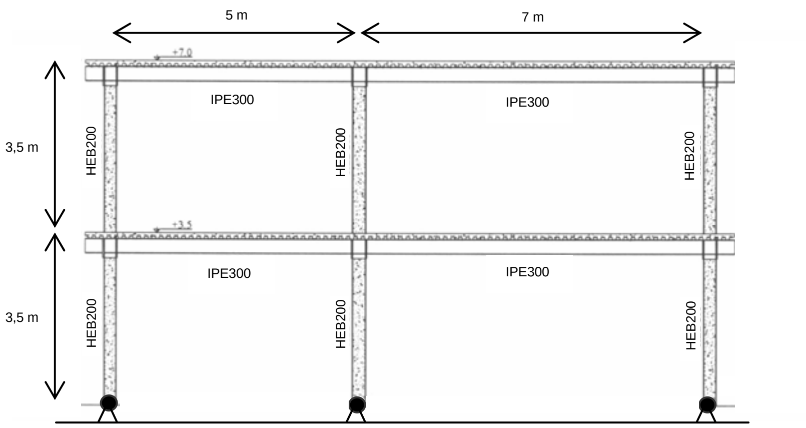


Figure 3-3: isolated composite frame of the "Ispra" building

3.3 *"Bochum" building*

It is a 2-D full-scale structure tested in Bochum (Germany) under static loading in the field of the above-mentioned ECSC project *[22]*. Also tests on joints in isolation have been performed but all the relevant experimental results were not available when the numerical investigations were performed.

The "Bochum" building (*Figure 3-4*) is a two-bay two-storey frame. The total height is 4,99 m and the total width is 9,76 m. We have designed the latter in strong collaboration with Bochum University so as to fail by global in-plane instability *[23]*.

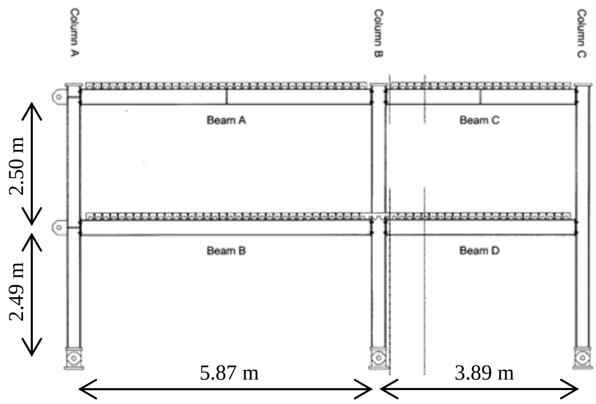


Figure 3-4: general layout of the "Bochum" frame test

Columns A and C are made of HEB260 profiles and column B of a HEB280 one. The IPE300 beams have their upper flange connected to the composite slab by means of shear studs. The composite slab thickness and width are 120 and 1200 mm respectively. The slab reinforcement is constituted of a T6x6x150x150 mesh with four additional rebars with a diameter of 12 mm at the first storey internal composite joint vicinity; the covering of the rebars is equal to 25 mm. The collaborating hollow rib is the same than the one used for the "Ispra" building (EGB210 from BROLLO).

All the moment resistant joints are steel ones except the internal joint at the first storey which is a composite one (*Figure 3-5*); these joints were designed so as to develop ductile modes of collapse with account of over-strength effects *[23]*. The configuration of the steel components of these joints are the same. According to Eurocode 3 and 4 and to the classification criteria given in *§ 2.3.5.3*, all the joints are classified as semi-rigid and partial-strength ones. Main details of these joints are given in *Figure 3-5*.

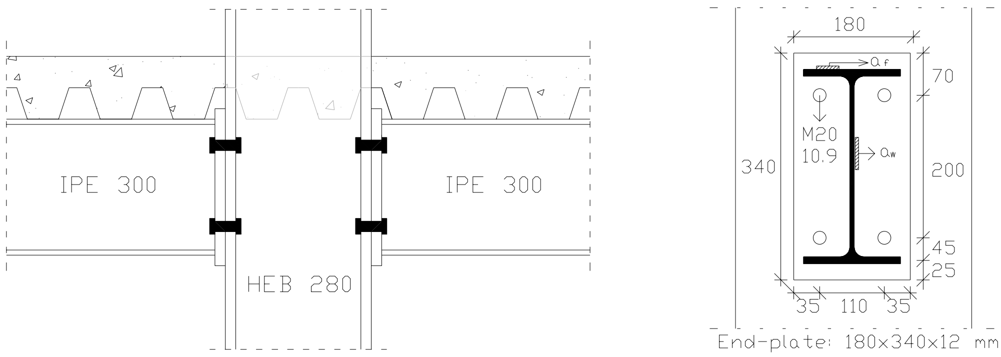


Figure 3-5: internal composite joint and joint details of the "Bochum" building

The steel and concrete grades of the different structural elements are the same than the ones used for the "Ispra" building (*§ 3.2*).

In accordance with the experimental facilities at Bochum University, the applied loads on the frame are as follows:

- a load of 400 kN applied at the top of each column; it is supposed to represent the gravity loads transmitted by the upper storeys;
- uniform and concentrated gravity loads as indicated in *Figure 3-6*;
- horizontal loads of 50 kN applied at both floor levels.

For testing, the loading sequence was the following: all the gravity loads are first increased up to their nominal values; they are then kept constant while the horizontal loads are progressively magnified by a load factor λ till failure (see *Figure 3-6*). This loading sequence is also the one used for the numerical analyses performed on this building in *§ 5.4.3*. More details about this structure are given in *[22]*.

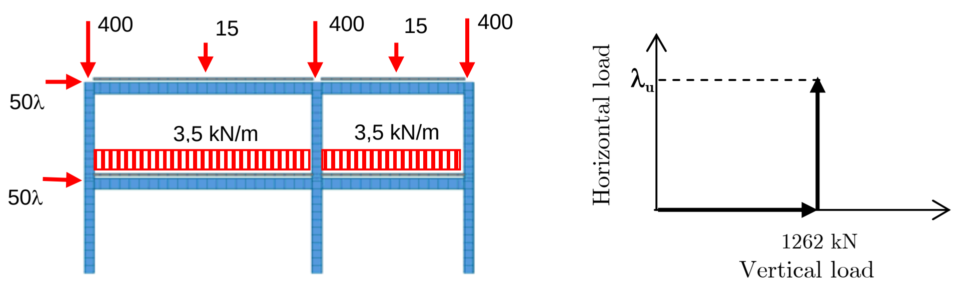


Figure 3-6: loading conditions & loading sequence for the "Bochum" frame (kN)

3.4 "U.K." building

The "U.K." building is a 3-D structure tested at BRE (Building Research Establishment), U.K. The test report is well documented (yield strengths, dimensions, type of loading); in particular, the behavioural curves of the structural joints are given (see *[24]* and *[25]*). In consequence, this building is the one used for the benchmark study presented later on in *§ 5.3*.

The structure (*Figure 3-7*) is composed of two parallel two-storey two-bay main frames (namely "Frame A" and "Frame B") connected by secondary beams. The "203x203 UC 46" bare steel columns support floors consisting in composite slabs; the latter are connected by shear studs to the top flanges of the sole "254x102 UB 25" primary beams. As the main purpose of the frame tests was to investigate semi-rigid joint effects on overall frame behaviour, a finite width of concrete slab (1 m) was used instead of the full floor slab layout; in the latter four 12 mm and four 10 mm high yield bars were used as longitudinal reinforcement. Flush end-plate joint configurations were used for all beam-to-column joints.

In Frame A, all the columns are bent about their major axis, while they are about their minor axis in Frame B. Both frames are subjected to concentrated loads F applied at one third and two thirds of each beam span (*Figure 3-7*). These ones are proportionally increased (λ load factor) until failure is reached, except for the lower right beam where these loads are kept constant as equal to "F". For Frame A, the nominal value of F is equal to 37 kN; for frame B, the latter is equal to 39 kN.

More details about this structure are available in *[24]* and *[25]*.

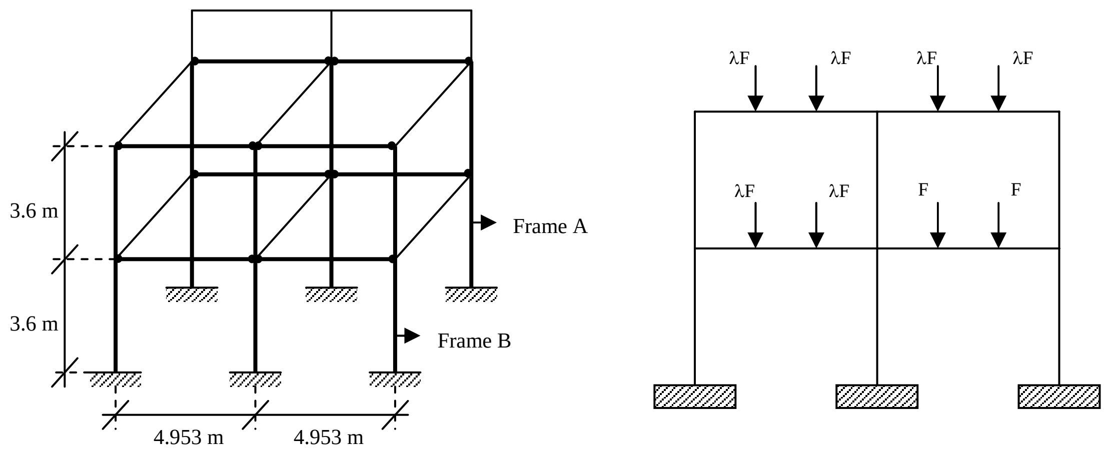


Figure 3-7: "UK" building - general layout and applied loading

3.5 "Eisenach" building

This structure is an unbraced factory building erected in Eisenach (Germany) and designed by Wuppertal University (*[26]* and *[27]*). The relevant data have been kindly provided by

ARCELOR Group (formerly ProfilARBED Division). The building is 240 m long, 54 m width and 30.6 m height. The latter is composed of two or three-storey three-bay transversal frames which are spaced out 12 m apart (*Figure 3-8.a*); they resist in-plane loading by frame action without exhibiting out-of-plane deformation because they are braced in the perpendicular direction.

The slabs are composite ones (concrete slab with a collaborating sheet) and lean on secondary beams with an interval of 3 m. These secondary beams are supported by the principal beams represented in *Figure 3-8*. The columns are partially encased WPG profiles.

The available data only concern the two-storey two-bay frames. One of these frames has already been studied by M. Pecquet in his diploma work *[3]*. To respect the field of his work (i.e. study of sway composite structures), he realized some modification on the actual frame (represented in *Figure 3-8.b*); the main modifications made in the frame were to substitute the second level steel trusses by composite beams (so as to get a "full" composite frame) and to increase the thickness of the column flanges to resist to the additional dead loads brought by the new composite beams. The static scheme of the so-obtained substitute frame are given in *Figure 3-8.b* (g = permanent loads and q = variable loads); the latter is used for the numerical investigations presented in *§ 5.4.4*. More details about this frame are available in *[3]*.

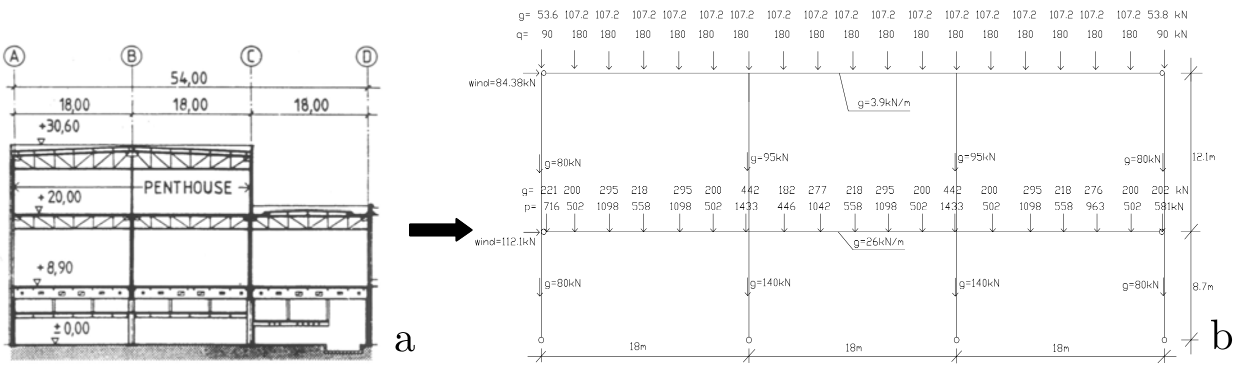


Figure 3-8: from the actual "Eisenach" structure to the substitute frame

The steel grade ordered for the structural elements is S355. The concrete grade for the slab and for the composite column is C30/37 with S500 rebars. The external joints of the frame are assimilated to hinges and the internal joints to fully rigid ones.

3.6 "Luxembourg" building

This is a bank office building located in Luxembourg (Grand-Duchy of Luxembourg). The relevant data have also been kindly provided by ARCELOR Group. This building is constituted of composite frames linked together by steel profiles. The height-storey three-bay frame is 38.8 m width and 26.2 m height (*Figure 3-9.a*). The actual "Luxembourg" structure is a braced one; indeed, concrete cores are present in the building and ensure its stability under transversal loading. Nevertheless, a frame of this structure has been isolated and assumed to be unbraced so as to satisfy the theme of the work.

As well as the slab, the beams or the columns are composite ones. This building was studied by Ms. Majkut in her diploma work *[2]*; she realized some modifications on the actual frame (*Figure 3-9.a*) so as to obtain a composite frame which presents a sway behaviour. The obtained frame is presented in *Figure 3-9.b*; this is the latter which is studied in *§ 5.4.6*.

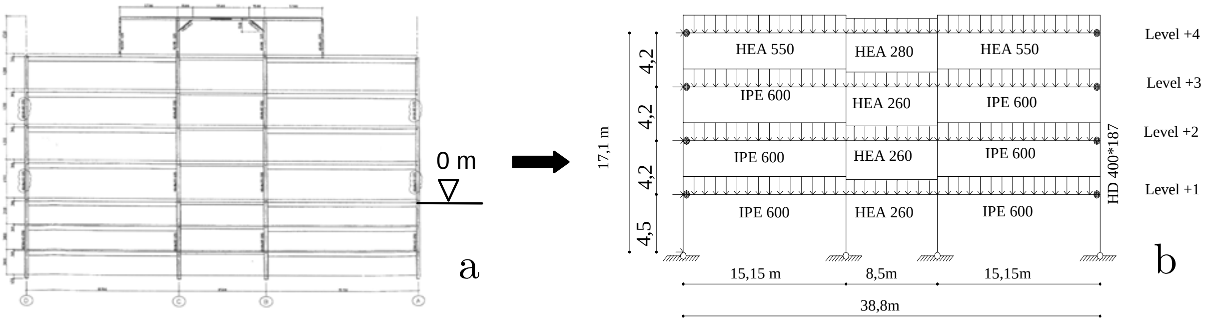


Figure 3-9: from the actual "Luxembourg" structure to the simplified substitute frame

The frame is loaded under permanent and variable uniform gravity loads and under concentrated horizontal loads at each storey level idealising the wind action; the values of these loads are presented in *Table 3-1*. The self-weight has to be added to the latter.

Table 3-1: permanent and variable loads applied on the substitute frame of "Luxembourg" structure

Level	Permanent load (kN/m)	Variable load (kN/m)	Wind (kN)
1	31.5	22.5	13.99
2	31.5	22.5	20.4
3	31.5	22.5	20.91
4	36.45	21.38	7.59

Three different steel grades have been chosen for the beam and column profiles: S235, S355 and S460. The concrete class is C30/37 for all the concrete elements of the building, with S500 rebars for the reinforce concrete. More details about this structure are given in *[2]*.

3.7 Chapter conclusions

In this chapter, we have presented five actual composite buildings in which sway effects are likely to occur under static loading; two of them (the "Ispra" and the "Bochum" buildings) were designed by us in the field of the above-mentioned European project (see *[20]* and *[23]*).

From these buildings, 2-D composite frames are isolated and will be numerically and analytically investigated in *Chapter 5* and *Chapter 6* respectively.

Chapter 4 : Tools for the evaluation of the response of structural elements in composite building (according to EC4)

4.1 Introduction

This chapter presents three software that we developed by means of Excel sheets and Visual Basic modules. These software have been developed for an easy characterisation of the structural elements of composite structures. They will be used when required in the next chapters. There are listed here below:

- the first software defines the properties of bolted beam-to-column composite joints with flush end-plate at the top flange and extended or flush end-plate at the bottom flange (*§ 4.2*);
- the second one evaluates the properties of composite beams (steel profile with the upper flange fully connected to a concrete or composite slab by means of shear studs *(§ 4.3);*
- the third one computes the resistance of partially encased composite columns (*§ 4.4*).

4.2 "Beam-to-column composite joint" software

This software allows to compute the main properties (see *§ 2.3.5.2*) for hogging and sagging loading of beam-to-column composite joint configurations presented in *Figure 4-1*:

- the initial stiffness ($S_{j,ini}$);
- the plastic bending resistance (M_{Rd});
- the elastic bending resistance (M_e);
- the shear resistance (V_{Rd});
- the collapse mode and ductility.

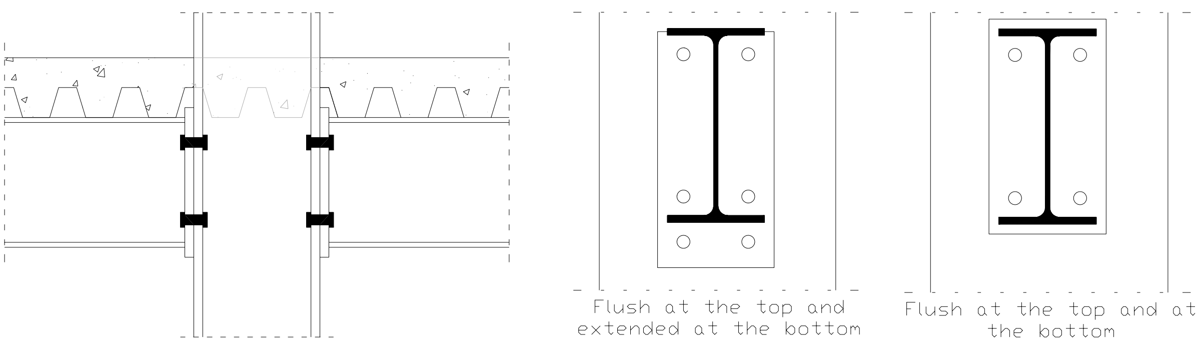


Figure 4-1: examples of joint configurations which can be computed by means of the software

These properties are computed according to the component method described in *§ 2.3.5*, which is the recommended method in the new draft of Eurocode 4 *[9]*. The latter only enables to compute the flexural properties of composite joints under hogging moments. However, in the field of the present study, it is also necessary to compute the properties under sagging moments; this situation may appear in structural joints when the structure is loaded under horizontal loads. A specific procedure has therefore been developed for joints under sagging moments.

This procedure has been easily defined with the help of the component method. Only one new component ("concrete slab in compression", at the contact joint between the concrete slab and the column flange) needs to be characterised; all the other components activated under sagging moment loading being already defined in Eurocode 3 (see *Table 2-4* in *§ 2.3.5.2* and *Table 4-1*). The resistance and the stiffness properties of this new component are computed by assuming that all the height of the slab concrete is in compression on the width of the column flange; the resistance force is assumed to be applied at mid-height of the concrete slab. These formulas have been developed through an analogy with the component "concrete in compression" useful in column base joints *[28]*. This characterisation needs to be validated in future studies (through comparisons with test results); however, the fact that this new component characterisation will be validated or not later on does not influence the validity of the results presented in this work as the objective of the latter is to investigate the global behaviour of composite sway frames and not the local one. The different components considered in the software for the design of the composite joints are given in *Table 4-1*.

Table 4-1: components used for the joint design

Hogging moment	**Sagging moment**
Column web panel in shear	Column web panel in shear
Column web in compression	Column web in compression
Column web in tension	**Concrete slab in compression***
Column flange in bending	Bolts in tension
End plate in bending	Column web in tension
Beam flange in compression	Column flange in bending
Beam web in tension	End plate in bending
Bolts in tension	
Longitudinal slab reinforcement in tension	

* new component to be validated

In the software, the user can choose between four different joint configurations to which correspond different responses of the joint (component "web panel in shear" being or not considered):

- single-sided joint configuration;
- double-sided joint configuration symmetrically loaded;
- double-sided joint configuration non-symmetrically loaded with balanced moments;
- double-sided joint configuration non-symmetrically loaded with unbalanced moments.

All the data and the main results are presented on the same sheet with the conventional non-linear moment-rotation curve computed as explained in *[14]*. All the detailed calculations are provided on separate sheets. An additional sheet with the meaning of most of the parameters is also given. The main sheet is presented in *Annex A.1.1*.

To validate the software, the results of its application have been compared to hand calculations.

4.3 "Composite beam" software

The developed tool allows to compute geometrical and mechanical properties of composite beams (steel profile with the upper flange fully connected to the concrete or composite slab by stud connectors) according to the rules given in Eurocode 4 *[9]*; sagging and hogging moments are considered. The computed properties are the following:

- equivalent steel cross section (A_{eq});
- position of the gravity centre (y_g) by reference to the lower face of the profile bottom flange;
- equivalent moment of inertia (I_e);
- class of the cross section to check wether the plastic resistant moments can develop or not;
- plastic bending resistances for sagging ($M_{pl,rd,+}$) and hogging ($M_{pl,rd,-}$) loading;
- vertical shear resistance ($V_{pl,rd}$);
- resistance of the transversal rebars to shear forces ($V_{rd,+}$ for sagging loading and $V_{rd,-}$ for hogging loading).

The user can decide to compute or not the shear stud resistance for the connection between the upper flange of the profile and the slab (P_{rd}).

The data and the results are presented on the same Excel sheet. The meaning of all the parameters used on this sheet is given by means of "comment windows". All the computation details are given on a separate sheet. An example of main sheet, with the data and the results, is given in *Annex A.1.2*.

4.4 "Partially encased composite column" software

The mechanical properties of the partially encased composite column (*Figure 4-2*), about major and minor axes, are computed according to the simplified analytical method (see *§ 2.3.4*) given in Eurocode 4 *[9]*. Some geometrical properties are also evaluated. The computed geometrical and analytical properties are the following:

- self-weight by length unit (g);
- equivalent steel cross section (A_{eq});
- equivalent moment of inertia (I_e);

- normal plastic resistance (with and without account of the buckling phenomena) ($N_{pl,rd,buckling}$ and $N_{pl,Rd}$ respectively);
- plastic bending resistance without considering the "M-N" interaction ($M_{pl,rd}$);
- "M-N" interaction curve with a graphical representation (with account of buckling phenomena);
- resistant bending moment according to the value of the design compression force in the column (M_{rd}).

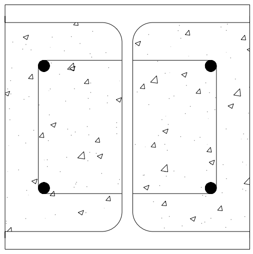


Figure 4-2: cross section of a partially encased composite column

The field of application of the simplified analytical method is checked before computing the mechanical properties.

