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Practical Design Method for Semi-Rigid Composite Joints with Double Web Cleat Connections

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Abstract

This paper presents a practical and simple method for the prediction of bilinear moment-rotation characteristics of composite beam-to-column joints using double web cleat connections. This method is similar to an existing one for composite joints with end plate connections based on models for the plastic analysis of composite cross-sections. It is verified by comparison to experimental results and computer simulation results based on non-linear finite element analysis. These comparisons demonstrate good and conservative predictions of the semi-rigid behaviour of composite joints with standard steel beam-to-column double web cleat connections.

1. Introduction

Current design practice for steel building frames normally considers beam-to-column joints as either pinned or rigid (Fig. 1). Frames are thus either pin-jointed and require diagonal bracing or rigid and do not in general need to be braced.

Pinned beam-to-column joints transfer principally shear forces with perhaps a small axial force. For this reason these joints are designed such that only the web of a beam is connected to a column. One of the most popular types of connection used in these pinned joints is the bolted double web cleat, with the following advantages :

- fabrication tolerances can be absorbed by the play in bolt holes,
- straight cuts at the ends of beams,
- welds are not required,
- simple erection.

Rigid beam-to-column joints transfer a bending moment, shear force and an axial force. A common type of connection is an end plate, welded to the end of the beam, which is bolted to the column. Fabrication and erection is more complicated in comparison to a pinned beam-to-column joint if a completely rigid joint is required. In many cases it can be necessary to add stiffeners and to reinforce the column web. Nevertheless, rigid beam-to-column joints also have certain practical advantages :

- straight cuts at the ends of beams,
- end plates can be attached with fillet welds and no special treatment of cut ends of beams is needed,

– simple erection.

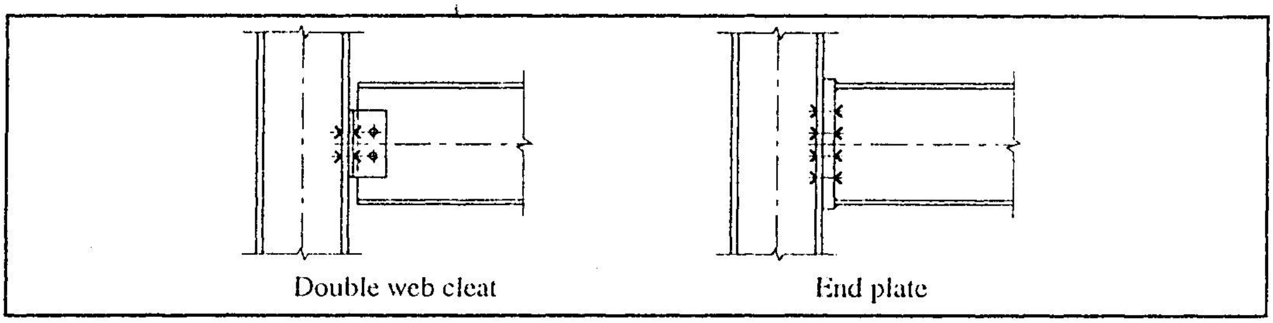

Fig. 1. Typical steel beam-to-column joints.

Floor slabs in traditional steel building frames are either of reinforced concrete or composite construction and are linked to beams with shear connectors. These slabs are continuous over the region of a beam-to-column joint and have therefore a significant influence on the behaviour of the beam-to-column joint. This influence is greatest in the case of a so-called pinned connection, for which the presence of continuous slab reinforcement has been demonstrated to provide the following advantages :

– increase in the moment resistance of the joint, leading to a reduction in midspan moment,
– higher rigidity of the joint reduces rotation at the column as well as midspan deflection.

In practice the majority of joints are therefore semi-rigid with partial resistance. The semi-rigid behaviour of a joint, somewhere between pinned and completely rigid, influences the distribution of load effects throughout a frame and should be considered during structural analysis.

The contribution of the floor slab is thus very important since it can justify a reduction in material (a lighter beam can be used) whilst employing a simple and economic connection (a double web cleat) as well as traditional construction methods (a composite or reinforced concrete floor slab).

The aim of this contribution is to give a simple and practical hand calculation method to determine the bilinear characteristics used in the design of semi-continuous composite structures.

2. Double web cleat connections

Both types of joint mentionned above have been standardized in Switzerland [1] and in many other countries. This standardization reduces the number of variations, has enabled the development of design tables and simplifies the fabrication of standard elements used in the connections.

