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EVALUATION OF THE LOAD BEARING CAPACITY OF THIN-WALLED STEEL MEMBERS FOR LONG SPAN COMPOSITE FLOORS

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Abstract: *This paper presents an efficient semi-analytical method based on both Eurocode 3 Part 1-3 and a finite strip program and aimed at evaluating the strength of a thin-walled profile with deep waves used for composite floors. Finite strip methods allow to determine the critical buckling stress corresponding to local, distortional or lateral-torsional buckling of a given thin-walled member. Through this approach, the complex iterative analytical procedure of the Eurocode 3 Part 1-3 is bypassed. By means of the Winter's formula, a so-called "stress reduction factor" is then evaluated, as a function of the cross-section slenderness; this one is finally used to derive the ultimate bending resistance of the cross-section.*

1 INTRODUCTION

In today's steel construction, the use of thin-walled cold formed steel members has become more and more frequent. For example, in the residential or car park market, cold formed profiles are used as formwork for fresh concrete in composite floors. However, the strength of these thin-walled steel members remains difficult to evaluate because of complex interactions between phenomena such as local and distortional instabilities.

Some design codes, such as Eurocode 3 Part 1-3 [1], and design recommendations [3], allow the engineer to determine the strength of thin-walled members for cross-sections, the dimensions of which are within a given range. Deep wave profiles used for floors are outside this range of dimensions. It is therefore necessary to perform tests in order to evaluate these mechanical properties.

According to the analytical method recommended in Eurocode 3 Part 1-3, each wall composing the cross-section is likely to buckle locally. Designers have thus to examine the slenderness of each wall before studying the entire behaviour of the thin-walled cross-section. Effective widths are defined for each wall, so leading to a new resistant cross-section in which stresses have to be redistributed in agreement with the new geometry. It is thus necessary to perform an iterative analytical calculation.

Another point is that cold-formed profiles used for floors aim at reaching long spans, in order to reduce the global cost of the structure. Researchers have thus to examine the effect of a range of parameters (such as the thickness of the walls or the dimensions of the cross-section) on the strength in the perspective of increasing the performance of the member. Numerical models are of particular importance to achieve this goal. Nevertheless, such models are rather difficult to implement and are rather .ime consuming, in particular in a pre-design stage.

This paper presents the work done on a particular type of cold formed profile with deep waves in order to estimate its structural strength in bending. First, finite element simulations made with ABAQUS [4], a finite element program developed by Hibbit, Karlsson and Sorensen, and tests performed in laboratory are described. Secondly, numerical and test results are compared to a semi-analytical method developed for the quick and easy evaluation of the ultimate bending moment resistance.

2 CONSIDERED COLD FORMED THIN-WALLED STEEL PROFILE

Cold formed profiles used for floors are shaped into deep waves and large upper flanges, the goal being to minimise the consumption of the concrete and to increase the strength of the composite floor [6].

The cross-section of the profile considered in this paper is schematically depicted in Figure 1. The member is made of traditional flat carbon steel (thickness: 1mm; f_y=320MPa). The upper flange of the profile is stiffened both longitudinally and transversally during the cold rolling process. Lower flanges of adjacent profiles are connected before the floor concreting.

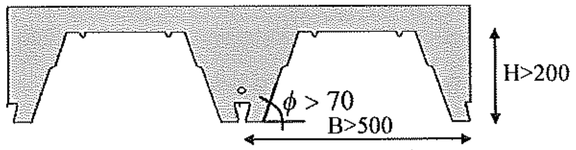

Figure 1: Cold formed steel profile used for floors.

It is important to notice that the dimensions of the profile are just outside the range in which recommendations of Eurocode 3 Part 1-3 are valid. Consequently tests have to be performed on such profiles to derive their strength. Moreover, the effect of transverse stiffeners on the behaviour of the profile cannot be taken into account through the formula given in the European standards.

3 FULL-SCALE TESTS [7][8]

3.1 Introduction

As explained before, Eurocode 3 proposes to evaluate the ultimate bending resistance of such a cold formed thin-walled steel profile by means of tests as those presented in this section.

The test specimens have been fabricated by bending in a press and their length is up to 6m. In the upper part of the actual profile, transverse folds are realised so as to increase the load bearing capacity in the transverse direction, during the concreting of the composite floor. But it is impossible to obtain these folds by press bending. Therefore, their have been substituted in the test specimens by transverse cuts in the upper flange, such that the behaviour of the profile during the tests is similar to the actual one. Figure 2 indicates how the actual transverse folds have been replaced through cuts.