The user has the possibility to take account or not for the influence of the long-term loading and for the second order effects.

The data and the results are presented on the same Excel sheet. The meaning of all the parameters used on this sheet is given by means of "comment windows". All the computation details are given on a separate sheet. An example of main sheet, with data and results, is given in *Annex A.1.3*.

4.5 Conclusions

In this chapter, we have presented three software that we have developed so as to facilitate the computation of the properties of structural elements in composite frames (see *Chapter 3*). These tools are Excel sheets and the computations are in agreement with the design recommendations given in Eurocode 4; they have been validated through comparisons with hand calculations.

Chapter 5 : Numerical modelling

5.1 Introduction

In this chapter, numerical investigations performed on sway composite frames are presented and analysed so as to highlight their behavioural particularities under static horizontal and vertical loads; the studied frames are isolated from the actual buildings presented in *Chapter 3*.

As for all numerical investigations, assumptions are needed to idealize and to model the studied structures; these assumptions depend mainly on the numerical tool capacities and on the type of the performed structural analysis. Accordingly, a first paragraph (*§ 5.2*) details the numerical tool used for the numerical investigations and the needed modelling assumptions.

Then, a benchmark study realized on 2-D frames isolated from the "UK" building (see *§ 3.4*) is presented in *§ 5.3*; it aims at validating the applicability of the non-linear homemade FEM software FINELG to above structures before starting numerical analyses with the latter.

Finally, *§ 5.4* details the performed numerical analyses on reference frames with the highlighting of particular behavioural aspects.

5.2 Numerical tool and modelling assumptions

5.2.1 Introduction

All the numerical investigations presented in this chapter are performed through the homemade finite element software FINELG. First, the latter is briefly described in *§ 5.2.2*. Then, the common assumptions of all the presented numerical investigations performed through FINELG are introduced as follow:

- first, *§ 5.2.3* presents the general assumptions relative to the modelling of the structures, the materials, the constitutive members and the joints;
- then, *§ 5.2.4* introduces the particular assumptions which depend on the performed numerical analysis.

Finally, information concerning the combinations of the applied loads are given in *§ 5.2.5*.

5.2.2 Presentation of the numerical tool

As said in *§ 5.2.1*, the tool used for the numerical investigations is the finite element software FINELG. The latter is a geometrically and materially non-linear finite element software developed at Liège University (M&S Department) and at Greisch office (Liège, Belgium) and especially used for research purposes. It enables to follow the behaviour of a structure under increasing loading up to the ultimate and even beyond. This software also enables to perform first-order and second-order elastic analyses (*§ 2.2.5.2* and *§ 2.2.5.4*) and to compute the critical load factor of a structure through a critical elastic analysis (*§ 2.2.5.3*).

More details about the FINELG software are given in *Reference [29]*.

5.2.3 Assumptions relative to the modelling of the structures

5.2.3.1 Introduction

For all the studied 3-D buildings, a 2-D frame is isolated as allowed in Eurocode 4 and a 2-D modelling of these isolated frames is performed; this procedure is based on the assumption that the frames are braced in the out-of-plane direction which is the case for the studied buildings (see *Chapter 3*).

The beam and column members are modelled by means of plane beam elements with three nodes (*Figure 5-1*). Node 1 and 3 present three degrees of freedom (u, v and θ - see *Figure 5-1*); node 2 only presents one degree of freedom (u) which allows to take into account of an eventual relative displacement between the concrete and the steel profile. This type of elements does not permit to involve the cross section local buckling phenomenon. As a 2-D numerical analysis is performed, the out-of-plane buckling phenomena as lateral-torsional buckling are not taken into account in the computation.

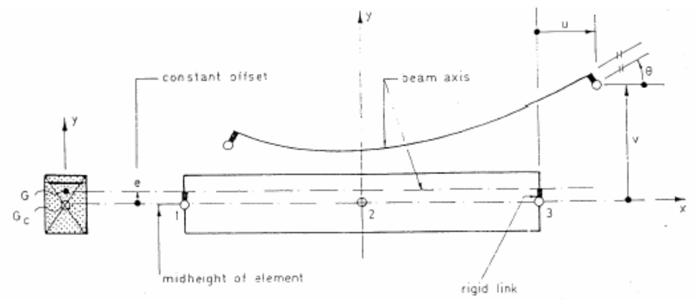


Figure 5-1: plane beam finite element with three nodes

5.2.3.2 Modelling of the materials

For all the materials, the characteristic values for the resistance are used (security coefficients for the materials equal to 1) as the objective of the study is not to perform a frame design but to investigate the behaviour of composite sway frames.

For the steel elements (steel profiles and steel rebars), a linear law ("Hooke's law") is used for the elastic analyses and a bilinear one for the non-linear analyses (*Figure 5-2*). The FINELG software also permits to take into account of the influence of the residual stresses; the latter are not introduced in the computations presented herein as the objective of the presented studies is to investigate the global behaviour of the structure (the residual stresses only influence the local behaviour of the members).

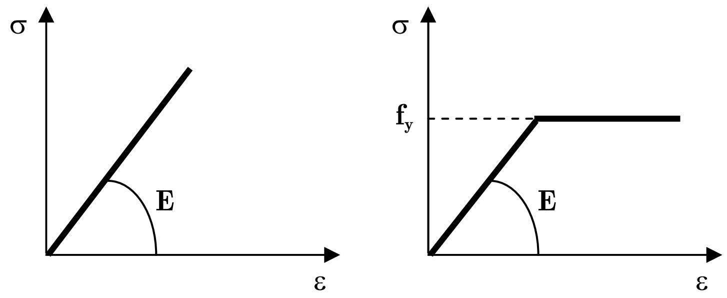


Figure 5-2: linear and bilinear laws for the steel elements

For the concrete material, a parabolic law with tension stiffening is introduced in the modelling (*Figure 5-3*). Shrinkage and creep phenomena are not introduced in the computation; the long term loading effect is taken into account by multiplying the concrete resistance by a factor equal to 0.85 (α coefficient).

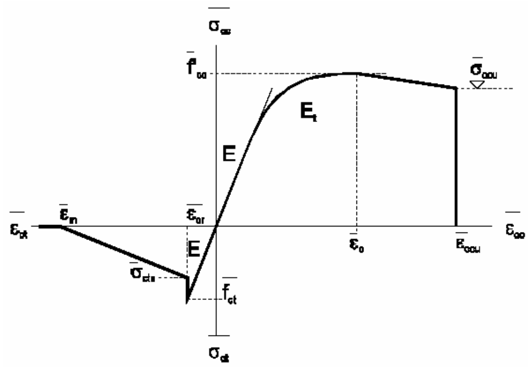


Figure 5-3: parabolic law with tension stiffening for the concrete elements

5.2.3.3 Modelling of the constitutive members and joints

The concept of effective width is used for the composite beam modelling (see *§ 2.3.3*). The connection between the concrete or composite slab and the steel profiles is assumed to be complete. All the columns are assumed to be continuous on all the height of the studied frames.

Concerning the semi-rigid and partial-strength joint modelling, the simplified approach described in *§ 2.3.5.4* is used, i.e. the deformability of the joints is concentrated at the intersection between the beam and the column axes. Rotational springs with tri-linear behaviour laws (*Figure 5-4*) are introduced in the frame modelling to model and to idealize the joint behaviour; with such idealisation, the post-limit behaviour of the joint (including the strain hardening effects, the eventual membranar effects, ...) is neglected (see *Figure 2-24* in *§ 2.3.5.4*). For the critical elastic analysis performed later on, these tri-linear laws are replaced by linear ones as presented in *Figure 5-2* with a stiffness equal to $S_{j,ini}$ (as recommended in the Eurocodes).

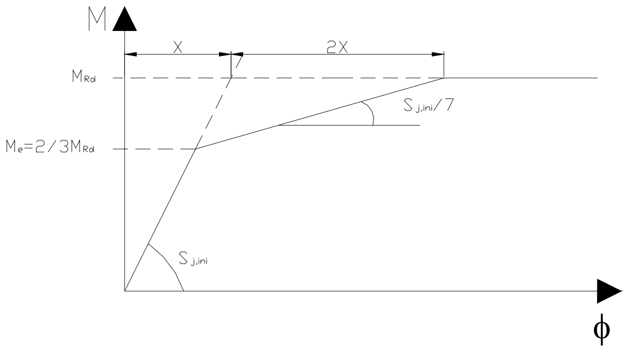


Figure 5-4: trilinear idealisation of the non-linear joint behaviour

As the behaviour of the constitutive joints of the "UK" building are available from test results presented in *[24]*, these test results are used to idealize the joints. For the other structures, the properties of the composite joints are computed by means of the software presented in *§ 4.2* and the properties of the steel ones with the help of the software COP *[30]* (COnnection Program developed at Aachen and Liège Universities) based on the component method concept presented in *§ 2.3.5*.

As said in *§ 4.2*, hogging and sagging loadings can occur at a joint under the combination of horizontal and vertical loads and the response of this joint can be different for these two types of loading. However, the FINELG software doesn't permit to introduce behaviour laws of rotational springs with two different behaviours under hogging and sagging loadings. So, the properties of the joints are assumed to be the same for hogging and sagging loadings (*Figure 5-5*); the choose between the "hogging or sagging properties" will depend of the type of loading which is predominant in the support region.

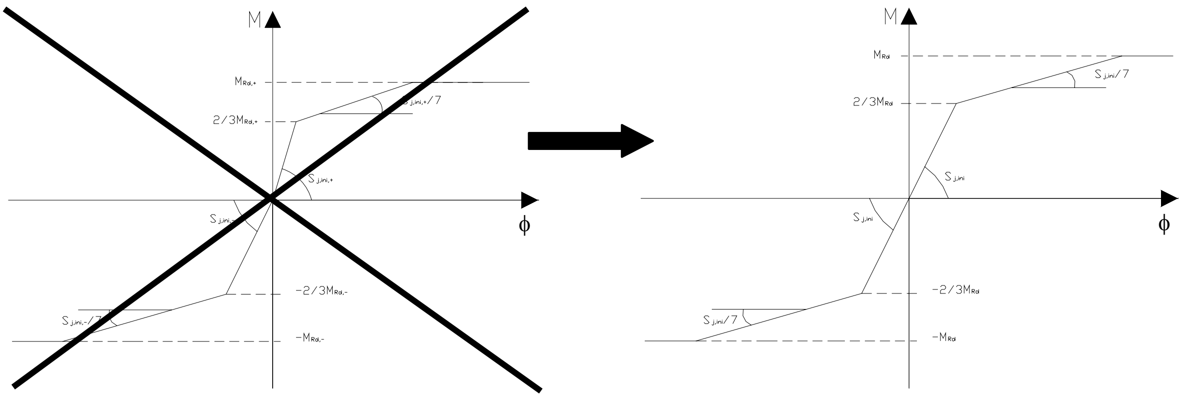


Figure 5-5: modelling assumption for a joint loaded under hogging and sagging moment

5.2.4 Assumptions relative to the performed analysis

5.2.4.1 Introduction

In the following numerical investigations, three types of analyses are performed through the FINELG software: critical elastic analyses, first-order rigid-plastic analyses and non-linear analyses.

This paragraph introduces the different assumptions which are needed to perform these analyses. In particular, a special assumption for the computation of the critical load factor is introduced so as to account of the additional effects due to the concrete cracking (described in *§ 1.2*).

5.2.4.2 Critical elastic analysis

As said in *§ 2.2.5.3*, this analysis permits to obtain the critical load factor λ_{cr} corresponding to a global frame instability phenomenon. Having this load factor, it is possible to classify the frame as sway or non-sway by means of the $\lambda_{Sd}/\lambda_{cr}$ ratio value. For this type of analysis, as well as the materials than the joints are assumed to be indefinitely elastic.

In the introduction (*§ 1.2*), the concrete cracking was introduced as a particularity of composite structures with respect to steel ones, which induces additional sway displacements. In *Reference [2]*, two different assumptions concerning the concrete cracking were considered for the computation of the critical load factor of composite frames:

- In the first case, the concrete is assumed to be uncracked all along the beam: the concrete is assumed to have the same stiffness in tension than in compression, which is not the case in reality. The so-obtained critical load factor called "uncracked critical load factor $\lambda_{cr,uncracked}$" is a purely theoretical concept which doesn't reflect the actual behaviour of the structure.
- In the second case, the concrete is assumed to be cracked in the support regions: this cracking is taken into account by assuming that the concrete has no stiffness in tension. The so-obtained load factor is called "cracked critical load factor $\lambda_{cr,cracked}$".

In the present chapter, these two different assumptions are adopted for the computation of the elastic critical load factors. These values will be used in *Chapter 6* investigating the applicability of simplified analytical methods; the results obtained through the latter will be compared to see which assumption permits to get the most accurate prediction through the simplified analytical methods.

5.2.4.3 First-order rigid-plastic analysis

This analysis is presented in *§ 2.2.5.5*. The first-order rigid-plastic load factor λ_p can be obtained easily by hand-calculation, or by using appropriate software. The FINELG software requires the use of a trick for the computation of λ_p as the latter always accounts for the second-order effects; this trick consists simply in increasing sufficiently and proportionally the flexural stiffness of all the constitutive frame elements so as to avoid significant sway displacements.

When plastic hinges formed in columns, the "M – N" interaction (see *§ 2.3.4* for composite columns) is taken into account in the computation as the latter may significantly decrease the available plastic bending resistance of a column.

5.2.4.4 Non-linear analysis

This type of analysis is the one which produces the closest results according to the actual behaviour of a studied frame. In this analysis performed with the FINELG software, the P-Δ and P-δ second order effects (*§ 2.2.4.2*) are included in the computation. All the non-linearities of the materials and of the joints are also included by means of the definition of bi-linear or tri-linear behaviour laws.

As recommended in Eurocode 4, an initial deformation is introduced in the computation (see *Figure 2-5* in *§ 2.2.4.2*). A formula is proposed in Eurocode 4 *[9]* so as to estimate a value for the initial out-of-plumb ϕ to impose to the structure:

$$\Phi_{ini} = k_c\ k_s\ \Phi_0 \tag{5.1}$$

where :

$$\Phi_0 = 1/200 \tag{5.2}$$

$$k_c = (0{,}5 + \frac{1}{n_c})^{0{,}5} \text{ but } k_c \leq 1 \tag{5.3}$$

$$k_s = (0{,}2 + \frac{1}{n_s})^{0{,}5} \text{ but } k_s \leq 1 \tag{5.4}$$

with n_c the number of full height columns per plane and n_s the number of storeys.

The shape of the initial deformation introduced in the computations is proportional to the first global instability mode obtained with a critical elastic analysis (which is in agreement with Eurocode recommendations); this permits to introduce at the same time a global initial deformation (to initiate P-Δ effects) and local initial deformations for the members (to initiate P-δ effects). In practice, the FINELG software produces a first global instability mode with a displacement at the top of the structure equal to the unity; so, to introduce the initial deformation in the computations, this mode is multiplied by $\phi_{ini}.H$ where ϕ_{ini} is the initial out-of-plumb computed through *Formula (5.1)* and H is the total height of the studied structure.

5.2.5 Load combinations

For the verifications of the serviceability limit states ("SLS"), no security coefficients are applied to the loads in agreement with Eurocode recommendations; so, the following load combination is used:

$$1\,G + 1\,Q \tag{5.5}$$

where G represents the self-weight and the permanent loads and Q the variable ones.

For the verification of the ultimate limit states ("ULS"), a reference combination must be chosen as the FINELG software allows to consider only one load combination for some types of analyses (the elastic critical analysis for instance). As a general rule, the following load combination is chosen for the further investigations:

$$1.35\,G + 1.5\,Q \qquad (5.6)$$

So, it means that all the loads, as well as the permanent ones as the variable ones, are proportionally increased by a factor “λ” till failure as following:

$$\lambda\,(1.35\,G + 1.5\,Q) \qquad (5.7)$$

However, two exceptions to this general rule will be made in the further numerical investigations:

- The first one concerns the benchmark study performed on the “UK” building and presented in *§ 5.3*. As the scope of this study is to compare the results numerically obtained to the test ones, the load sequence for the numerical investigations of the benchmark study will be the same than the one followed during the test (see *Figure 5-6* in the following paragraph).
- The second one concerns the numerical investigations performed on the “Bochum” building. As said in *§ 3.3*, this structure has been tested in Bochum and the loads were not proportionally increased during the test (see *Figure 3-6*). The load sequence applied for the numerical investigations described in *§ 5.4.3* will be the same than the one used for the test.

Remark: through these assumptions concerning the frame loading, it is implicitly assumed that the loading history can be neglected as the self-weight and the variable loads are both proportionally loaded at the same time; it would be interesting to investigate the influence of this assumption on the global behaviour of the composite frames (see *§ 7.3*).

5.2.6 Conclusions

In *§ 5.2*, we have introduced the main assumptions used for the numerical investigations presented herein: in one hand, the assumptions relative to the structure modelling and, in the other hand, particular assumptions which depends on of the performed numerical analysis.

5.3 Benchmark study

5.3.1 Introduction

In this paragraph, a benchmark study aimed at validating the use of several finite element software for the numerical simulation of the non-linear behaviour of composite structures is summarized. More details may be found in the common report *[31]*.

The Institutions which contributed to the benchmark study were partners involved in the numerical studies of the European project presented in *§ 1.1*:

- LABEIN (Spain) – abaqus 6.2 software;

- RWTH Aachen (Germany) – DYNACS software;
- Pisa University (Italy) – ADINA 7.5 software;
- Liège University (Belgium) – FINELG software.

The reference structure for the benchmark study is the "U.K." building (described in *§ 3.4*) because both the detailed data and test results are available (*[24]* and *[25]*). The validation is subordinated to a successful comparison of the results obtained numerically by the above partners with the ones recorded during the tests.

The structure (*Figure 3-7*) is composed of two parallel two-storey two-bay main frames (namely "Frame A" and "Frame B") connected by secondary beams. The loading of the frames and the loading sequence is described in *§ 3.4* and reminded in *Figure 5-6*. Though the reports *[24]* and *[25]* are well documented, some data are nevertheless missing; therefore reasonable assumptions *[31]* have been agreed on so as to ensure a complete similarity of the data used by the above partners for performing their respective numerical simulations.

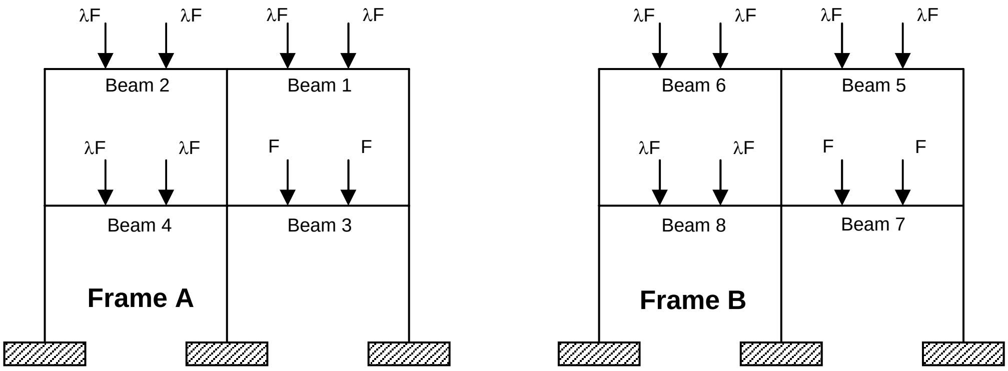


Figure 5-6: static schemes of Frame A and Frame B extracted from the "UK" building

This paragraph is organised as following: a first paragraph (*§ 5.3.2*) introduces some differences between the assumptions adopted by the different partners for the modelling of the frames. Then, a second paragraph (*§ 5.3.3*) presents the main results and comparisons between the numerical and the experimental results. Finally, different reasons that could explain some differences between the numerical results, in one hand, and between the numerical and test results, in the other hand, are developed in *§ 5.3.4*.

5.3.2 Main differences in the modelling assumptions adopted by the partners

The partners carried out non–linear FEM analyses, with due account taken of second-order effects and material non-linearities. Frames A and B have been modelled as plane frames and investigated separately up to collapse so as to get the ultimate load factor, the corresponding failure mode as well as load-deflection curves.

As said in the previous paragraph, reasonable assumptions have been agreed on so as to ensure a complete similarity of the data used by the above partners for performing their respective numerical simulations. Nevertheless, each partner is limited by the performances of the chosen software; so, some differences in the modelling assumptions still present. The main ones are developed herein.

- Initial out-of-plumb: Labein and Liège have considered an initial out-of-plumb displacement of the frame of 2.7 cm at the top of the columns (calculated according to EC4 through *Formula (5.1)*) while Pisa and Aachen did not introduce such initial out-of-plumb.
- Width of the concrete slab: two different approaches have been used to define the width of the concrete beams in the modelling:
  - Labein and Liège have used the concept of effective width (as recommended in EC4 – see *§ 2.3.3*) with different widths for the concrete slabs according to the position on the beam length;
  - Pisa and Aachen, on their side, have defined a constant slab width of 1 m all along the length of the composite beams, which corresponds to the actual physical width of the slab in the laboratory test (*§ 3.4*).
- Material and mechanical laws: the material and mechanical laws used by the different partners are presented in *Annex A.2.1*; the steel, concrete and joint characteristics have been defined in accordance with test results presented in *[24]*. The main differences in the chosen behaviour laws are the followings:
  - all the joints are modelled as semi-rigid and their characteristic moment-rotation curves are defined using a tri-linear law except for Aachen where a parabolic law is used (see *Figure A.2-2*);
  - elastic-plastic laws with strain hardening are used for the profile steels except for Labein where strain hardening effects are not considered (see *Figure A.2-1*);
  - parabolic laws in compression and tension stiffening effects are taken into account for the concrete with some differences in the behaviour curve shape (see *Figure A.2-3*).

Remark: for the numerical investigations performed through the FINELG software, the different assumptions presented in *§ 5.2* have been applied. However, as said previously, the strain hardening effect is introduced in the definition of the steel behaviour law (which constitutes a deviation with regards to these assumptions) so as to be as close as possible to the actual behaviour of the steel material.

5.3.3 Analysis of the results

5.3.3.1 Ultimate load factors

In general, the numerical simulations are in a rather good agreement with the test results got at BRE *[25]*. The comparison between FEM simulations and test for the ultimate load factor is presented in *Table 5-1* and *Table 5-2*.

Table 5-1: ultimate load factor λ_u – Frame A

Partner	Simulations	Test Results	Difference (%)
Labein	4.59	6.05	24.1
Pisa	6.32	6.05	-4.5
Aachen	5.31	6.05	12.2
Liège	5.48	6.05	9.4

Table 5-2: ultimate load factor λ_u – Frame B

Partner	Simulations	Test Results	Difference (%)
Labein	3.97	5.23	24.1
Pisa	5.03	5.23	3.8
Aachen	4.66	5.23	10.9
Liège	5.02	5.23	4.0

Table 5-1 and *Table 5-2* values show that the ultimate load factors are under estimated in all cases (except for Pisa in Frame A). The maximum difference with the test results is obtained by Labein (24,1 %).