Bolted double web cleat connections are both practical and economic. This type of connection can be used between cross and main beams as well as between main beams and columns. In the latter case the connection can be made to either the web or flange of a column (Fig. 2).

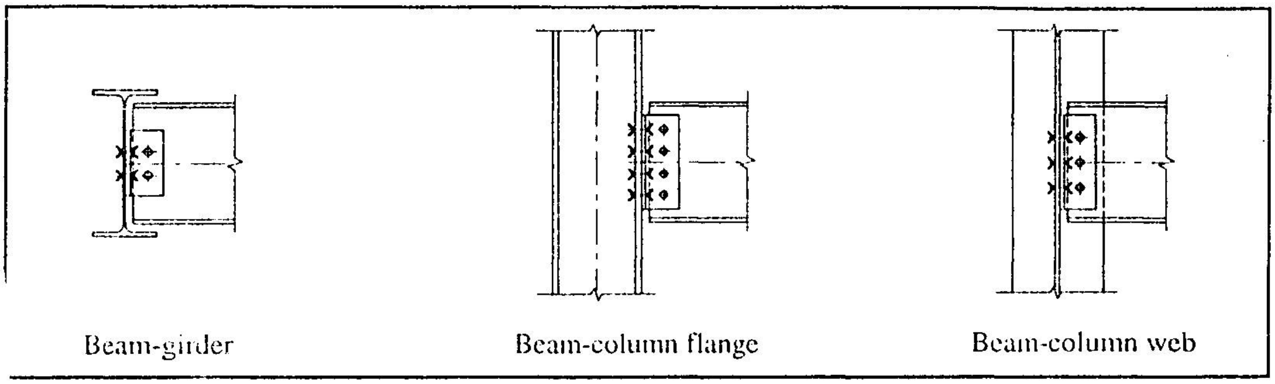

Fig. 2. Three typical double web cleat connections.

Since double web cleat connections have a wide range of application, standard configurations are used in order to simplify design, detailing and fabrication. For each configuration, design tables provide the following data :

- section size and cleat length,
- number, diameter and position of bolt holes,
- thickness of the connecting web or flange,
- vertical shear resistance.

Figure 3 shows several examples of composite beam-to-column joints of different sizes using the standard web cleats mentioned above.

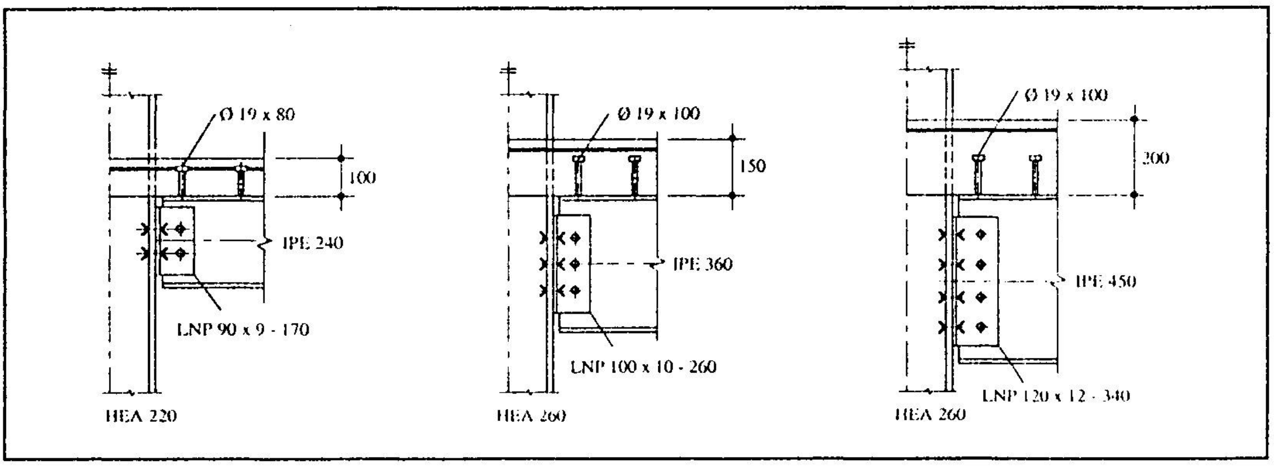

Fig. 3. Typical detailing of composite joints with double web cleat connections.