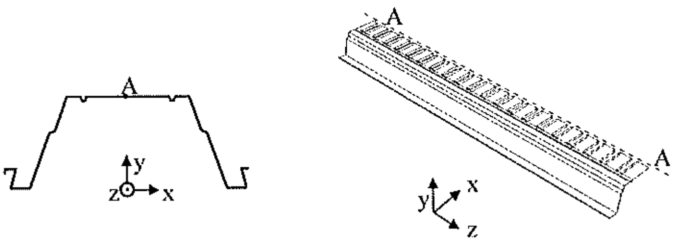

Figure 2: Transverse folds in the cold formed profile.

In the reality, the profile is subjected to an uniformly distributed load (fresh concrete). To approach it, the test specimens are submitted to bending by applying an equal transverse force P at 4 different ocations along the member, as depicted at Figure 3. The forces are applied through two jacks. If the distance between each force P is the span divided by four, the central part of the beam undergoes a constant bending moment given by PL/2.

In fact, the two jacks apply equal forces at 8 points (4 points by flange) located along the lower flanges of the profile through an appropriate load distribution system. So as to resist to the locally applied forces, load distribution plates are used to reinforce the lower flanges of the cross-section as shown at Figure 4.

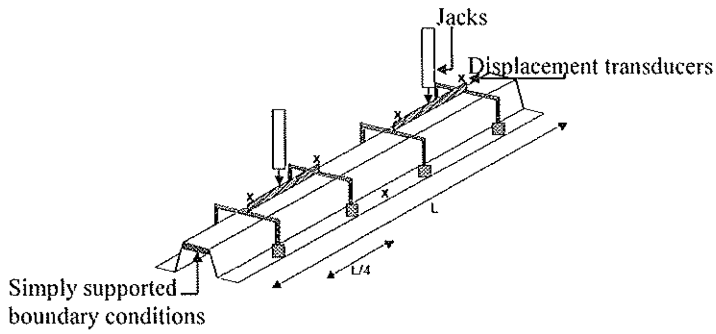

Figure 3: Test set-up.

At the end of the beam, the profile is supported by a system allowing longitudinal displacements and rotations. To avoid the failure of the webs under transverse end reactions, support is provided to the upper flange (see Figure 3).

In composite floors, steel profiles are connected together to form a surface designed to support the fresh concrete. Because of this continuity, transverse displacements and longitudinal rotations of the lower flanges are not permitted. To simulate this effect, Eurocode 3 recommends the use of local transverse links fixed to the lower flanges of the profile (see Figure 4).

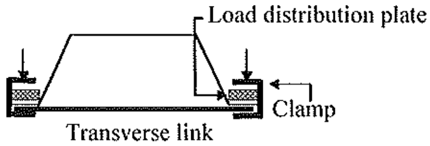

Figure 4: Loading device for the tests: transverse links and load distribution plates.

During the tests, the load and the member deflections have been measured in various places along the member (See small crosses in Figure 3).

3.2 Results

3.2.1 Deformations

Deformations (Figure 5) consist in local buckling waves in the upper flange of the profile at mid-span and in an overall bending deformation of the member. Finally, web buckling appears and rapidly causes the failure of the profile.

3.2.2 Load bearing capacity

Three tests have been carried out on fully similar profiles. This is obviously not sufficient to compute mean and standard deviation values rigorously; nevertheless few discrepancies between the results are observed and therefore mean values obtained during the tests are reported in Table 1.

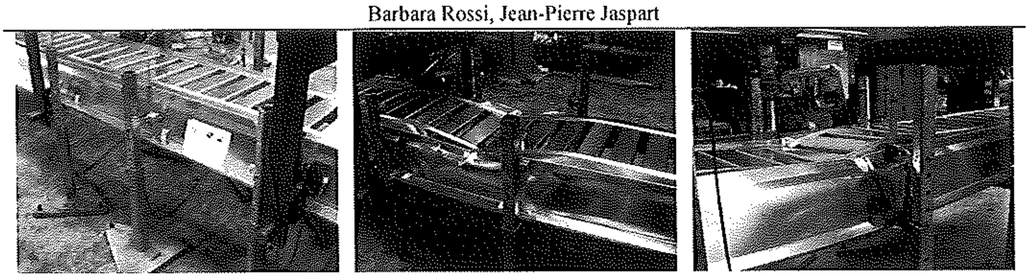

Figure 5: Deformations at failure.