The numerical results fit better to the test for Frame B than for Frame A (except for Labein where no difference is observed). The failure of the two frames results from the development of beam plastic mechanisms in all the numerical simulations as in the test but for different beams; indeed, for the numerical simulations, the beam mechanisms develop in beam 4 and beam 5 for Frame A and Frame B respectively when, for the test, they develop in beam 1 and beam 8 (see *Figure 5-6* for the position of the beams). The reasons which could explain this difference are developed later on in *§ 5.3.4*.

5.3.3.2 Displacements in the frames

During the test, the mid-span relative deflections of the beams (relative to the displacement of the extremities) have been measured. These deflections are reported in *[25]* as a function of the total applied load on the beam (sum of the two applied concentrated loads).

Figures A.2-4 to A.2-9 in *Annex A.2.2* present comparisons between load – deflection curves numerically derived and experimentally measured for the primary beams of Frame A and Frame B.

It may be seen in these figures that, for the two frames, the ultimate loads are under estimated (except for the University of Pisa – Frame A) as it was already shown in *§ 5.3.3.1*. The response of the beams is quite similar in the test and in the simulations (except for Labein where the simulations are always below the test curves). In Frame B, at the beginning of the loading, the curves obtained by the simulations are more linear than in the test curves. But at the yielding phase, the simulation curves and the test curves are quite alike. Nevertheless, it can be observed that, at this phase, for the University of Pisa simulations (see *Annex A.2.2*), the stiffness remains bigger than the other simulations. This can be explained by the type of steel constitutive law chosen for the modelling (without a plateau in the yielding phase) (see *Figure A.2-1* in *Annex A.2.1*).

Again, some differences between the simulations and the test results may be explained by different reasons developed in *§ 5.3.4*.

5.3.3.3 Bending moment diagrams

In *Figure 5-7*, the diagrams of bending moments at failure obtained from the test and the simulations are compared.

Again, the numerical simulations predict rather well the actual frame response (except for Labein). The maximum difference between the test results and the numerical results (in percentage) is observed for the "almost non-loaded" beams; it is not surprising as the moments are rather small in these beams.

5.3.4 Reasons explaining the result differences

5.3.4.1 Differences between the simulation results

These differences can be justified by the differences in the assumptions adopted by each partner for their modelling. These differences have been developed in *§ 5.3.2*.

5.3.4.2 Differences between the simulations and the test results

A detailed analysis of the test results (*[24]* and *[25]*) allows to identify unclear and doubtful points which can explain some differences with the simulation results:

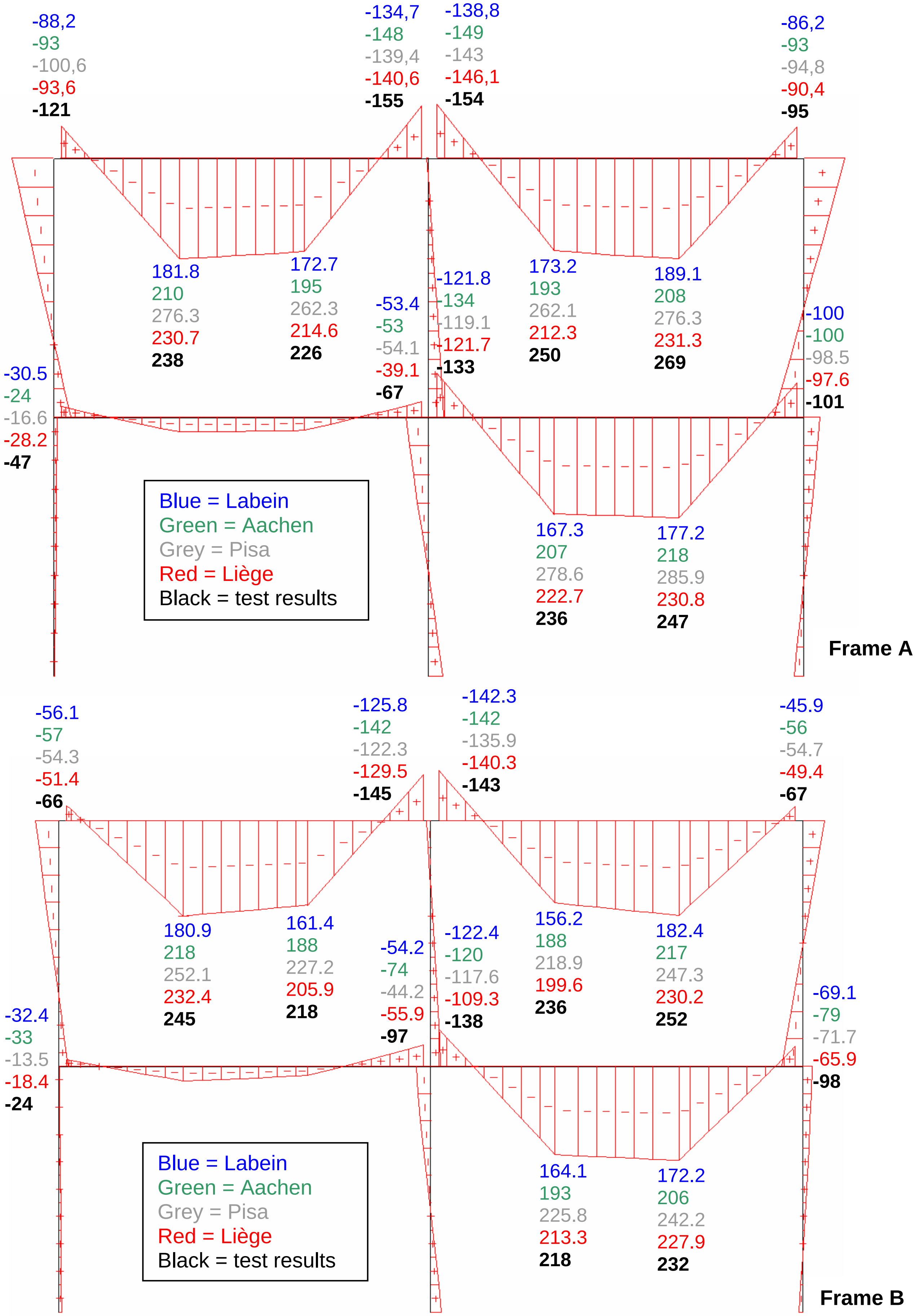


Figure 5-7: diagram of bending moments at failure for Frame A and Frame B

- The applied loads on Frame A at failure are not the same for all the beams what is in contradiction with the definition of the applied loads in *[24]* which should be proportionally loaded.
- Test results for the joints in Frame A look inconsistent: joints 4 and 8 have the same mechanical properties but joints 1 and 4 don't have at all the same behaviour (*Figure 5-8*).

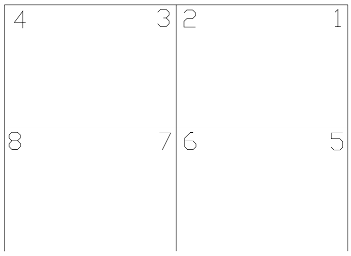


Figure 5-8: position of the joint in Frame A

- In the test, rotations with the joints were calculated by taking the difference between the measured beam end rotations and column rotations for every load increment. The moments at beam end were also evaluated through strain measurements and both have been used to build the joint moment-rotation curves. But, it is well known that it is very difficult to obtain a correct moment-rotation curve by this technique; so, the experimentally obtained moment-rotation curves used in the numerical simulations may be not so close to the actual ones.

These points can explain the greater difference between the numerical and experimental failure load factor for Frame A and the fact that the failure appears in beam 4 (in the simulations) instead of beam 1 (in the test).

5.3.5 Conclusions

In this paragraph, we have presented a benchmark study that we coordinated (definition of common assumptions, comparison of the results, redaction of the common report, ...). Through the latter, we have demonstrated that the simulations conducted with different software show a reasonably good agreement with tests except for ABAQUS; LABEIN still investigate their modelling method to find eventual mistakes.

In a last paragraph, we have pointed out some reasons that could explain small differences between the simulation results one hand, the numerical simulation results and the test ones in the other hand.

More especially, we have demonstrated the validity of the FINELG software for the study of composite structures, what justifies its further use when investigating sway composite frames.

5.4 *Application to reference frames*

5.4.1 Introduction

This paragraph introduces and analyses the numerical investigations performed on sway composite frames through the FINELG software; the applicability of the latter to composite structures has been demonstrated in the previous paragraph.

Five frames are examined herein; the latter have been isolated from actual buildings presented in *Chapter 3*: the "ISPRA", "Bochum", "UK", "Eisenach" and "Luxembourg" buildings. The assumptions presented in *§ 5.2* are adopted for the modelling.

The serviceability limit states have been previously checked according to the limitations proposed in Eurocode 4 (which are the same than the one proposed in Eurocode 3 – see *Table 2-3* in *§ 2.2.5.1)*; all these limitations are respected for the frames studied herein except for the "Bochum" building where the top sway displacement, which is equal to "$H/270$", is higher than the "$H/500$" limitation with H equal to the total height of the structure (see *Table 2-3*).

In fact, as said in *§ 3.3*, the design of the latter was performed so as to obtain a failure of the frame by global instability during the test; as the general geometry of the frame and the properties of the constitutive elements were fixed, the only parameters which could be adapted during the design were the applied loads and it was impossible to find a frame loading which respected at the same time the SLS limitations and the condition of a global instability failure. However, the fact that the SLS conditions are not satisfied has to be moderated; indeed, the drift of the "Bochum" building has been computed through assumptions which can lead to an over-estimation of the lateral displacement. For instance:

- the column bases have been assumed to be perfectly pinned joints (which is not the case in an actual building);
- the frame has been modelled without considering the non-structural elements, present in actual buildings, which can lead to an increasing of the sway stiffness;
- ...

The out-of-plane stabilities have also been checked as the latter are not included in the numerical analyses presented herein. This point will not be reminded later on.

Remark: part of the presented results comes from *Reference [2]* and *[3]*.

5.4.2 "Ispra" building

Only the "Ispra" building designed for static loading is examined herein (see *§ 3.2*). One of the constitutive frames is reminded here below (*Figure 5-9*); as said in *§ 5.2.3.1*, a 2-D modelling of the latter is realised.

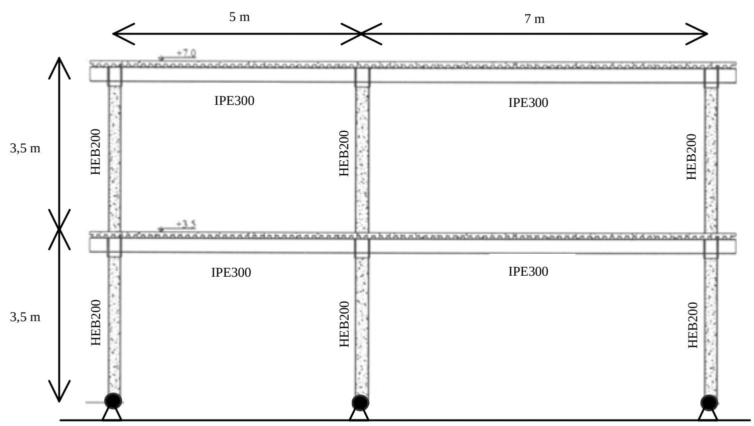


Figure 5-9: isolated 2-D composite frame of the "Ispra" building

The properties of the joints introduced in the modelling has been computed through the homemade software presented in *§ 4.2*. The column bases are assumed to be nominally pinned.

All the cross sections of the constitutive members are class 1 ones; so, plastic analyses can be performed on this structure.

Critical elastic analyses performed with the adoption of the two different assumptions described in *§ 5.2.4.2* provide critical load factors $\lambda_{cr,uncracked}$ and $\lambda_{cr,cracked}$ respectively equal to 6.49 and 5.95; the corresponding ratios $\lambda_{Sd}/\lambda_{cr,uncracked}$ and $\lambda_{Sd}/\lambda_{cr,cracked}$ are equal to 0.154 and 0.168, with the consequence that the frame would be sway if the criterion of Eurocode 3 is generalized to composite structures (see *§ 2.2.4.2*).

According to the first-order rigid-plastic analysis performed through hand calculations (the mechanical properties of the frame constitutive elements are computed by means of the software presented in *Chapter 4*), failure corresponds to the formation of a local beam mechanism in the 7 m span of the lower level; it is achieved for a load factor $\lambda_p = 1.84$ (see *Table 5-3*). This value is in accordance with a similar computation performed through the FINELG software.

Table 5-3: results from a first-order rigid-plastic analysis

Type of mechanism	λ_p
Panel mechanism (minimum obtained for a first-storey mechanism)	4.87
Beam mechanism	1.84
Combined mechanism	2.26

For the non-linear analysis, an initial out-of-plumb is introduced in the modelling; the latter corresponds to a displacement at the top equal to 0.0267 m (computed by means of *Formula*

(5.1) in accordance with Eurocode 4 *[9]*). As said in § *5.2.4.4*, the deformed shape of the first global instability mode computed through the critical elastic analysis is used to introduce the initial deformation. The load-sway displacement curve obtained from the so-performed non-linear analysis is shown in *Figure 5-10*. The ultimate load factor is λ_u = 1.786 while the first plastic hinge is formed at λ_e = 1.605.

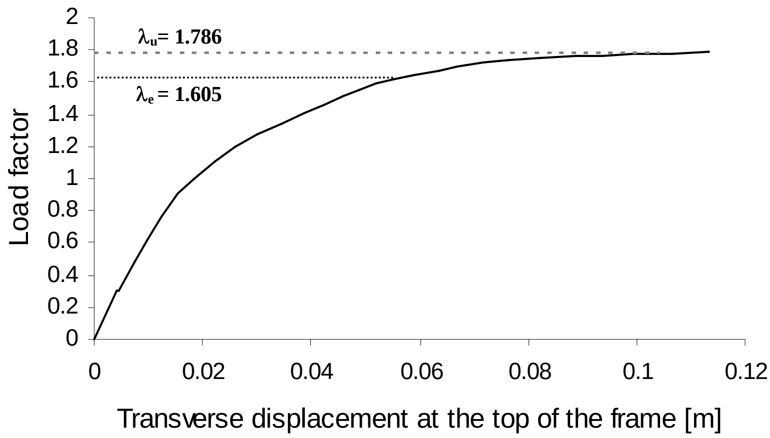


Figure 5-10: behaviour of the "Ispra" frame

The ultimate load factor λ_u is not much different from the plastic load factor λ_p. However the respective predicted failure modes are different (*Figure 5-11*). As the number of plastic hinges is not sufficient to develop a full plastic mechanism, failure is due to global instability and results from the progressive decrease in the sway stiffness of the frame when the plastic hinges successively develop.

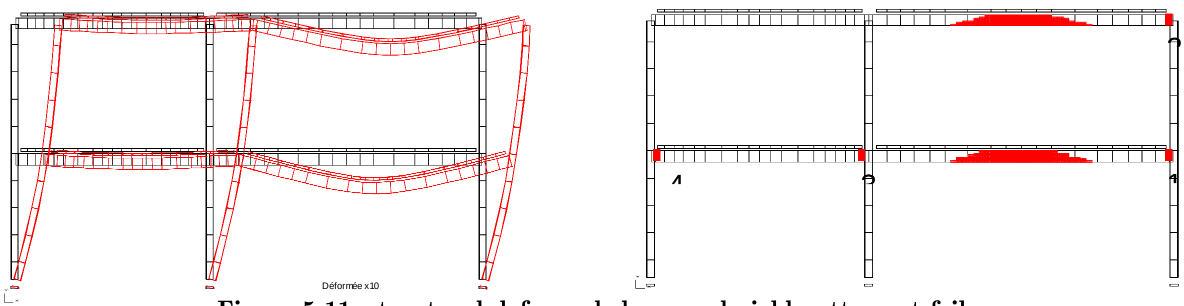


Figure 5-11: structural deformed shape and yield pattern at failure

5.4.3 "Bochum" building

The "Bochum" building is described in *§ 3.3*; the general layout of the 2-D frame is reminded in *Figure 5-12*.

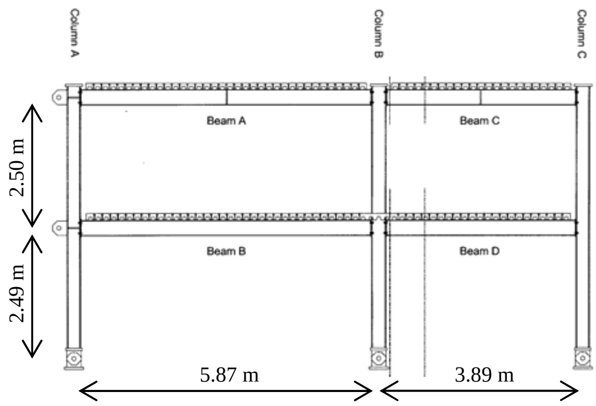


Figure 5-12: general layout of the 2-D frame test

A 2-D modelling of the "Bochum" frame is performed. The properties of the joints are computed through the homemade software presented in *§ 4.2* for the internal composite joint and through the software COP *[30]* for the steel joints. The column bases are assumed to be nominally pinned as for the "Ispra" building modelling.

The particular loading sequence applied to this structure during the test has been described in *§ 3.3* (*Figure 3-6*): the gravity loads are first increased until their nominal value and are then kept constant while the horizontal loads are progressively magnified by a load factor λ till failure. The same loading sequence is used herein for the numerical investigations.

Concerning the material modelling, a modification for the steel modelling is made according to the assumptions presented in *§ 5.2.3.2*: the non-linear analysis of the frame was performed in the field of the above-mentioned ECSC project (*§ 1.1*) with the objective of predicting the actual behavioural response of the frame before the performance of the test in Bochum. In order to achieve this goal, a tri-linear law with strain hardening effect (*Figure 5-13*) for the profile steel material was used for the non-linear analysis instead of the bi-linear one as described in *§ 5.2.3.2*.

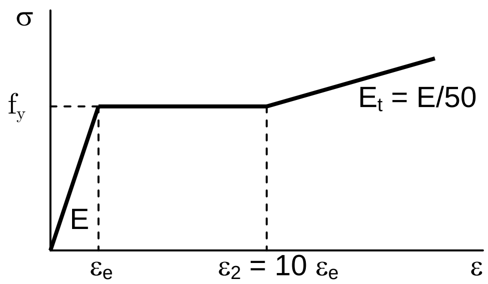


Figure 5-13: tri-linear law used for the profile steel modelling for the "Bochum" building

All the cross sections of the constitutive members are class 1 ones; so, plastic analyses can be performed on this structure.

The elastic critical analyses gives the following critical load factors:

- $\lambda_{cr,uncracked} = 9.83 \rightarrow \lambda_{sd}/\lambda_{cr,uncracked} = 0.102$

- $\lambda_{cr,cracked} = 9.42$ → $\lambda_{sd}/\lambda_{cr,cracked} = 0.106$

The latter values are just slightly larger than 0.1 with the result that it corresponds to the sway/non-sway boundary in the criterion introduced in *§ 2.2.4.2*.

A first-order rigid-plastic analysis has also been performed. When conducting hand calculations (with the help of the software presented in *Chapter 4*), only the following basic independent plastic mechanisms must be first considered prior to their possible further combination:

- panel mechanisms;
- plastic beam mechanism.

The latter may however be disregarded because the vertical loads, once applied, are kept constant. The so-obtained value of λ_p is equal to 1.82 which corresponds to the formation of a global panel mechanism. That is in accordance with a similar FINELG computation.

An initial deformation is introduced in the modelling for the non-linear analysis; it corresponds to the latter an initial horizontal top displacement of 0.019 m.

The non-linear analysis provides an ultimate load factor $\lambda_u = 1.41$ to which corresponds a top sway displacement of 0.085 m (initial out-of-plumb non-included) (*Figure 5-14*). The response curve starts at an abscissa, which represents the initial out-of-plumb of the frame. The general shape of this curve, especially the descending branch in the post-limit regime, means that failure results from a global frame instability. *Figure 5-15* shows that the number of plastic hinges at failure is smaller than the one required to form a full plastic mechanism.

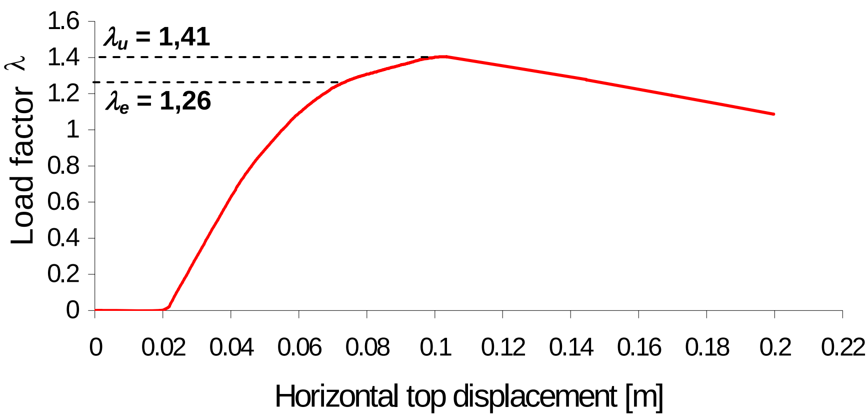


Figure 5-14: top displacement – total horizontal load for the "Bochum" structure

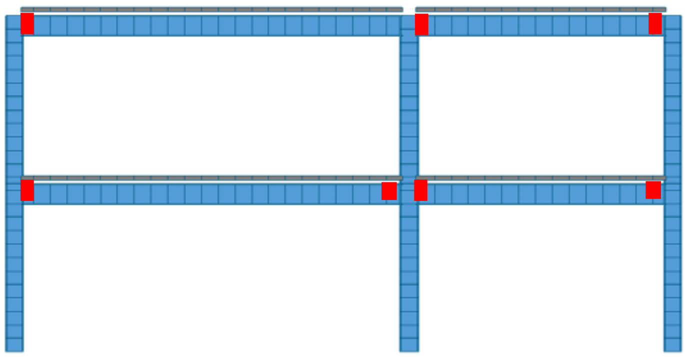


Figure 5-15: location of the plastic hinges at failure for the "Bochum" structure

As said in *§ 2.2.5.6*, panel mechanisms are significantly influenced by second-order effects. A second-order rigid-plastic analysis is conducted through hand calculations in order to evaluate the influence of the geometrical non-linearities on the value of the first-order rigid-plastic load factor λ_p. The relevant results are given in *Figure 5-16*.