3. Experimental and analytical investigations

The behaviour of the steelwork connection of a composite joint is more complex than that of a bare steel connection. It is subjected to a very high axial force transferred by the shear connectors between the concrete slab and the steel beam. Existing knowledge about the moment-rotation behaviour of steel connections with the effect of axial force was very poor, so further research was undertaken [2]. Both experimental and analytical studies have been conducted as part of this research, and tests have been performed on bare steel connections as well as composite joints.

The first part of the research treated bare beam-to-column joints without composite slabs but with the effect of axial force. In this way, the contribution of the bare steelwork connection of a composite joint was assessed, using tests and an analytic approach. The second part of the research treated composite joints. The purpose was to verify the assumption that composite action increases both the strength and stiffness of bare steel joints.

A macro element model of composite joints has been developed and implemented in a computer program. Comparisons between test results and calculation predictions have been made to demonstrate the validity of this numerical model.

3.1 Steel connection tests

Four kinds of loading arrangement were used to apply different moment and axial load combinations to the steel connection. The loading arrangement is shown in Figure 4, together with the test specimen, the test set-up and the test results. Test results in terms of moment-rotation curves for the first three loading arrangements ($\alpha = 90°$, $60°$ and $30°$ or $N = 0$, $0.5\ P$ and $0.87\ P$) are shown in Figure 4c.

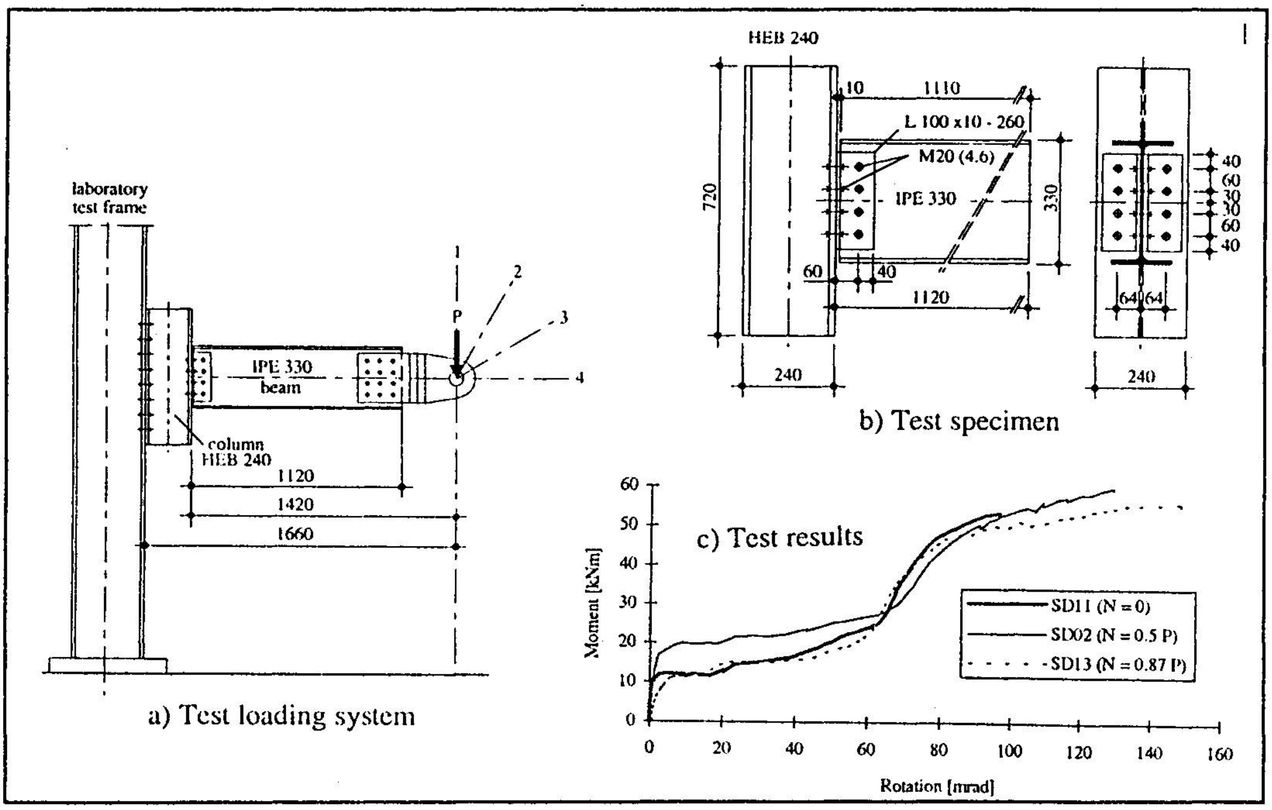

Fig. 4. Bare steel connection tests.

