# Experimental and analytical investigations on single-sided composite joint configurations

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ABSTRACT: For three years, in the framework of a European research project funded by the European Community for Steel and Coal (ECSC) in which the Liège University was deeply involved, intensive experimental, numerical and theoretical investigations have been carried out on the behaviour of composite sway buildings under static and seismic loading.
This paper presents experimental and analytical studies carried out at Liège University, as part of the above European project, with the objective to investigate the behaviour of a single-sided composite joint configuration under static loading. A particular failure mode occurred during the experimental tests, which had not yet been detected previously; it is described in details. Then, an analytical formula covering this particular phenomenon and based on a theoretical model is presented; it is validated through comparisons with experimental results.

*Keywords: single-sided composite joint configuration, test results, component method*

## 1 INTRODUCTION

Most 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.

Moment resisting frames offer a flexible solution to the user of the buildings, especially for the internal arrangement and the exploitation of the building. 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 [1] devoted to earthquake engineering in which high values of the behaviour factor are recommended. But these frames are prone to second-order effects; the latter have to be predicted carefully because they may govern the design.

As far as the European codes are concerned, Eurocode 4 [2], which deals with composite construction under static loading, 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). 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. The rotational behaviour of the beam-to-column composite joints is one of the key aspects of the problem to which special attention has been paid.

In this project, two main tests were performed: a 3-D dynamic test on a two-bay two-storey composite building in Ispra, Italy (namely "Ispra" building) [3] and a 2-D static test on a two-bay two-storey composite frame in Bochum, Germany (namely "Bochum" frame) [4]. At Liège

University, four tests on isolated single-sided joints belonging to these two structures have been performed [5]:

- TEST 1 - static test on the single-sided composite joint configuration at the first storey of the “Ispra” building (hogging moment);
- TEST 2 - cyclic test on the single-sided composite joint configuration at the first storey of the “Ispra” building;
- TEST 3 - static test on the single-sided steel joint configuration at the first storey of the “Bochum” frame (hogging moment);
- TEST 4 - static test on the single-sided steel joint configuration at the top of the “Bochum” frame (hogging moment).

In the present paper, only the experimental and analytical investigations performed on the single-sided composite joint configuration of the Ispra building under static loading are presented (TEST 1). The other tests are presented in [5].

First, the tested specimen is described in § 2. Then, § 3 introduces the test results with the description of a particular failure mode that occurred during the tests and that had not yet been detected previously. Finally, the test results are compared in § 4 to those analytically obtained by means of the component method approach amended to cover the new identified failure mode.

## 2 DESCRIPTION OF THE TESTED SPECIMEN

### 2.1 *General layout*

In the “Ispra” building [6], semi-rigid and partial-strength composite joints with sufficient rotational capacity (minimum 0.035 rad) have been selected so as to enable the development of plastic hinges and the dissipation of energy in the joints under seismic loading. To achieve this goal, as many ductile components as possible are activated in the joint at plastic failure (i.e. web panel in shear, reinforcement in tension and end-plate in bending). Figure 1 presents the so-obtained configuration of the single-sided composite joint tested at Liège University (TEST 1). The beam is an IPE300 one, and the column a HEB260 one (partially encased). The slab is 150 mm thick and the hollow rib is an EGB 210 one from BROLLO (Italy), with ribs perpendicular to the beam axis. The composite slab is connected to the upper beam flange by means of shear studs. The layout of the rebars in the slab is given in Figure 2. The mesh is made of rebars with a diameter of 6 mm and a spacing of 150 mm. The column is surrounded by two 12 mm rebars. Additional transversal rebars of 12 mm of diameter are placed close to the column. More details about these specimens are given in [5].