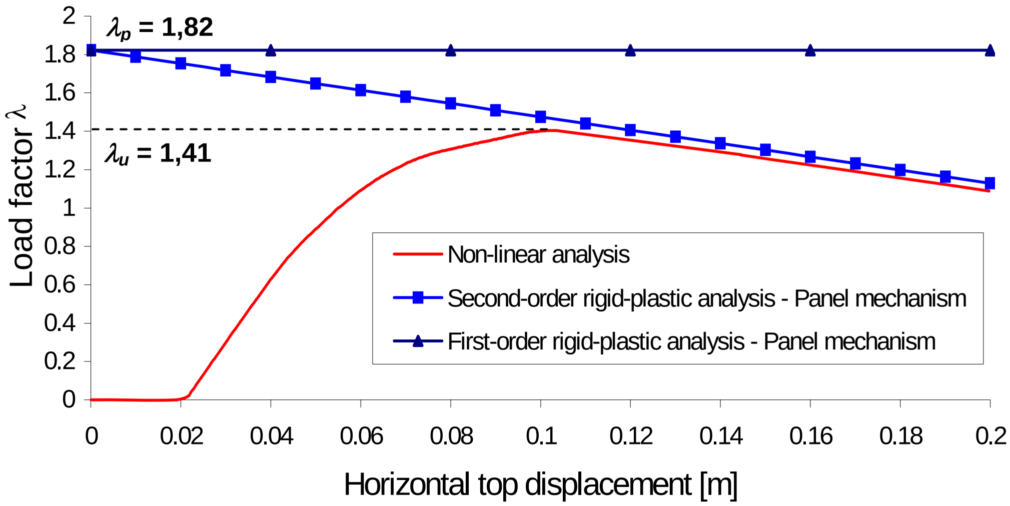


Figure 5-16: second-order rigid-plastic analysis for the "Bochum" structure

The descending branch of the frame response obtained from the non-linear analysis is found close to the one deduced from the second-order rigid-plastic analysis even if, as shown in *Figure 5-15*, the corresponding failure load is not, strictly speaking, associated to a full plastic mechanism, which would need the formation of eight plastic hinges. That is due to the fact that: i) only one plastic hinge at one beam end (right handside of the left upper beam) is missing before a global panel mechanism is formed (*Figure 5-15*), and ii) the bending moment in this cross section when the last hinge (the seventh) forms is only 10% lower than the plastic moment resistance of the cross section.

Finally, an elastic critical analysis is performed on the frame in which seven perfect plastic hinges are introduced at the beam ends and located as shown in *Figure 5-15*. It gives an elastic critical load resultant, which amounts 95 percent of the total applied vertical loads. This last analysis confirms the above prediction that failure is due to a global frame instability. This failure mode is also the one that has actually been observed during the test carried out in Bochum.

5.4.4 "Eisenach" building

This building is described in details in *§ 3.5*. The column bases as well as the beam-to-column joints connecting the beams to the outer columns are assumed to be pinned while the inner beam-to-column joints are assumed to be rigid. *Figure 5-17* reminds the substitute frame with its idealized static system and corresponding loading pattern defined in *[3]*. Notation "g" is used for the permanent loads (including self-weight of the structure) and "q" for the variable loads.

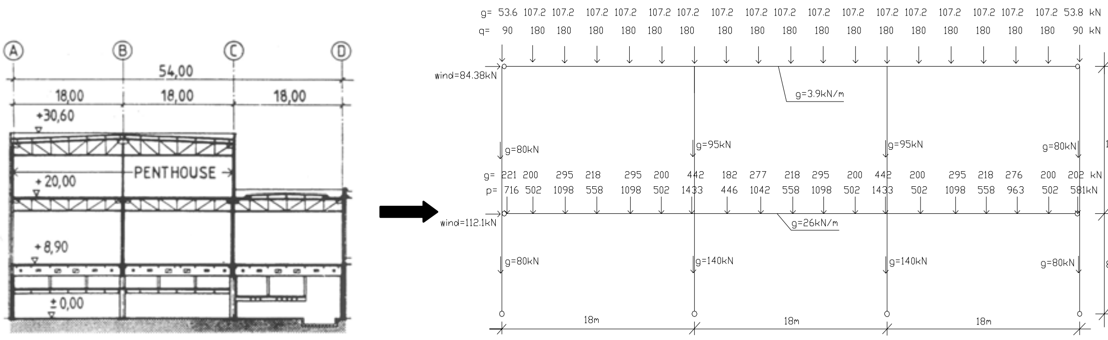


Figure 5-17: from the actual "Eisenach" structure to the substitute frame

All the cross sections of the constitutive members are class 1 ones; so, plastic analyses can be performed on this structure.

The elastic critical load factors obtained through elastic critical analyses are:

- $\lambda_{cr,uncracked} = 4.33$ → $\lambda_{Sd}/\lambda_{cr,uncracked} = 0.23$
- $\lambda_{cr,cracked} = 4.27$ → $\lambda_{Sd}/\lambda_{cr,cracked} = 0.23$

Consequently, a significant influence of the second-order effects on the structural response should be expected if reference is made to the limitation criterion defined in Eurocode 3 (see *§ 2.2.4.2*).

The first-order rigid-plastic load factor is reached with the formation of a local beam mechanism – in the lateral span of the first level – at a load factor $\lambda_p = 1.545$. This value has been computed through a FINELG numerical analysis and confirmed by hand calculations.

The curve representing the frame behaviour, which results from a non-linear analysis, is presented in *Figure 5-18*. An initial deformation has also been introduced in the modelling for this analysis (initial out-of-plumb at the top equal to 0.075 m). After the maximum load is reached, the curve goes down rapidly; that may be explained by the fact that all the plastic hinges form at the same time, so that $\lambda_e = \lambda_u = 1.136$ (*Figure 5-18*). These plastic hinges enable a panel mechanism to form in the first storey – a phenomenon which is usually designated as "soft floor" – while the rest of the structure remains nearly undeformed (*Figure 5-19*).

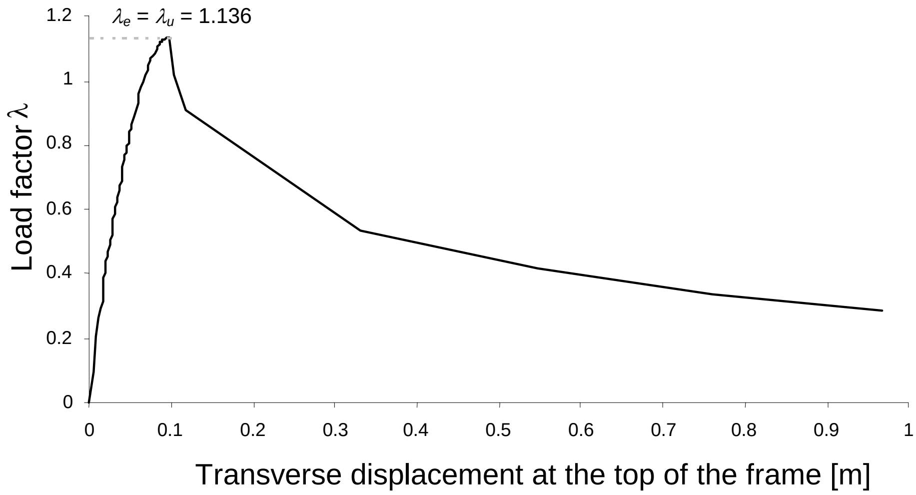


Figure 5-18: non-linear behaviour of the "Eisenach" frame

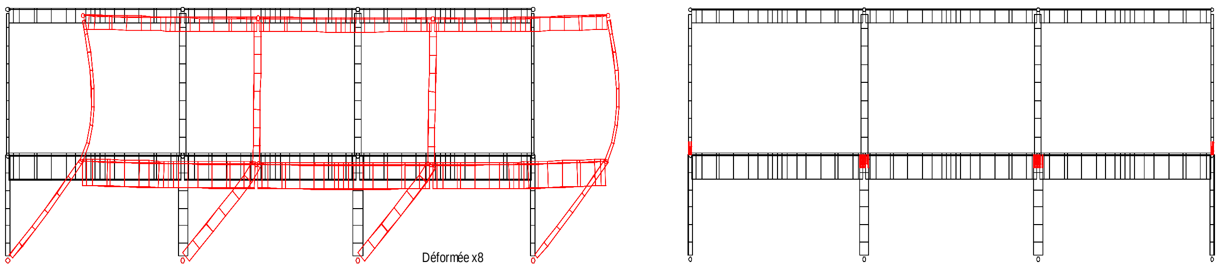


Figure 5-19: deformed shape and yield pattern at failure for the "Eisenach" frame

To get confirmation of the failure mode, two second-order rigid-plastic analyses were performed on a structure in which the first-order rigid-plastic failure mode is assumed to be either a beam mechanism – what is the case in reality – or a panel mechanism. The corresponding response curves as well as the one obtained from the non-linear analysis are shown in *Figure 5-20*. The comparison is quite conclusive:

- second-order sway effects do not affect the development of a beam mechanism but influence significantly the formation of a panel mechanism;
- failure is reached when a panel mechanism develops: the ultimate load is much lower than the one obtained based on a first-order rigid-plastic analysis. Consequently the ultimate load factor is reached where non-linear response and second-order rigid-plastic "panel" response join each other.

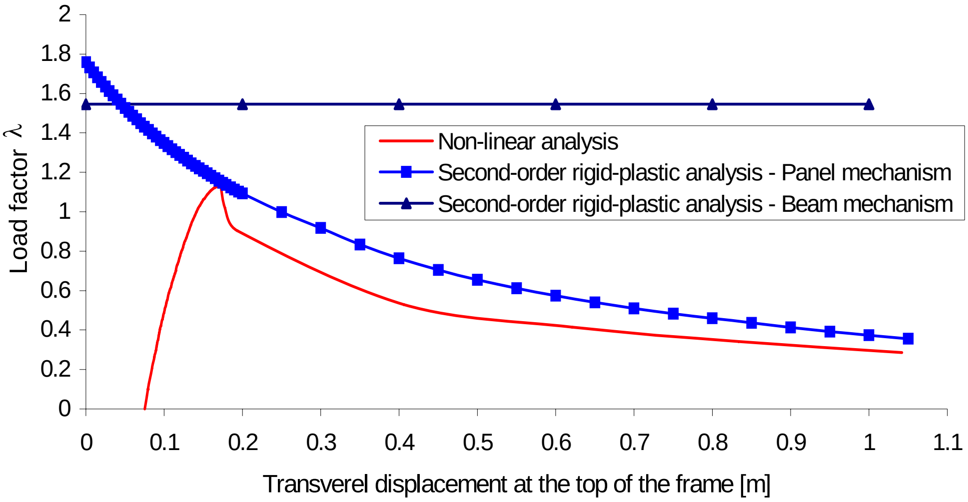


Figure 5-20: second-order effects in the Eisenach frame

5.4.5 "UK" building

The "UK" building is described in *§ 3.4*; the latter was tested at BRE, UK and used to perform the benchmark study presented in *§ 5.3*. This structure is composed of two two-bay two-storey composite frames (namely "Frame A" and "Frame" B); only the behaviour of "Frame A" is investigated herein. The general layout of the latter is reminded in *Figure 5-21*.

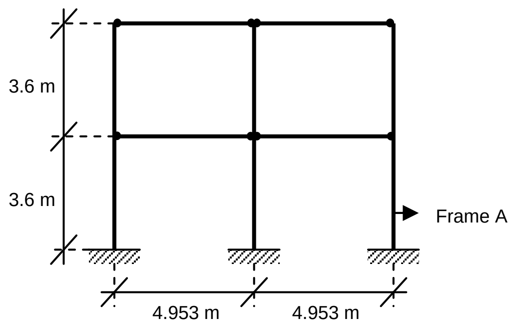


Figure 5-21: general layout of "Frame A" of the "UK" building

A 2-D modelling of the "Frame A" is realised. The *M-ϕ* behaviour curves of the joints were registered during the test and reported in *Reference [24]*; the latter have been idealised through tri-linear behaviour curves (see *Figure 5-4* in § *5.2.3.3*) and introduced in the frame modelling as done for the benchmark study. The column bases are assumed to be fully rigid.

All the cross sections of the constitutive members are class 1; so, plastic analyses can be performed on this structure.

In *§ 3.4*, the loading applied to the frame during the test is described. Under this loading, the frame cannot be classified as a sway one if reference is made to the classification criterion given in *§ 2.2.4.2* ($\lambda_{Sd}/\lambda_{cr} \leq 0.1$). In order to enter in the field of the present study (i.e. study of composite sway frame), modifications in the frame loading have been realised so as to obtain a $\lambda_{Sd}/\lambda_{cr}$ ratio which is higher than 0.1. The "Frame A" loading finally obtained is presented in *Figure 5-22*; the horizontal loads are supposed to represent wind actions and the vertical ones applied at the top of the columns to represent the gravity loads transmitted by upper storeys.

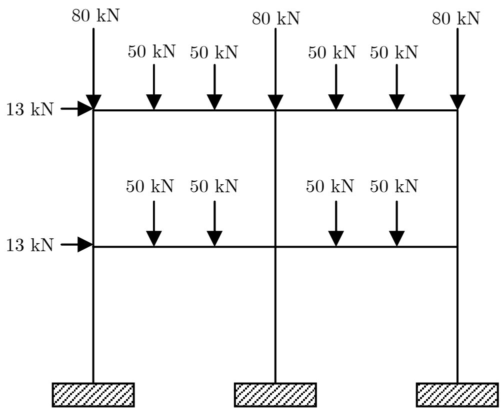


Figure 5-22: new frame loading for "Frame A"

This is this loading which is used in the further investigations; all the loads are assumed to be proportionally increased up to collapse.

The performed elastic critical analyses gives the following critical load factors:

- $\lambda_{cr,uncracked} = 9.31$ → $\lambda_{sd}/\lambda_{cr,uncracked} = 0.107$
- $\lambda_{cr,cracked} = 8.77$ → $\lambda_{sd}/\lambda_{cr,cracked} = 0.114$

The latter values are just slightly larger than 0.1.

A first-order rigid-plastic analysis has also been performed through hand calculations. The corresponding λ_p values are listed in *Table 5-4*.

Table 5-4: results from a first-order rigid-plastic analysis

Type of mechanism	λ_p
Panel mechanism (minimum obtained for a first-storey mechanism)	5.93
Beam mechanism	2.38
Combined mechanism	2.36

In accordance with the first-order rigid-plastic analysis, the failure of the structure is due to the formation of a combined mechanism; the plastic load factor corresponding to the formation of the latter is closed to the one corresponding to the formation of a beam plastic mechanism. That is in accordance with a similar FINELG computation.

An initial deformation is introduced in the modelling for the non-linear analysis; it corresponds to an initial transversal top displacement of 0.027 m.

The non-linear analysis provides an ultimate load factor $\lambda_u = 2.01$ to which corresponds a top sway displacement of 0.134 m (initial out-of-plumb non-included) (*Figure 5-23*). The response curve starts at an abscissa, which represents the initial out-of-plumb of the frame. The general shape of this curve, especially the descending branch in the post-limit regime, means that failure results from a global frame instability. *Figure 5-24* shows that the number of plastic hinges at failure is smaller than the one required to form a full plastic mechanism; three plastic hinges are missing at the column bases to form a combined plastic mechanism.

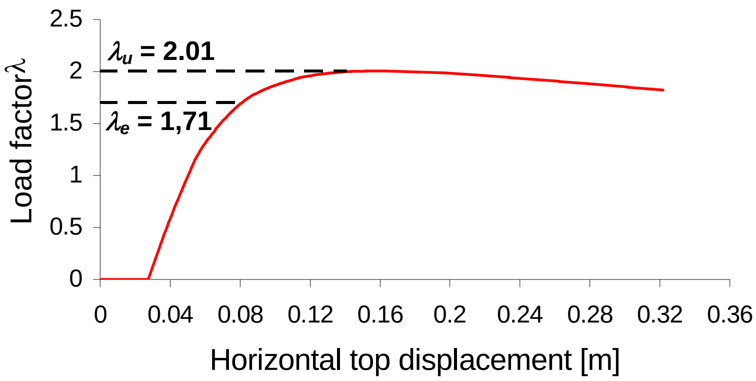


Figure 5-23: top displacement – total horizontal load for "Frame A" of the UK building

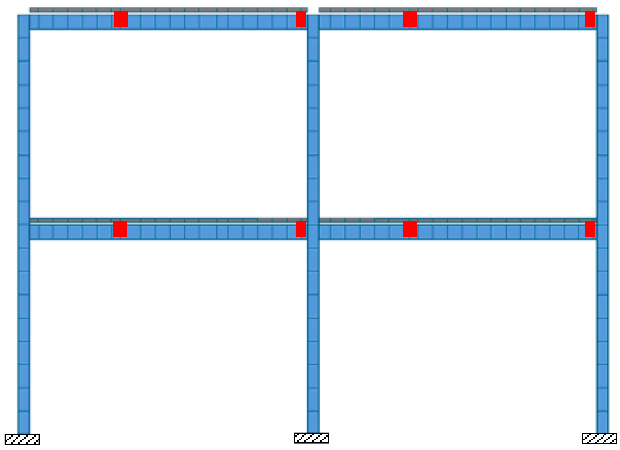


Figure 5-24: location of the plastic hinges at failure of "Frame A" of the "UK" building

As said in *§ 2.2.5.6*, combined mechanisms are influenced by second-order effects. Second-order rigid-plastic analyses are then conducted through hand computations so as to evaluate the influence of the geometrical non-linearities on the value of the first-order rigid-plastic load factor λ_p as it was done for the "Bochum" frame. The relevant results are given in *Figure 5-25*.

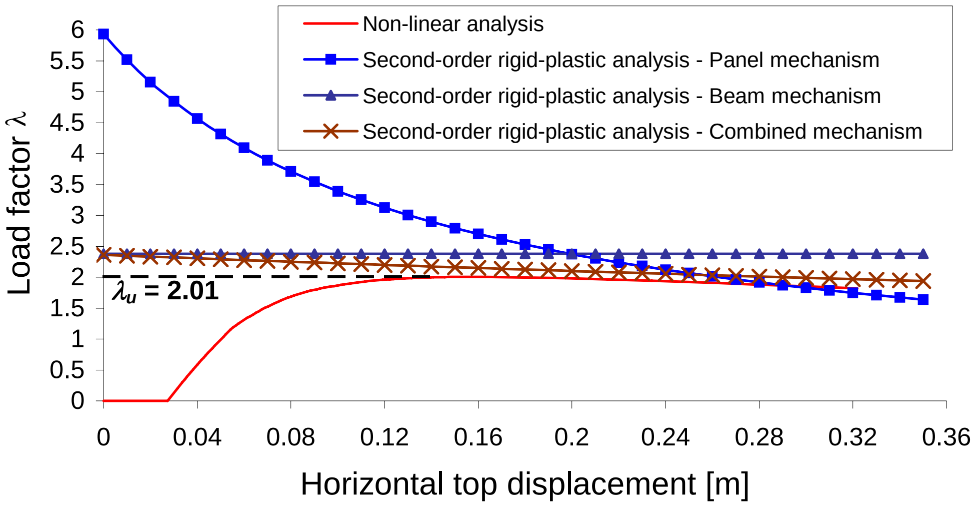


Figure 5-25: second-order rigid-plastic analyses for "Frame A" of the "UK" building

The descending branch of the frame response obtained through the non-linear analysis is found close to the one deduced from the second-order rigid-plastic analysis even if, as shown in *Figure 5-24*, the corresponding failure load is not associated to a full plastic mechanism, which would need the formation of eleven plastic hinges. That is due to the fact that the bending moments in the cross sections where the last hinges are missing (at the column bases) are close to the plastic moment resistance of these cross sections.

It can also been observed on this figure that the panel plastic mechanism is much more influenced by the second order effects than the combined one; it is illustrated by the very sloping descending branch corresponding to the panel mechanism with regards to the one corresponding to the combined mechanism.

Finally, two elastic critical analyses are performed with a difference in the number of perfect plastic hinges introduced in the modelling:

- the first one is conducted with eight plastic hinges located as shown in *Figure 5-24*;
- the second one is conducted with seven plastic hinges which represents the situation just before failure when the last plastic hinge has not yet formed (plastic hinge at one third of the top right beam).

These two analyses give elastic critical load factors respectively equal to 1.69 and 2.72; the first one is smaller than the ultimate load factor obtained through the non-linear analysis while the other is much higher. These last analyses confirm the above prediction that failure is due to a global frame instability widely influenced by the yielding phenomena.

5.4.6 "Luxembourg" building

This building is described in *§ 3.6*. One of its constitutive frames is reminded in *Figure 5-26* with the simplified substitute frame defined in *Reference [2]*; it is the latter which is studied herein.

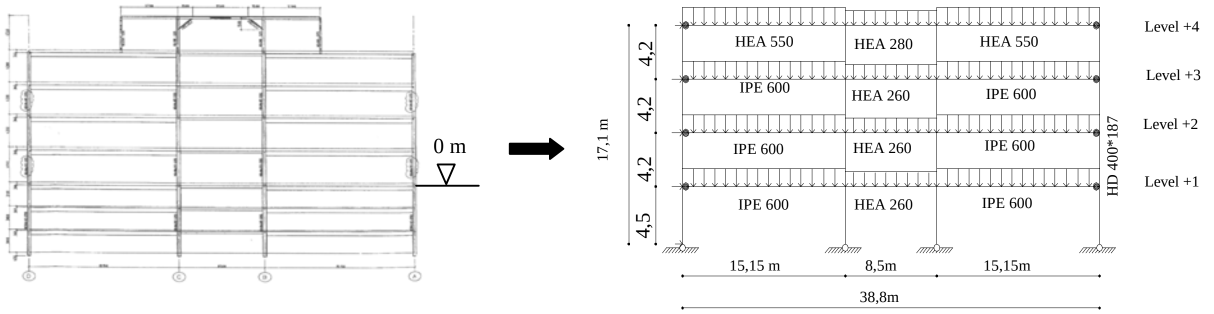


Figure 5-26: from the actual "Luxembourg" structure to the simplified substitute frame

The column bases are assumed to be nominally pinned. The internal joints are composite ones and are characterised by a great stiffness; consequently, they are assumed to be fully rigid. Only the external steel joints are modelled through a tri-linear behaviour curves; their properties are computed through the software COP *[30]*.

The loads applied to the studied frame are given in *Table 3-1* in *§ 3.6*.

All the cross sections of the constitutive members are class 1 ones; so, plastic analyses can be performed on this structure.

The obtained elastic critical load factors obtained for this frame are as following:

- $\lambda_{cr,uncracked} = 5.15$ → $\lambda_{Sd}/\lambda_{cr,uncracked} = 0.194$
- $\lambda_{cr,cracked} = 4.62$ → $\lambda_{Sd}/\lambda_{cr,cracked} = 0.216$

The frame seems therefore prone to significant sway.

According to the first-order rigid-plastic analysis, failure occurs when a local plastic beam mechanism is formed in a lateral beam of the first storey for a load factor $\lambda_p = 1.58$ (see *Table 5-5*).

Table 5-5: results from a first-order rigid-plastic analysis

Type of mechanism	λ_p
Panel mechanism (minimum obtained for a first-storey mechanism)	2.45
Beam mechanism	1.58
Combined mechanism	1.64

The behaviour of the frame predicted by a non-linear analysis is shown in *Figure 5-27*. An initial deformation has been introduced in the modelling which is represented by the initial abscissa in *Figure 5-27* (initial top displacement equal to 0.05 m).