The moment-rotation behaviour of these three tests can be divided into three phases. The first was a initial elastic stage where no clear deformation occurred in the connection. The second phase was characterised by a very long plateau with almost no change in moment resistance as rotation increased. The top of the beam clearly moved away from the column and the bottom of the beam clearly moved towards the column. Vertical slip was observed between the beam web and double cleats, without significant deformation of the cleats. The third phase started when the bottom of the beam touched the column flange. The stiffness increased and the web cleats started to deform significantly. The three tests were stopped by excessive deformation.

It is apparent from the moment-rotation curves that the joint in test SD11 (moment only applied), with the weakest components in the tension zone, had the lowest moment resistance. During test SD02 (low axial force applied) the neutral axis moved towards the critical components in the tension zone and the moment resistance was thus higher. Finally, during test SD13 (high axial force applied) the critical components moved from the tension to compression zone, so that moment resistance decreased again.

3.2 Composite joint tests

Two cruciform composite joint specimens were tested. Each specimen was composed of one column and two adjacent beams connected to the column using the same standard steel connection already described. The details of composite joint specimens are shown in Figure 5.

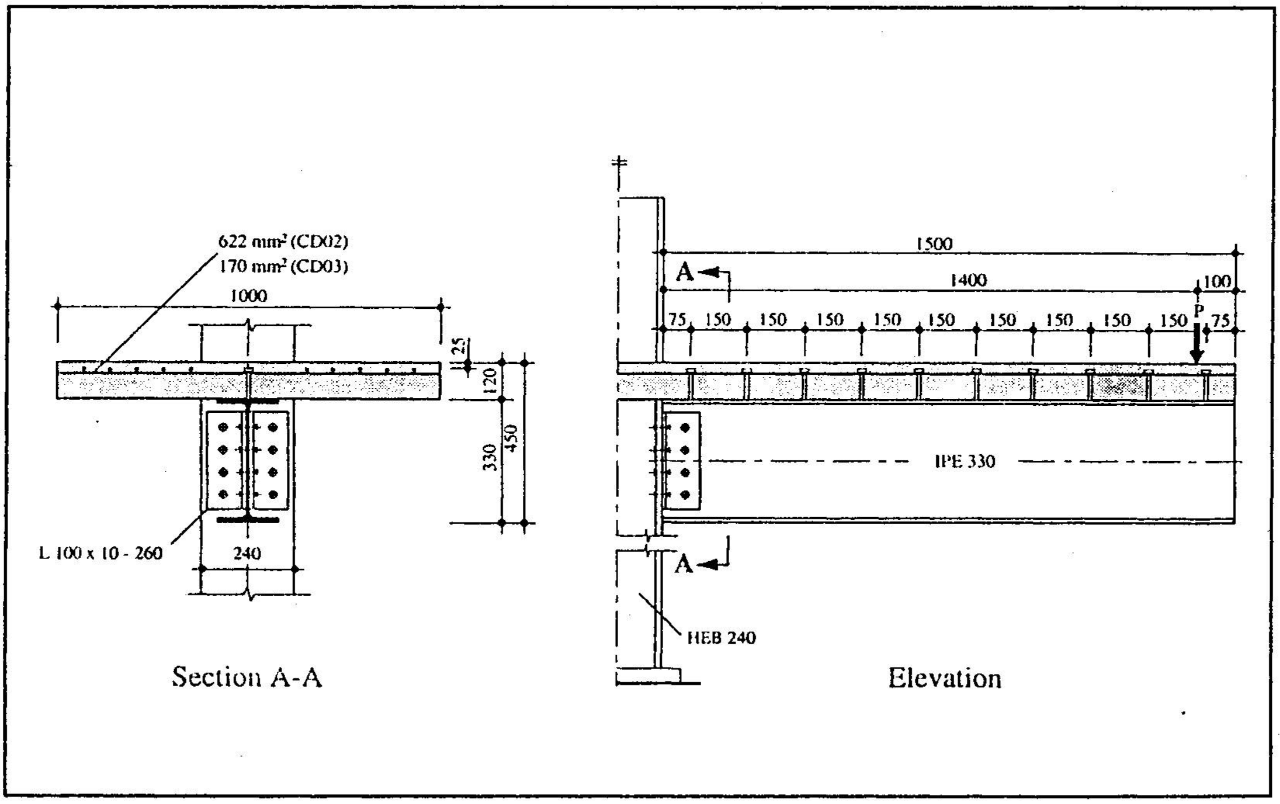

Fig. 5. Composite joint specimen.