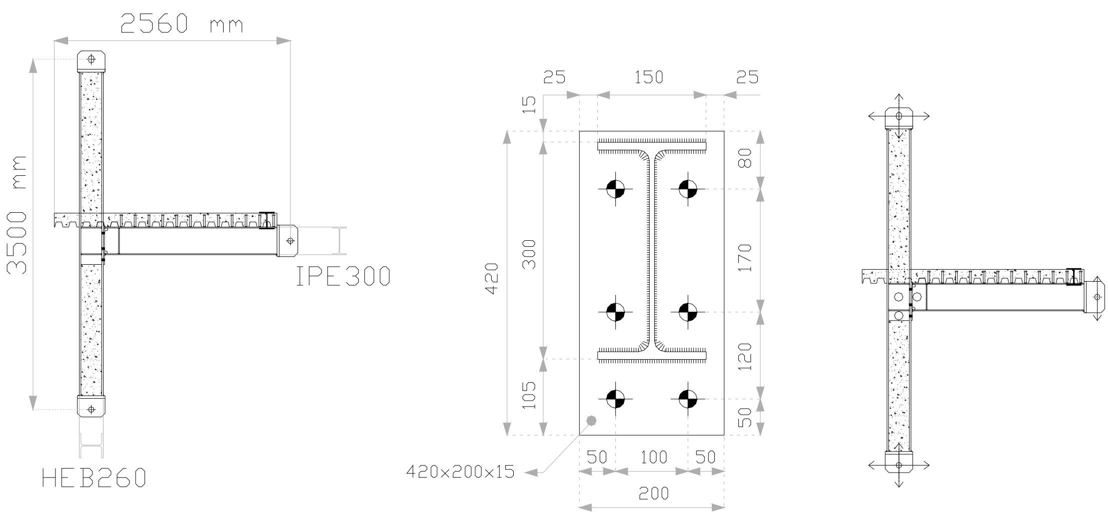


Figure 1. Properties (in mm) and instrumentation of the single-sided composite joint specimen

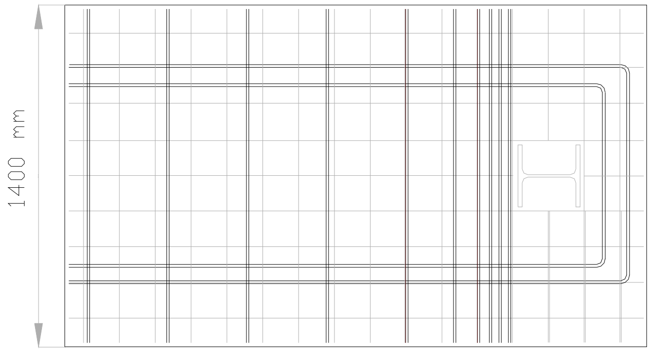


Figure 2. Distribution of the rebars in the slab

## 2.2 *Material properties*

S235 steel grades have been ordered for the beams, the columns the beam end-plates; rebars are made of S500C steel. C25/30 concrete is used for the slabs and the composite columns is C25/30. The bolts are M24 10.9 ones; the are preloaded at 75 % with an additional rotation of 60°.

All the steel elements used to test the Ispra building and the corresponding isolated joints in Liège come from the same producer and from the same production so as to reach a full adequacy between the experimental results.

Compression tests on concrete cubes have been performed on the same day as the experimentation on the joint specimens. Coupons have also been extracted from the different steel elements. All the so-measured mechanical properties for steel and concrete are reported in [5].

## 2.3 *Instrumentation*

Two independent measurement systems have been used to derive the moment-rotation curve of the joints to be tested: inclinometers and extensometers, as shown in Figure 1 (respectively circles and arrows). The way on how moment-rotation curves are obtained in both cases is explained in [5]. The objective of this rather complex instrumentation is to draw two independently measured joint moment-rotation curves, to compare them and, in the case of good agreement, to ensure the accuracy of the reported experimental results.

# 3 TEST RESULTS

Figure 3 presents the M-$\phi$ behavioural curves respectively obtained for joint "TEST 1" by means of the extensometer and inclinometer measurements; in [5], the moment-rotation curve for the connection and the shear-rotation curve for the web panel are available. In accordance with EC3 and EC4, the moment is evaluated at the connection level. Main joint properties are listed in Table 1.

During the test, first cracks in the concrete slab appeared i) transversally, close to connection and ii) longitudinally, just behind the column, as shown in Figure 4. The transversal cracks result from the tension forces in the longitudinal rebars while the longitudinal ones are due to shear forces. Then, at a higher loading step, new cracks developed until a shear failure occurred behind the column (hatched part in Figure 4) for a bending moment of 201,6 kNm and a rotation of 31 mrad (less than the requested one, i.e. 35 mrad). In addition to these cracks, significant yielding developed in the steel joint components : column web panel in shear and end-plate in bending first, beam flange in compression and beam web in tension after later on. Photographs of this test are available in [5].

The shear failure of the concrete slab behind the column was not expected and had therefore not been considered in the design phase (as for TEST 2 (§ 1) under hogging moments). This aspect will be further investigated in § 4.

