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# SESSION 1

# EXPERIMENTAL RESULTS ON JOINT CHARACTERISTICS

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# Composite Connections - Experimental Results

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## Summary

Moment resisting joints have to transfer moments and forces between structural members. They should be stiff enough, should have much bending resistance and should provide a rotation capacity as large as possible to redistribute bending moments from negative to positive moment regions. The properties of composite joints are assessed by means of the so-called component method. Main components are the steel connection, the column web panel and the reinforced concrete flange. They influence the joint behaviour very much while the joint itself influences the behaviour and design of the whole structure. In this paper, the behaviour of composite joints with finplate connections is investigated up to failure.

## 1. Introduction

The behaviour of composite structures is mainly influenced by stiffness and strength of the joints, if these properties are less than the corresponding properties of the adjacent cross sections of the connected members. If redistribution of bending moments is intended also rotation capacity of the joints has to be considered. Until now joints are assumed as nominally pinned or rigid. In reality both assumptions are quite conservative and may be uneconomic. Thus in view of economy and safety it is necessary to incorporate the realistic behaviour of composite joints into analysis and design of composite beams and frames.

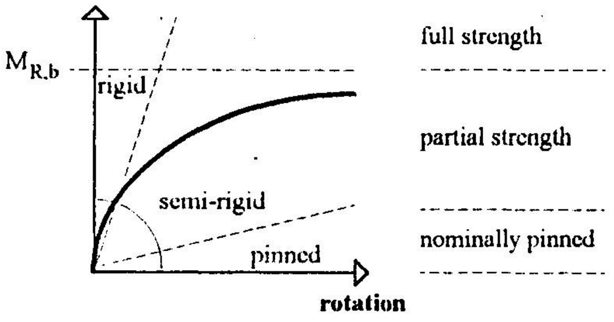



*Fig. 1 Non-linear moment-rotation behaviour of a joint*

Figure 1 shows the non-linear moment-rotation behaviour of a joint in principle, together with the classification regarding stiffness and strength.

The intention is to develop design rules which make allowance for the whole range between nominally pinned and rigid or full strength connections. As a first step the behaviour of so called semi-rigid or partial strength joints has been investigated. In a second step the influence of such joints on the behaviour of semi-continuous composite beams and frames shall be considered.

## 2. Types of joints

Composite joints consist of structural steelwork connections, a continuous reinforced concrete slab and - in case of beam-to-column connections - the column web panel. Figure 2 compiles usual types of moment resisting composite beam-to-column joints, the strength of which becomes smaller and smaller, from full strength (case a) to partial strength (cases b, c, d).

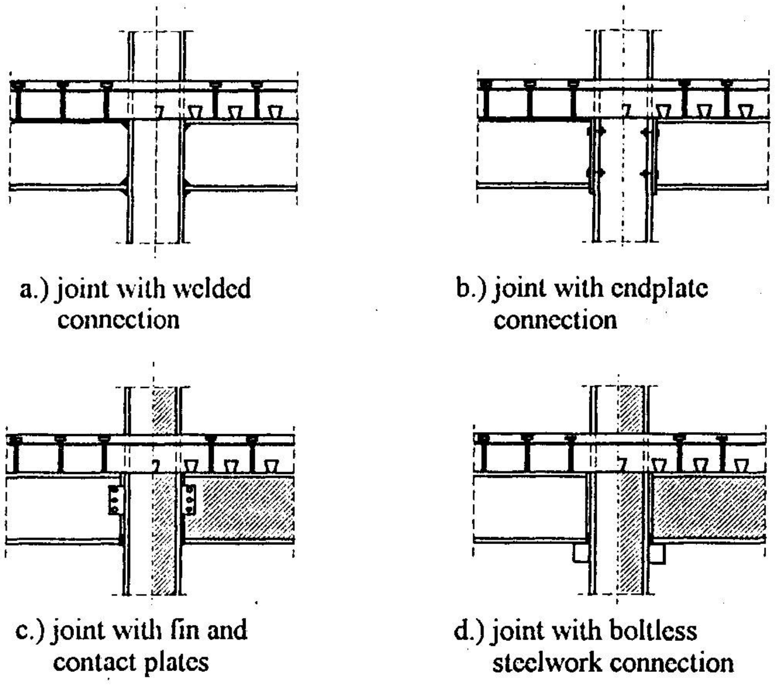



*Fig. 2 Types of moment resisting beam-to-column joints*

The different detailing of steelwork connection is decisive for the destinction between these types of joints. As far as no instabilities in the column web panel or the adjacent beam sections occur welded connections as shown in fig. 2a provide a high stiffness and strength and result in a behaviour in accordance with continuous beams. But they are very cost intensive in fabrication and erection and require small tolerances. Alternatively endplates (fig. 2b), finplates (fig. 2c) or cleated connections can be used as well as connections without bolts (fig. 2d). The last two types with contact plates can be assumed to be pinned during erection, but after the concrete has hardened they provide a high degree of stiffness and moment resistance. This effect leads to a change in the structural system from simply supported during erection to semi-continuous after hardening of the concrete.

Most of these beam-to-beam or beam-to-column joints can be carried out with partially encased beams and columns. This additional concrete part increases the fire-resistance as well as the strength and stiffness of several joint components. The reinforced slab can be a solid slab as well as a composite slab when profiled steel sheeting is used.

## 3. Behaviour of composite joints

Since the late seventies world-wide research focused on the behaviour of composite joints. Early results are summarized in [1]. Since 1990 the European research is co-ordinated by the COST-Project C1 'Control of the Semi-Rigid Behaviour of Civil Engineering Structural Connections'. Within this project the behaviour of composite joints as well as the behaviour of steel, concrete, timber and polymeric joints is being investigated. The work within material-related groups is supported by numerical and seismic groups and supplemented by the development of data bases. Previous results of this work are summarized in [2], [3] and [7].

Stiffness, strength and rotation capacity of a composite joint depend on the load-deformation relationships of the single joint components, thus leading to the so-called component method for the prediction of the joint behaviour. In Eurocode 3, Annex J [4] this method is used to determine the strength and stiffness of steelwork joints. In addition to the components for bare steel joints, the reinforced concrete slab and contact plates have to be included when composite joints are considered.

The behaviour of composite joints is - apart from the behaviour of the bare steel connection and the column web panel - mainly influenced by the type, amount and distribution of reinforcement embedded in the concrete flange and by the shear connectors [6, 9 and 10]. Also the type of loading (balanced or unbalanced), the joint configuration (single or double sided joints), the loading history (propped or unpropped during concreting) and possible changes in the structural system have influence on the joint behaviour, which is no longer neglegible.

## 4. Research at Kaiserslautern University

In 1991 a research group has been established at Kaiserslautern University by the DFG (German Research Foundation) to investigate the behaviour of composite structures. Results of two projects, which are dealing with the behaviour of partial strength composite joints and the contribution of the reinforced concrete component, are given below.

### 4.1 Reinforced concrete behaviour - first test series

In hogging moment regions, the concrete flange of the composite beam and as part of the composite joint is stressed in tension and a certain amount of bending. Therefore it is useful, to study first the behaviour of a concrete member under bare tension. While strains and deformations are still elastic, the tensile strength of concrete and its variation as well as the amount, type and distribution of reinforcing bars have great influences on first cracking, crack pattern and the so-called tension stiffening effect. Figure 3 shows the simplified stress-strain relationship for embedded reinforcing steel given in the CEB-FIP Model Code (1990). But

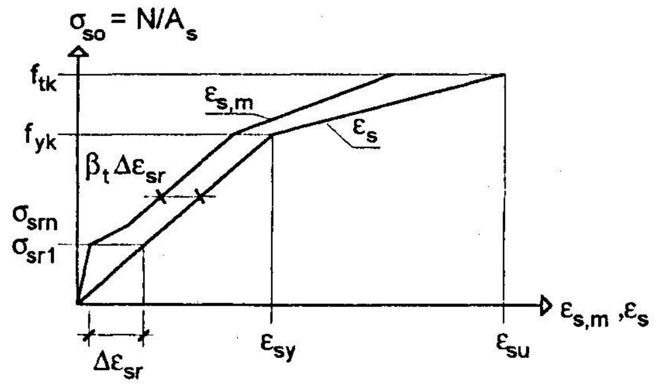



*Fig. 3 Simplified stress-strain relationship of embedded reinforcing steel*

after yielding, in the plastic range with strain hardening, ductility and deformation capacity depend mainly on the reinforcement ratio and the stress-strain behaviour of the embedded reinforcing steel.

Figure 4 shows stress-strain curves of tensile tests with reinforced concrete members, which have reinforcement ratios of 0.2 % and 0.6 %, respectively. The very low value of 0.2 % results in only a few cracks, in which the concrete can yield, thereby reducing the deformation capacity of the concrete member. In case of members 1 and 2 with 0.6 % reinforcement ratio, however, the deformation capacity is much larger: there are more cracks, with plastic deformations in and near the cracks, particularly when elongation of the reinforcing steel is as large as in case 2. Excellent bond properties between concrete and steel would again reduce the deformation capacity.

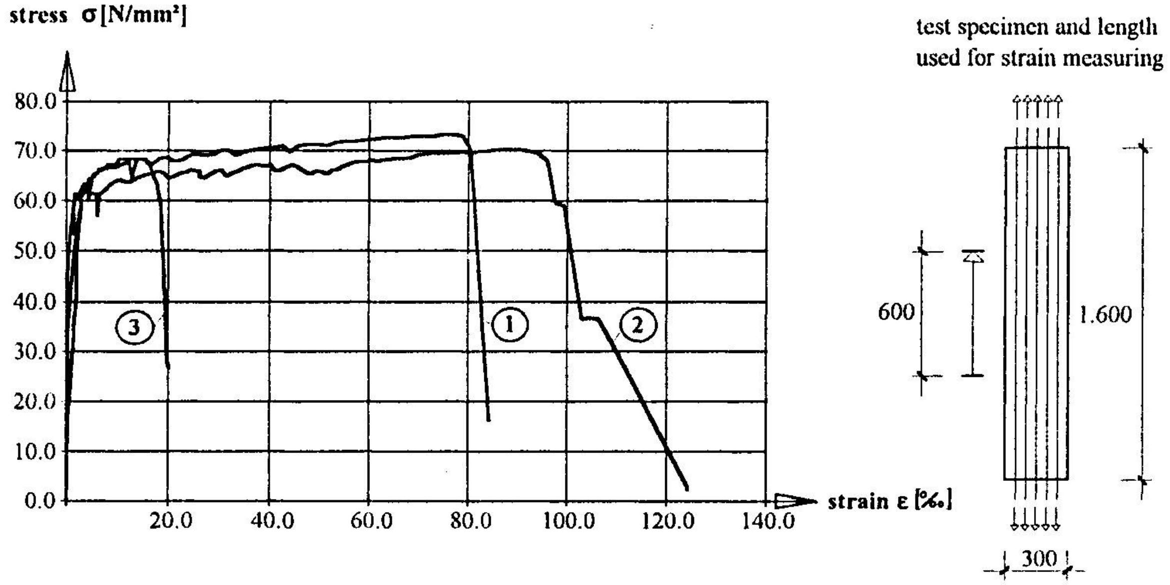



*Fig. 4 Stress-strain curves of tensile tests with reinforced concrete*

| Test No. | reinforcement ratio | diameter of rebars | ultimate strain $\varepsilon_{su}$ | ratio fu/ft |
|---|---|---|---|---|
| 1 | 0.6 % | 10 mm | 12.0 % | 1.14 |
| 2 | 0.6 % | 8 mm | 15.2 % | 1.14 |
| 3 | 0.2 % | 8 mm | 15.2 % | 1.14 |

*Table 1 Test parameters of the concrete tensile members and the reinforcing bars*

The knowledge about this behaviour of a concrete member in tension is very helpful, because the concrete flange is a main component of moment-resisting composite joints.

## 4.2 Beam-to-column connections - second test series

In a second test series 4 beam-to-column joints, symmetrically loaded, have been investigated in order to find out the contribution of the reinforced concrete flange component to the total joint behaviour. Figure 5 shows the test specimen and the test setup for this test series with finplate connections. Forces and bending moments were applied by means of a hydraulic jack, tie rods, cross beams and anchors. The cruciform test specimen was positioned 'concrete side up' in order to make observation and measurements of crack widths easier.

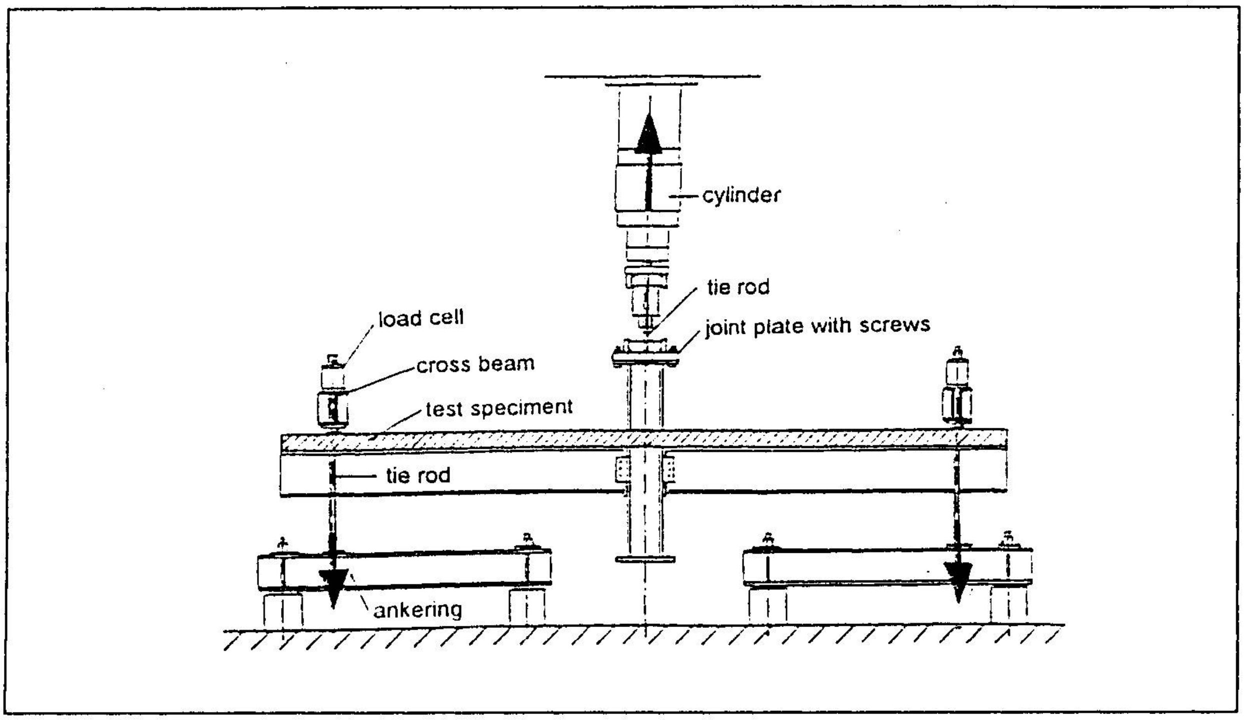



*Fig. 5 Setup of the second test series*

During the test, first cracks appeared above the steel connection at the column corner due to stress concentration. These cracks opened immediately and spread over the whole width of the concrete slab. While the cylinder load was increased, further cracks formed in the slab besides the column. Afterwords the development of cracking continued from the section with maximum bending to the free ends of the cantilevers. Due to horizontal shear in the concrete flanges, additional diagonal cracking occured, which can be seen too in figure 6.