The moment-rotation curves of the specimens tested are shown in Figure 6. Bare steel connection SD11 test results are given as a reference in the same figure.

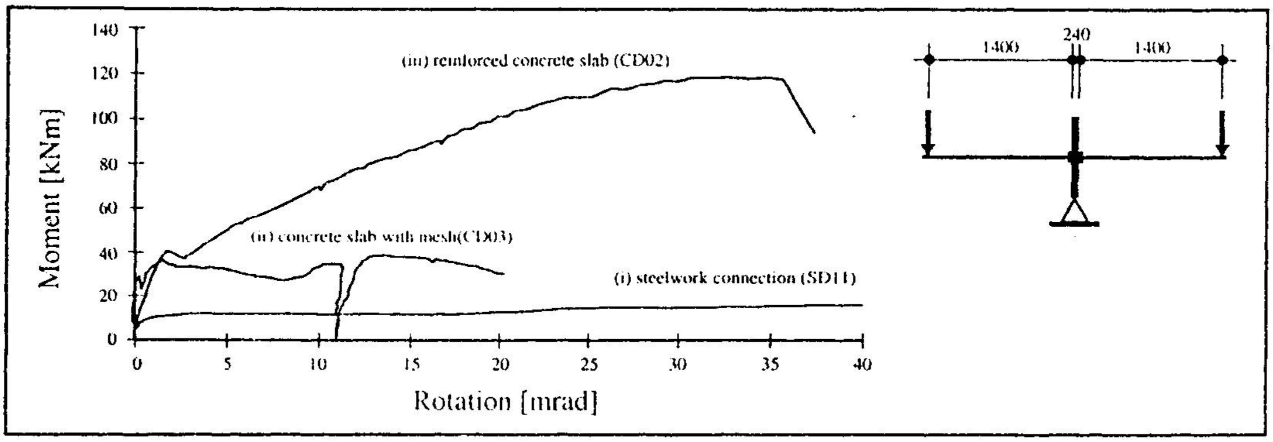

Fig. 6 Moment-rotation curves for composite joint tests.

Moment-rotation results of the first test (mesh and 4 x 12mm diameter rebars) exhibited a high moment resistance and good rotation capacity. With respect to much more deformation occurred in the steelwork connection, most of it being beam horizontal movement rather than rotation. Failure occurred also by the fracture of reinforcement at the cross-section through the column flange.

In the second test (mesh only), moment-rotation behaviour was characterised by a low moment resistance and small rotation capacity. Visible cracking in the slab was limited. No significant deformation occurred in the steelwork connection. Failure was due to mesh fracture at the cross-section through the column flange.

3.3 Analytical investigation

A numerical model of the non-linear composite joint behaviour including all flexibilities governing the characteristics of this type of joint has been developed [2]. By incorporating this numerical model into an existing composite beam analysis program [3], a new non-linear finite element (NLFE) program named COJOINT has been developed to simulate the semi-rigid behaviour of composite joints. This program has been verified with 14 tests of specimens having a wide range of member geometries, types of steelwork connections, degrees of horizontal shear interaction and reinforcement ratios [4]. The comparisons demonstrated a very good agreement between the tests and the numerical simulations. The relative importance of the parameters affecting composite joint behaviour has been identified by a parametric study with the use of the program COJOINT. This study enabled the development of the following simplified method.

4 Simplified method

4.1 Moment resistance

The method proposed for the prediction of the moment resistance of composite joints with double web cleat connections is similar to that for flush end plate connections, based on research undertaken in England [5, 6 and 7]. It is possible to use a plastic analysis (stress block method)

provided that the components of the joints have a ductile behaviour. The following simplifications are made when calculating negative moment resistance :
- tensile strength of concrete is neglected,
- moment resistance of the reinforced concrete slab is neglected,
- interaction between steel beam and concrete slab is complete,
- bolts in compression do not contribute to moment resistance, but they provide sufficient shear resistance,
- the resistance of the upper part of the connection in tension (bending and shear of the cleats, shear, bearing and tension of the bolts) is ignored,
- the centre of compression lies in the lower row of bolts,
- the resistance of the column web in compression is not critical.