Table 1: Mean experimental values.

Slope P/fmax (kN/mm)	Maximum load (kN)	Maximum bending moment (kN.m)
0,3445	15,344	11,508

The graph representing the applied bending moment versus the deflection at mid-span is given in Figure 6.

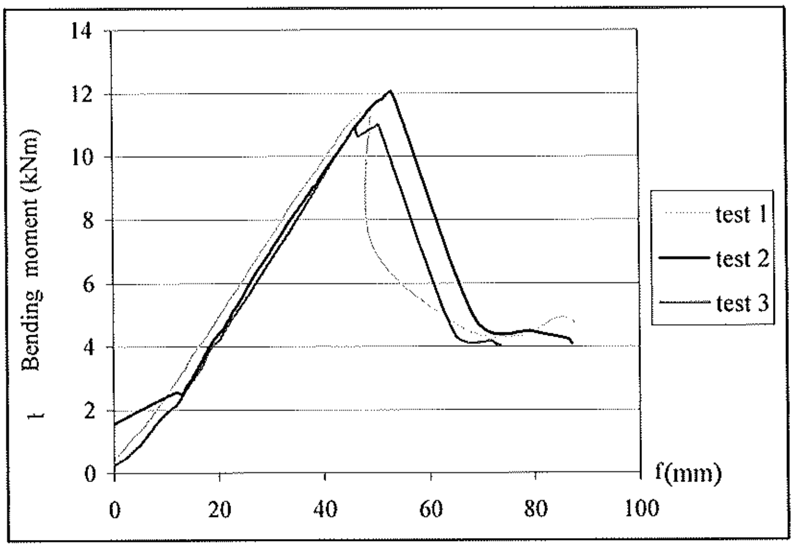

Figure 6: Experimental results: bending moment versus mid-span deflection.

4 FINITE ELEMENT ANALYSIS [8]

ABAQUS [4] is used to conduct this study.

4.1 Model

The geometry of the studied profile is the one of the test specimens (which, as said before, slightly differs from the actual one). For symmetry reasons (profile, loading, geometry, deformation mode), only one quarter of the profile is modelled (see Figure 7).

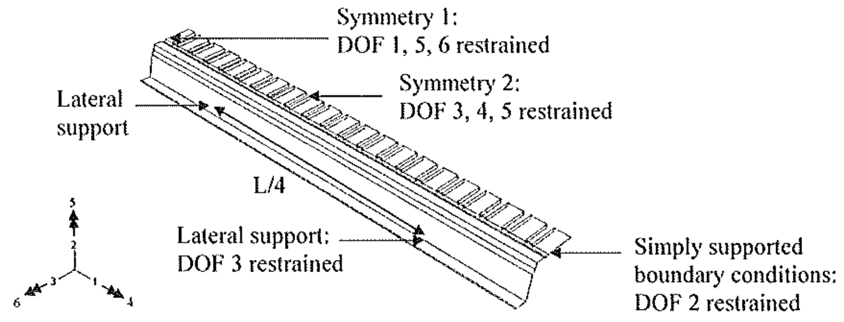

Figure 7: Boundary conditions for FE model.

At its extremities, the profile is simply supported, what means that DOF 2 (Degree of Freedom 2 - see reference axes in Figure 7) is restrained.

Moreover, two conditions of symmetry are imposed:

-At mid-span (symmetry 1): DOF 1, 5, 6 are restrained;

-Along the beam (symmetry 2): DOF 3, 4, 5 are restrained.

Transverse links placed in each loading zone are modelled by keeping DOF3 equal to zero at the level of the lower flange of the profile. Linear quadrilateral "general purpose" shell elements are used. The aspect ratio of the finite elements is kept in the ½-2 range and the mesh has been refined until an acceptable converged solution has been reached.

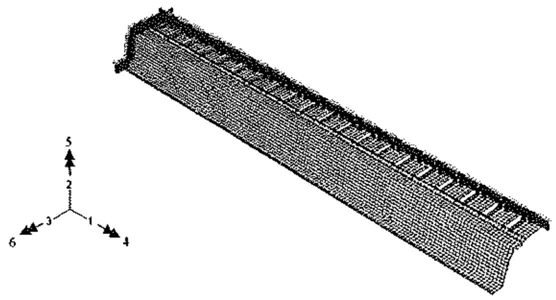

Figure 8: FEM mesh.