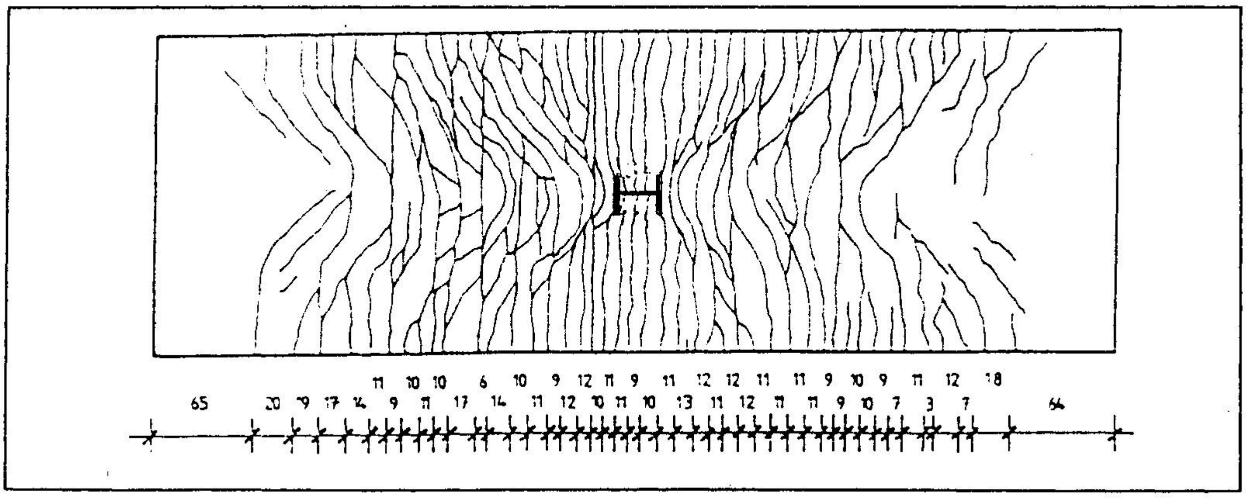


*Fig. 6 Crack pattern at the end of the test with a reinforcement ratio of 1.3 %*

In the middle of the test specimen the beam as well as the joint are loaded in negative bending. But comparing the global load-deflection behaviour of this test specimen with the behaviour of a corresponding beam specimen without column hole and connections (see fig. 7), a clear dif-

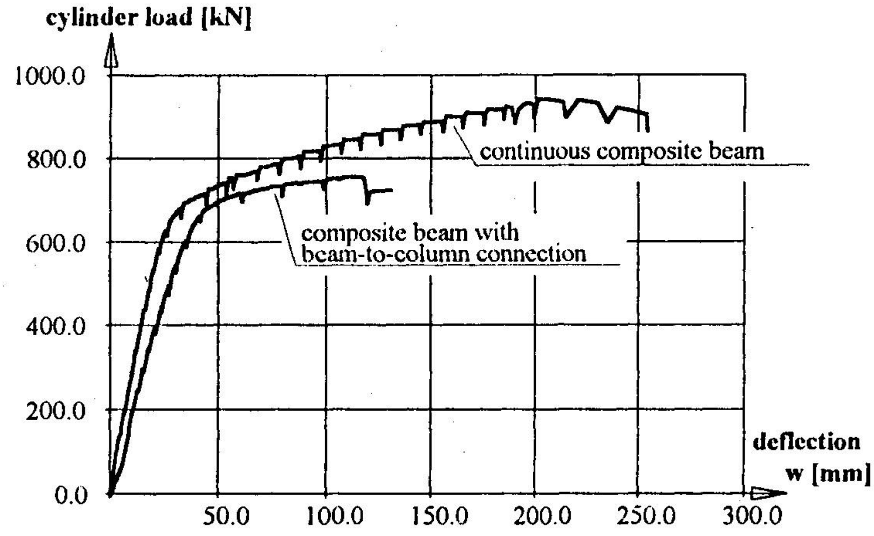



*Fig. 7 Load-deformation curves of composite beams with connection and continuous (reinforcement ratio 0.9 %)*

ference can be identified. Particularly, the deformation capacity is reduced, if the slab is holed due to the column. In this beam test the reinforcement was the same (0.9 %), but the rebars were stressed differently. The test specimen with column hole and connections failed by fracture of the reinforcement, while in the composite beam the lower steel flange buckled at much larger deformations.

## 4.3 Beam-to-column connections - third test series

A third test series was performed with cruciform test specimen as shown in figure 5, but now with different ratios of reinforcement from ρ = 0.4 % up to 1.3 %. The moment-rotation behaviour is shown in figure 8. The moment resistance was provided by means of contact plates at the bottom flange, with the exception of test No. 4. Failure of test No. 1 was initiated by buckling of the lower steel flange close to the column, while the other specimen failed due to rupture of the reinforcing bars. The more reinforcement is used, the larger is the rotation capacity of the considered connections. Without contact plates, the behaviour is similar to a pinned connection, but a sudden increase in bending resistance is provided as soon as the gap between the column flange and the lower beam flange closes. Due to the low degree of reinforcement, however, the rupture of bars limits the rotation capacity.

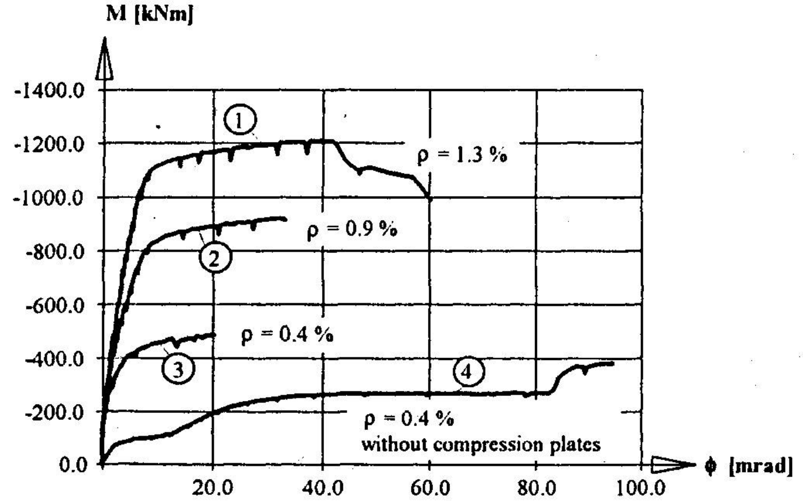



*Fig. 8 Finplate connections - moment-rotation curves*

Further test series are reported in the following, where the cooperation of the components 'reinforced concrete flange' and 'steel connection' is considered in detail with regard to the shear connection along the composite beams, together with further variations. In all these further tests, a 12 cm thick concrete slab was cast on Holorib type profiled steel sheeting with

120 cm slab width, while in the above mentioned tests a 14 cm thick solib slab with a width of 200 cm had been used.

### 4.4 Beam-to-column connections - fourth test series

The fourth series included three beam-to-column tests with finplate connections as shown in figure 9. For such connections it is assumed generally that the moment-resistance is provided by the reinforced concrete slab stressed in tension and a contact plate between the column flange and the lower steel flange.The vertical shear force is transferred by the finplate. In practice the contact plate will be provided after concrete has hardened to establish a pinned connection during erection and to achieve significant moment-resistance and stiffness at the final composite stage. This leads to a change in the structural system during the loading history.

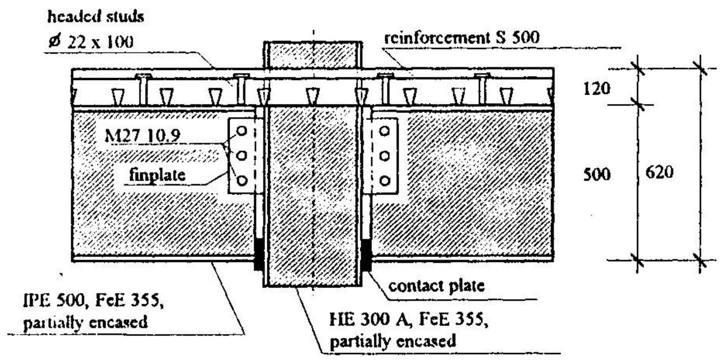
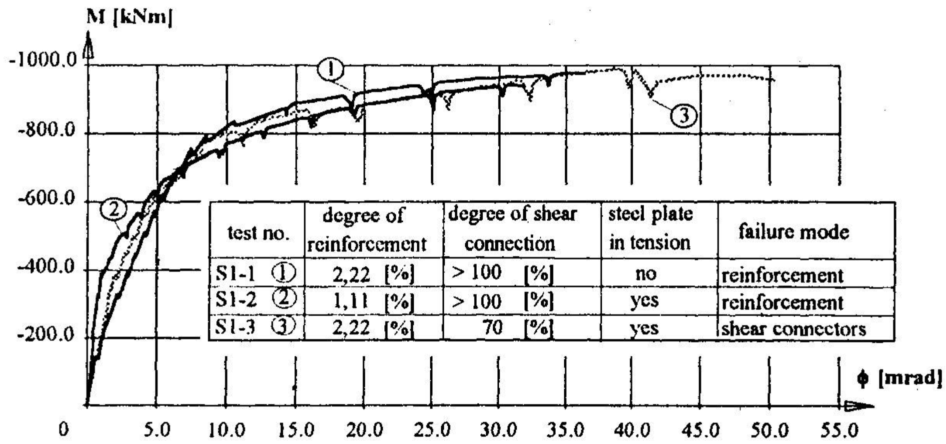



*Fig. 9 Tests with composite joints with finplate connection: joint detailing*

In the first test S1-1 the reinforcement within the slab had to transfer the tensile forces alone. In the second test part of the tensile force has been transferred by additional steel plates welded to the upper steel flanges, while the amount of reinforcement has been reduced in order to provide the same moment resistance as in test S1-1. In the third test the same amount of reinforcement as in the first test has been combined with the same number of shear connectors as in the second test. This leads to partial shear connection with slip and to a ratio of shear connection of about 70 %. In this test steel plates welded to the upper steel flanges have been used also to provide the same moment resistance.




| test no. | degree of reinforcement | degree of shear connection | steel plate in tension | failure mode |
|---|---|---|---|---|
| S1-1 ① | 2,22 [%] | > 100 [%] | no | reinforcement |
| S1-2 ② | 1,11 [%] | > 100 [%] | yes | reinforcement |
| S1-3 ③ | 2,22 [%] | 70 [%] | yes | shear connectors |

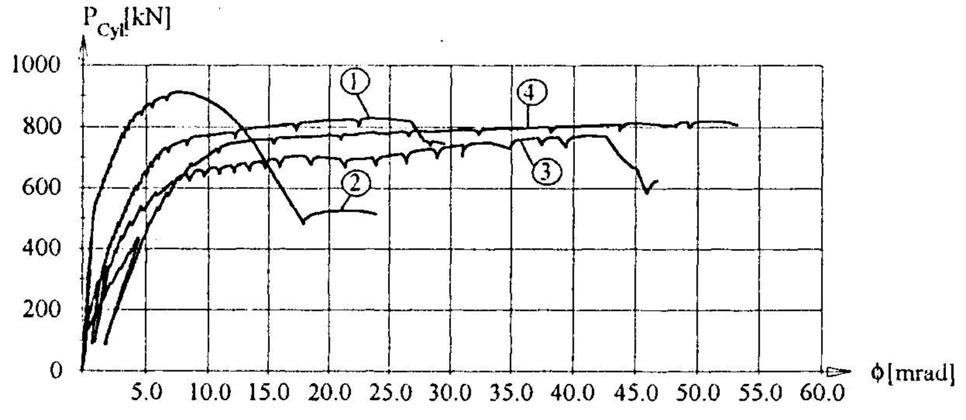
*Fig. 10 Tests on composite joints with finplate connection*

Test S1-1 showed the lowest initial stiffness, but a moment resistance of about 1000 kNm has been reached, see figure 10. Recalculation of the moment resistance with the actual material properties showed that only 60 % of the moment resistance has been provided by the reinfocement, the rest by the bolts within the finplate. In this test reinforcement rupture occured at a rotation of 36 mrad. Due to the steel tensile plates test S1-2 showed the highest initial stiffness, but the steel plates began to yield at an applied moment of 400 kNm which resulted in loss of stiffness. In this test the reinforcement ruptured also, at a rotation of 32 mrad. In test S1-3 with slip due to partial shear connection a maximum rotation of 40 mrad has been measured before the first shear connectors failed. The initial stiffness is between the other tests. This is a result of the lower longitudinal stiffness of the welded tensile steel plates and the slip between steel beam and concrete slab. As can be seen from figure 10 these three tests provided nearly the same initial stiffness and moment resistance while the rotation capacity varied a little (up to 11 %).

### 4.5 Beam-to-beam connections - fifth test series

In a fifth test series beam-to-beam connections have been investigated. This type of boltless steel connection (figure 11) can be used for example for staggered beams where the composite floor beams are supported by main beams. Moment resistance is established by reinforcement in the slab and contact plates at the bottom.