Equilibrium is evaluated for the following internal forces (Fig. 7) :
- Tensile resistance of the reinforcement placed within the effective width of the concrete slab, F_s.
- Resistance of the zone in compression, F_c, represented by the shear or bearing resistance of the lower bolt in the web cleat connection.

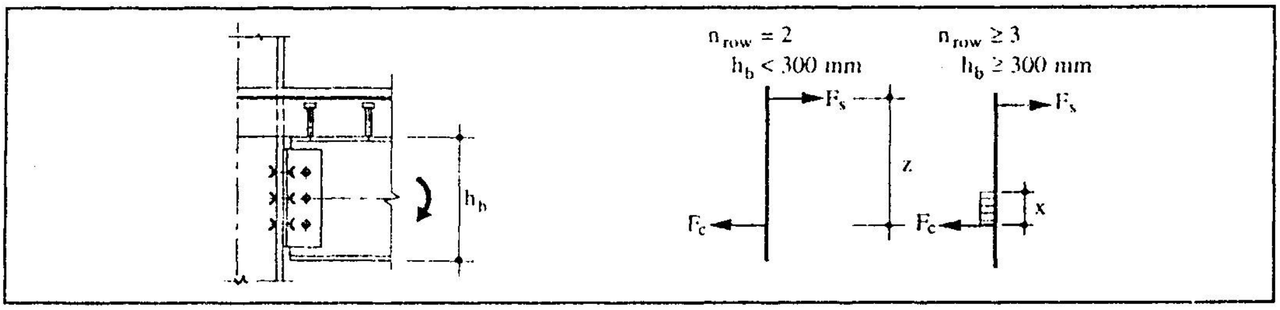

Fig. 7. Simplified model for composite joints with double web cleat connections.

Two cases can be defined according to the size of the elements.

First case : $h_b < 300$ mm ($n_{row} = 2$)

The moment resistance is then given by :

$$M_{Rd} = F \cdot z \tag{1}$$

F : minimum value of F_s or F_c
z : lever arm between compression and tension forces

Second case : $h_b \geq 300$ mm ($n_{row} \geq 3$)

If $F_c \geq F_s$, the moment resistance is given by Eq. (1), with $F = F_s$.
If $F_c < F_s$, a depth x of the web above the lower bolt is required to provide equilibrium :

$$x = \frac{F_s - F_c}{t_{wb}\ f_{yb}} \tag{2}$$

The moment resistance is then :

$$M_{Rd} = F_s \cdot z - (F_s - F_c)\frac{x}{2} \tag{3}$$

The transfer to the column flange of the force over the depth x is made using the bolts in the row above the lower row. The detailing of the third connection in Figure 4 illustrates a good solution for this force transfer. On the subject of the cross-sectional area of the total reinforcement (mesh + longitudinal bars), it should be chosen such that the neutral axis of the composite section lies within the web of the steel beam. In case of unbalanced moments, the area of the longitudinal reinforcement should be chosen such that failure of the section is ductile (Eurocode 4, Annex J [8]).

4.2 Rotational stiffness

The calculation of the rotational stiffness of composite joints is based on the following expression, derived from the formula proposed in the component method defined in Eurocode 4, Annex J [8]:

$$S_j = \frac{z^2}{\frac{1}{k_s} + \frac{1}{k_v} + \frac{1}{k_c}} \tag{4}$$

The stiffness of the reinforced concrete slab k_s is given by :

$$k_s = \frac{E_s\ A_s}{l_s} \tag{5}$$

E_s : modulus of elasticity of reinforcing steel
A_s : cross-sectional area of the total reinforcement (mesh + bars)
l_s : length of reinforcement under consideration (for example $l_s = h_c/2$) [7]
h_c : depth of the steel column

The stiffness of the shear connection k_v is given by :

$$k_v = \frac{\alpha F_v}{\delta_i} \tag{6}$$

α : reduction factor corresponding to the load at initial slip (for example $\delta_i = 0.1$ mm, see Figure 8)
F_v : longitudinal shear over the length of composite beam subjected to a hogging moment :

$$F_v = \min\{F_s\ ,\ \sum P_{Rd}\} \tag{7}$$

$\sum P_{Rd}$: sum of the resistances of the shear connectors over the length of composite beam subjected to a hogging moment
δ_i : slip at the steel-concrete interface corresponding to the load $\alpha\ F_v$