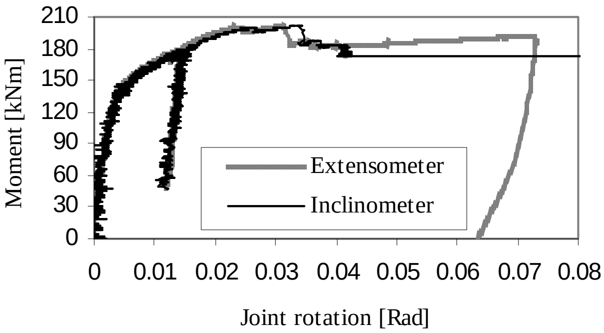


Figure 3. M-$\phi$ joint behaviour curves

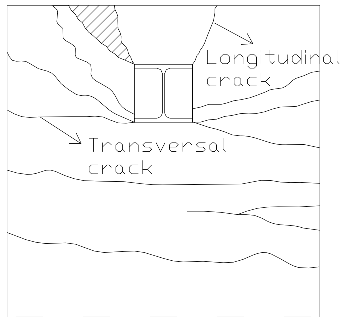


Figure 4. Cracks in the concrete slab

Table 1. Main mechanical properties of the "TEST 1" joint

| | Test results |
|---|---|
| $S_{j,ini}$ [kNm/rad] | 65,000 |
| $M_u$ [kNm] | 201.58 |
| $\phi_u$ [rad] | 0.031 |

## 4 ANALYTICAL INVESTIGATIONS

The present paragraph presents analytical investigations on the TEST 1composite joint configuration. The objective of this paragraph is to validate the applicability of the component method, as proposed in EC3 and EC4, through a comparison of the experimental and analytically predicted M-$\phi$ joint behavioural curves.

The following main values may be derived by means of EC4 regulations for joints: the initial stiffness $S_{j,ini}$, the plastic resistant moment $M_{Rd}$ and the elastic resistant moment $M_e$ (Fig. 5).

In order to draw a full analytical curve, three other values are also computed with the help of a Eurocode-compatible analytical method described in the Ph.D. thesis of Jaspart [7]: the strain hardening stiffness $S_{sh}$, the ultimate resistant moment $M_u$ and the ultimate rotation of the joint $\phi_u$ (Fig. 5).

The failure mode associated to the analytically predicted plastic and ultimate resistant moments involves two components: "rebars in tension" and "end-plate in bending".

The comparison between the test result and the prediction by the component method (CM) is shown in Figure 6. Key properties are reported in Table 2.

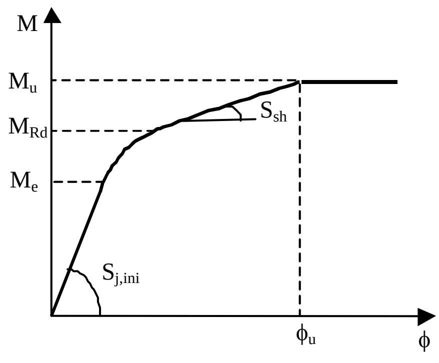


Figure 5. Characteristic values of a moment-rotation curve

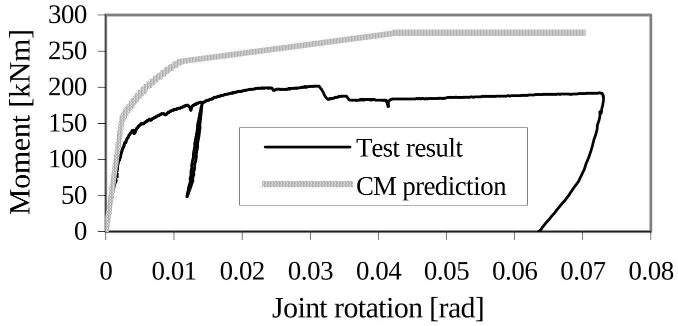

Figure 6. Comparison between test result and component-method (CM) prediction

Table 2. Main values obtained through the test and through the CM prediction

| | Test result | CM prediction |
|---|---|---|
| $S_{j,ini}$ [kNm/rad] | 65,000 | 64,312 |
| $M_u$ [kNm] | 201.58 | 275.4 |
| $\phi_u$ [rad] | 0.031 | 0.043 |

From Figure 6, it may be observed that the initial and the strain hardening stiffnesses are well estimated while the elastic, plastic and ultimate resistant moments and the ultimate rotation are overestimated in the analytical approach. To explain this difference, reference is made to § 3 where experimental observations have been reported. As already stated, first cracks develop in the concrete slab, transversally in the vicinity of the connection and longitudinally just behind the column. The transverse cracks are due to the tension forces acting in the longitudinal rebars, this failure mode is covered by Eurocode 4 ("slab rebars in tension" component). But nothing is said in the normative documents as far as longitudinal shear cracks are concerned. Actually, EC4 prescribes only a minimum section area for the transversal rebars to be placed behind the column (see Formula (6) below) so as to avoid the failure of the concrete slab at this specific place under hogging moments (i.e. to avoid the concrete crushing against the column).