In the first, second and fourth test tensile forces have been transferred by reinforcement only. Test S2-1 is the reference test with full shear connection and uniformly distributed shear connectors. In test S2-2 part of the reinforcement has been replaced by a steel plate welded to the upper steel flanges. A reduced amount of reinforcement leads to a reduced number of shear connectors while still providing full shear connection. In the third test (S2-3) the same amount of reinforcement as in test S2-1 has been combined with the number of shear connectors from test S2-2, which leads to partial shear connection. In the last test S2-4 the same amount of reinforcement and number of shear connectors as in the first test have been used, but all the shear connectors have been placed at the end of the cantilevers, thus forming a uniformly stressed tensile tie.




| test no. | r [%] | h [%] | failure mode |
|---|---|---|---|
| S2-1 ① | 2,42 | > 100 | reinforcement |
| S2-2 ② | 1,45 | > 100 | tensile plate |
| S2-3 ③ | 2,42 | 70 | shear connectors |
| S2-4 ④ | 2,42 | > 100 | reinforcement |

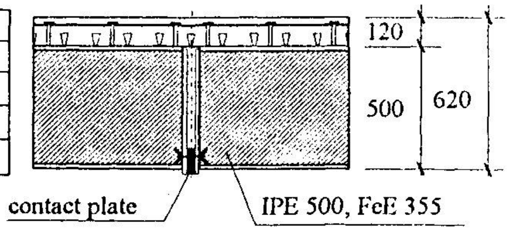



*Fig. 11 Fifth test series: type of connection and experimental load-rotation curves*

Figure 11 shows the detailing of the connection and the load-rotation curves measured in the tests. The reference test S2-1 provides a significant stiffness and moment resistance. The reinforcement ruptured at a rotation of 27 mrad. Test S2-2 with the additional steel plate stressed in tension reached the highest stiffness and moment resistance, but this is accompanied by the lowest rotation capacity. The very high stiffness is a result of the detailing of the welds between steel plate and steel flanges: the welds are running through to the end of the steel beams. This means, the free length for elongation of the steel plate (after yielding) was limited to the small gap between the steel beams. Therefore fracture of the steel plate occured at a rotation of 18 mrad only, while the ultimate moment resistance has been achieved at a rotation of 8 mrad. From the P-$\phi$-curve of test S2-3 it can be seen, that partial shear connection with slip can lead to a large rotation capacity, in this case to a rotation of 43 mrad, where the first shear connector failed. On the other hand, the stiffness of the composite joint decreased significantly due to slip between the steel beam and the concrete slab. In comparison with the other tests of this series a slip ten times higher was measured. In the last test of this series all shear connectors had been placed at the end of the cantilever. Due to this arrangement a long tensile tie within the slab could develop, which leads to a decrease of stiffness and an increase in rotation capacity. The rebars failed at a rotation of 53 mrad. These test results show very clearly, how and to which extend the moment rotation behaviour of a joint depends on the structural detailing.

## 5. Summary and Conclusions

From these tests and further reseach at Kaiserslautern University, the following conclusions can be drawn:

- The structural detailing governs the M-$\phi$-behaviour of composite joints.
- The component method is the standard procedure to predict stiffness and strength of composite joints without testing.
- The total rotation consists of two parts: the rotation in the joint itself and the rotation along the composite beam under negative bending moments. These two parts have to be separated strictly.
- The reinforced concrete flange provides a significant influence on the joint behaviour. This influence decreases with high stiffness and strength and low flexibility of the steelwork connection.
- If composite beams are connected to a column, first single cracks form early in the stress concentration areas at the column hole corners.
- To reach the same bending moment resistance and rotation capacity as in case of continuous composite beams without column hole, more reinforcement is necessary.
- The deformation capacity can be increased to a certain extend, if more reinforcement is used (more cracks), which consists of bars with ductile behaviour.
- The crack pattern is significantly influenced by the transverse reinforcement.
- If joints are rigid and of full strengh, the behaviour of semi-continuous composite beams appraoches that of continuous span beams, and the well known EC 4-rules based on class 1 to 4 cross sections apply. But the question is: is there any additional resistance neccesary, to prevent plastic hinges in joints, if global plastic analysis is applied?

- Composite beams provide their large resistance in sagging moment regions. On the other hand, the resistance under negative bending is smaller, and it is further reduced at partial strength joints.
- In order to make use of the high resistance at midspan, elastically calculated bending moments should be redistributed up to values, which can be determined by means of global plastic analysis. This requires rotation capacity in the negative moment region.
- The smaller the resistance in the joint, the larger is the required rotation capacity. Due to the fact, that cracking of concrete and yielding of steel in the adjacent beam cross sections is diminishing simultanuously, the joint rotation itself has to be enlarged.

To increase the rotation capacity of partial strength composite joints, several means are possible and outlined in the following:

- Partial shear connection, flexible connectors, use of profiled steel sheeting, spacing of connectors such that a uniformly stressed tensile tie within the reinforced concrete flange can form out.
- If additional steel plates are used, they should have enough free length for unrestrained elongation.
- The concrete flange shall be reinforced adequately with reinforcing bars which have high ductility.
- Bolts in the tensile zone of the structural steel connection should not be decisive for the rotation capacity. If such bolts are used, end- as well as finplates should be thin enough, to provide high ductility of this component.

## 6. Acknowledgement

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## 7. References

[1] Zandonini, R.: Semi-Rigid Composite Joints. Stability and Strength: Structural Connections, Elsevier Applied Science, London - New-York, 1989

[2] Semi-Rigid-Behaviour of Civil Engineering Connections. COST-C1. Proceedings of the first state of the art workshop, Strasbourg, 1992

[3] Semi-Rigid-Behaviour of Civil Engineering Connections. COST-C1. Proceedings of the second state of the art workshop, Prague, 1994

[4] Eurocode 3, Part 1.1, CEN, 1993

[5] Eurocode 4, Part 1.1, CEN, 1993

[6] Anderson, D. , Najafi, A.: Performance of Composite Connections: Major Axis End Plate Joints. Research Report CE 44, Department of Engineering, University of Warwick, 1993

[7] Proceedings of the Second International Workshop on Connections in Steel Structures, Pittsburgh, ASCE, 1990

[8] CEB-FIP Model Code 1990. Comité Euro-International du Beton, Lausanne, 1993

[9] Bode, H. , Kronenberger, H.-J.: Zum Tragverhalten von Verbindungen und Anschlüssen im Stahlverbundbau. 31. DAfStb-Forschungskolloquium, Deutscher Ausschuß für Stahlbeton, Kaiserslautern, 1995

[10] Aribert, J.-M.: Influence of Slip of the Shear Connection on Composite Joint Behaviour. Proceedings of the Third International Workshop on Connections in Steel Structures, ECCS and AISC, Trento, May 1995

[12] Tschemmernegg, F. et al.: Zur Nachgiebigkeit von Verbundkonstruktionen. Stahlbau 63 (1994), Heft 12 and Stahlbau 64 (1995), Heft 1

[13] Roik, K., Hanswille, G.: Rißbreitenbeschränkung bei Verbundträgern. Stahlbau 60 (1991), Heft 12

[14] Maurer, R.: Grundlagen zur Bemessung des Betongurtes von Stahlverbundträgern. Ph. D. Thesis, Technische Hochschule Darmstadt, 1992

[15] Kreller, H., Zum nichtlinearen Trag- und Verformungsverhalten von Stahlbetonstabtragwerken unter Last- und Zwangeinwirkung. Heft 409 des DAfStb, Verlag Ernst und Sohn, Berlin, 1991

[16] Ramm, W., Elz, S.:Tragverhalten und Rißbildung von Gurtplatten von Verbundträgern im Bereich negativer Momente. 31. DAfStb-Forschungskolloquium, Deutscher Ausschuß für Stahlbeton, Kaiserslautern, 1995

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# Bolted Beam-to-Column Steel Connections

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## Summary

The "Parallel Beam" approach to steel framing makes use of composite and continuous construction to give a cost effective answer for industrial and commercial structures. In this study, the actual behaviour of parallel beams having double channel profiles as beam is investigated. The aim of the study is to provide moment-rotation characteristics and corresponding parameters of this kind of connections.

## 1. Introduction

The steel frame is ideally suited to satisfy the most stringent commercial, architectural and engineering demands for quality, speed, economy and flexibility. Several alternative systems for providing "Fast-Built" construction have been developed over the past few years. The Parallel Beam approach to steel framing makes use of composite and continuous construction to give a cost effective answer for industrial and commercial structures. It is especially beneficial for buildings with high service contents. The aims are to:

- -Reduce the number of members,
- -Reduce the weight of the beams by means of continuity,
- -Simplify connections,
- -Allow simple service integration,
- -Reduce cost.

A building profits from a steel frame designed in this way because it gives flexible service layouts and can be fabricated and erected quickly, at reduced cost.

The response of steel frames is also influenced by the mechanical properties of the joints (strength, stiffness, rotation capacity). In practice, the joints are usually considered as either rigid or pinned. Research has shown that frames with semi-rigid joints can be more economical than frames with rigid or pinned joints.

In recent years, many researchers have published papers discussing the influences of connection rigidity on steel frame structures for different types of connections. In this study, the actual behaviour of Parallel Beams having double channel profiles as beams is investigated. The aim of the study is to provide moment-rotation characteristics and corresponding parameters of the this kind of connections. As known for the pre-design stage there is a need for simple rules to estimate the mechanical properties of the joints. Because of the shortage of the knowledge about the real behaviour of this kind of joints and the non-existance of the simple rules for pre-designed in the Codes, the moment-rotation curves of the tested joints are obtained to be the starting point of a general research protect aimed to extend for this type of connections. The test programme performed at the Structural Laboratories of Istanbul Technical University and consisted of six full-scale connections. Both strong and weak axis connections were tested.

## 2. Experimental Programme

In the programme six specimens were tested. In each specimen, the beam is composed with double [ 300 profiles and the column has HE 280 B profile as cross-section. A schematic view of the test set-up for the full-scale tests is shown in Fig.1. The testing frame was anchored on the testing floor of the laboratories and aimed to resist the applied load and the support reactions. Accessories clamped on the testing slab whose rigidity can be considered as infinite. A load-cell which has been stated at the end point of the cantilever was used to measure the vertical load.

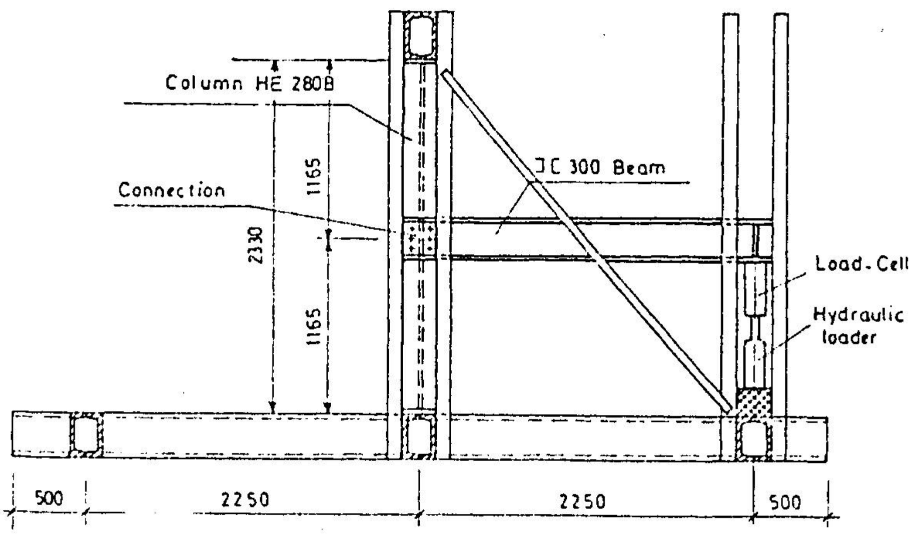


*Fig. 1. The test set-up*

The material of the specimens is Fe 360 (St 37) structural steel and the strength of the material is tested and proved at the laboratories. The bolts used in connections are in 10.9 quality. In strong axis connections eighteen and in weak axis connections twelve electronic strain-gages were mounted on the beam web to determine the vertical and horizontal variations of the strains. In each specimen eleven comparameters were used to measure the displacements. One of them was located under the load point and the rest at the connection.

Three of the samples are strong axis connections in which the webs of the twin channels are connected by nine bolts to the two plates welded to the column flanges by fillet welds. Other three samples are weak axis connections and in these the webs of the beams are directly connected by six bolts to the flanges of the column (Fig.2).

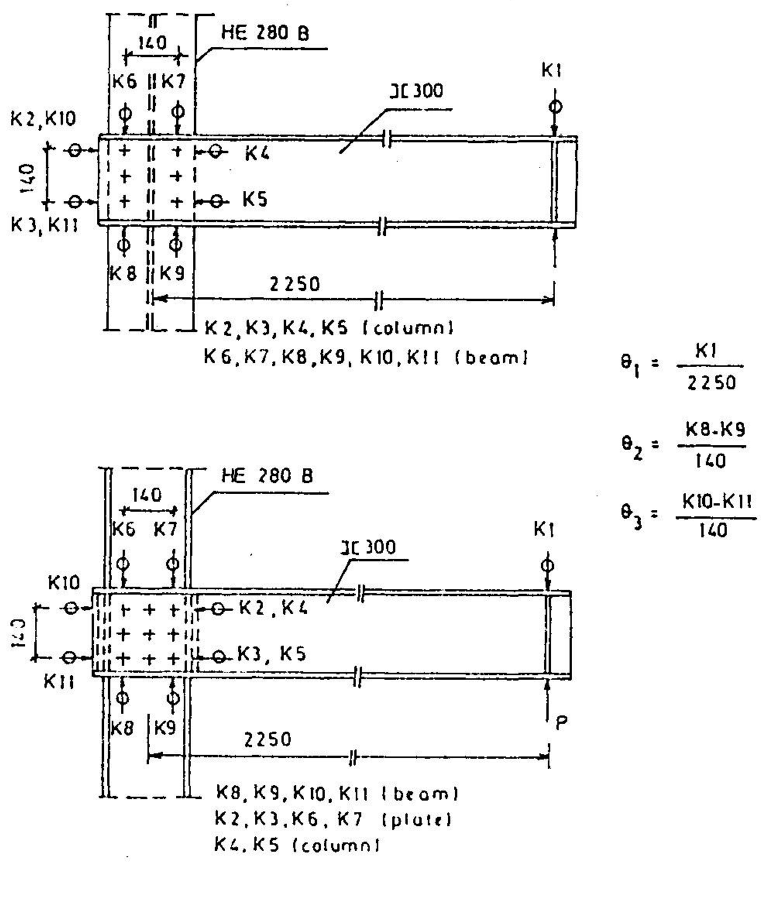


*Fig.2. Test specimens*

## 3. Test Procedure

Due to the acting load (V), (L) being the distance from the edge to the connection, a shearing force equal to the value of (V) and a bending moment equal to the value of (VL) occured at the connection. The load (V) was increased step by step and in each step of the loading, displacements were measured by comparameters. After each displacement, the load was allowed to stabilize until there was no further movement. First the load was increased progressively until the bending moment reaches sixty percent of the elastic moment resistance of the cross-section. Then the samples were unloaded. The specimens were loaded and unloaded in the same way for a second time. In the third loading case, the load was increased until the connection collapses. The strains were recorded during the tests for all levels of the load by strain-gages mounted at the edges of the bolts.