Finally, the stiffness of the steel connection is given by :

$$k_c = \frac{E_a\, A_l\, n_{row}}{h_c / 2} \tag{8}$$

A_l : lateral bearing area of a bolt :

$$A_l = t_{wb} \cdot d \tag{9}$$

d : bolt diameter

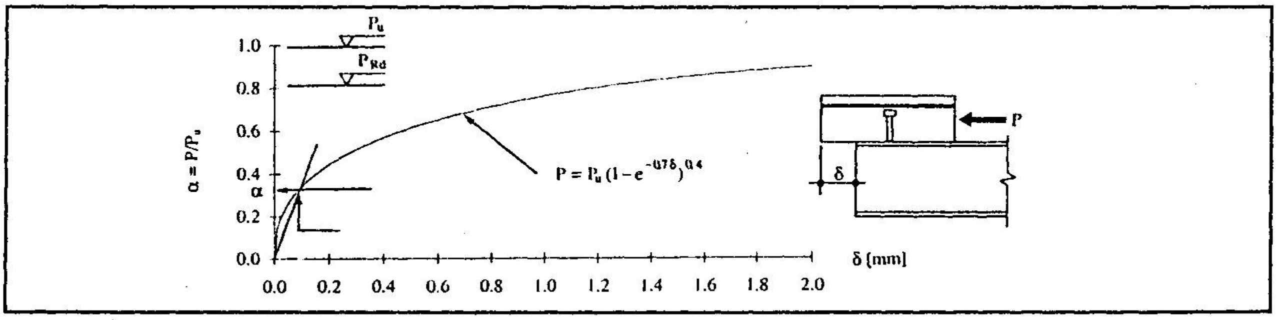

Fig. 8. Load-slip behaviour of a shear stud connector ([9]).

5. Comparisons

Figure 9 shows a typical comparison between test result of specimen CD02 and moment-rotation curves calculated with the NLFE COJOINT program and with the simplified method. The correlation between the experimental and the analytical behaviour is good in terms of initial rotational stiffness and moment resistance. However, the trilinear behaviour demonstrated in the test is difficult to reproduce by the "more exact" numerical model [2]. It can also be seen that the simplified method is conservative with respect to this model.

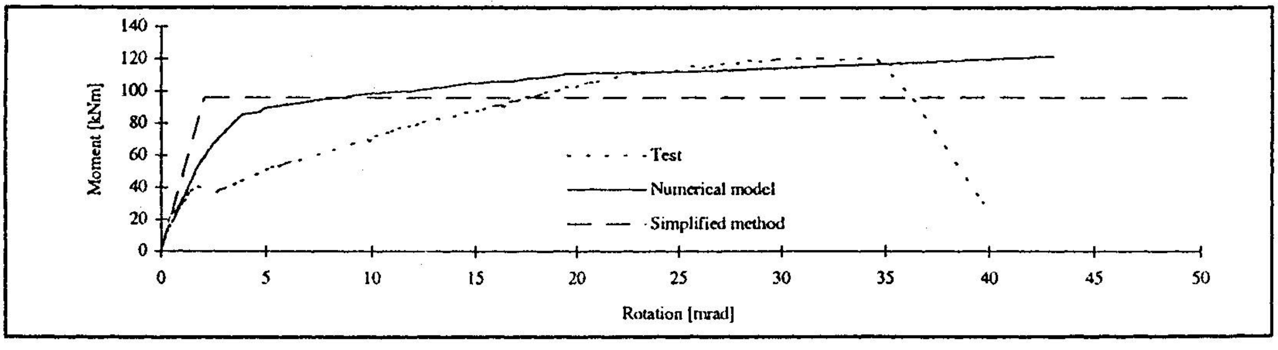

Fig. 9. Comparison between the test result, the numerical model and the simplified method.

Further comparisons have been conducted with several typical composite joints using different sizes of steel beam, web cleat, slab depth and reinforcement area (Fig. 10). The margin of safety is greater for small connections with only two rows of bolts than for large connections with three or more rows. This is due to the fact that the supplementary resistance existing in the actual connection is not taken into account in a small connection but is considered through the use of the x value in a larger one.