In reality, as soon as the longitudinal cracks in shear develop, the concrete no more contributes to the slab resistance. So the transversal rebars are alone to resist the forces acting along the cracks. These forces can be divided in two parts (Fig. 7):

- tension forces resulting from the "truss behaviour" (phenomenon which is described in details in [8] and [9]);
- shear forces induced by the longitudinal rebars in tension.

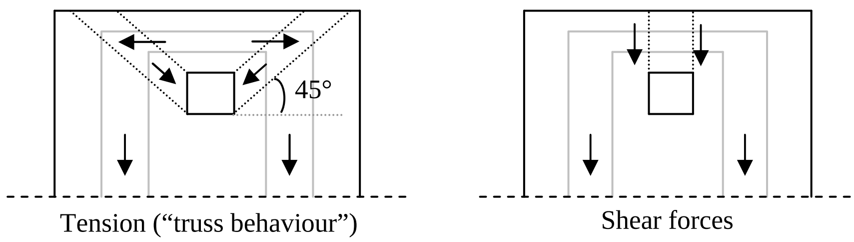

Figure 7. Forces acting along the shear cracks in the slab

Obviously, an interaction between the tension and the shear forces must be considered. This interaction, which may be roughly estimated by means of the Von Mises criterion:

$$(\sigma^2 + 3\,\tau^2) \leq f_y \tag{1}$$

where $\sigma$ = tension stresses induced by the "truss behaviour"; $\tau$ = shear stresses induced by the shear forces; and $f_y$ = yield strength of the rebars.

In the present case, the truss rods are assumed to be inclined by 45° (Fig. 7), which is close to the reality; the tension and shear stresses are then equal ($\sigma = \tau$) and Formula (1) becomes:

$$\sigma \leq f_y / 2 \tag{2}$$

According to Formula (2), half of the resistance is allocated to tension, and half to shear. To take this into account, a modification of the resistance of the rebars is suggested for single-sided joint configurations.

In EC4 (or [10]), the resistance of "slab rebars in tension" component is defined as:

$$F_{Rd,13} = \frac{A_s \, f_{sk}}{\gamma_s} \tag{3}$$

where $A_s$ = total area of the longitudinal slab rebars in tension with a diameter higher than 6 mm; $f_{sk}$ = yield strength of the rebars in tension; and $\gamma_s$ = safety coefficient for the rebars (equal to 1,15 according to EC4).

The formula which is here suggested to include the shear-tension interaction is the following:

$$F_{Rd,13} = \min\left[\frac{A_s \, f_{sk}}{\gamma_s}; \frac{2\, A_{s,2} \, \left(f_{sk}/2\right)}{\gamma_s}\right] \tag{4}$$

where $A_{s,2}$ is the total area of the transverse slab rebars behind the column; the factor "2" in front of $A_{s,2}$ is justified by the presence of two sections of failure (one at each side of the column – Fig. 7).

Another solution would be to define new requirements for the minimum area of transverse rebars so as to avoid the failure of the concrete slab behind the column (by assuming the truss rods are inclined by 45°):

$$A_{S,2} \geq A_S \tag{5}$$

instead of:

$$A_{S,2} \geq \frac{A_s}{2} \tag{6}$$

as stated in EC4 for such joint configurations (to avoid concrete crushing against the column).

The CM computed moment-rotation curve obtained by substituting Formula (4) to Formula (3) for the evaluation of the resistance of the "slab rebars in tension" component is given in Figure 8. Key values are reported in Table 3. Two assumptions have been used to perform the computations as far as the area of the rebars is concerned:

– total area of the rebars ($A_{s,tot}$);
– area (as recommended in EC4) of the rebars with a diameter higher than 6 mm only ($A_{s,12\,mm}$).