## 4. Behaviour and Failure of Connections

The study of the bolt holes after completion of each test indicated that all of the connections failed by bolt hole yielding. From the observation of the bolt holes, it is found that the centre of rotation of the connection is located nearly on the centre of gravity of the bolts. The design moment of the connection obtained from the test results is equal to the moment value obtained from the evaluation of the moment resistance using the method given in Ref [1]. The centre of rotation at the connection can approximately be determined from the strain distribution. As seen from the examination of the bolt holes, the most deformed bolt hole is the one which located at

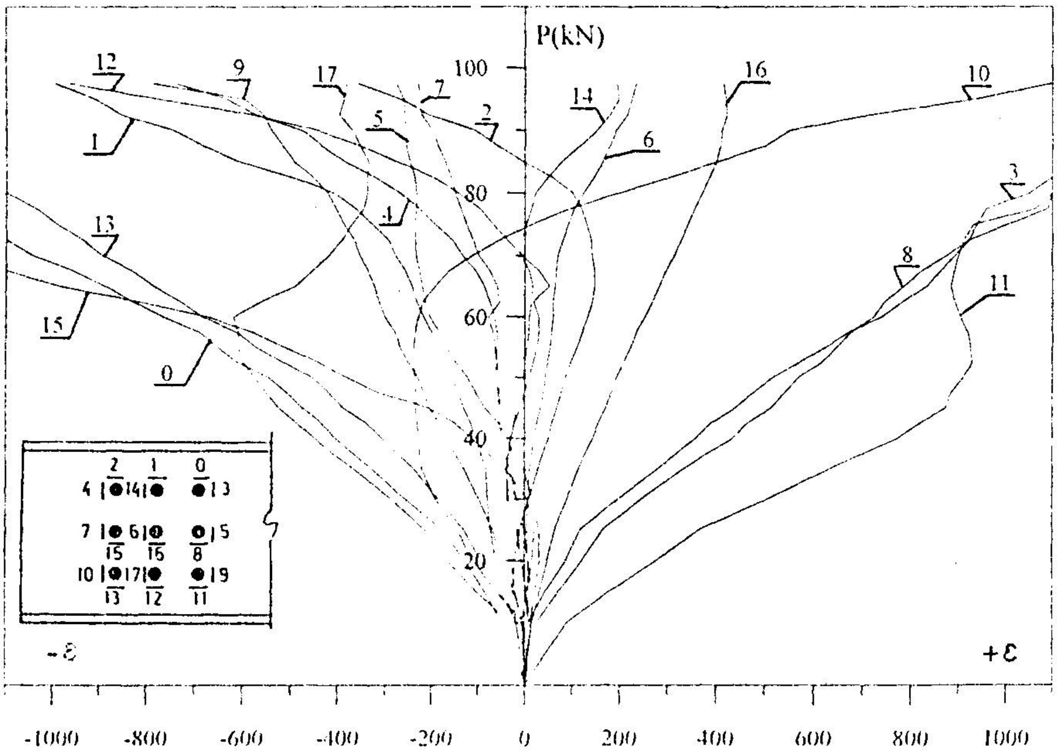


*Fig.3. Strain-gages records*

the farest distance from rotation centre of the connection. No deformation has been observed for the bolt hole located at the centre. The load-deformation relationships at the edges of the holes were obtained from the records of strain-gages during the tests. One of the records for strong axis connections is given in Fig.3.

In the strong axis connections, the fillet welds showed no sign of yielding during the tests.

An evaluation of the test datas showed that, in general, a plot of applied moment versus to the rotation of the beam gave the best quantative description of the connection behaviour. Location of the comparameters used in the tested samples is shown in Fig.2. The rotation ($\phi$) of the beam is obtained from dividing the difference between the measured and calculated deflection of its vertical displacement by the length (L). The calculated deflection is occured due to the bending and shear of the beam. The moments and corresponding rotations of the connections obtained from three test specimens for strong and other three for weak axis connections are plotted on two separate (M-$\phi$) diagrams. Then a continuous curve is plotted for each type of the connection using these tests prints (Figs.4a-4b). As seen from Figs.4a-4b, unlike the strong and weak axis connections designed by I profiles, there is not much difference between the diagrams of such the strong and weak axis connections, designed by double channel profiles, Ref [2].

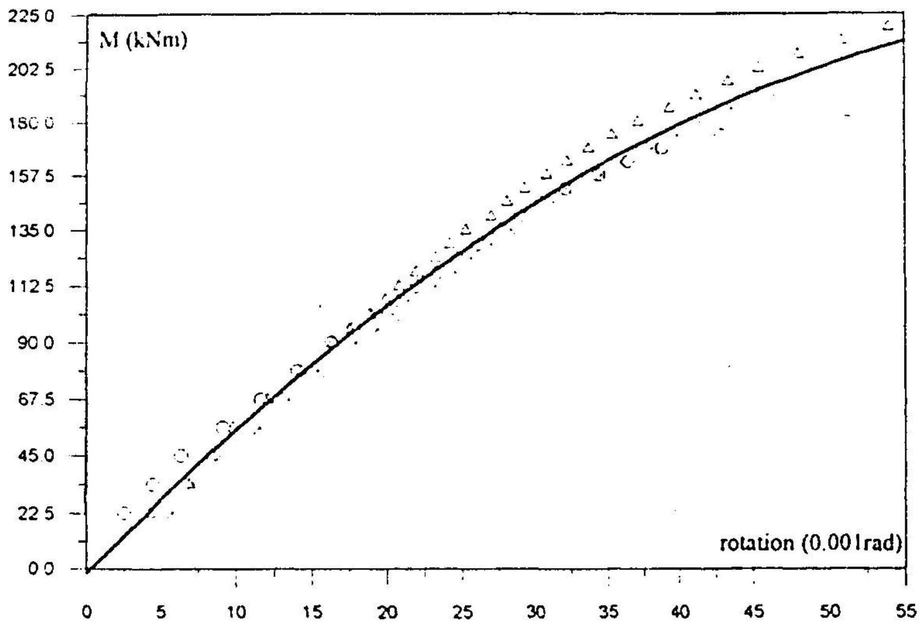


*Fig.4a.* (M-$\phi$) *diagram of the strong axis connections*

The rotations of the joints evaluated using the corresponding comparemeters are seen in Figs.5a-5b. It is seen from these diagrams that the column is not effective on the behaviour of the tested connections. The bolt hole yielding is the most effective component on the moment resistance of all the connections so, there is not much difference between the diagrams of strong and weak axis connection types tested in this study.

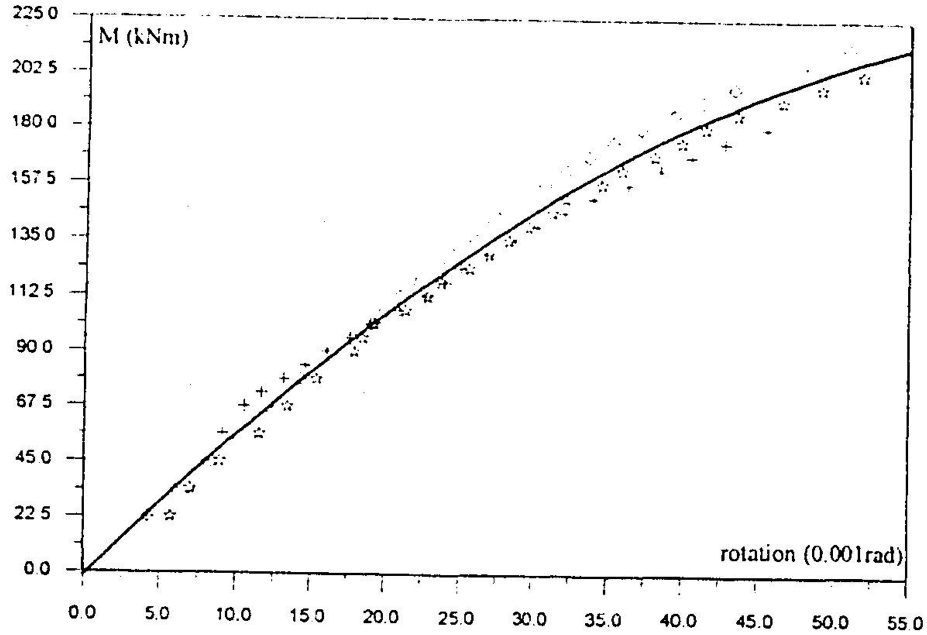


*Fig.4b.* (M-$\phi$) *diagram of the weak axis connections*

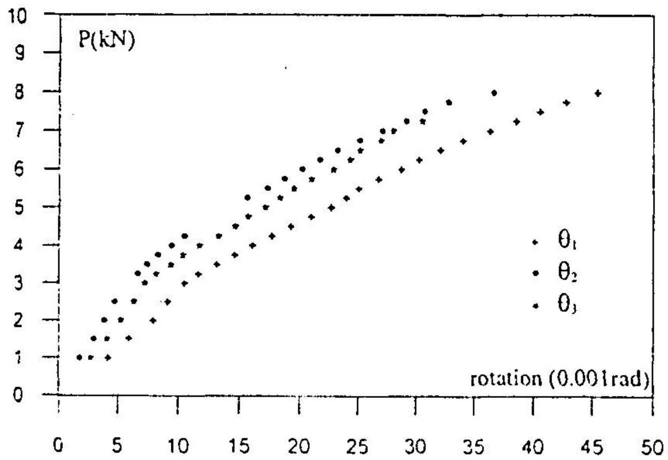


*Fig.5a. Rotation of the strong axis joints*

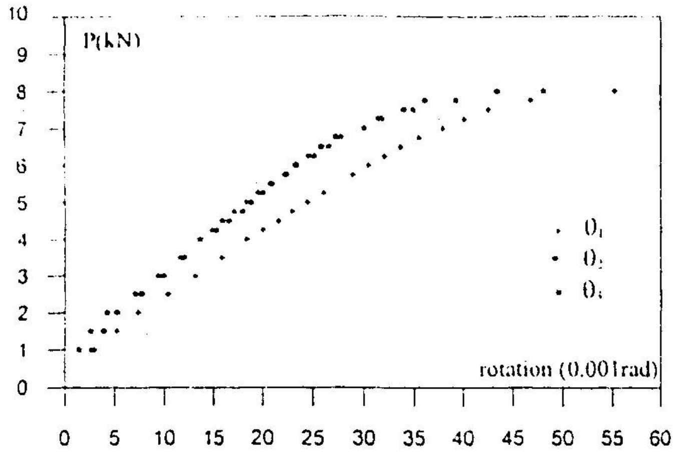


*Fig.5b. Rotation of the weak axis joints*

Fig.6 shows the moment-rotation behaviours of the typical steel connections. The classification boundary according to Eurocode 3 is included in the figure, Ref [3]. Using (M-$\phi$) diagrams, $\bar{m}$ and $\bar{\phi}$ values are obtained for the two types of the tested connections and plotted on the same diagram. The behaviour of tested connections are situated somewhere between the pinned and semi-rigid boundary lines.

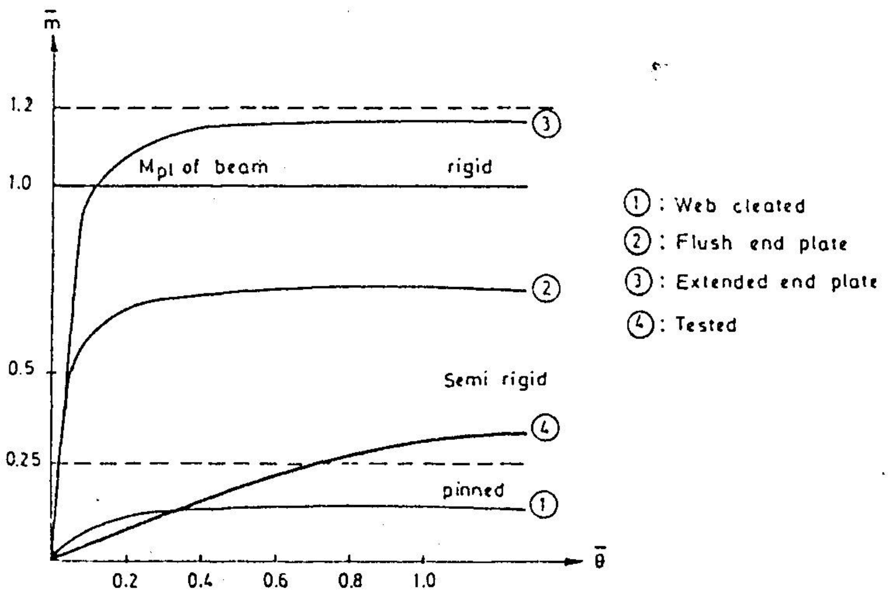


*Fig.6. ($\bar{m}$-$\bar{\phi}$) diagram*

The non-dimensional parameters used in this diagram are, Ref [4]

$$\bar{m} = \frac{M}{M_{pl\,Rd}}, \qquad \bar{\phi} = \frac{EI\phi}{LM_{pl\,Rd}}$$

Here,

$M_{pl\,Rd}$ : the plastic moment capacity of the beam
E : the modulus of elasticity of steel
I : the moment of inertia of the cross-section of the beam

The higher values of $\frac{M}{M_{pl\,Rd}}$ ratios and connection stiffness can be achieved if the webs of the profiles are strengthened.

## 5. Conclusions

In the (M-$\phi$) diagrams, the relations between the values of moments and rotations are increasing linearly up to the yield moment level and then tending to curve until the connection collapses.