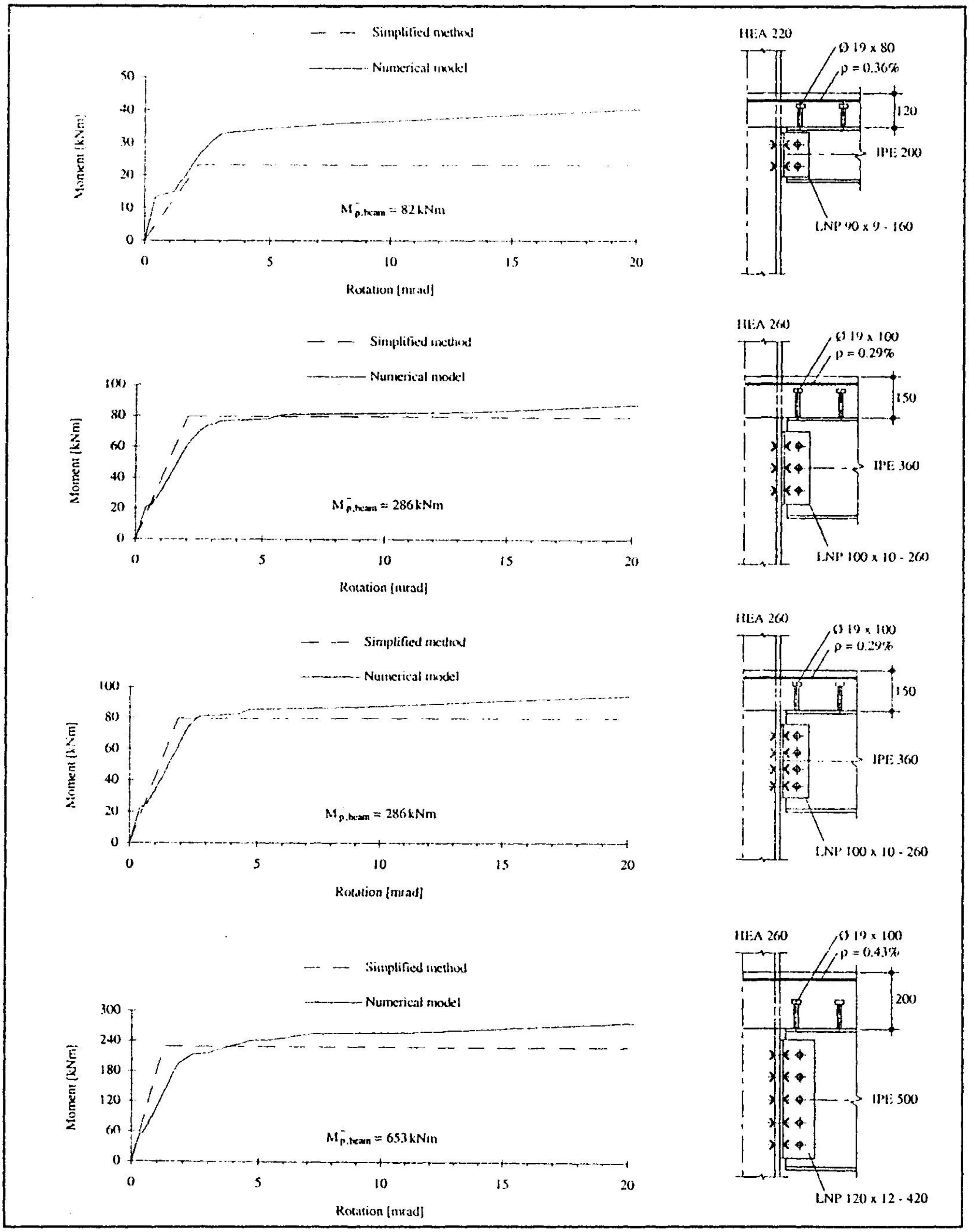

Fig.10. Comparison between the numerical model and the simplified method for four typical composite joints.

6. Conclusions

A method is proposed for the simple calculation of the moment resistance and rotational stiffness of standard double web cleat beam-to-column connections, taking account of the continuous reinforced concrete slab. This method is based on an existing method for end plate connections. Comparisons with test results and numerical simulations using a non linear finite element analysis show that the predicted behaviour is satisfactory. The contribution of the concrete slab enables the standard pinned steel beam-to-column connection to be considered as a semi-rigid composite joint. Bilinear characteristics determined by simple hand calculations can be easily used in the design of semi-continuous composite structures.

7. Acknowledgement

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