Three finite element analyses have been carried out:

- a first order elastic analysis in which all geometrical and material non-linearities are disregarded (Young's modulus and Poisson's coefficient respectively equal to 210000N/mm² and 0.3);
- a first-order elastic-perfectly plastic analysis (yield strength equal to 320N/mm²);
- a second-order elastic-perfectly plastic analysis where both geometrical and material non linearities are accounted for.

For the last one, an initial geometrical imperfection is considered. Its shape is homothetic to the first eigen mode obtained through a preliminary elastic buckling analysis. Its maximum magnitude is controlled to be equal to the width of the upper flange divided by 200 [2].

4.2 Results

In Figure 9, the results of the three analyses are reported in the form of moment-deflection curves. Besides that, on the basis of the analytical pre-evaluation of the properties of the profile (moment of inertia and plastic cross-section resistance), the theoretical elastic and rigid plastic responses of the member in bending has be obtained; these two values are also reported in Figure 9.

First of all, a significant difference is seen between the theoretical and numerical values of the elastic stiffness; this discrepancy results from the fact that transverse cuts in the upper flange are not considered in the theoretical stiffness evaluation.

The same conclusion applies as far as the first-order plastic analysis is concerned. The hand-calculated plastic resistance of the cross-section (Figure 9: hand calculations 1) disregards the presence of the cuts and is again seen to be much higher than the numerically-evaluated one. If the cuts are taken into account, the hand-calculated plastic resistance (Figure 9: hand calculations 2) is much closer than the numerically-evaluated one.

In the third type of analysis, second-order effects are taken into consideration and therefore, during the loading, buckling waves progressively develop in the upper flange and in the webs. The effect of these local instabilities is known to be strongly dependent on the level of the initial imperfections. In Figure 9, it may be seen that the second-order effects lead rapidly to a significant decrease of the initial elastic stiffness (obtained through the first analysis). The failure occurs by flange and web buckling in the plastic range, as in the experimental tests, which are also reported in Figure 9.

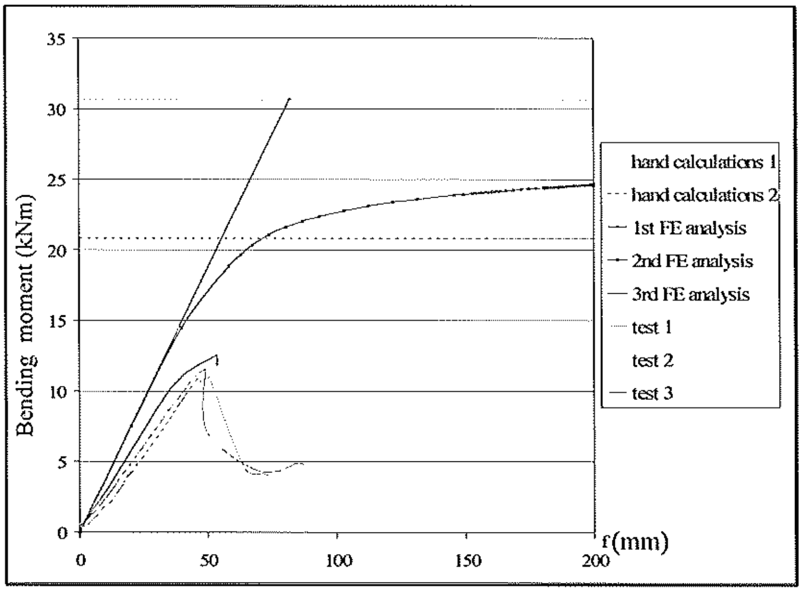

Figure 9: Comparison between tests and various design approaches.

Finally, from Figure 9, a quite reasonable agreement between the tests and the finite element analysis may be seen. Nevertheless, the achievement of such non linear finite element studies including complex material and geometrical behavioural features requires a lot of time. Moreover, if the researcher has to examine the effect of a range of parameters such as the cross-section dimensions, such models are rather difficult to adapt. That is why an analytical design procedure is suggested in the next section.

5 SEMI-ANALYTICAL METHOD [8]

The finite strip program developed by Schafer and called CUFSM [5] may be used to determine the elastic buckling resistance and the corresponding eigen modes for any simply supported thin-walled member subjected to arbitrary stress conditions at its ends. As soon as the cross-section is defined, the program examines a range of different lengths for the member and records the stress at which buckling occurs and the related deformation mode. The use of this program ensures a saving of time in comparison with the iterative method proposed in the normative documents and design recommendations.