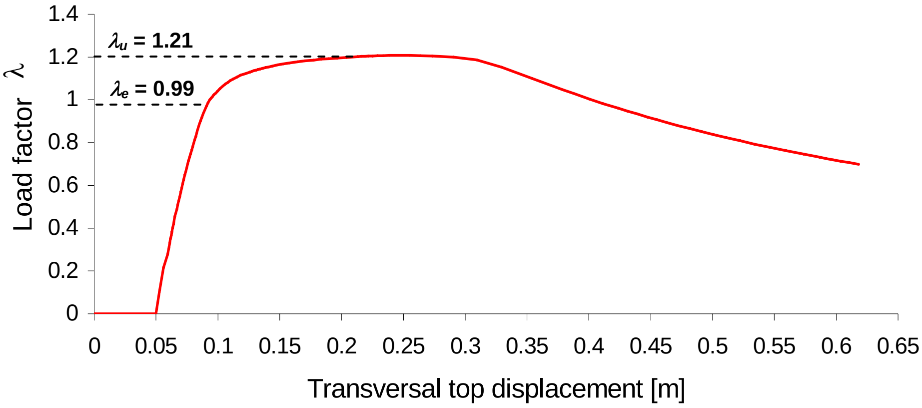


Figure 5-27: behaviour of the "Luxembourg" frame

The ultimate load factor λ_u is equal to 1.21 and the one corresponding to the formation of the first plastic hinge amounts 0.99. The top transversal displacement at failure is 0.21 m, including the initial out-of-plumb. The structure fails when a panel mechanism forms in the first storey (see *Figure 5-28*), even if the one obtained through a first-order rigid-plastic mechanism is a beam one.

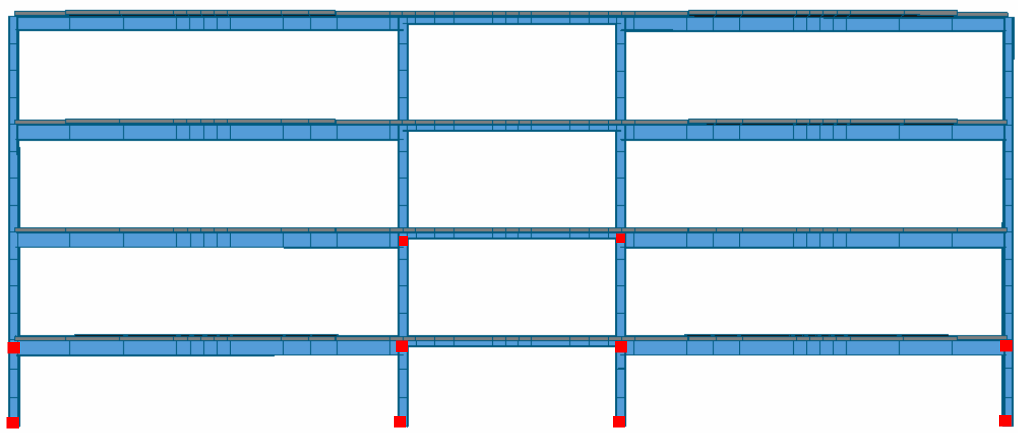


Figure 5-28: position of the plastic hinges at failure of the "Luxembourg" building

Second-order rigid-plastic analyses are performed (*Figure 5-29*) so as to understand how the structure behaves at failure. *Figure 5-29* results in similar conclusions to those drawn previously for the "Eisenach" building; it can also been observed that the panel plastic mechanism is more affected by the second order effects than the combined plastic one as observed for the "UK" building in the previous paragraph.

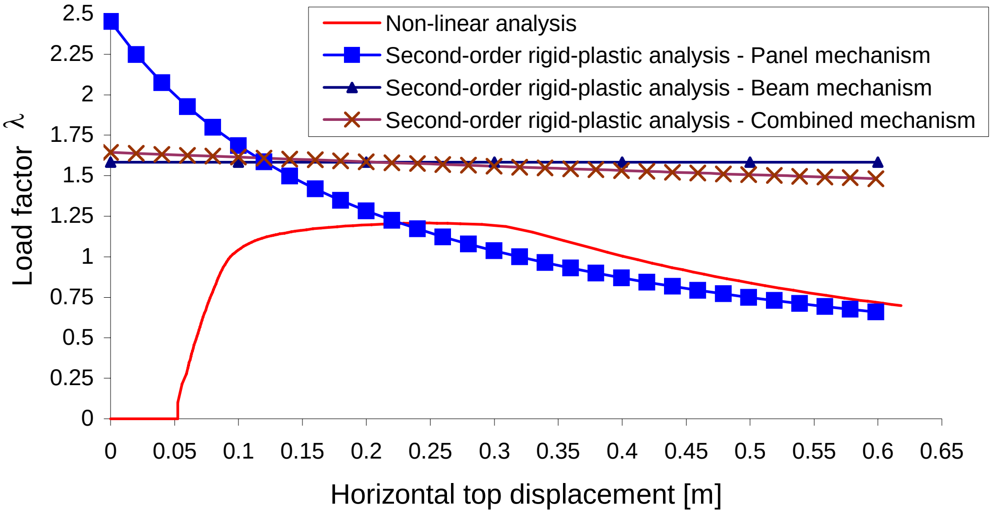


Figure 5-29: second-order effects in the Luxembourg building

5.4.7 Conclusions

Through the numerical investigations presented herein, we have demonstrated that:

- The plastic failure mechanism obtained through a non-linear analyses may not coincide with the one occurring through a first-order rigid-plastic analysis; this phenomenon was illustrated through the "Eisenach" and "Luxembourg" frame investigations where the "non-linear" plastic mechanism is a panel one while the "first-order rigid-plastic" mechanism is a beam one. This illustrates the great influence of the second order effects on the frame behaviour and, in particular, on the yielding phenomena.

- The second-order effects have no significant effects on the beam plastic mechanism in opposition to the panel and combined ones which are widely influenced by the latter with different proportions; indeed, the influence of the second-order effects is greater for a panel plastic mechanism than for a combined one.

- The criteria used for the steel frame classification as sway or non-sway may be transferred to composite frames.

Consequently, it may be concluded from the study of the five composite frames that the general behavioural response of such structures to static vertical and horizontal loads is quite similar to the one exhibited by steel sway frames. This conclusion justifies the following chapter investigations on the applicability to composite sway frames of simplified analytical methods initially developed for steel ones.

5.5 Chapter conclusions

We have presented, in this chapter, different numerical investigations performed on reference composite sway frames under static loading.

We have first proceeded to the description of the homemade finite element software called FINELG used for the numerical investigations presented herein and of the adopted assumptions which are needed for the frame idealizations and modelling. Then, we have described a Benchmark study that we coordinated aiming at validating the FINELG software used for the numerical investigations. Finally, we have presented numerical investigations performed on composite sway frames isolated from the actual buildings presented in *Chapter 3*; through these investigations, we have concluded that the general behavioural response of such structures to static vertical and horizontal loads is quite similar to the one exhibited by steel sway frames.

As a main conclusion, the application, to composite sway building frames, of simplified design methods available for steel buildings (see *§ 2.2.6*) may be a priori contemplated. Investigations aimed at validating these statements are presented in the next chapter. In the latter, some of the presented results obtained through the non-linear analysis will be used as "reference" results for the validation of the simplified analytical method predictions.

Chapter 6 : Applicability of simplified analytical methods

6.1 Introduction

Several simplified analytical methods exist for the study of steel sway frames and some of them are presented in *§ 2.2.6*. The objective here is to investigate whether and how these design procedures can be generalized to composite sway frames. Two of these methods are focused on in the following paragraphs: the amplified sway moment method (*§ 2.2.6.2*) and the Merchant-Rankine approach (*§ 2.2.6.4*).

The applicability of the other simplified analytical methods presented in *§ 2.2.6* is not investigated herein, what is justified here below:

- The applicability of the wind moment method (see *§ 2.2.6.6*) to composite sway frames has already been investigated in a Ph.D thesis of Nottingham University *[1]*; in the latter, it was demonstrated that the remarks concerning the application of this method to steel sway frames presented in *§ 2.2.6.6* are still valid for composite ones. In addition, this method is closer to a pre-design method than to an analytical one and it is difficult to use it so as to predict a failure load factor. So, this method is no more investigated herein.
- The sway-mode buckling length method (see *§ 2.2.6.3*) gives too safe results for steel sway frames, which explains the fact that this method is rarely used for such frames. Accordingly, this method is not investigated herein.
- In *Reference [3]*, it was demonstrated that the simplified second-order plastic analysis is equivalent to the Merchant-Rankine approach if this method is used so as to determine the ultimate load factor λ_u.

As said in *§ 1.2*, composite structures present a particularity according to steel structures: the concrete cracking. This phenomenon leads to an amplification of the lateral deflections and, consequently, to an amplification of the second order effects, which 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 respectively, larger sway displacements are reported in the composite one.

This additional phenomenon (with regard to steel structures) should be taken into account in the simplified design methods. In *Reference [2]*, it is proposed to take account of the concrete cracking through the computation of the critical load factor: the concrete is assumed to be cracked in the support regions by assuming that the concrete has no stiffness when loaded in tension (→ computation of $\lambda_{cr,cracked}$ – see *§ 5.2.4.2*). The accuracy of this proposition was investigated in *[2]* through a parametrical study performed on the “Luxembourg” building: the latter showed that a better agreement between the Merchant-Rankine approach prediction and the numerical non-linear analysis results is obtained when using this assumption for the critical load factor computation. *Figure 6-1* shows an example of the results obtained through the parametrical study (the parameter which is varied here is the column base stiffness of the “Luxembourg” building – see *[2]*); it can be observed that the use of the $\lambda_{cr,cracked}$ permits to be closer to the Merchant-Rankine curve and to come in the safe side for some points (i.e. in the upper part of the graph according to the Merchant-rankine curve).

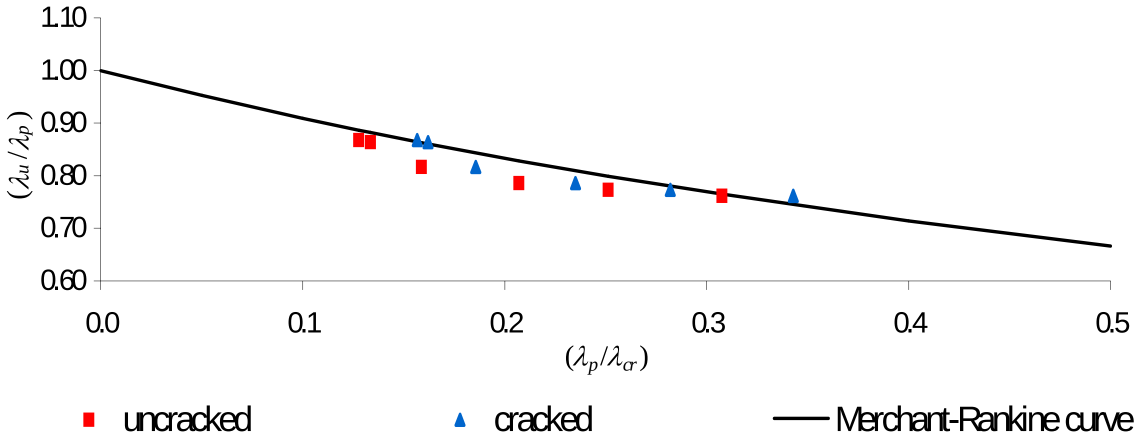


Figure 6-1: parametric study performed on the “Luxembourg” building *[2]*

The structures used as references herein are those studied in the previous chapter; in the latter, the two different assumptions concerning the concrete cracking which are described in *§ 5.2.4.2* were used for the computation of the critical load factors (i.e. computation of $\lambda_{cr,uncracked}$ and $\lambda_{cr,cracked}$). The simplified analytical methods studied here will be applied with these two values so as to compare the so-predicted results and to know if the observations made in *Reference [2]* can be extended to the structures studied herein. The predicted values will also be compared to the “non-linear” ones obtained through the numerical investigations presented in the previous chapter (considered as “reference” results). These comparisons are presented in *§ 6.2* and *6.3* for the amplified sway moment method and the Merchant-Rankine approach respectively.

The Merchant-Rankine approach cannot be applied to the “Bochum” structure in a straightforward way, as the latter is based on the concept of “proportional loading”; an alternative method is nevertheless developed and analysed in *§ 6.3.3* to assess the ultimate load factor of the “Bochum” structure.

Remark: part of the presented results comes from *Reference [2]* and *[3]*.

6.2 *Amplified sway moment method*

6.2.1 Introduction

The Amplified Sway Moment Method ("ASMM") is proposed in Eurocode 3. In this method, first-order linear elastic analyses are first carried out; then, the resulting internal forces are amplified by a "sway factor" which depends of the critical load factor so as to ascertain for second-order sway effects. The "ASMM" is described in details in *§ 2.2.6.2*.

This method has been applied to the frames which have been studied in the previous chapter so as to determine the elastic load factor λ_e for which the first plastic hinge forms; the criterion of applicability of this method ($V_{Sd}/V_{cr} \in [0.1; 0.25]$) is respected by all the studied frames. The first-order elastic analyses needed by this method have been performed through the FINELG software as the modelling of these frames were already available from the previous computations.

6.2.2 Comparison of the results

Table 6-1 and *Table 6-2* show how are the results obtained in *Chapter 5* through a full non-linear analysis compare with those got from the "ASMM" with the use of $\lambda_{cr,uncracked}$ and $\lambda_{cr,cracked}$ for the computation of the "sway" factor (see *Formula (2.5)* in *§ 2.2.6.2*).

Table 6-1: comparison between the non-linear analysis results and the "ASMM" predictions (with $\lambda_{cr,uncracked}$)

Structures	$\lambda_{cr,uncracked}$	$\dfrac{1}{1-\dfrac{V_{sd}}{V_{cr}}}$	λ_e **"ASMM"**	λ_e **Non-linear analysis**	**Difference (in %)**
"Ispra" building	6.49	1.18	1.56	1.61	3.1
"Bochum" building	9.83	1.11	1.21	1.26	4
"Eisenach" building	4.33	1.3	1.18	1.14	-3.4
"UK" building	9.31	1.12	1.63	1.71	4.7
"Luxembourg" building	5.15	1.24	0.96	0.99	3.4

Table 6-2: comparison between the non-linear analysis results and the "ASMM" predictions (with $\lambda_{cr,cracked}$)

Structures	$\lambda_{cr,cracked}$	$\dfrac{1}{1-\dfrac{V_{sd}}{V_{cr}}}$	λ_e **"ASMM"**	λ_e **Non-linear analysis**	**Difference (in %)**
"Ispra" building	5.95	1.20	1.55	1.61	3.7
"Bochum" building	9.42	1.12	1.20	1.26	4.9
"Eisenach" building	4.27	1.31	1.18	1.14	-3.4
"UK" building	8.77	1.13	1.63	1.71	4.7
"Luxembourg" building	4.62	1.28	0.95	0.99	4

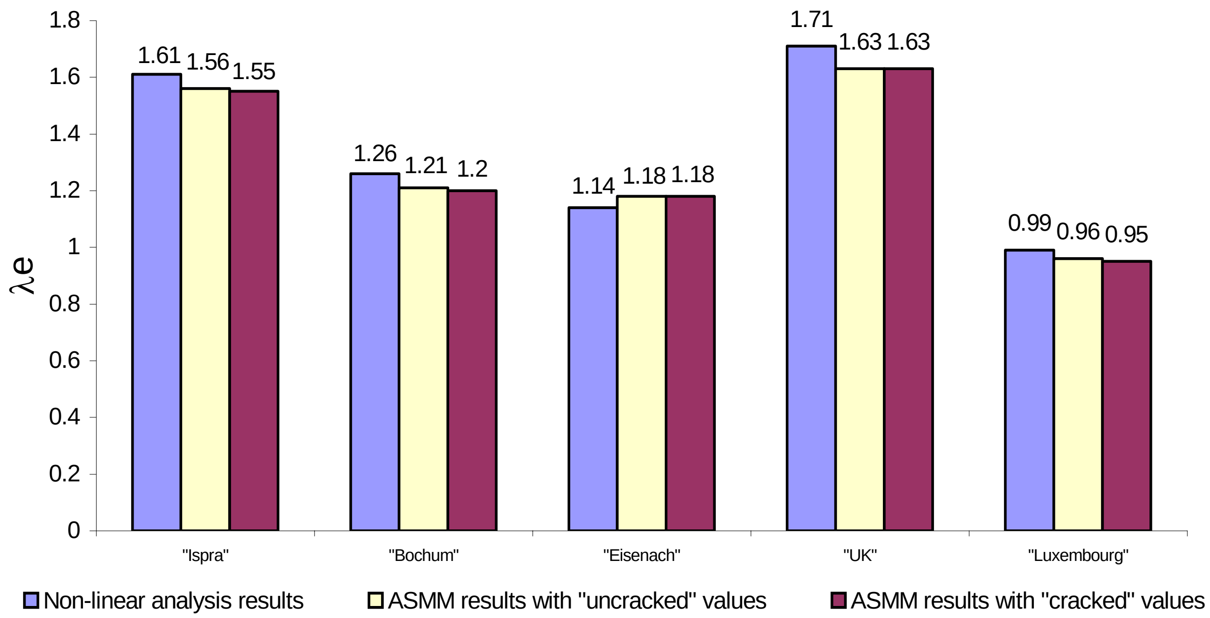


Figure 6-2: comparison between the results numerically obtained and the "ASMM" predictions

From this table, it may be concluded to a pretty good agreement between the "ASMM" predictions and the numerically obtained results; indeed the maximum difference amounts only 4.9 % for the "Bochum" building. Only the application of this method to the "Eisenach" building gives an unsafe result according to the non-linear analysis result, but with a small difference.

In addition, the formation of the first plastic hinge occurs at the same place in the frame through the "ASMM" than through the non-linear analysis, and that for all the studied frames.

Concerning the use of $\lambda_{cr,cracked}$ or $\lambda_{cr,uncracked}$ in the "ASMM", it can be observed in *Figure 6-2* that the chosen assumption has not a significant effect on the predicted value of λ_e.

According to these results, it can be concluded that the amplified sway moment method can be applied with confidence to sway composite structures; the adopted assumption with regards to the concrete cracking for the computation of λ_{cr} has no significant influences on the predicted value λ_e for the studied cases.

An important remark is that the critical load factors used in the ASMM are the ones numerically obtained through the FINELG finite element software; more investigations should be performed in future to know if the accuracy of this method is conserved if the critical load factor is computed through simplified methods (as the Grinter procedure for instance).

6.3 *Merchant-Rankine approach*

6.3.1 Introduction

The Merchant-Rankine and the Modified Merchant-Rankine approaches ("MR" and "MMR" respectively) are introduced in *§ 2.2.6.4* (see *Formulas (2.7)* and *(2.8)*); the latter are second-order elasto-plastic methods which allow to assess the ultimate load factor of a steel structure with account of interactions between plasticity (λ_p) and instability (λ_{cr}) in a simplified and empirical way.

As said in *§ 6.1*, the MR approach cannot be applied to the "Bochum" structure in a straightforward way as this approach is based on the concept of "proportional loading". Nevertheless, an alternative method will be developed and analysed in *§ 6.3.3* to analytically assess the ultimate load factor of this structure.

For the modelling of the frames presented in the previous chapter, the strain hardening of the steel material and the post-critical behaviour of the joints were not considered (see *§ 5.2.3.2* and *5.2.3.3*); so, it is the "MR" approach and not the "MMR" approach which will be used in this chapter as the "MMR" formula was developed in order to take into account of strain hardening effects. Nevertheless, the alternative method developed for the "Bochum" frame will be based on the "MMR" approach as the steel strain hardening was introduced in the modelling of this frame presented in the previous chapter.

6.3.2 Comparison of the results

The critical and first-order rigid-plastic load factor values used herein for the "MR" approach are the ones computed and presented in the previous chapter. *Table 6-3* and *Table 6-4* show how are the predictions of the Merchant-Rankine approach (with the use of $\lambda_{cr,uncracked}$ and $\lambda_{cr,cracked}$ respectively) compare with the numerically derived ultimate load factors λ_u.

Table 6-3: comparison between the non-linear analysis and the "MR" results (use of $\lambda_{cr,uncracked}$)

Structures	λ_p	$\lambda_{cr,uncracked}$	λ_u "MR"	λ_u Non-linear analysis	Difference (in %)
"Ispra" building	1.84	6.49	1.43	1.79	20.1
"Eisenach" building	1.545	4.33	1.139	1.138	-0.1
"UK" building	2.36	9.31	1.88	2.01	6.5
"Luxembourg" building	1.58	5.15	1.210	1.209	-0.2

Table 6-4: comparison between the non-linear analysis and the "MR" results (use of $\lambda_{cr,cracked}$)

Structures	λ_p	$\lambda_{cr,cracked}$	λ_u "MR"	λ_u Non-linear analysis	Difference (in %)
"Ispra" building	1.84	5.95	1.41	1.79	21.2
"Eisenach" building	1.545	4.27	1.137	1.138	0.1
"UK" building	2.36	8.77	1.86	2.01	7.5
"Luxembourg" building	1.58	4.62	1.181	1.209	2.3

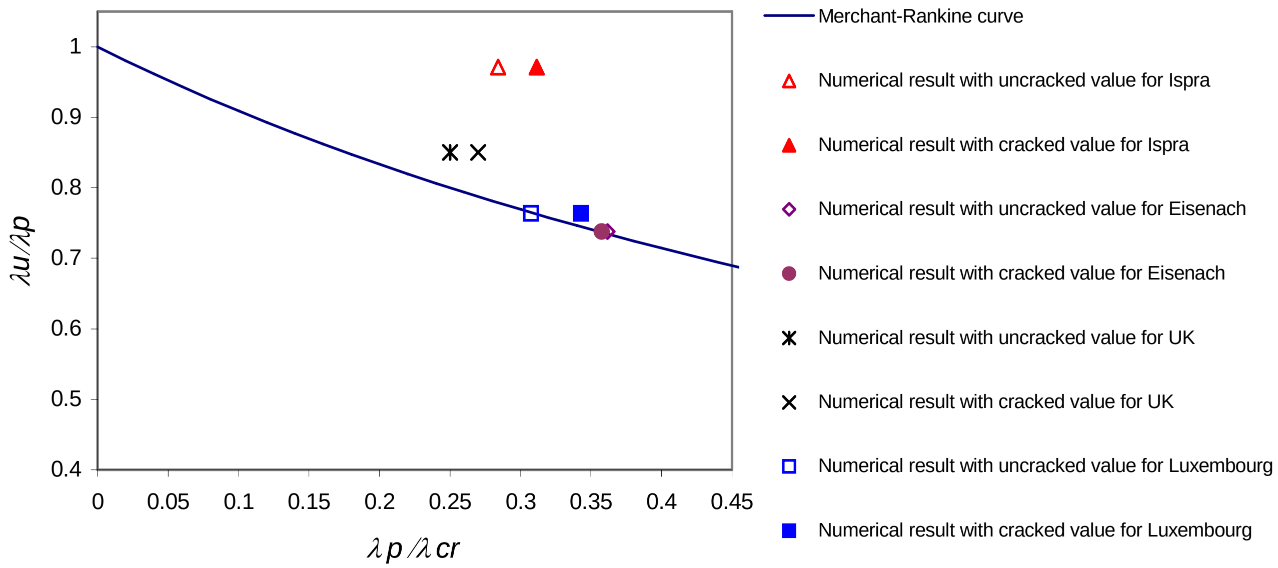


Figure 6-3: comparison between the "MR" curve and the numerically obtained results

First, it can be observed on this figure that all the points does not enter in the field of application of the Merchant-Rankine approach as defined in *§ 2.2.6.4* ($\lambda_p/\lambda_{cr} \in [0.1;0.25]$); however, the studied frames are not too far from this domain.