There is not any given procedure in the literature to evaluate the moment resistance of this kind of connections to compare by test results. The most similar connection to the tested ones in the study is the web cleat connections composed by I profiles. So the tested connections compared by these kind of connections and the classification conditions given in Eurocode 3 for cleat connections are used to decide the type of the connections used in the study.

Column does not affect to the moment resistance of the connection; the bolt hole yielding is the most effective component on the behaviour of the joint. To obtain a greater moment resistance for the connection, the webs of the profiles should be strengthened.

As mentioned above, the tested connection has a very low strength and stiffness and connections are situated somewhere between the pinned and semi-rigid boundary lines. When a composite beam is used in the connection the strength and stiffness of the specimen is obviously enhanced.

In the practice, this kind of beams are mainly used as continuous beams. So, the values of the rotations encountered in the tests with cantilever samples show greater values compared by the beams used in practice.

This study is to be intended as the starting point of a general research project aimed to extend for the connections in which beams designed by double channels.

## References

[1] Yardımcı, N., Yorgun, C., Arda, T.S., Eyrekci, O., Bayramoğlu, G., "Tests on Full-Scale Steel Bolted Beam-to- Column Connections", Structural Assessment the Role of Large Scale and Full Scale Testing, City University, London,England, July 1-3, 1996.

[2] Yardımcı, N., Yorgun, C., Arda, T.S., "An Experimental Investigation on the Behaviour of Semi-Rigid Welded Beam-to-Column Connections", Proc. of the Nineth National Congress of Mechanics, Ürgüp, Turkey 4-8 September, 1995, pp.748-757.

[3] Eurocode 3, Design of Steel Structures, Part 1.1, General Rules and Rules for Buildings, ENV, 1993-1-1.

[4] Yu, C.H., Liew, J.Y.R., "Design of Semi-Rigid Frames Using Second-Order Inelastic Analysis, Proc. of the Third International Conference on Steel and Aluminium Structures, ICSAS, Istanbul, Turkey, 1995, pp.351-358.

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# Use of threaded studs in joints between I-beam and R.H.S. column

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Didier Vandegans (1971) got his Civil Engineering Degree at the University of Liège in 1994. His main domain of activity is the semi-rigid connections.

## Summary

This paper shows that the component method, described in Annex J of Eurocode 3 can be applied in the case of studded joints, by using different existing models. Numerical simulations make it possible to determine the deformation of the bended face of the section, on which the studs are fixed. Therefore the curves of behaviour of these joints can be drawn and compared with the experimental curves.

## 1. Introduction

The concrete-filled R.H.S. (Rectangular Hollow Section) technique has many advantages in the building domain. The bearing load, the stiffness of joints between beams and columns and the fire stability are increased, the floor space required is smaller, the aesthetic is improved, the maintenance is easier, and in comparison with concrete column, no shuttering is needed.

The stiffness of joints with steel beams or composite steel-concrete beams, and their resistance to bending moment are relatively high. In order to calculate the resistance of different joint elements, some codes or models are developed. However, no model exists to determine the deformability of the face of the hollow section when it is in bending.

The aim of this paper is to show that, with the rules given in Annex J of Eurocode 3 (EC3) [1], based on the component method, with different design models and with numerical simulations, it is possible to determine the characteristics of the behaviour curve of joints between a I-beam and a concrete filled R.H.S. column.
Furthermore, experimental tests show that important membranar effects develop within the face of the hollow section. Preliminary studies are now carried out to see if this effect can be neglected with accuracy.

## 2. The stud technique.

The presence of concrete within the hollow section make it impossible to fix a connection element with bolts on the face of this section. The stud technique is useful to solve this problem. This technique consists in welding with the help of a special gun, a threaded stud on the face of the section on which the connection is to be realised [3]. The other elements of the connection are fixed to the studs with nuts, as done for classical bolts. The studs are then subjected to traction and shearing, as for usual joints. The studs to be used are threaded studs with reduced base. So, the welds have approximately the same diameter as the threaded part.

## 3. Test results.

Eight tests have been performed in the laboratory. Four different joint configurations were considered (web cleats, extended and flush end-plates, flange cleats). The full description of the experimental program is given in [5].
The experimental curves, as well as the rigid and pinned classification domain, are given in figure 1. For the classification, a braced frame is considered with a beam span equal to 6 meters.

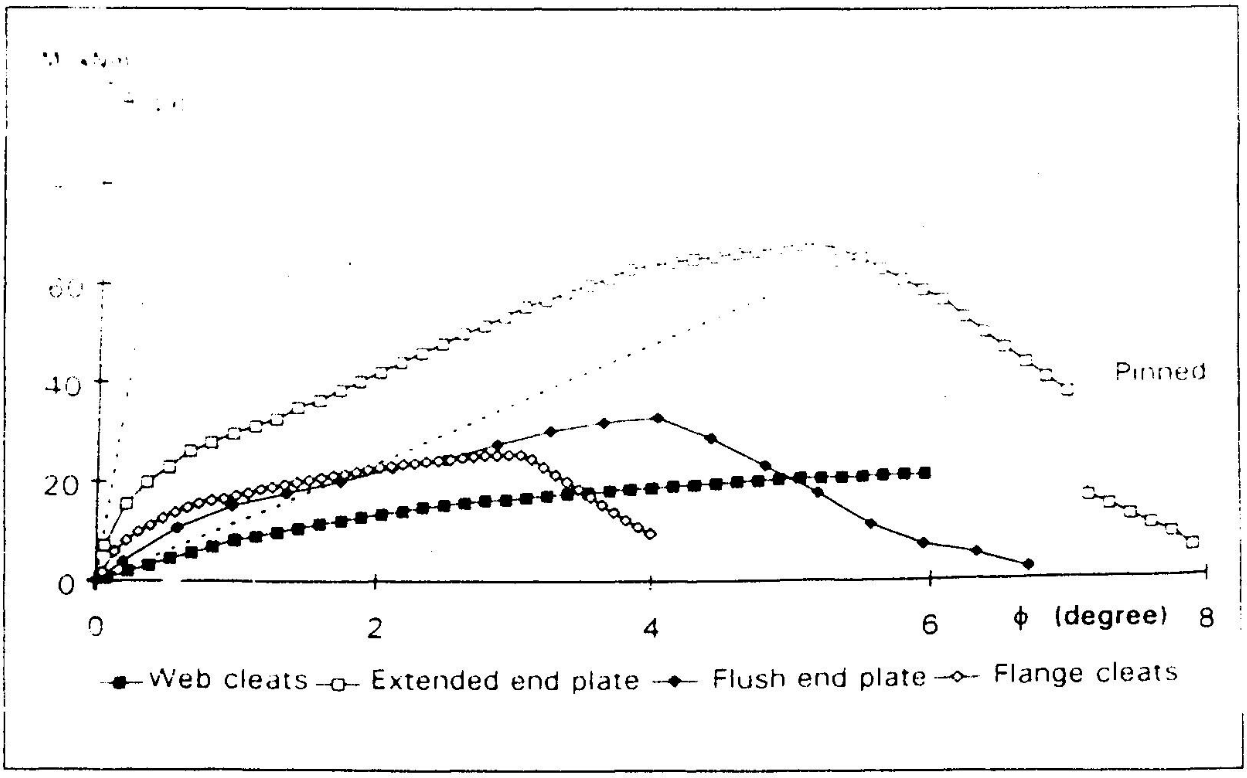


*Fig. 1. Experimental curves and classification*

# 4. Method of calculation used.

## 4.1. Annex J of EC3 method.

For the design of the joint, the approach described in Annex J of Eurocode 3 is used. This is based on the component method, which considers a joint not as a unit, but as a set of individual components each with its own strength and stiffness. EC 3 Annex J allows to calculate the characteristics of the following components :

- end-plate in bending;
- flange cleats in bending;
- flange of the beam in compression;
- web of the beam in tension;
- flange of the beam in bearing;
- bolts in shearing;
- studs in tension.

## 4.2 Naveau model.

An earlier research carried out by NAVEAU in CRIF in Belgium [4] has established some design rules for the resistance of the face of the rectangular hollow section. These rules cover the following modes of failure :

- shearing of the face of the section :

$$N_{max} \leq 0{,}95.\pi.d.t.\frac{f_{yt}}{\sqrt{3}}\Big/\gamma_{mo} \qquad (1)$$

- lamelar pull out :

$$N_{max} \leq 0{,}95.\frac{\pi\, d^2}{4}\, f_{yt}/\gamma_{mo} \qquad (2)$$

$f_{yt}$ is the yield strength of the hollow section, $t$ is the thickness of the face, $d$ is the nominal diameter of the studs.

## 4.3 Gomes model.

In order to calculate the resistance of the R.H.S bended face, the so-called "Gomes Model" is used. Gomes has studied the weak axes beam to column joints and has deduced a model to design the column web. A part of this model predicts a local failure of the column web due to the bolts in tension. He substitutes the plastic mechanism of the web by an equivalent rectangle of b x c dimensions as shown on figure 2.

The local resistance of the web is :

$$F_{local} = M_{pl}.\alpha.k \qquad (3)$$

in which $M_{pl}$ is the plastic moment of the web.

$$M_{pl} = \frac{1}{4}\, t_w^2\, f_{yw} \tag{4}$$

$$\alpha = \frac{4}{1 - b/L}\left(\pi\sqrt{1 - b/L} + 2c/L\right) \tag{5}$$

$$k = \begin{cases} 1 & \text{if } (b+c)/L \geq 0.5 \\ 0.7 + 0.6\,(b+c)/L & \text{if } (b+c)/L \leq 0.5 \end{cases} \tag{6}$$

The validity range of this model is as follows :

$$\begin{cases} b/L < 0.8 \\ 0.7 \leq h/(L-b) \leq 10 \end{cases} \tag{7}$$

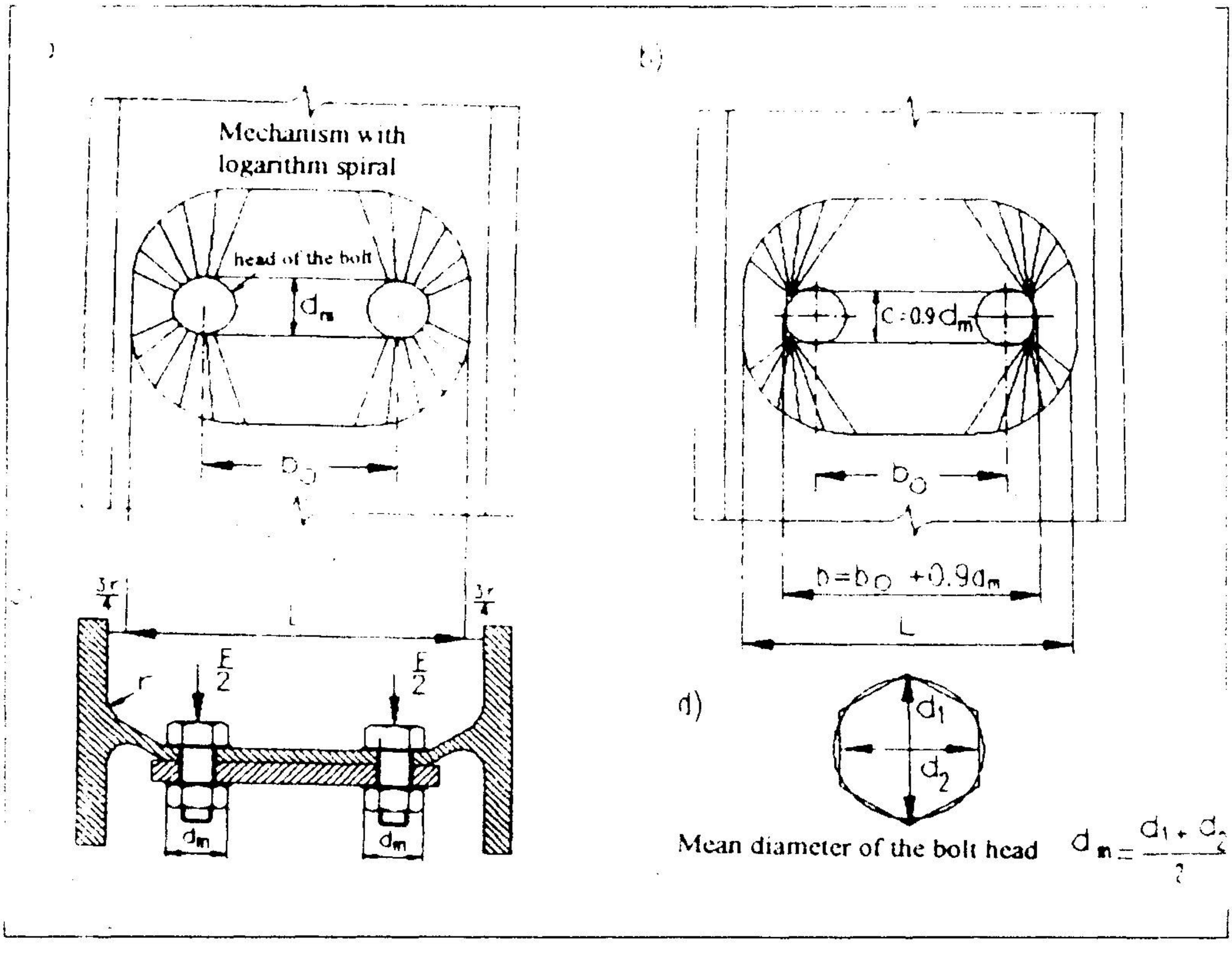


*Fig. 2. Local mechanism for bolted connections:*
*i) yield line pattern*
*ii) yield line pattern for equivalent rectangle b x c*
*iii) section view*
*iv) mean diameter of the bolt head ( or nut )*

This model can be extended to the bended face of the rectangular hollow section. If two stud rows are present, the model might also be extended, by taking group plastic mechanism into account.

### 4.4 Numerical simulations.

So far, no model exists to evaluate the deformability of the bended face of the hollow section. Numerical simulations, with shell elements, have been done with a non-linear finite elements program (FINELG [9]), in order to find a numerical value of the deformation.

## 5. Hypotheses.

During the calculation, the following elements have been considered:

1. The presence of concrete stiffens the sheared part of the connection very strongly and increases the compression zone resistance of the joint considerably. Because of lack of information, it is not possible to calculate with a high degree of precision the shear and compression resistance and the corresponding deformations. But it is obvious that the resistances which would have been found are largely higher than those of the other components, and in addition, the deformability of these zones is extremely low.