The semi-analytical method suggested here is twofold:

- a preliminary evaluation of the elastic buckling stress through the use of this finite strip program;
- the derivation of an ultimate stress by means of the Winter's formula; this one allows to define a so-called "stress reduction factor" as a function of the cross-section slenderness.

The slenderness of the cross-section is given by the following formula (1).

$$\lambda = \sqrt{\frac{f_y}{\sigma_{cr}}} \tag{1}$$

where fy is the yield stress and σcr is the critical elastic buckling stress evaluated with CUFSM. In the profile, the longitudinal rigidity is increased by the presence of transverse stiffeners (as depicted in Figure 1). In CUFSM, it is impossible to model these transverse stiffeners. In order to simulate their effects for the evaluation of σcr in CUFSM, the Young's modulus Ez of the upper flange of the profile is increased such that the transverse moment of inertia is equal to the real one with the transverse stiffeners. σcr is equal to 142N/mm².

If λ is lower than 0.673, Eurocode stipulates that the ultimate bending moment is equal to the elastic bending moment. But if λ is larger than (or equal to) 0.673, the ratio ρ between these two bending moments is given by the formula (2).

$$\rho = \frac{1-\frac{0.22}{\lambda}}{\lambda} \tag{2}$$

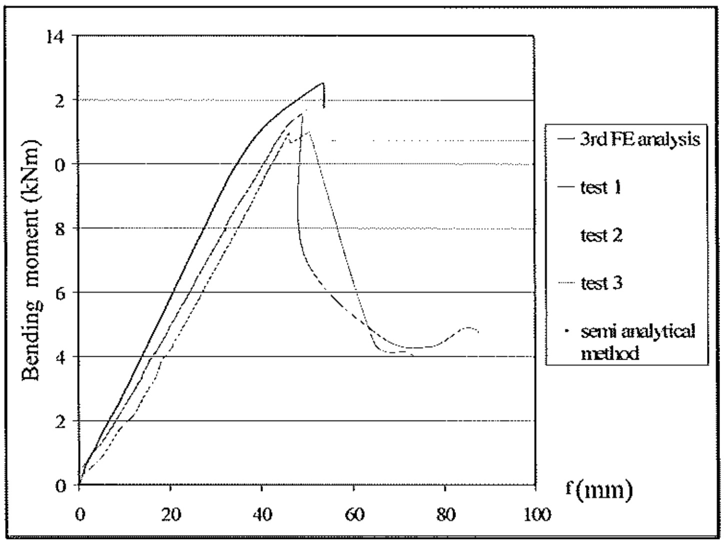

Figure 10: Comparison between FEM analysis and semi-analytical method.

This semi-analytical method applied here on thin-walled members used in long span composite floors is in good agreement with both the full-scale tests and the finite element analysis already performed.

In fact, the maximum bending moment obtained with that method (10.8kNm) is seen in Figure 10 to be quite close to one got from the tests (11.5kNm).

6 CONCLUSIONS

The analytical method proposed in Eurocode 3 Part 1.3 is not adequate at all to evaluate the maximum bending moment resistance of the profiles considered in this paper. First because of the presence of transverse stiffeners in the upper flange of the profiles; and then because of the particular dimensions of the profile cross-section. So, according to Eurocode 3, experimental tests have to be achieved. In the present study, such full scale experimental tests have been performed so as to determine the strength and visualise the deformation mode. But, from an economical point of view, this approach has to be recognised as quite expensive and time consuming:

- for the evaluation of the design properties of existing cross-sections;
- but more especially in the development phase of new cross-sections.

FEM numerical approaches offer, from that point of view, a quite interesting alternative. In the present investigations, finite element simulations have been made and a good agreement with experimental tests has been shown. Anyway, finite element modelling is also time-consuming and requires a certain expertise.

As a complement to the two previously described approaches, a semi-analytical design procedure is therefore suggested in the present paper. It semi-analytical character is justified by the rather extreme complexity of the full analytical procedures usually found in the literature for cold-formed steel members.

In this semi-analytical design procedure, a finite-strip program is first used to determine the elastic buckling strength of the member. Its use is of great interest, for instance, in the frame of parametrical studies, in which the thickness of the profile, the length of the member or even the whole geometry of the cross-section are varied. In a second step, Winter's formula is used to finally derive the ultimate member resistance. This approach allows, in a quite simple way, to tackle quite well the strength of the members, as shown through comparisons with FEM and experimental performed studies.

7 REFERENCES

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