A rather good agreement between numerical simulations and the "MR" calculation model seems to be obtained, except for the "Ispra" building. In *Figure 6-3*, it can be observed that the use of the $\lambda_{cr,cracked}$ value instead of the $\lambda_{cr,uncracked}$ one permits to pass from the unsafe to the safe side of the graph (the points corresponding to the "Luxembourg" and the "Eisenach" buildings pass from the lower to the upper part of the graph according to the "MR" curve), which confirms the observations of *Reference [2]*.

However, the obtained results have to be discussed:

- The somewhat too safe character of the "MR" approach for the "Ispra" building results from the nature of the first-order rigid-plastic mechanism, which corresponds to the plastic failure of a beam; the same phenomenon was also observed for steel structures presenting a beam plastic mechanism through a first-order rigid-plastic analysis (see *§ 2.2.6.4*). In fact, the formation of plastic beam mechanisms is not affected in a significant way by the second order effects which can justified the too conservative result obtained through the "MR" approach.

- The good prediction of the ultimate load factor through the "MR" approach for the "UK" building results of the nature of the first-order rigid-plastic mechanism, which is a combined one. Again, the same phenomenon was observed for steel structures presenting such first-order rigid-plastic mechanism.
- It can be observed that the "MR" approach gives very good results for the "Eisenach" and "Luxembourg" buildings although the plastic mechanism associated to a first-order rigid-plastic analysis is a beam one. This observation seems to be in contradiction with the obtained results for steel structures. In fact, if reference is made to the numerical investigations performed in the previous chapter, it was shown that the failure of these frames through a non-linear analysis is associated to the formation of panel mechanisms influenced by the second order effects. So, the first-order rigid-plastic load factor associated to a beam plastic mechanism, which is involved in the "MR" approach, does not correspond to the plastic mechanism occurring at failure. It can be then conclude that the good agreement between the "MR" predictions and the non-linear analysis results for these two structures occurs quite by chance; for instance, if the second-order effects had a greater influence on the behaviour of these structures and especially on the formation of the panel plastic mechanism, the "MR" approach could have given unsafe results.

A solution would be to develop different formulas, one for each type of first-order rigid-plastic mechanism (i.e. beam, panel and combined mechanism) to analytically predict different ultimate load factors; the ultimate load factor of the studied structure would be the smallest one. This proposition will be detailed later on in *§ 7.3*.

6.3.3 Development of an alternative method for the "Bochum" building

This paragraph presents analytical investigations performed on the non-proportionally loaded "Bochum" structure with the development of an alternative method which enables to estimate the ultimate load factor with account of the loading sequence.

This alternative method consists:

- in deriving a "MMR" interaction curve in a "$V - H$" diagram (*Figure 6-4*), V and H being respectively the total vertical and horizontal applied loads at failure;
- in reporting in this diagram the actual loading path followed during the test (see *§ 3.3*);
- in defining the failure load at the intersection between the "MMR" interaction curve and the one representing the actual loading path.

This approach is based on the assumption that the ultimate load factor of a structure is independent of the loading history; this is not theoretically exact but it usually appears as acceptable.

In practice, the "MMR" interaction curve is obtained as follows:

- first, the V and H loads in *Figure 6-4* are normalised by dividing them by their values V_{serv} and H_{serv} at serviceability limit state, respectively 1262,4 kN and 100 kN (see *§ 3.3*);
- in a second step, different load combinations between V_{serv} and H_{serv} are considered (i.e. 0,5 V_{serv} + H_{serv} or V_{serv} +0,5 H_{serv} or ...);
- for each particular load combination, the corresponding vertical and horizontal service loads are then assumed to be proportionally increased until failure (load factor λ_{prop}); through this assumption, the critical load factor ($\lambda_{prop,cr}$) and the first-order rigid-plastic load factor ($\lambda_{prop,p}$) are computed and an estimation of the ultimate load factor ($\lambda_{prop,u}$) is derived, for each load combination, by means of the "MMR" approach;
- finally, the ultimate load factors are reported in the "$V - H$" diagram so as to obtain the MMR interaction curve.

The "MMR" interaction curve computed for the "Bochum" structure is presented in *Figure 6-4* (curve "DIAC"). The points reported on this graph have been computed with $\lambda_{cr,cracked}$ values as it has been demonstrated in the previous paragraph that the use of the latter in the MR approach sometimes permits to pass from the unsafe to the safe side; however, a comparison between the so-obtained "MMR" interaction curve and the one computed with $\lambda_{cr,uncracked}$ values is presented and discussed later on in *Figure 6-5*.

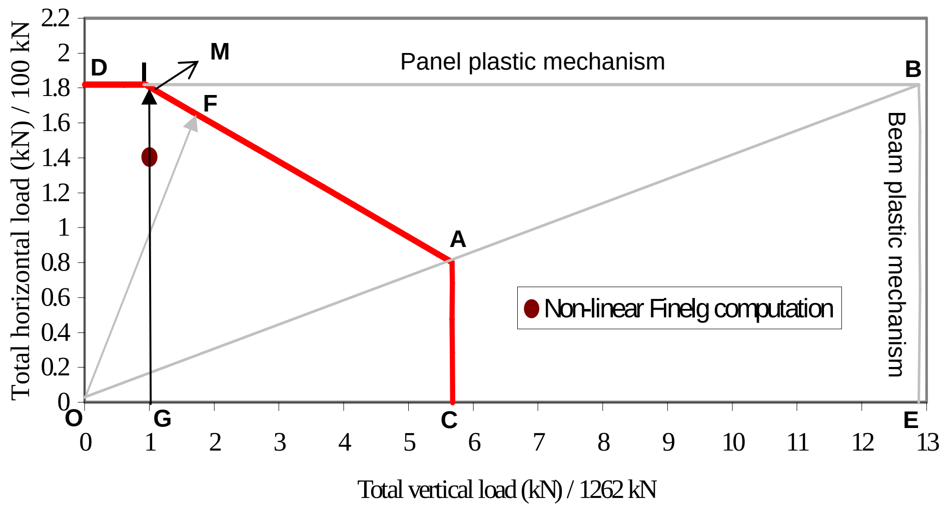


Figure 6-4: "MMR" interaction curve

The curve "DBE" corresponds to the first-order rigid-plastic resistance interaction curve; the horizontal line "DB" relates to the development of a first-order panel plastic mechanism ($\lambda_{prop,p}$ only depends on the horizontal loads) and the vertical line "BE" to the development of a beam plastic mechanism in beam A (see *Figure 3-4* in *§ 3.3*) ($\lambda_{prop,p}$ only depends on the vertical loads). In *Figure 6-4*, the diagram is seen to be separated in two zones by the line "OAB":

- development of a first-order rigid-plastic panel mechanism for load combinations relative to the upper part of the diagram;
- development of a first-order rigid-plastic beam mechanism in beam A for load combinations relative to the lower part of the diagram.

Figure 6-4 shows that no combination of V and H leads to the development of a combined plastic mechanism, as far as the "Bochum" structure is concerned.

The shape of the "MMR" interaction curve presented in *Figure 6-4* can be explained as follows.

- When no vertical loads are applied to the structure, the ultimate load factor $\lambda_{prop,u}$ corresponds to the development of a panel plastic mechanism (point "D"); no instability phenomena occur as no vertical loads are applied. In this case, $\lambda_{propr,u}$ is equal to 1,82 ($H = 182$kN), which is equal to the first-order rigid-plastic load factor λ_p computed in *§ 5.4.3*.
- For small vertical loads, a panel plastic mechanism still appears at failure (line "DI"). This indicates that second-order effects are quite negligible in this loading range.
- Beyond point "I", second-order effects can no more be neglected and the "MMR" computed values $\lambda_{prop,u}$ reduce when the importance of the vertical loads in the load combinations increases. At point "I", the ratio $\lambda_{cr,cracked}/\lambda_p$ is equal to 10.
- When no horizontal loads are applied to the structure, the first-order rigid-plastic load factor corresponds to the development of a beam mechanism ($\lambda_{prop,p}$ = 12,88); the "MMR" ultimate load factor $\lambda_{prop,u}$, which is equal to 5,68, takes into account the interaction between the plasticity and the instability phenomena under high vertical loads.
- The value $\lambda_{prop,u}$ is constant and equal to 5,68 when the first-order rigid-plastic mechanism is a beam one (vertical line "CA" of the "MMR" interaction curve) as, in this specific case, λ_p, and therefore the "MMR" load factor, are strictly depending on the vertical loads.

For the "Bochum" structure, if the service loads were proportionally increased (loading path "OF"), an ultimate load factor $\lambda_{prop,u}$ = 1,66 would be found by means of the "MMR" approach ($V = 2095{,}6$ kN and $H = 166$ kN). But, as stated in *§ 3.3*, this is not the case and therefore the $\lambda_{prop,u}$ = 1,66 load factor can not be compared with the one (λ_u = 1,41) obtained in *§ 5.4.3* by means of FINELG; indeed, the latter has not been computed with a proportional loading, but with the actual one.

The actual loading path is represented in *Figure 6-4* by the arrow "OGM". At its intersection with the interaction curve, a "MMR" estimated failure load multiplier $\lambda_u = 1{,}8$ is derived, which may be now compared to the FINELG numerical result. The difference between the two approaches is equal to 22 % and the analytical predicted value is seen to be quite unconservative. Such a conclusion has already been drawn (see *§ 2.2.6.4*) from previous studies on steel structures characterised by the development of a first-order rigid-plastic panel mechanism. Furthermore, the fact that "M" is very close to "I" in *Figure 6-4* confirms the importance of the sway effects in the "Bochum" frame (in accordance with the numerical results where it is observed that only one plastic hinge is missing to form a full panel plastic mechanism at failure, see *Figure 5-15* in *§ 5.4.3*).

In *Figure 6-5*, a comparison between the "MMR" interaction curve computed with $\lambda_{cr,cracked}$ values and the one computed with $\lambda_{cr,uncracked}$ values is presented. It can be observed in this figure that the same value of λ_u is obtained through the two curves for the "Bochum" frame loading sequence. The difference between the two curves increases when the vertical loads are increasing; this can be justified by the fact that the more these loads are increasing, the more the instability phenomenon (and consequently the assumption made for the computation of λ_{cr}) influences the value of the ultimate load factor λ_u.

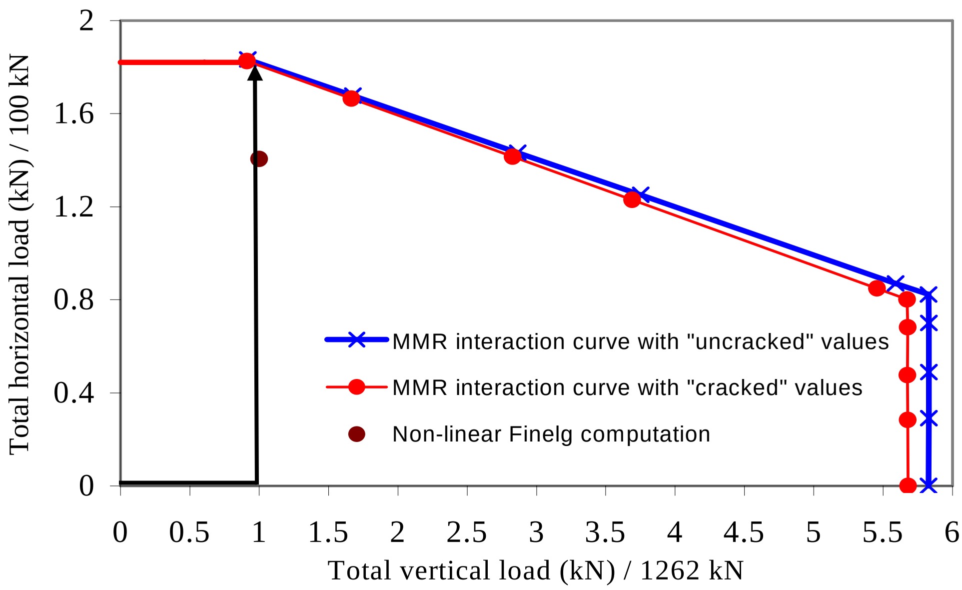


Figure 6-5: comparison between the "MMR" curves computed with $\lambda_{cr,cracked}$ and $\lambda_{cr,uncracked}$

6.4 Chapter conclusions

In this chapter, we have investigated the applicability of the "Amplified sway moment method" and "Merchant-Rankine" approaches, respectively based on elastic and plastic design philosophies, to composite sway structures. Also, the influence on the accuracy of these methods of the chosen assumption with regards to the concrete cracking in the computation of the elastic critical load factor has been investigated.

First, for the “amplified sway moment method”, we have demonstrated that:

- The chosen assumption for the computation of λ_{cr} has no significant influences on the predicted value through this method for the studied cases.
- A good accuracy of this method is obtained when applied to sway composite structures; the maximum difference between the amplified sway moment method results and the numerically obtained results (results obtained through a non-linear analysis) is equal to 4.9 %, which is reasonable.

More investigations should be performed in this field so as to verify if the accuracy of this method still be good when the critical load factor is predicted through simplified analytical methods (for instance the Grinter method); indeed, for the presented results, the critical load factors used in the amplified sway moment method were numerically computed through the FINELG software which constitutes a relatively precise estimation of the actual critical load factor.

Secondly, we have investigated the applicability of the Merchant-Rankine approach and we have shown that:

- The conclusions concerning the accuracy of this method which were drawn for steel sway structures *[8]* still appropriate for the composite sway structures:
  - safe for beam plastic mechanisms;
  - adequate for combined plastic mechanisms;
  - unsafe for panel plastic mechanisms.
- The use of the $\lambda_{cr,cracked}$ value instead of the $\lambda_{cr,uncracked}$ one in the Merchant-Rankine approach permits, for some cases, to pass from the unsafe to the safe side for the estimation of the λ_u with respect to the non-linear analysis results; this result is in agreement with the conclusions of a previous work (*Reference [2]*).
- 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. Complementary works should be performed in order to improve the Merchant-Rankine approach through a more accurate consideration of the nature of the first-order rigid-plastic mechanism.

Finally, we have proposed an alternative method for the estimation of the ultimate load factor of frames subjected to a non-proportional loading, which is the case of the “Bochum” structure.

The developed alternative method consists in deriving a Merchant-Rankine interaction curve in a “$V - H$” diagram (*Figure 6-4*), V and H being respectively the total vertical and horizontal applied loads. The procedure to follow to compute the interaction curve was detailed and applied to the "Bochum" building with satisfactory results; in fact, we have obtained an unsafe result through this method but this observation is consistent with previous results obtained for steel sway frames characterised by a first-order rigid-plastic panel mechanism (as it is the case for the "Bochum" building).

Chapter 7 : General conclusions

Since few years, the construction of taller buildings and larger industrial halls without wind bracing systems becomes quite popular; this is susceptible to make global instability a relevant failure mode. This is not yet covered by Eurocode 4 which mainly deals 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 only and proposes design rules for composite slabs, beams, columns and joints. To compensate for the lack of knowledge in this field, a three years research project on global instability of composite sway frames, in which we were widely involved, has been funded by the European Community for Steel and Coal (ECSC) in 2000.

The studies presented in this thesis reflect our contribution to the above-mentioned European project. The objective of this work was to analyse the behaviour of sway composite frames under static loading through numerical analyses and to investigate the applicability of simplified analytical methods initially developed for steel sway frames to composite ones.

To achieve this goal, we have first coordinated a benchmark study aimed at validating the finite element software used by the different partners involved in the numerical investigations of the above-mentioned European project. This study was performed on a full-scale composite building tested at the “Building Research Establishment” in United Kingdom for which details data and results were available; the numerically obtained results were compared to the test ones. As a conclusion, we have demonstrated through this study that the homemade finite element software called FINELG used for the numerical investigations presented herein can be used with a good confidence to investigate the behaviour of composite structures.

Secondly, we have performed different numerical analyses on actual composite sway buildings submitted to horizontal and vertical static loads with the so-validated FINELG software so as to understand their behaviour and to highlight some particularities. We will summarise the main results in *§ 7.1*.

Then, we have investigated the applicability to composite sway structures of two simplified analytical methods: the “Amplified sway moment method” and “Merchant-Rankine” approaches, initially developed for steel structures and respectively based on elastic and plastic design philosophies. The latter cannot be applied in a straightforward way to composite sway frames as these frames present an additional source of sway deflection with regards to steel structures: the concrete cracking. To bypass this problem, a method is proposed in *Reference [2]* which consists in computing a critical load factor of the composite structure with the assumption that the concrete is cracked in the support region; the validity of this method for the cases studied herein has been investigated. The two simplified analytical methods have been applied to above-mentioned numerically simulated building frames; moreover, the numerical results obtained through the non-linear analyses have been used as reference results to which we have compared the analytically predicted ones. In this part, we have also developed an alternative method based on the "Merchant-Rankine" approach formulas for the analytical prediction of the ultimate load factor of sway frames loaded by a non-proportional loading; indeed, the "Merchant-Rankine" approach, which allows to predict the ultimate load factor, cannot be applied in a straightforward way to such frames as it is based on the assumption of a proportional loading. The main conclusions concerning these investigations are drawn in *§ 7.2*.

Finally, we will introduce some suggestions for future works in *§ 7.3*; indeed, we have pointed out in the present work some interesting aspects which would require further developments.

7.1 Conclusions of the numerical investigations

Five actual composite buildings have been studied herein:

- the "Ispra" building;
- the "Bochum" building;
- the "UK" building;
- the "Eisenach" building;
- and the "Luxembourg" building.

The three first ones are full-scale buildings which have been tested in European laboratories, the "Eisenach" building is a factory in Germany and the last one is a bank in Luxembourg; these buildings have been described in detailed in *Chapter 3.*

The different analyses which have been performed on these structures are the following:

- Critical elastic analyses with two different assumptions with regards to the concrete cracking:
    - in the first case, the concrete is assumed to be uncracked all along the beam and to have the same stiffness in tension than in compression (→ $\lambda_{cr,uncracked}$);
    - in the second case, the concrete is assumed to be cracked close to the support by assuming that the concrete has no stiffness in these zones (→ $\lambda_{cr,cracked}$).
- First-order and second-order plastic analyses (→ λ_p)
- Non-linear analyses (→λ_e when the first plastic hinge is formed and λ_u when the failure of the structure is reached)

The numerical tool used to perform these analyses is the geometrically and materially non-linear finite element software FINELG developed at Liège University (M&S department) and at Greisch office (Liège, Belgium); it has been validated through the benchmark study presented in *§ 5.3.* The different results obtained through these analyses are summarized for each studied structures in *Table 7-1.*

Table 7-1: summary of the obtained results through the numerical investigations

Building	$\lambda_{cr,uncracked}$	$\lambda_{cr,cracked}$	λ_p	**Plastic mechanism associated to λ_p**	λ_e	λ_u	**Failure associated to λ_u**
"Ispra"	6.49	5.95	1.84	beam	1.61	1.79	global instability
"Bochum"	9.83	9.42	1.82	panel	1.26	1.41	global instability
"Eisenach"	4.35	4.27	1.55	beam	1.14	1.14	panel plastic mechanism
"UK"	9.31	8.77	2.36	combined	1.71	2.01	global instability
"Luxembourg"	5.15	4.62	1.58	beam	0.99	1.21	panel plastic mechanism

From this table, the following remarks may be drawn:

- All the computed critical load factors are smaller than 10; in other words, the V_{Sd}/V_{cr} $(= \lambda_{Sd}/\lambda_{cr})$ ratios for all the studied structures are higher than 0.1. So, these structures can be classified as sway if reference is made to the Eurocode 3 criterion.
- When the failure associated to λ_u is a global instability phenomenon, we can observe that the load factor at which it occurs is significantly smaller than the elastic critical load factor. This demonstrates the great influence of the yielding of the structure on the instability phenomena.
- For the "Eisenach" and the "Luxembourg" buildings, the failure associated to λ_u is a panel plastic mechanism while the one associated to λ_p is a beam one. This can be explained by the fact that panel plastic mechanisms are strongly influenced by the second order effects while the latter have no significant effects on the beam ones. We have illustrated this observation in *Figure 5-25* and *Figure 5-29* of *Chapter 5*; indeed, we have shown on these figures that the results obtained through the second-order rigid-plastic analyses and the non-linear ones are in good agreement (i.e. the failure of the frame obtained through a non-linear analysis is reached when the "non-linear curve" in a load – top displacement graph intersects the "second-order rigid-plastic curve" associated to the panel mechanism).

From these numerical studies of the five above composite frames, we have demonstrated that the general behavioural response of such structures to static vertical and horizontal loads is quite similar to the one exhibited by steel sway frames; the results presented here tend to prove that the classification criterion (i.e. $V_{Sd}/V_{cr} \geq 0.1$ to be considered as sway) used for the steel structures and proposed in Eurocode 3 could be extended to the composite ones.

Starting from this observation, we decided to investigate the applicability to composite sway frames of simplified analytical methods initially dedicated to steel ones.

7.2 *Conclusions of the analytical investigations*

7.2.1 Amplified sway moment method

This method has been applied to the here-above mentioned frames so as to predict the elastic load factor λ_e at which a first plastic hinge forms. As said previously, the influence of concrete cracking on the results predicted by this method has been investigated through the computation of 2 critical load factors: "cracked" and "uncracked". The results are reminded in *Figure 7-1* where the so-predicted values are compared to the ones numerically obtained through the previously mentioned numerical investigations.

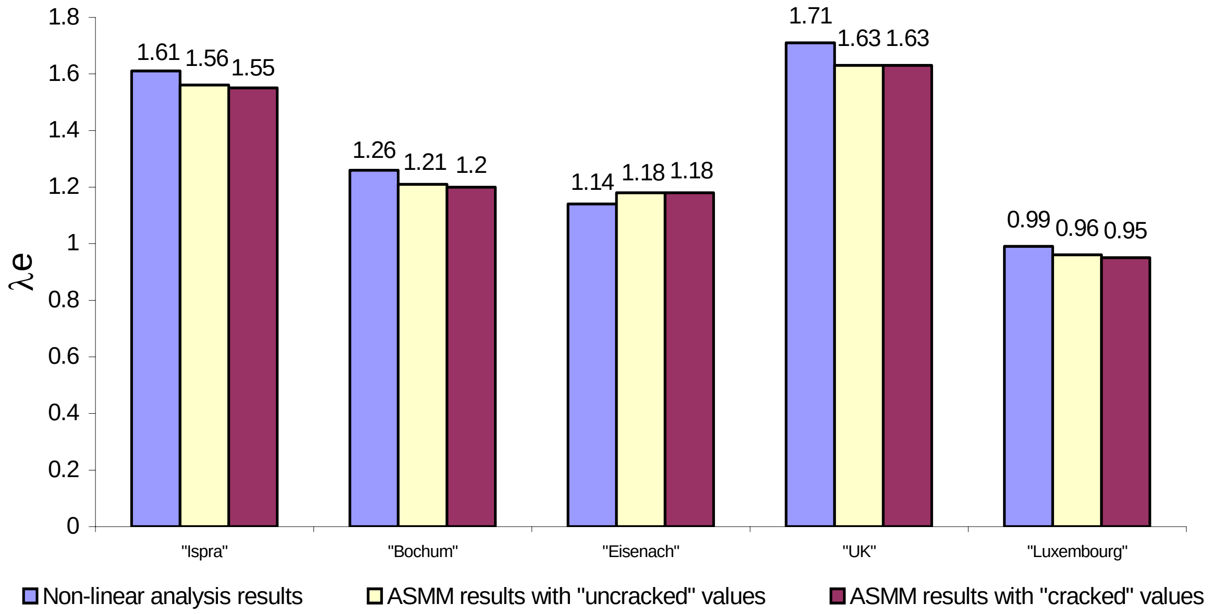


Figure 7-1: comparison between the results numerically obtained and the "ASMM" results

Through these investigations, we have demonstrated that:

- The values predicted by the amplified sway moment method are in very good agreement with the ones numerically obtained; indeed, the maximum difference is equal to 4.9 % for the "Bochum" building.
- All the predicted values are on the safe side with regards to the numerical ones (except for the "Eisenach" building).
- The formation of the first plastic hinge occurs, for all the studied frames, at the same location through the "ASMM" and through the non-linear analysis.
- The assumption concerning the concrete cracking has no significant influence on the values predicted by the "ASMM".