2. Lateral faces of the section are submitted to tensile forces, acting on a defined length corresponding to the efforts diffusion length. The value of this diffusion length cannot be determined with a high degree of precision. However, the "web" of the R.H.S. may be considered as largely over-dimensioned in comparison with its equivalent in a I-beam. The reason is that, in this case, the faces of the section have the same thickness, as opposed to the difference in thickness between the flanges and the web of an I-beam. In addition, adherence may occur between the lateral faces and the concrete. Therefore, this source of deformation is supposed to be negligible. This hypothese is confirmed by the numerical simulations.

3. When pulled out by the studs, the face of the section comes out of its plane. The angles of the rectangular section are rigid, and thus, while the section is empty, the deformation of one induces a deformation of the other lateral faces, as shown on the figure 3. The presence of concrete within the hollow section prevents these lateral faces from deforming and therefore reduces the deformation of the face connected to the studs when they are submitted to traction. This fact stiffens this component, and consequently, during the numerical simulations, the face of the section is modelised by means of a infinitely long plate, imbedded on 2 faces and subjected to two "concentrated" forces. The reason why a numerical simulation has to be done is that, to our knowledge, no analytical solutions are available.

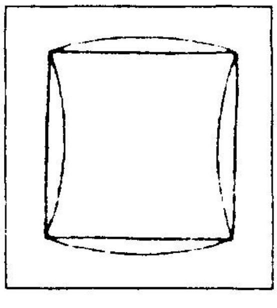

*Fig. 3. Free out of plane deformation of the section's faces*

## 6. Results of the calculation.

Without giving all the details, which can be found in [5], the results of the stiffness and strength calculations are given in table 1, as well as the corresponding mode of failure.

| | $M_{Rd}$ (in kN.m) | $M_u^{exp}$ (in kN.m) | Failure mode | $S_{ji}^{calc}$ (in kN.m/deg) |
|---|---|---|---|---|
| Extended end-plate | 35.2 | 67.10 | Face of the section | 78.97 |
| Flush end-plate | 16.5 | 33.15 | " | 34.85 |
| Flange cleats | 17.7 | 25.50 | Cleat | 37.23 |

*Table n° 1 : Stiffness and strength calculation results*

Figure 4 shows the experimental curves and their equivalent given by calculation.
The experiments show that important membranar effects develop in the bended face of the column. Because of the importance of these effects, the ratio between the initial and secant stiffnesses, $S_{ji}$ and $S_{js}$, as well as between the elastic moment $M_e$ and the design moment resistance $M_{Rd}$, have to be slightly modified.

The formulae given in EC 3 Annex J are as follows :

$$S_j = \frac{E.z^2}{\mu.\sum_i \frac{1}{k_i}} \qquad (8)$$

where
$E$ = the Young modulus;
$z$ = the lever arm of the external forces;
$k_i$ = the stiffness of the individual components;
$\mu$ = the stiffness ratio.

$$\mu = \left[1.5\,\frac{M_{j,Sd}}{M_{j,Rd}}\right]^{\psi} \qquad (9)$$

and $\psi$ = 2.7 for welded and bolted end-plate joints;
= 3.1 for flange cleated joints.

For the determination of the secant stiffness $S_{js}$, $M_{j,Sd}$ equals $M_{j,Rd}$ and thus

$$\mu = (1.5)^{\psi} \qquad (10)$$

$$\text{and } S_{js} = S_{ji} / \mu \qquad (11)$$

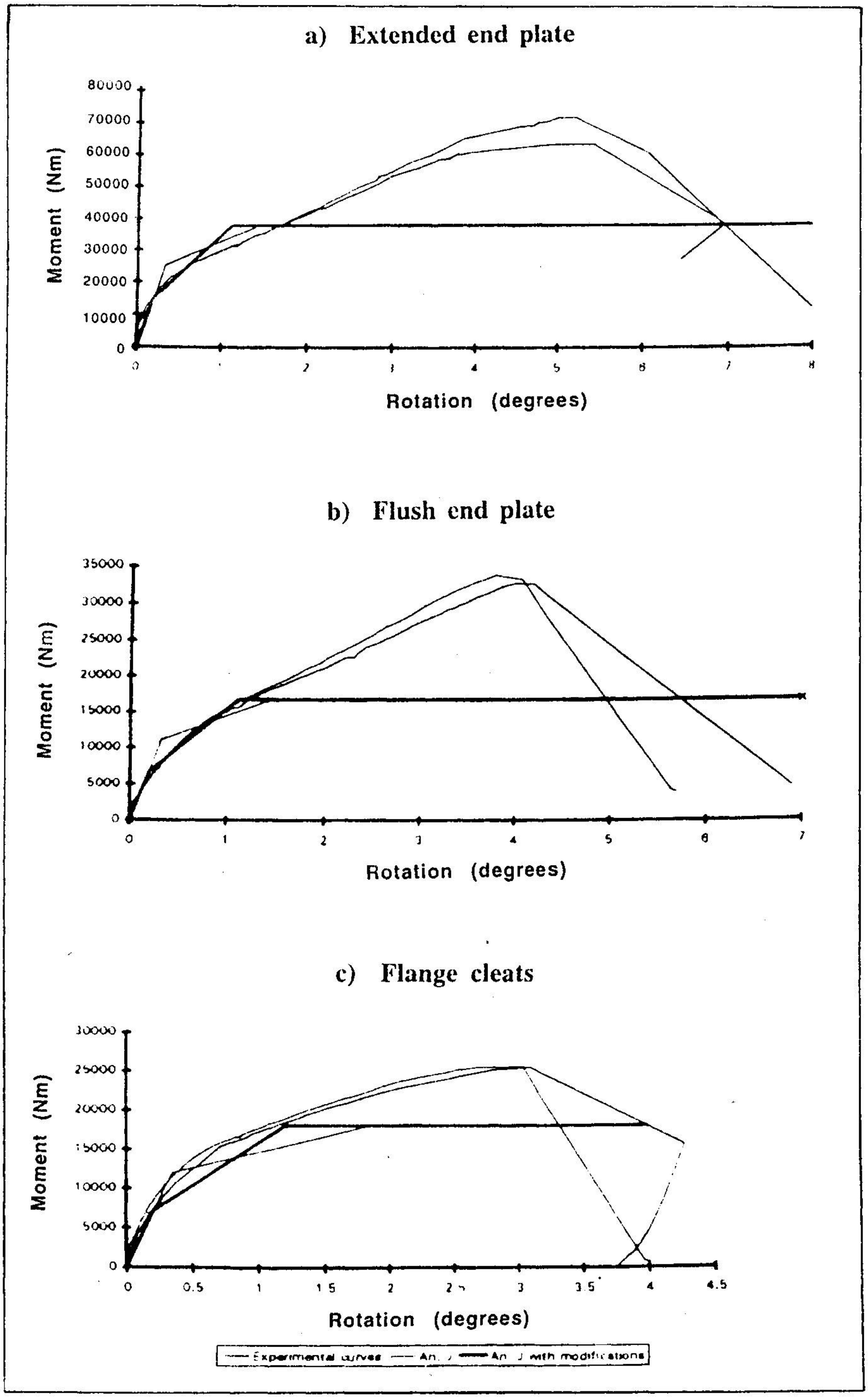


*Fig. 4. Experimental curves and comparison with the curves calculated according to EC3 Annex J (with and without modifications)*

Comparing the value of $S_{js}$ (calculated according to EC3 Annex J and deriving from the value of the initial stiffness $S_{ji}$ deduced from the calculation and numerical simulations) and the measurements on the experimental curve, leads to the results given in table 2:

| | $S_{ji}$ (in kN.m/deg) | $S_{js}^{EC3}$ (in kN.m/deg) | $S_{js}^{exp}$ (in kN.m/deg) | $\mu^* = S_{ji} / S_{js}^{exp}$ |
|---|---|---|---|---|
| Extended end-plate | 78.97 | 26.43 | 24,59 | 3,21 |
| Flush end-plate | 34.85 | 11.66 | 14.36 | 2.43 |
| Flange cleats | 37.23 | 10.59 | 16.80 | 2.21 |

*Table 2 : Stiffness coefficients*

As a first approximation and to keep a simple solution, it's possible to give the following ratios :

$$M_e = M_{Rd}/2,5 \qquad (12)$$

$$S_{js} = S_{ji}/\mu \qquad \text{with} \qquad \mu = 2.3 \qquad (13)$$

Figure 4 shows the tri-linear curves drawn with these values.

## 7. Influence of the membranar effect on frame response.

As already said, membranar effects develop in the bended face of the column section. The ultimate moment $M_u$ is therefore largely higher than the design moment resistance $M_{Rd}$. However, even if this sort of connection have a large resistance reserve, the rotations which correspond to the ultimate value $M_u$ are relatively high and not acceptable for the serviceability limit state. $M_u$ cannot be considered for the design [7]. Nevertheless, the rotation capacity is very high.

It is important to study the influence of neglecting this so-called joint post-limit stiffness on the frame response [8]. In a plastic analyse, a yield plateau is considered, but the actual behaviour of the joint is different. That leads to a different internal forces repartition within the frame.

That also means that the load factor at the ultimate state will be higher in the reality than the one given by a plastic analyse, as well as the actual moment in the joint for a given external load.

If the structure is submitted to the ultimate loads, derived by a plastic analyse, the joint rotation is assumed to be equal to $\varnothing_l$ and the corresponding moment in the joint is equal to $M_{Rd}$, in case of a plastic mechanism collapse mode for the beam (see figure 5). In fact, for the same load level, because of the actual internal forces distributions, the actual joint rotation is equal to $\varnothing^*$ and the corresponding moment to $M^*$.

In EC 3 Annex J, a rather limited post-limit effect mainly due to strain hardening is safely considered. The ratio between the post-limit and initial stiffness considered in the code is about 1/50. When membranar effect occurs, the experimental curves shown in figure 4. give a ratio close to 1/6, which is quite different.

The influence of the post-limit effect is important for the design of the welds between the beam and the column or end-plate. EC 3 Annex J states that they have to be design for 1.4 $M_{Rd}$ (or for the plastic design resistance of the beam flange).
If the ratio between the actual moment $M^*$ in the joint and $M_{Rd}$ is higher than 1.4 for braced frames, the welds design is unsafe for this particular connections and a brittle failure might occur. It is too early to give results, but studies are under developments.

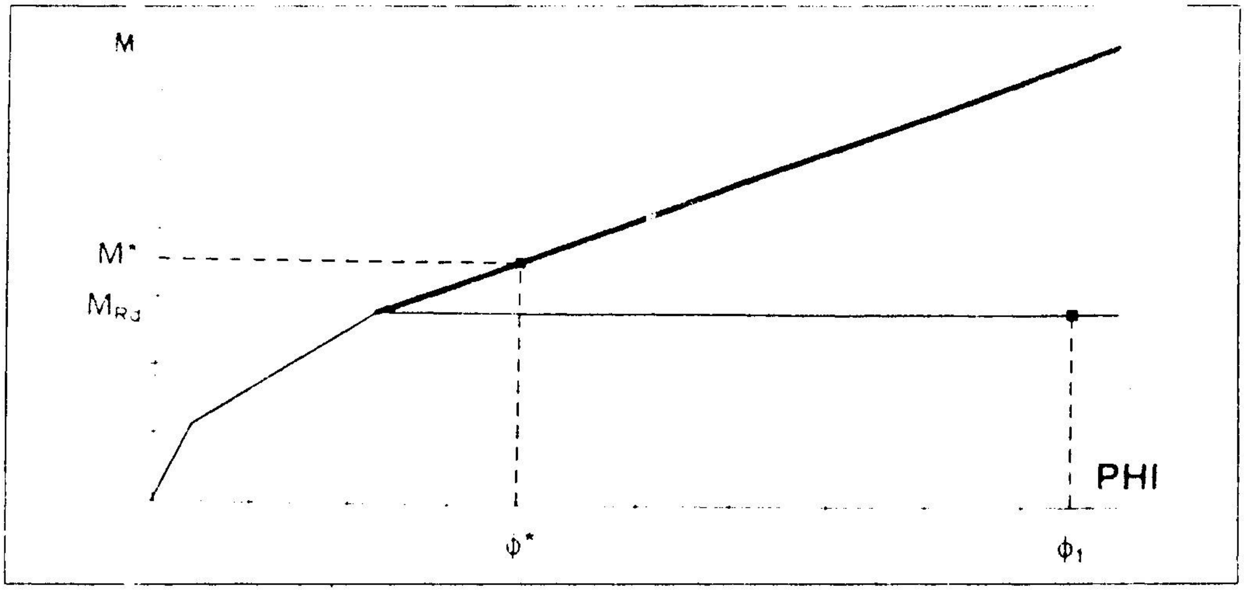


*Fig. 5. Variation of the joint response with the post-limit stiffness*

## 8. Conclusions.

The stud technique makes it easy to build a connection between a I-beam and a R.H.S. column filled with concrete. Reliable design models exist to determine a design value of the joint resistance.

The component method described in EC 3 Annex J can be extended to this sort of connections, without any difficulties, what is of great interest. However, small changes have to be made in the assembly procedure of the different components.

The only component for which the deformation cannot be derived analytically is the bended face of the column section. Numerical simulations have shown that it is possible to find the initial stiffness of the joint with a good degree of precision. The deformations of the lateral faces of the section can be neglected.

Important membranar effects occur in the face of the column. Studies are nowadays under development to ensure that it is safe to neglect that particular effect.

## 9. References.

[1] EUROCODE 3 : Design of steel structure, part 1-1; General rules and rules for buildings, ENV 1993-1-1, 1992. Annex J : Joints in building frames, 1994.

[2] Gomes F.C.T., Jaspart J.P., Maquoi R. : COST C1 semi-rigid behaviour : Behaviour of minor axis joints and 3-D joints - Proceedings of COST C1 workshop, Prague, October 1994.

[3] Jaspart J.P. : Le boulon dans la construction métallique, perspective d'avenir, CRIF, 1994.

[4] Maquoi R., Naveau X., Rondal J. : Beam-column welded stud connections. Journal of constructional steel research, vol. 4 n° 1 - CRIF, Belgium, 1984.

[5] Vandegans D. : Liaison entre poutres métalliques et colonnes en profils creux remplis de beton, basee sur la technique du goujonnage (goujons filetés). CRIF, MT 193, 1995.