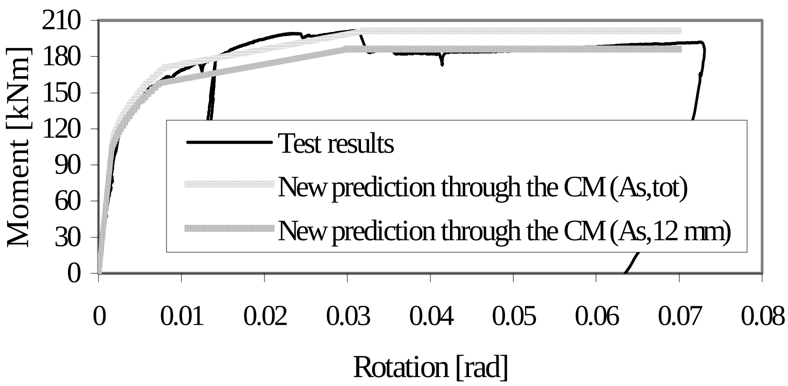


Figure 8. Comparison between test result and modified CM analytical approach

Table 3. Key values obtained experimentally and through the new prediction approach

| | Test results | New formula: $A_{s,tot}$ | New formula: $A_{s,12\ mm}$ |
|---|---|---|---|
| $S_{j,ini}$ [kNm/rad] | 65,000 | 64,312 | 64,312 |
| $M_u$ [kNm] | 201.58 | 201.54 | 186.4 |
| $\phi_u$ [rad] | 0.031 | 0.032 | 0.03 |

Figure 8 shows good agreements between the test result and the CM predictions using the new resistance formula for the "slab rebars in tension" component. The validity of this new formula has also been demonstrated through further comparisons with experimental tests carried out by Pisa University on a single-sided composite joint configuration under cyclic loading.

EC4 indicates that the only rebars to consider in the joint calculation are those with a diameter higher than 6 mm. On one side, this is safe as it leads to a smaller resistance of the joint than the actual one; but, on the other side, this is unsafe to determine the minimum percentage of transverse rebars. When the TEST 1 joint configuration has been designed (before testing), the actual presence of a mesh with rebars of 6 mm has been neglected (as proposed in EC4) and the minimum area of transverse rebars has been defined accordingly ($A_s/2$ with $A_s$ the total area of the longitudinal rebars in tension with a diameter higher than 6 mm). But it may be observed in § 2.1 (Fig. 2) that 8 longitudinal rebars from the mesh (diameter of 6 mm) are in tension (4 each side of the column) while only 2 transverse rebars from the mesh are present behind the column. So, if the presence of the mesh is considered to estimate the minimum area of transverse rebars, the $A_s/2$ EC4 requirement is not respected; to respect it, 4 transverse rebars with a diameter of 6 mm and not 2 would be needed behind the column. A proposal to avoid this unconsistency would be to consider all the rebars in the vicinity of the connection to determine the minimum percentage of transverse rebars to place behind the column.

Remarks:

- For double-sided joint configurations, to neglect the rebars with a diameter smaller or equal to 6 mm has not the same importance as, generally, rebars with such a diameter belong to a mesh. So, for a double sided joint configuration, the percentage of rebars with a diameter smaller or equal to 6 mm around the column will be equal what is not the case for single sided joint configurations because of the limited length of the cantilever part of the concrete slab behind the column.
- In the tests on double-sided composite joint configurations performed by other partners involved in the above European project (see [11] and [12]), the appearance of shear cracks in the concrete slab has not been observed. This can be explained by the amount of concrete available behind the column to resist to the shear loads that is higher than in a single-sided joint configuration.
- In the literature, few tests on single-sided composite joint configurations may be found; for those available, the rather high percentage of transverse rebars behind the column seems to explain why shear cracks in the cantilever part of the concrete slab have not been reported.

## 5 CONCLUSIONS

In this paper, the behaviour of a single-sided composite joint configuration is presented and investigated through experimental and analytical approaches. During the test in laboratory, the joint exhibited an unexpected behaviour characterised by the development of longitudinal shear cracks in the concrete slab, behind the column. As a consequence, the expected rotation capacity of 35 mrad of the joint has not been reached.

This new failure mode is not actually taken into account in the recommendations for joint design included in Eurocode 4. An analytical expression covering this new failure mode has been developed and integrated in the component method approach suggested by Eurocode 4. It allows to reach a very good agreement with the test results as shown in the paper.

On the basis of this study, proposals for amendment of Eurocode 4 will be made in view of its next revision.

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