So, accordingly, it can be concluded that the amplified sway moment method can be applied with confidence to sway composite structures with no needs to introduce the concrete cracking in the computation of λ_{cr}. However, some additional investigations, which will be described in *§ 7.3*, should be performed in the future so as to check the influence of some other parameters on the accuracy of this method.

7.2.2 Merchant-Rankine approach

The Merchant-Rankine approach allows to assess the ultimate load factor through a formula that takes account of interactions between plasticity (λ_p) and instability (λ_{cr}) in a simplified and empirical way. Again, we have applied this method to the five reference frames. However, this method could not be applied to the "Bochum" frame in a straightforward way as it is characterised by a non-proportional loading; and this is in opposition with the Merchant-Rankine approach which covers proportional loading. To bypass this problem, we have developed an alternative method so as to estimate the ultimate load factor of such frames, with due account of their actual loading path. This method has been detailed in *§ 6.3.3*.

As for the amplified sway moment method, we have also investigated the influence of the concrete cracking on the critical load factor when predicting ultimate load factors through the Merchant-Rankine approach.

The results obtained for 4 of the 5 frames characterised by a proportional loading are reminded in *Figure 7-2*; those obtained for the "Bochum" building (through the proposed alternative method) are reminded in *Figure 7-3*.

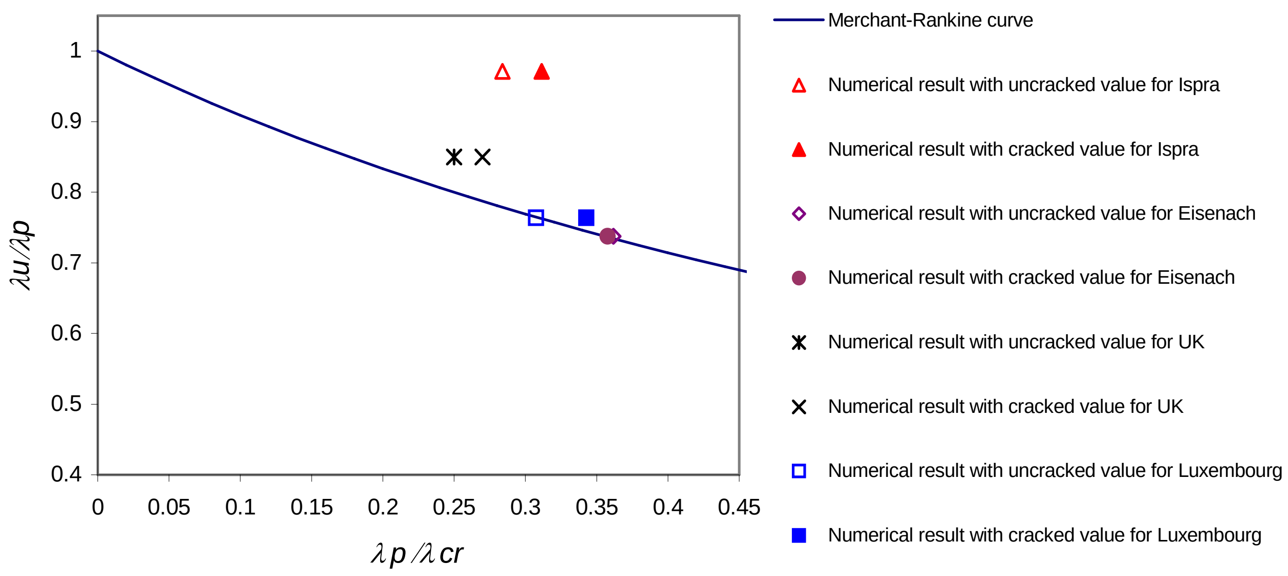


Figure 7-2: comparison between the "MR" curve and the results numerically obtained

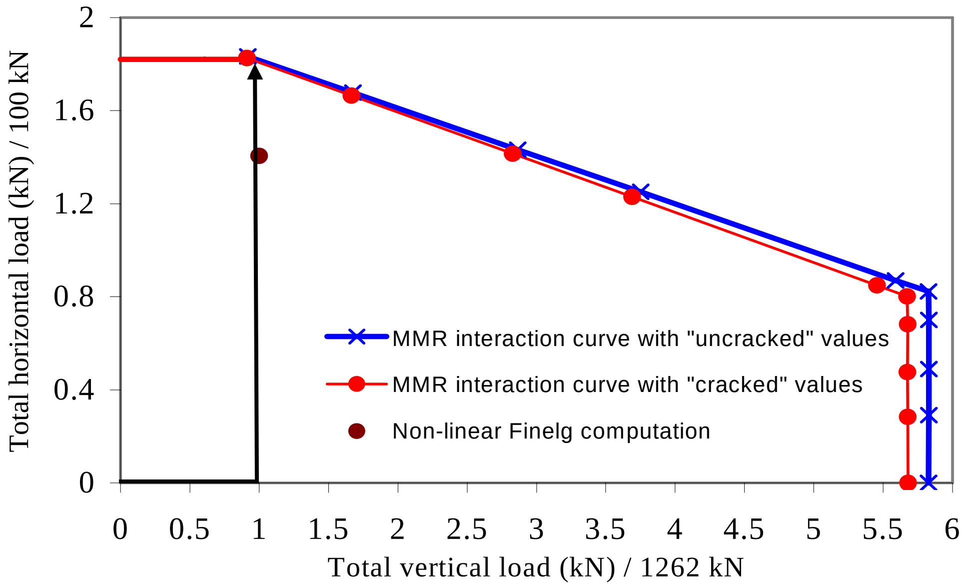


Figure 7-3: comparison between the "MMR" curves computed with $\lambda_{cr,cracked}$ and $\lambda_{cr,uncracked}$

Through these investigations, we have demonstrated that the conclusions which were drawn for steel sway structures *[8]* are still appropriate for the composite ones:

- safe for beam plastic mechanisms;
- adequate for combined plastic mechanisms;
- unsafe for panel plastic mechanisms.

As illustrated in *Figure 7-2*, the use of the "cracked" critical load factor instead of the "uncracked" one in the Merchant-Rankine approach permits for some cases ("Eisenach" and "Luxembourg" buildings) to pass from the unsafe to the safe side, i.e. from the lower to the upper part of the graph with regards to the Merchant-Rankine curve. This observation is in agreement with the results of the parametrical study presented in *Reference [2]*.

Also, we have pointed out through the numerical investigations that the plastic mechanism occurring at failure, obtained through a non-linear analysis, does not always correspond to the one reached through a first-order rigid-plastic analysis, because of the second order effects; so, the nature of the plastic mechanism considered in the Merchant-Rankine does not always correspond to the one occurring at failure. As we explained in *§ 6.3*, this phenomenon can lead to unsafe situations in cases where the load factor corresponding to a panel or a combined mechanism is found close to the smallest one corresponding to a beam plastic mechanism and where the second order effects are influential. A solution would be to develop different formulas, one for each type of first-order rigid-plastic mechanism (i.e. beam, panel and combined mechanism) to analytically predict different ultimate load factors; the ultimate load factor of the studied structure would be the smallest one. This proposition will be detailed in the following paragraph.

7.3 Perspectives

In the present work, we address some points which should be investigated in further studies; these ones should be covered in our future PhD thesis. These points are the following:

- In *§ 5.2.5*, we have described the different assumptions relative to the loading of the studied frames; through these assumptions, we have implicitly assumed that the loading history can be neglected. Indeed, the self-weight and the variable loads have been both proportionally loaded at the same time. Through further investigations, it should be interesting to investigate the influence of the loading history on the global structural responses of these frames by considering that:
  - first, the steel constitutive elements are alone to resist to the self-weight when the concrete is not yet hardened (self-weight of the steel elements + fresh concrete);
  - then, both the steel and concrete constitutive elements are resisting to the self-weight and to the variable loads through a "composite action" when the concrete is hardened.
- As said in *Chapter 5*, we have modelled the steel material without strain hardening and the joints without considering the post-limit behaviour. It should be interesting to introduce these phenomena in the modelling so as to see their influence on the global behavioural response of the studied frames and then, to investigate the applicability of the "Modified Merchant-Rankine approach".
- For the characterisation of composite joints under sagging moment, we have developed a new component ("concrete slab in compression" – see *§ 4.2*). This formula used for the characterisation of this new component need to be validated. This validation could be made through comparisons to test results performed in the field of the ECSC project presented in *§ 1.1*.
- As said in § , the "Bochum" building is a 2-D frame which was tested in Bochum; however, all the test results are not yet available. When the latter will be available, it could be interesting to see if the FINELG predictions are in good agreement with the test results.
- In the "Amplified sway moment method", we have used the critical load factor computed through the FEM FINELG software, which constitutes a precise way to compute this value; through this procedure, a good accuracy was obtained for all the studied frames. It should be of interest to know whether the accuracy of this method is still good if a simplified method is used for the computation of the critical load factor (for instance, the "Grinter frame procedure").

- For the Merchant-Rankine approach, we have shown that the plastic mechanism associated to the plastic load factor introduced in the Merchant-Rankine formula does not always correspond to the one occurring at failure of the frame which can lead to an unsafe prediction of the ultimate load factor. A solution would be to develop three formulas, one for each type of plastic mechanisms which could occur in the studied frame (i.e. beam, panel and combined mechanisms):
    - Formula1($\lambda_{p,beam}$, λ_{cr}) → $\lambda_{u,1}$;
    - Formula2($\lambda_{p,panel}$, λ_{cr}) → $\lambda_{u,2}$;
    - Formula3($\lambda_{p,combined}$, λ_{cr}) → $\lambda_{u,3}$.

  Through these formulas, we would get three predicted ultimate load factors; the smallest one would be the ultimate load factor of the studied frame: $\lambda_u = \min(\lambda_{u,1}, \lambda_{u,2}, \lambda_{u,3})$.

  These new formulas could be developed from the Merchant-Rankine one; in fact, the actual Merchant-Rankine formula could be used as "Formula3" as it was demonstrated herein and in previous studies on steel sway frames *[8]* that this formula gives satisfactory results for frames for which the first-order rigid-plastic mechanism is associated to a combined one. Nevertheless, we think that it would be more practical to develop these formulas from the Ayrton-Perry formulation (see *Table 7-2*), which is already used in the Eurocodes to treat the member instability phenomena (plane buckling, lateral buckling and lateral torsional buckling) and which would be in agreement with the recommendation of the last draft of Eurocode 3 *[32]*; indeed, it is stated in the latter 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 [32])*".

  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 apparition of a plastic mechanism (λ_u → λ_p);
    - when λ_p is very high, no yielding appears in the frame and the failure occurs through an instability phenomenon.

Table 7-2: from the Ayrton-perry formulation to the new simplified analytical design method

Ayrton-Perry formulation – new draft of Eurocode 3 *[32]*	New simplified design method for sway frames
$N_{b,Rd} = \dfrac{\chi \alpha_{ult,op}}{\gamma_{M1}}$ $\chi = \dfrac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{op}^{\,2}}}$ $\bar{\lambda}_{op} = \sqrt{\dfrac{\alpha_{ult,k}}{\alpha_{cr,op}}}$ $\phi = 0.5\left[1 + \alpha(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2\right]$	$\lambda_u = \chi \lambda_p$ $\chi = \dfrac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{op}^{\,2}}}$ $\bar{\lambda}_{op} = \sqrt{\dfrac{\lambda_p}{\lambda_{cr}}}$ $\phi = 0.5\left[1 + \mu(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2\right]$

Through this new method, only the parameter μ should be calibrated through parametrical studies; the parameter $\bar{\lambda}_0$, which represents the length of the plateau in a $\bar{\lambda}_0$ - χ graph, should be taken as equal to 0 when no strain hardening effects are considered and to $\sqrt{0.1}$ when the strain hardening effects are taken into account so as to have the same plateau than for the Merchant-Rankine approach.

In fact, it is not necessary, for each type of mechanism, to perform the parametrical studies on composite sway frames, which need big computation efforts, as we demonstrated in the present work that such frames present the same behavioural response than steel ones; so, the parametrical studies could be performed on steel sway structures so as to develop these two formulas, what is easier from a computational point of view, and to verify later on if these formulas show a good accuracy for composite sway frames.

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Annexes

A.1 Presentation of the main excel sheets of the software

A.1.1 Beam-to-column composite joint software

A.1.1.1 Data sheet

Beam-to-column composite joints with extended end-plates

ref: Design of Composite Joints for Buildings, CECM 109

External Joints - First Storey

Data

Geometrical characteristics

Joint		**Beam**		**Column**	
hp [mm]	385	Rolled ?	Yes	Rolled ?	Yes
aw [mm]	4	Name	IPE 300	Name	HE 200 B
af [mm]	6	hb [mm]	300	hc [mm]	200
tp [mm]	12	db [mm]	248.6	bc [mm]	200
bp [mm]	180	twb [mm]	7.1	twc [mm]	9
ep1 [mm]	50	tfb [mm]	10.70	tfc [mm]	15
ep2 [mm]	50	A [cm²]	53.81	rc [mm]	18
ep3 [mm]	40	Ia [cm4]	8356	Ac [mm²]	7808
pp [mm]	195	Wpby [cm³]	628.4	Encased?	No
p [mm]	50	**Slab**		**Stud**	
w [mm]	100	hcs [mm]	95	dsc [mm]	19
Bolts		hps [mm]	55	hsc [mm]	101
Grade	10.9	acs [mm]	20	Spacing [mm]	100
d [mm]	M20	As.l [mm²]	113.1	Stiffness [kN/ı	100
Asb [mm²]	245	As.r [mm²]	113.1	**Unbraced?**	Yes
hh [mm]	10	beff.b [mm]	320	span [m]	5
hn [mm]	50				
dw [mm]	20				

Mechanical characteristics

Strength [N/mm²]		**Modulus [N/mm²]**		**Partial Safety factors**	
Steel grade b.	235	Esb	210000	γM0	1
Steel grade c.	235	Esc	210000	γS	1
fywb	235	Esp	210000	γC	1
fyfb	235	Es	210000	γMw	1
fywc	235	Ecs	30500	γMb	1
fyfc	235	Ecc	30500		
fyp	235				
fuw	235				
fub	1000				
fsk	500				
fcks	25				
fckc	25				

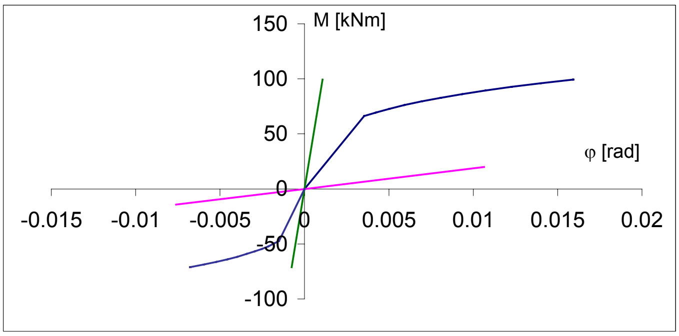
Loads

β	single sided configuration	
σ com,a,Ed [N/mm²]		54.6
σ com,c,Ed [N/mm²]		0
σ com,fc,Ed [N/mm²]		64.5
Nsd [kN]		330

A.1.1.2 Result sheet

Mechanical Properties of the Joint

Resistance			
Hogging moment		**Sagging moment**	
MRd [kNm]	99.4	MRd [kNm]	71.0
MelRd [kNm]	66.3	MelRd [kNm]	47.4
Vrd [kN]	313.6	Vrd [kN]	313.6
Stiffness			
Hogging moment		**Sagging moment**	
Sj,ini [kNm]	18736	Sj,ini [kNm]	31456
Sj [kNm]	9368	Sj [kNm]	15728
Failure			
Hogging moment		**Sagging moment**	
Column web panel in shear		End plate in bending	



Ductility no information

Remark **None**

Component results

Hogging moment	FRd [kN]	k [mm]	Sagging moment	FRd [kN]	k [mm]
Column web in compression	341.393	10.187	Column web panel in shear	303.20	2.42
Column web in tension	312.838	8.787	Column web in compression	1446.88	8.40
Column flange in bending	315.613	17.825	Concrete slab in compression	269.14	7.57109401
End plate in bending	234.635	5.251	Bolts in tension	882	13.754386
Beam flange in compression	510.453	infini	Column web in tension	391.21	13.4885821
Beam web in tension	439.515	infini	Column flange in bending	551.96	27.36
Bolts in tension	441	6.877	End plate in bending	178.84	4.87
longitudinal slab reinforcement in tension	113.1	0.25160243			
Column web panel in shear	303.198	3.404			

A.1.2 Composite beam software

Beam profile	
Type	IPE 220
hb [mm]	220
bb [mm]	110
twb [mm]	5.9
tfb [mm]	9.2
rb [mm]	12
Ab [cm²]	33.37
Iyb [cm^4]	2772
Wpl,yb [cm³]	285.4
Izb [cm^4]	204.9
Wpl,zb [cm³]	58.11
d [mm]	177.6

Slab properties			
General		*Longitudinal rebars*	
beff,- [mm]	320	Layer 1	
beff,+ [mm]	875	As1 [mm²/m]	188.5
h [mm]	340	Cover [mm]	20
hc [mm]	65	Layer 2 (hogging zone)	
hp [mm]	55	As2 [mm²/m]	706.86
Hollow rib properties		*Transversal rebars*	
Ribs	Perp. To the profile	Layer 1	
Ip [mm^4]	816300	As3 [mm²/m]	188.5
b0 [mm]	150	Cover [mm]	20
yg [mm]	24.8	Layer 2 (slab hogging zone)	
Ahr [mm²/m]	1666	As4 [mm²/m]	0
Studs			
Calculations?	YES		
ds [mm]			
hs [mm]			
fu,s [mm]			
Nr			

Mechanical properties	
Profile steel	S355
Hollow rib	S320
Concrete	C25/30
Rebars	S500
Security coefficient	
Type	Eurocode
γs	1.1
γc	1.5
γr	1.15
γhr	1.1
γstud	1.25

Mechanical property details	
Profile steel	
fy,s [Mpa]	355
Es [Mpa]	210000
Hollow rib	
fy,hr [Mpa]	320
Ehr [Mpa]	210000
Concrete	
fy,c [Mpa]	25
τc [Mpa]	0.3
Ec [Mpa]	30500
Rebars	
fy,r [Mpa]	500
Er [Mpa]	200000

Computation Results

Mechanical properties		
Bending moments	Mpl,rd,- [kNm]	116.23
	Mpl,rd,+ [kNm]	188.45
Shear force	Vpl,rd [kN]	295.90
Transversal rebars	Vrd,- [kN]	634.67
	Vrd,+ [kN]	634.67
Stud property	Prd [kN]	#DIV/0!

Geometrical properties		
Hogging zone	Aeq [mm²]	3623.5
	yg [mm]	126.6049233
	Ie [mm^4]	39356232.11
	Class section	1
Sagging zone	Aeq [mm²]	7467.208
	yg [mm]	219.2397733
	Ie [mm^4]	101169469.4
	Class section	1

A.1.3 Partially encased composite column software

DATA

Column profile		General geometry	
Type	HE 260 B	*Major axis*	
hc [mm]	260	Lfl [mm]	5000
bc [mm]	260	*Minor axis*	
twc [mm]	10	Lfl [mm]	5000
tfc [mm]	17.5	**Reinforcements**	
rc [mm]	24	As,tot [mm²]	314.2
Ac [cm²]	118.4	h_major axis [mm]	140
Iyc [cm^4]	14920	b_minor axis [mm]	150
Wpl,yc [cm³]	1283	**Long term loading?**	
Izc [cm^4]	5135	YES	
Wpl,zc [cm³]	602.2	Ng,sd [kN]	665
d [mm]	177	Nsd [kN]	995

Mechanical properties		Mechanical property details	
Profile steel	S355	*Profile steel*	
Concrete	C25/30	fy,s [Mpa]	355
Rebars	S500	Es [Mpa]	210000
Security coefficient		*Concrete*	
Type	Eurocode	fy,c [Mpa]	25
γs	1.1	τc [Mpa]	0.3
γc	1.5	Ec [Mpa]	30500
γr	1.15	*Rebars*	
γstud	1.25	fy,r [Mpa]	500
		Er [Mpa]	200000

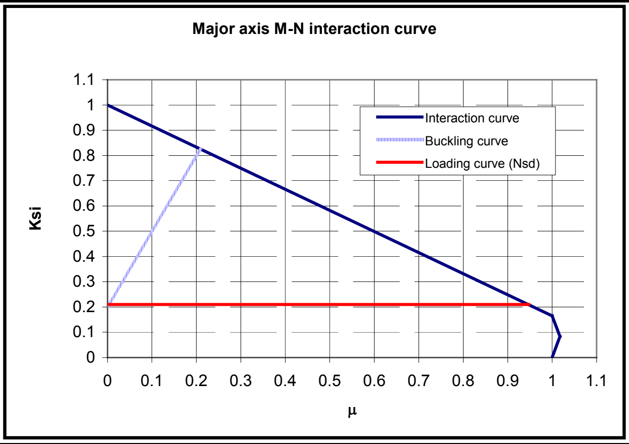
Second order effects?