[6] Vandegans D. : Application of the component method according to EUROCODE 3 to connections with threaded studs. COST C1/WD2/95-21, 1995.

[7] Jaspart J.P. : Design and ultimate resistance for joints ; COST C1/WD2/95-18, 1995

[8] Vandegans D., Jaspart J.P. : Influence of the post-limit stiffness of joints on the frame behaviour (first draft) : COST C1/WD2/96-03, 1996.

[9] FINELG, Non Linear Finite Element Analysis Program, version 6.2, Users Manual, Department MSM, University of Liège, BEG Design office, Belgium, 1994.

# Testing of Simple Flowdrill Connections

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## Summary

A series of tests has been conducted on joints where the connected members are open section beams and closed formed columns (SHS). The reported tests were specifically designed to investigate the use of flowdrill connectors for use in simple multi-storey frames. The details allow the use of traditional endplates, frequently used with open section columns, whereby the beam endplate is bolted directly to the face of the column section.

## 1. Introduction

The traditional steel framed building usually incorporates open sections for both the beam and column members. The use of open sections for columns where axial loads are predominant is recognised as being structurally inefficient compared with closed formed members (SHS-structural hollow sections) as the column section is usually determined by the buckling resistance about the weaker minor axis.

The success of steel framed buildings in maintaining market dominance in the UK, stems in part from the ease with which site bolted connections of the pre-fabricated elements of the steel frame can be achieved. A major problem which results from the use of SHS columns is the difficulty of devising a satisfactory joint detail. Previous examples of use of tubular columns have often been associated with areas of seismic activity where rigid and full strength joint details are normally adopted. This often involves site welding the open section beams directly to the face of the column which is then further reinforced by the addition of an internal or external diaphragm plate welded to the column and to the top flange of the beams. The use of welding in this manner greatly increases the fabrication costs and frequently outweighs the original benefit brought about by the use of SHS columns, especially in areas of non-seismic activity where simple site bolted connections are used in conjunction with braced frame construction.

### 1.1 Blind bolt connectors

A useful alternative to site welding is the use of blind bolt connectors which avoids the need to weld obtrusive fittings to the column exterior and enables traditional endplates to be site bolted from the outside of the SHS column. To date there are three systems (excluding the welded threaded stud) commercially available; Lindapter Hollo-bolt[1], Huck Ultra- twist[2] and Flowdrilling[3]. The first two are mechanical in operation which allows the column to be drilled by plain but oversized holes. Special bolts are then inserted through the endplate and drilled column. The tightening process expands the back of the bolts and clamps the endplate and column face together. If the wall thickness of the SHS is greater or equal to 16 mm, then the section can be drilled and tapped to accept ordinary structural bolts up to 20 mm in size. However the structural efficiency of the SHS column usually results in the selection of the largest practical plan size resulting in relatively thin walls which do not have adequate thickness to tap a thread into the section. In this case flowdrilling may be employed as an alternative to the mechanical bolts mentioned previously.

Flowdrilling is a system which locally increases the wall thickness of the closed section by rotating a tungsten carbide drill bit at high speed on the face of the tube. This heats the section locally by friction allowing the drill bit to be forced through the wall of the section to form a conical lobe on the inside of the tube which is of sufficient depth to allow a thread to be cold formed into the section allowing ordinary structural bolts to be used.

## 2. Experimental flowdrill research

A programme of flowdrill joint tests which incorporated the welded endplate detail has recently been completed. The endplates investigated were the flexible (partial depth- PD) endplate, the flush endplate (FE) and finally the extended endplate (EE). This selection enabled the full range of joint behaviour to be investigated, from assumed simple joint details

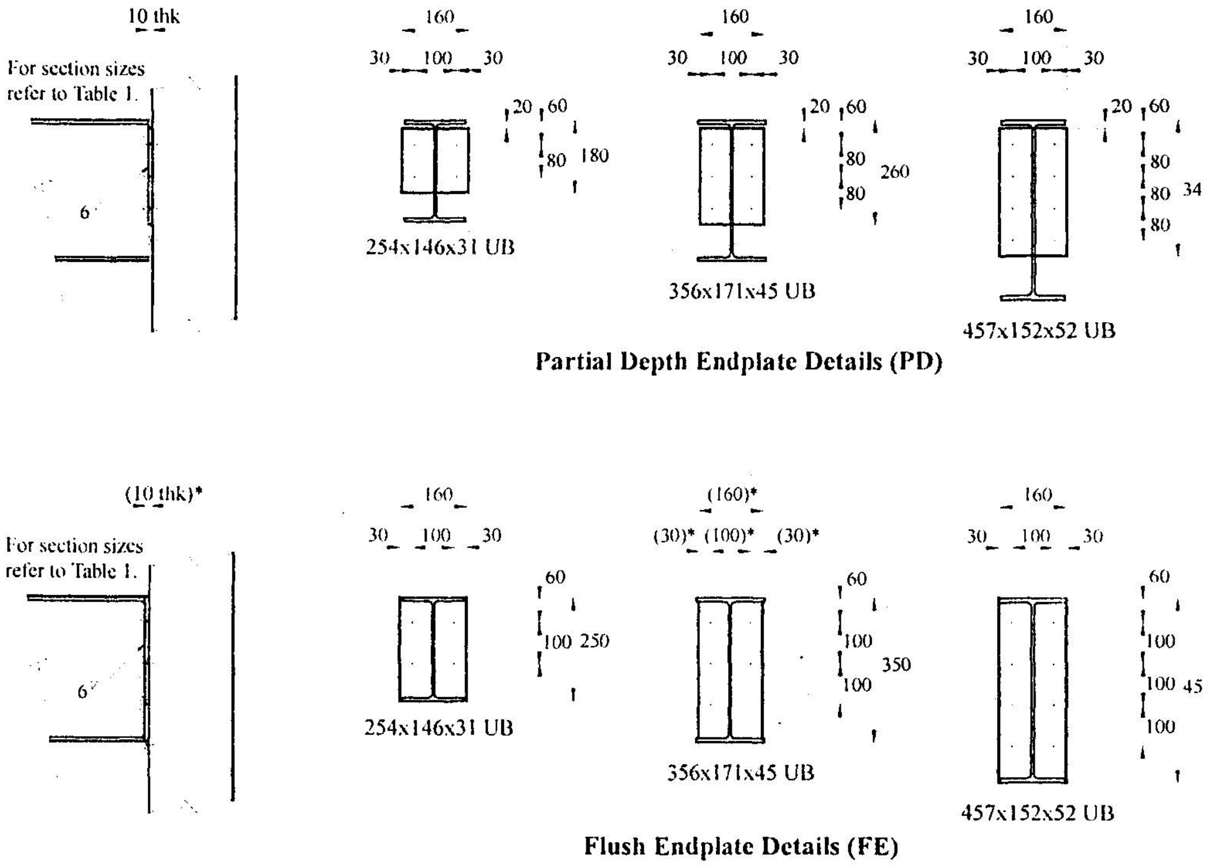
| Test Reference | Column Section | Column Yield Strength ($N/mm^2$) | Beam Section | Endplate Type | Endplate width and Thickness (mm) | Bolt Cross Centres (mm) |
|---|---|---|---|---|---|---|
| **PD-254-100/8** | 200x200x8.0 SHS | 318 | 254x146x31 UB | Flexible | 160x10 | 100 |
| **PD-356-100/8** | 200x200x8.0 SHS | 313 | 356x171x45 UB | Flexible | 160x10 | 100 |
| **PD-457-100/8** | 200x200x8.0 SHS | 313 | 457x152x52 UB | Flexible | 160x10 | 100 |
| **FE-254-100/8** | 200x200x8.0 SHS | 318 | 254x146x31 UB | Flush | 160x10 | 100 |
| **FE-356-100/8** | 200x200x8.0 SHS | 313 | 356x171x45 UB | Flush | 160x10 | 100 |
| **FE-457-100/8** | 200x200x8.0 SHS | 313 | 457x152x52 UB | Flush | 160x10 | 100 |
| **FE-356-80/8** | 200x200x8.0 SHS | 318 | 356x171x45 UB | Flush | 160x10 | 80 |
| **FE-356-120/8** | 200x200x8.0 SHS | 318 | 356x171x45 UB | Flush | 180x10 | 120 |
| **FE-356-100/6.3** | 200x200x6.3 SHS | 336 | 356x171x45 UB | Flush | 160x10 | 100 |
| **FE-356-100/12.5** | 200x200x12.5 SHS | 307 | 356x171x45 UB | Flush | 160x15 | 100 |

*Table 1. Flowdrill joint parameters*

(PD) to the rigid joints (EE), with the flush endplates acting in between these two extreme cases.

The aim of the work is to investigate the semi-rigid action of the joint under moment rather than the performance of the connector which has already been investigated by Sherman[4], Banks[5] and Balleria[6]. This paper presents the moment-rotation characteristics for simple joint details associated with braced frame construction, namely partial-depth and flush endplates. The flush endplate has been included within this selection in accordance with normal practice which assumes it to act as a simple shear connection even though substantial fixity and moment transfer may sometimes be provided at the column joint.

Figure 1. shows details of the test programme with joint parameters specified in table 1. All steel used in the tests was specified as grade S275 (nominally 275 N/mm$^2$ yield), subsequent coupon test results from the SHS members are also presented in table 1.




NOTES: (1) M20(8.8) bolts used throughout.
(2) Where values are bracketed thus ( )*, refer to table 1 for variations to dimensions shown.

*Figure 1. Details of tests specimens*

## 2.1 Experimental set-up and fabrication of specimens

The endplate and beam components of the specimens were fabricated in the departmental workshop. Endplates were attached to the beams with nominal 6 mm fillet welds. All the flowdrilling of the column specimens was carried out by an experienced fabrication company.

All the test specimens adopted the cantilever test arrangement with slow cyclic loading using a point load at a 1.0 m to 1.3 m leverarm (details of the test procedure together with further results can be found in France et al.[7]). Use of the cantilever arrangement simulated the joint of an edge column typically found in a frame. Such joint arrangements subject the column specimen to constantly increasing column moments when compared with the cruciform testing arrangement where some unloading may occur due to lack of symmetry. One disadvantage of the cantilever arrangement is the reduced severity of the buckling of the side walls induced by only one sided compression from the beam compression flange.

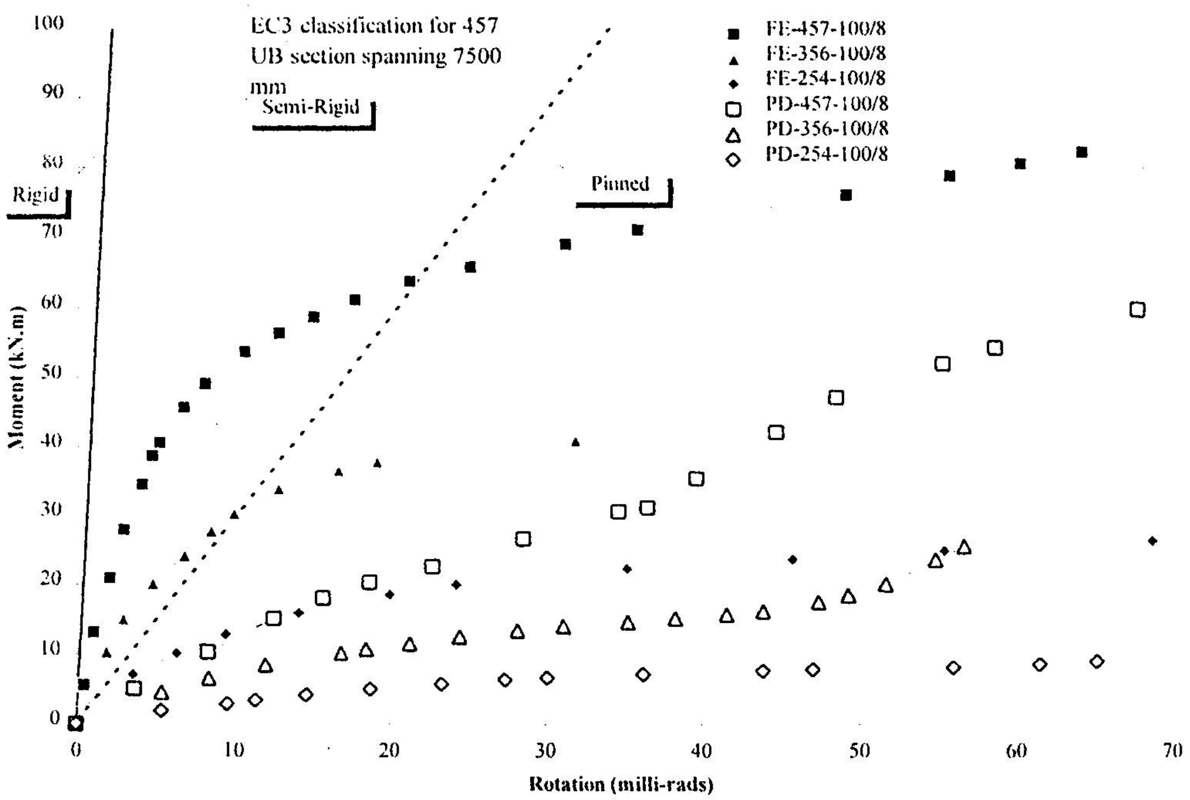



*Figure 2. Comparison of moment-rotation curves for flush and partial depth endplates.*

## 3. Moment-rotation response

Figure 2 indicates the moment-rotation responses for both the flush and partial depth endplates attached to the 254, 356 and 457 UB beam depths. All the joints showed a ductile and safe failure mechanism resulting from the top tension bolts deforming the SHS face. Although the deformation of the flowdrilled thread was significant at the end of the test, the bolts did not exhibit any signs of thread stripping.

The comparisons between the two types of endplates indicate that partial depth endplates exhibit reduced initial stiffness and moment resistance compared with their flush endplate counterparts. As the test proceeded, the moment rotation characteristic for the partial depth endplate connections changed abruptly when the rotation had increased to such an extent that the compression flange of the beam was bearing against the face of the box section, for the most flexible of these joints this occurred at a rotation in excess of 70 milli-radians. This would be unlikely to occur in practice as the beam member would fail before such large end rotations develop.