YES

Major axis		*Minor axis*	
M_extremity 1 [kNm]	0	M_extremity 1 [kNm]	0
M_extremity 2 [kNm]	90	M_extremity 2 [kNm]	45

RESULTS

Major axis				Minor axis			
Geometrical properties		*Resistance properties*		*Geometrical properties*		*Resistance properties*	
g [kg/m]	234.02	Npl,Rd [kN]	4743.16	g [kg/m]	234.02	Npl,Rd [kN]	4743.16
Ae [cm²]	161.81	Mpl,Rd [kNm]	437.79	Ae [cm²]	161.81	Mpl,Rd [kNm]	230.82
Ie [mm^4]	170788682	Npl,Rd,buckling [kN]	3908.82	Ie [mm^4]	67027468	Npl,Rd,buckling [kN]	2567.14



Interaction curve			*Interaction curve*		
Point	M [kNm]	N [kN]	Point	M [kNm]	N [kN]
A	0	4743.2	A	0	4743.2
B	437.8	0	B	230.8	0
C	437.8	785.5	C	230.8	785.5
D	445.5	392.7	D	231.3	392.7
Mrd associated to Nsd [kN]			*Mrd associated to Nsd [kN]*		
Nsd [kN]	995		Nsd [kN]	995	
Mrd [kNm]	414.05		Mrd [kNm]	195.32	

A.2 Detailed data and results for the benchmark study

A.2.1 Constitutive laws for the materials and moment-rotation curves for the joints

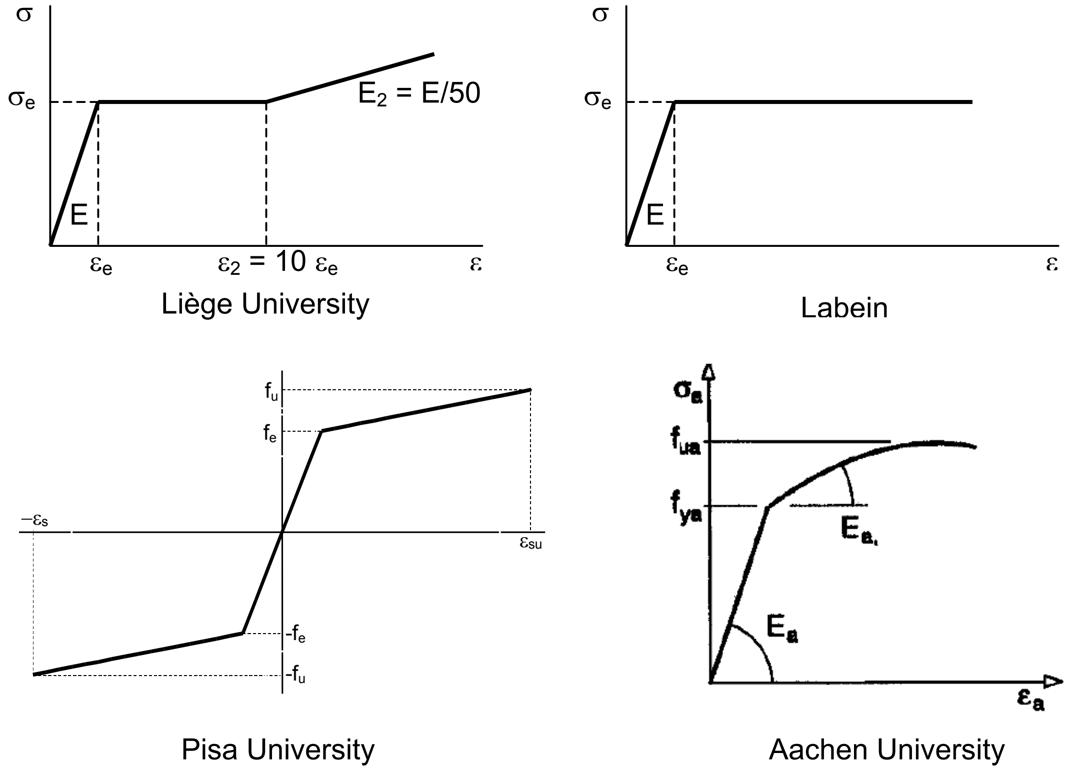


Figure A.2-1: constitutive laws for steel

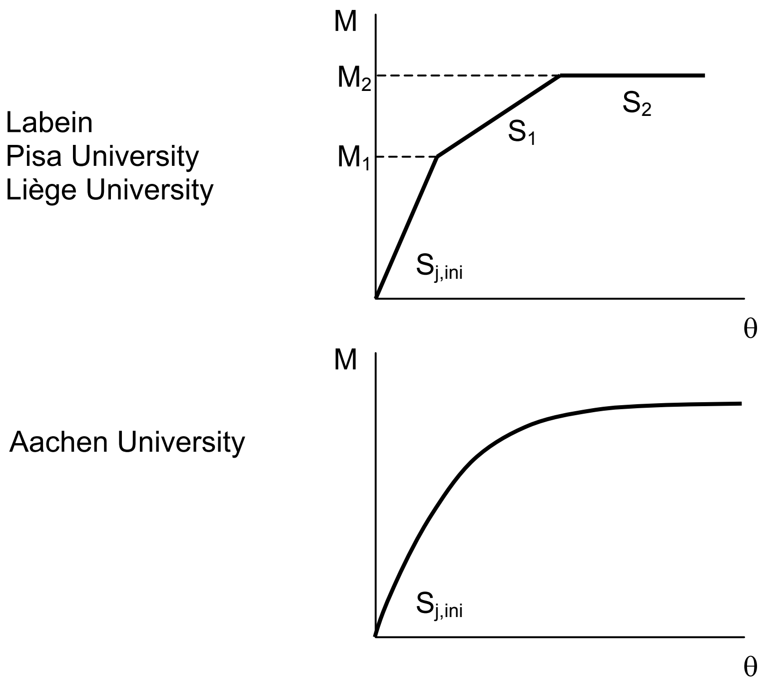
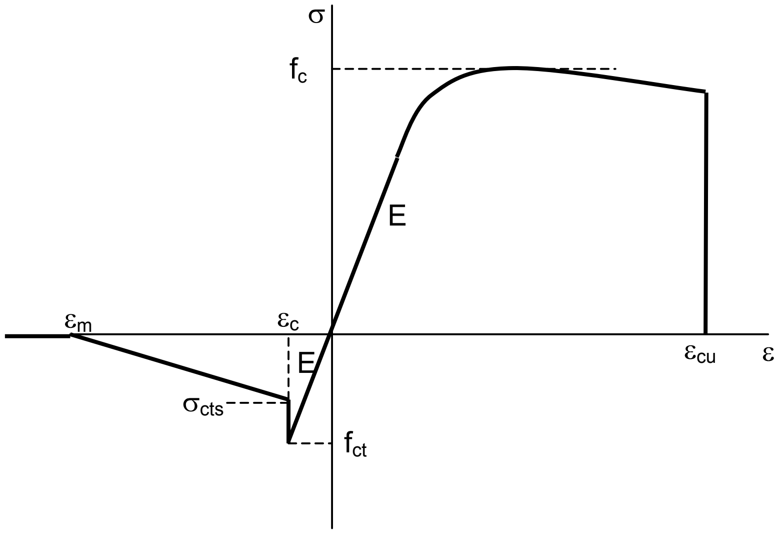
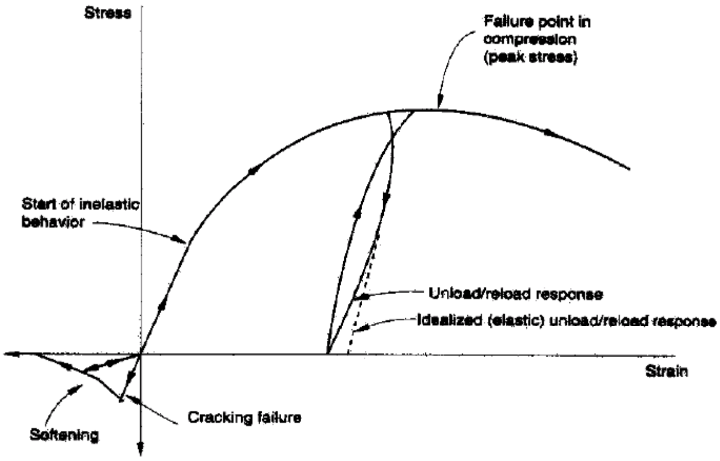


Figure A.2-2: moment-rotation curves for joints

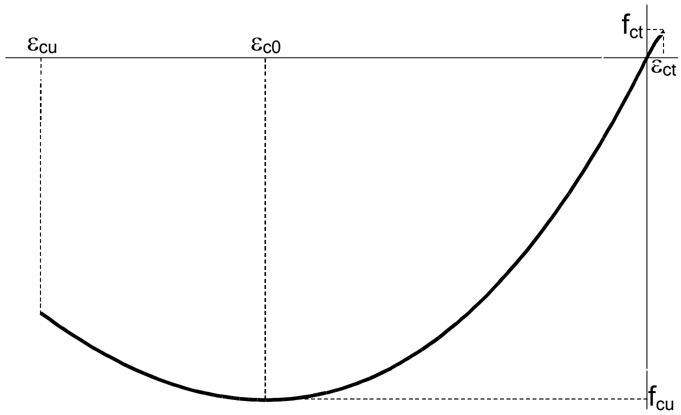
Liège University



Labein



Pisa University



Aachen University

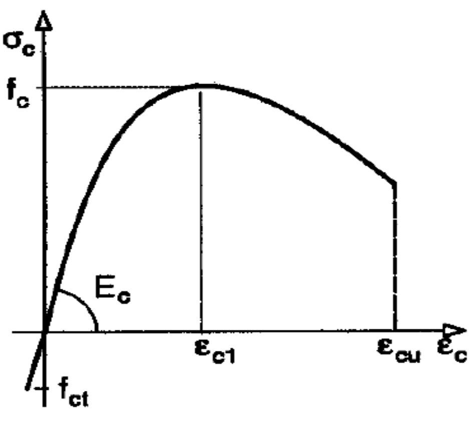


Figure A.2-3: parabolic laws for concrete

A.2.2 Structural response of the primary beams for Frame A and Frame B

A.2.2.1 Frame A

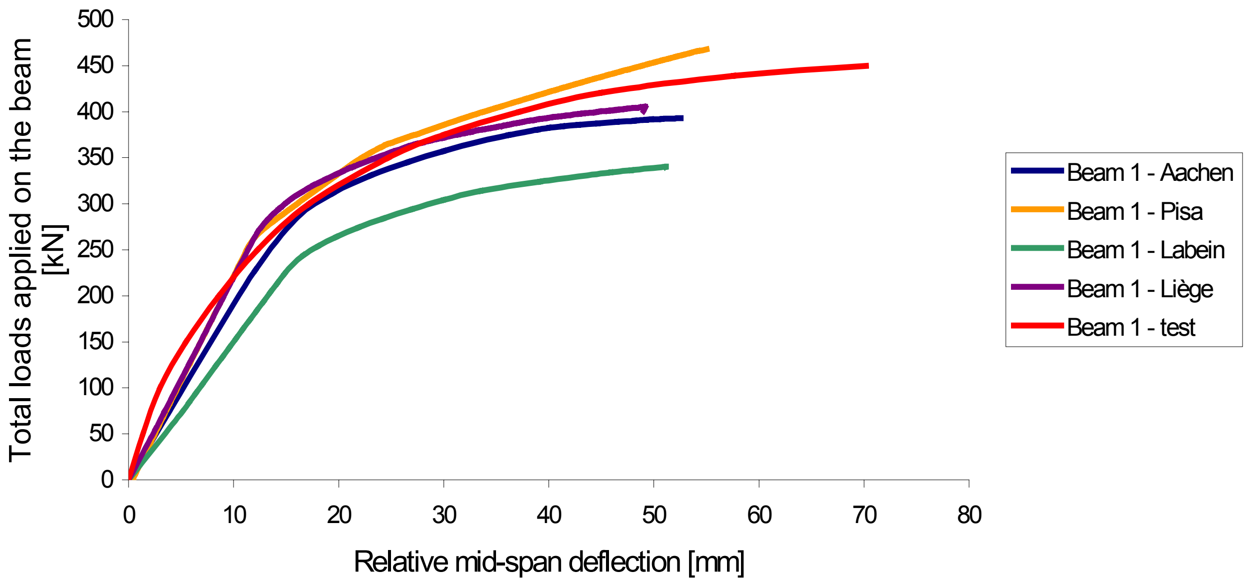


Figure A.2-4: relative mid-span deflection – beam 1

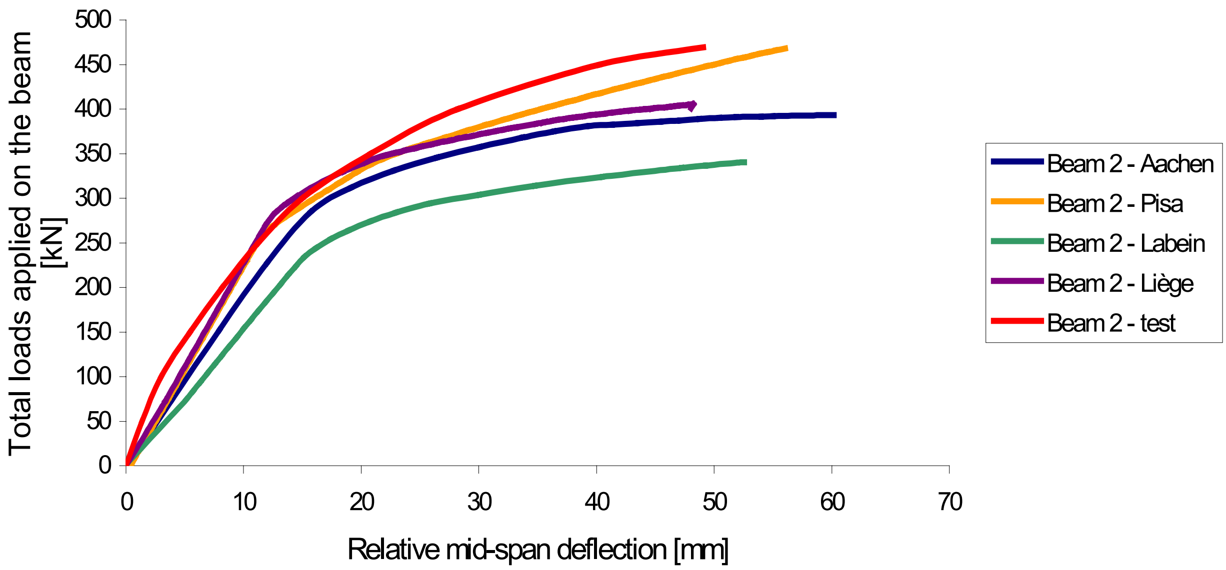


Figure A.2-5: relative mid-span deflection – beam 2

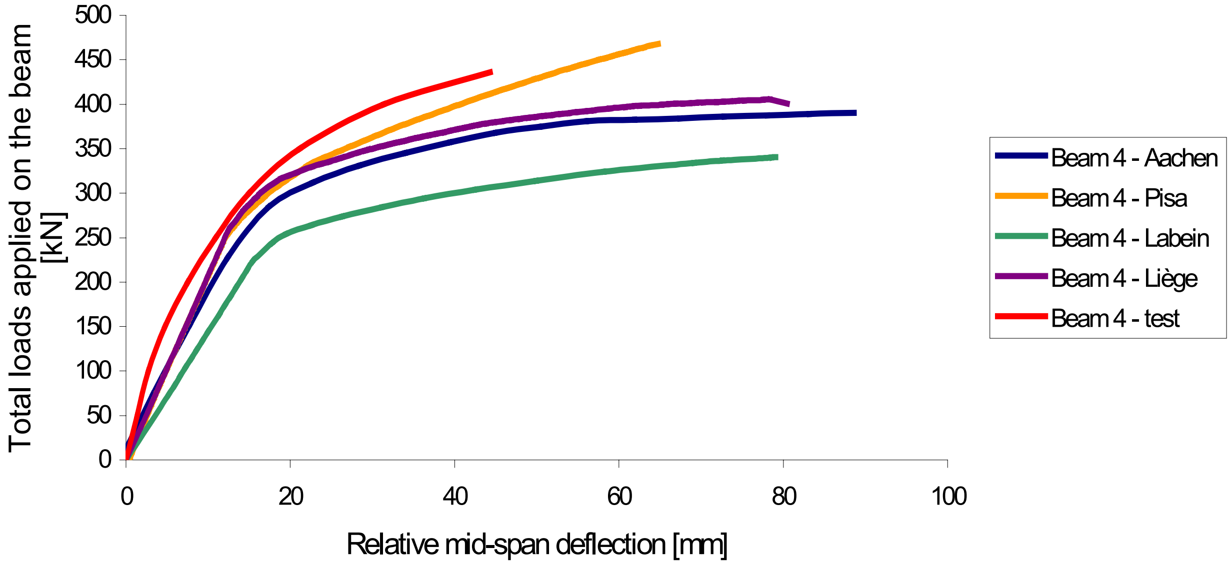


Figure A.2-6: relative mid-span deflection – beam 4

A.2.2.2 Frame B

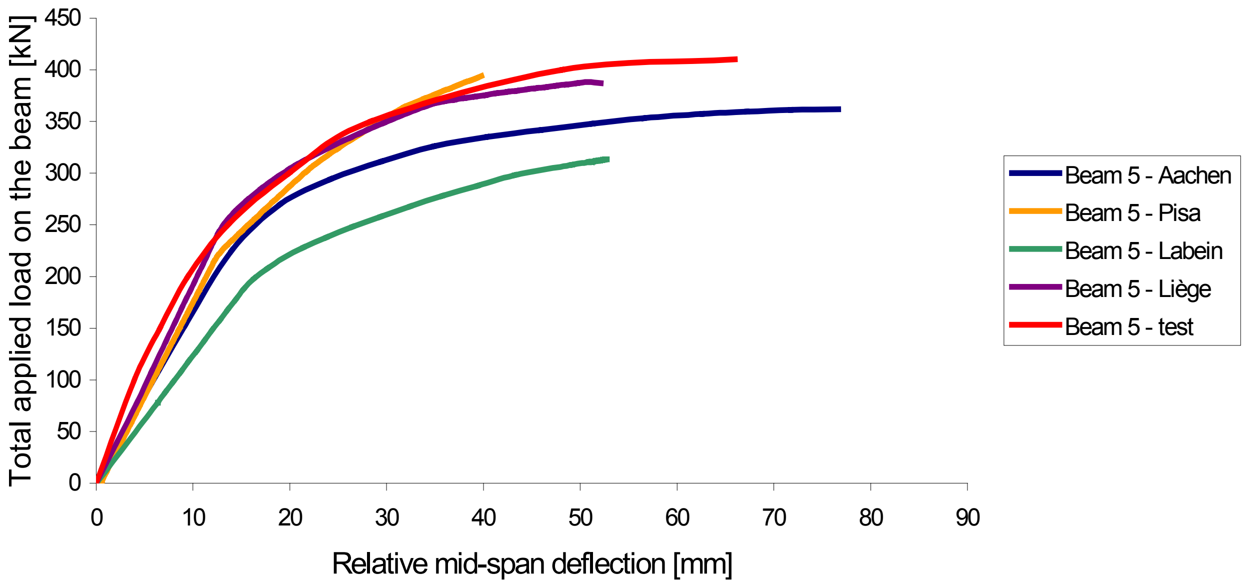


Figure A.2-7 relative mid-span deflection – beam 5

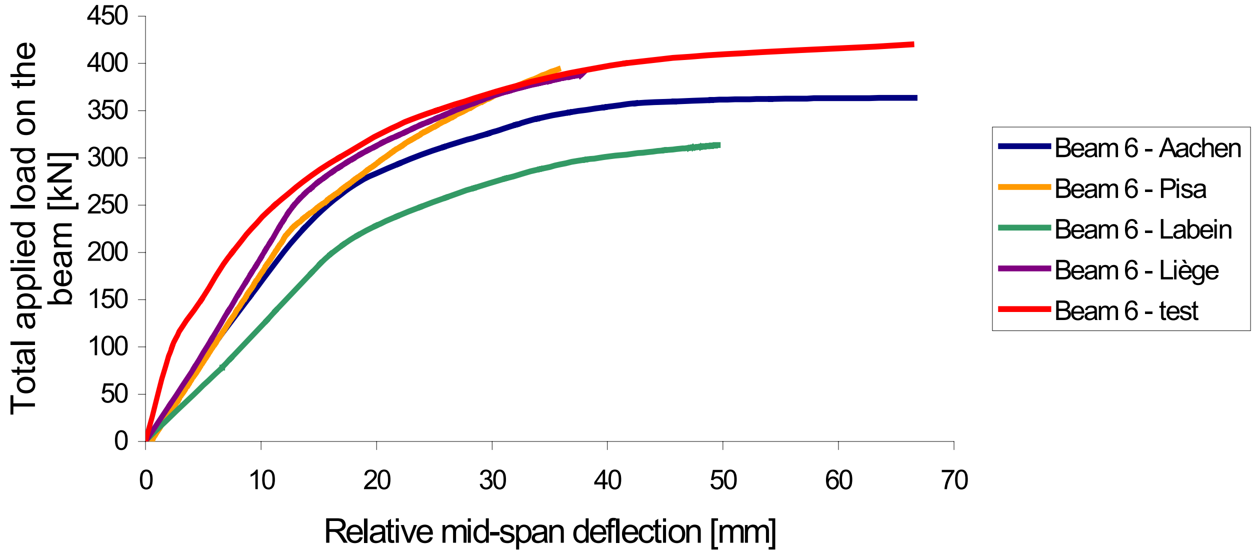


Figure A.2-8: relative mid-span deflection – beam 6

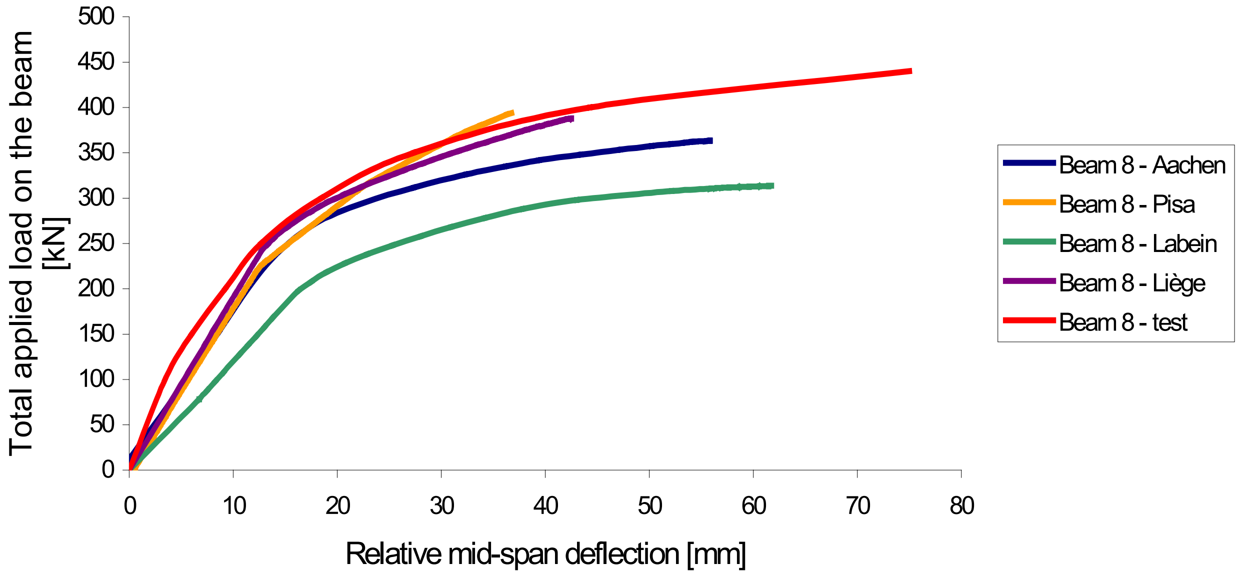


Figure A.2-9: relative mid-span deflection – beam 8