EC3 (revised annex J) gives guidance for acceptable limits to the stiffness and rotation capacity of joints. Limits to the joint capacity depend on the geometry of the frame and whether the frame is constructed as braced or unbraced. To highlight the differences between the two types of endplate the classification limits for the 457 UB beam spanning 7500 mm have been incorporated into figure 2. As seen, the flexible endplate response is suitable for pin-jointed frames whereas the flush endplate is placed in the semi-rigid category during the initial stages of the loading history.

In separate tests the wall thickness and bolt cross-centres were varied to investigate the sensitivity of the flush endplate moment rotation response to these changes. The 200x200x8 SHS column and 356 UB beam (test FE-356-100/8) with the 10 mm flush endplate was selected as the bench mark. Figure 3(a) illustrates the differences in the moment-rotation characteristics when tube wall thickness is varied between 6.3, 8 and 12.5 mm while figure 3(b) highlights the difference when 80, 100 and 120 bolt cross centres are adopted. The results show that the characteristic is highly sensitive to changes in tube wall thickness but less so to bolt cross-centres.

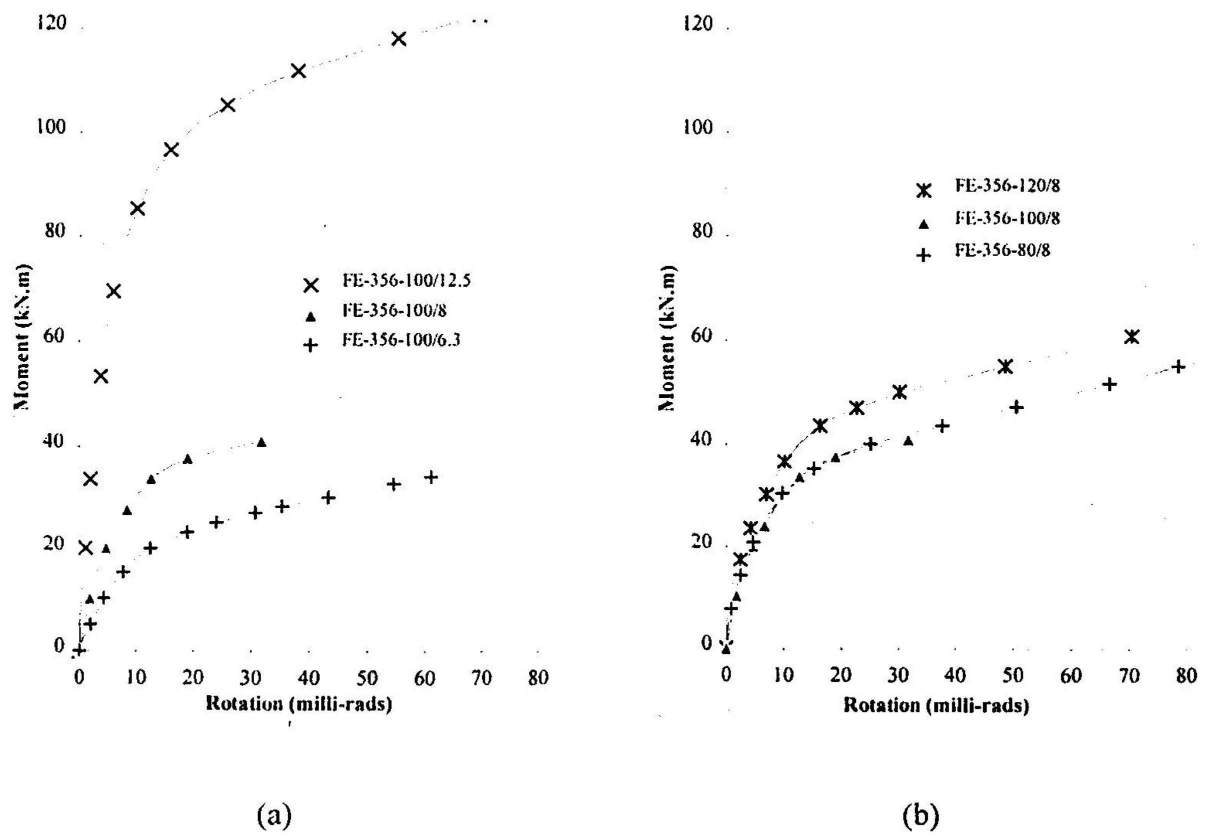



*Figure 3(a) and 3(b) comparison of Moment-rotation characteristics for variation to tube wall thickness and bolt cross centres respectively*

## 4. Effect of Endplate Bending on Moment-rotation Response

Frequently the flexibility of a joint is determined as the summation of the flexibilities of the component parts. To check the validity of this approach for joints to tubular columns, additional tests were undertaken.

### 4.1 Isolated Endplate tests

Where flush endplates are used as pin joints, the thickness of the endplate is selected to ensure that the majority of the deformation occurs in the endplate rather than in the column flange. Tests were conducted on endplates, nominally identical to those used in the flowdrill joint tests FE356-100 and FE356-120, to determine the contribution of endplate bending in the overall rotational deformation of the joint.

The two types of endplate with 100 mm and 120 mm bolt cross centres (nominally identical in construction to those used in the flowdrill joints) were each tested by either bolting the endplate directly to a rigid base or by testing the endplate with packs inserted between the flanges and the rigid base as shown in figure 4(a) and 4 (b). Packs used in this way allowed the edge of the endplate to be either restrained or unrestrained. Figures 5(a) and 5(b) show the moment rotation results of the tests for the 100 mm and 120 mm endplate bolt cross-centres respectively.

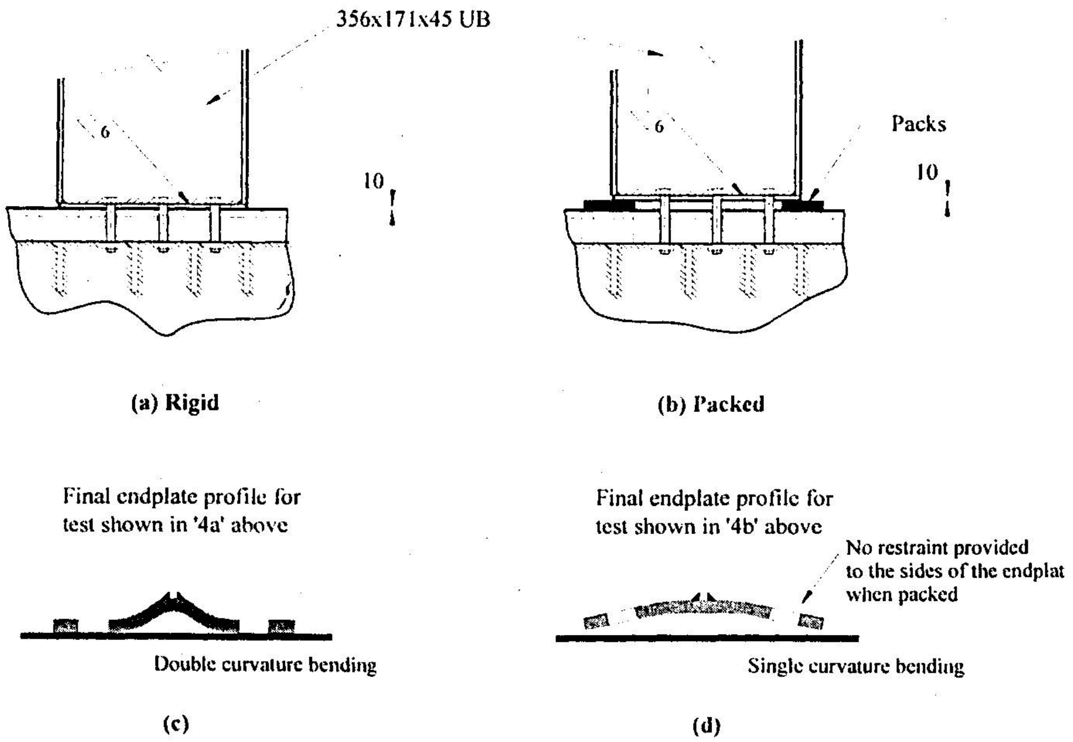
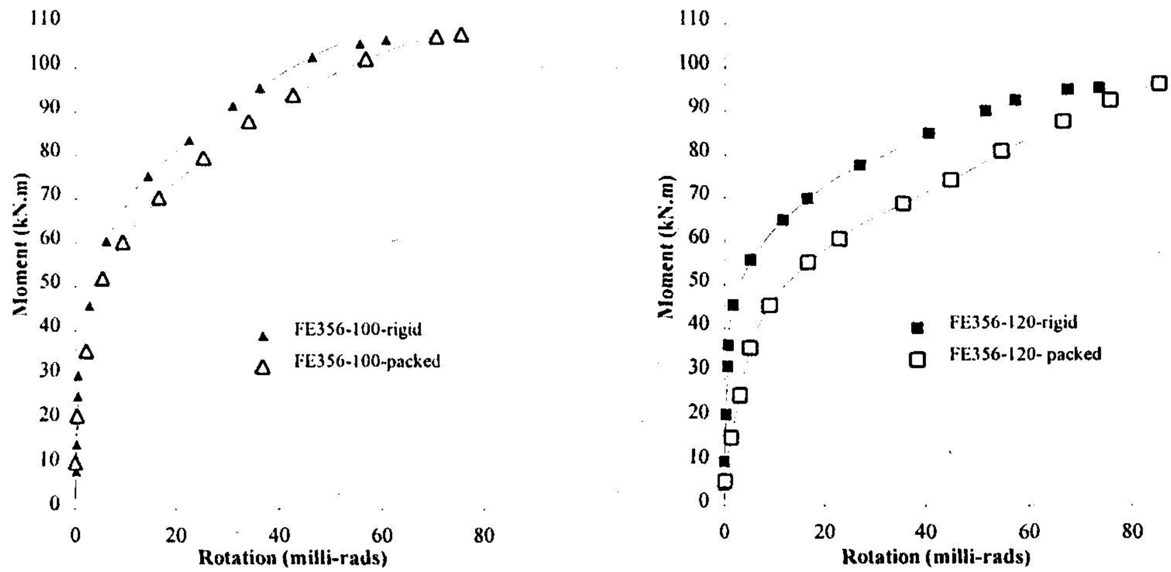



*Figure 4. Details of rigid base and endplate deformations*

The differences in the responses of each pair of tests shown in figure 5 are attributed to the restraint afforded to the edge of the endplate by the rigid base. Endplates in direct contact with the rigid base develop double curvature bending (figure 4c), increasing the initial stiffness of the joints in comparison to the endplates which are packed from the base (figure 4d). Both endplate details attained a similar ultimate moment of resistance regardless of being packed or unpacked.




(a) (b)

*Figure 5. Comparison of Endplate moment rotation characteristics*

## 4.2 Contribution of endplate bending to the flexibility of Flowdrill joints

Figure 6 plots the moment-rotation characteristics of test FE356-120/8 (from figure 3b) and the corresponding endplate test FE356-120-packed (from figure 5b). The 'packed' test was selected as the deformation pattern (figure 4d) most closely resembled that in the flowdrill test. The moment-rotation curve for this test is taken to represent the contribution of endplate deformation only. The upper curve shown on figure 6 is the moment rotation curve obtained in a flowdrill test with an overthick (25 mm) endplate (denoted as test 26). This curve represents the contribution of SHS deformation only to overall joint rotation. The summation of these two curves is also shown on figure 6 and may be taken as representing the total response using the concept of component distortion. It is immediately apparent that this curve does not correspond with the result of test FE356-120/8, resulting in a significant error at large rotations. Part of this discrepancy is due to the difference in endplate depth in test 26 which was 395 mm rather than 350 mm. This has the effect of increasing the lever arm to the upper most bolts by 30 mm (approximately 10% increase). A further 9% discrepancy was found in the yield strength of the two SHS columns. Whilst these differences are significant they will not account for the large discrepancy shown between the two moment rotation curves in figure 6.

A more likely explanation for the large difference in the two curves is that the isolated endplate tests did not pivot about the same point as the endplate in the flowdrill tests. Furthermore, the separation of the endplate and column face effects obviously removes the interaction of column face and endplate stiffness which is present in a real flowdrill

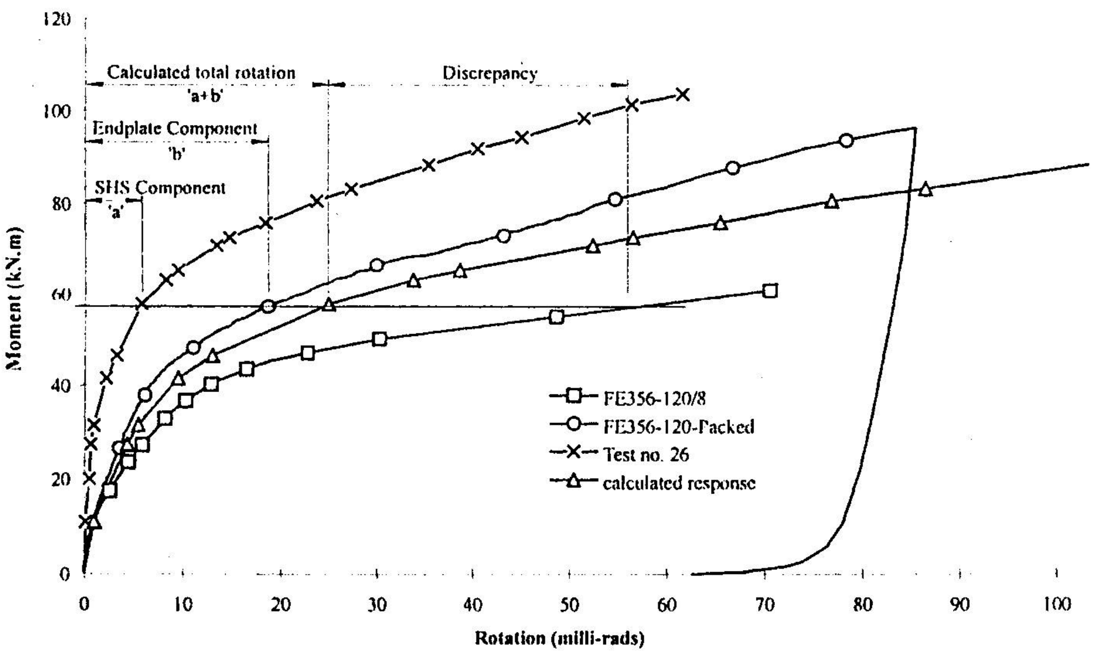



*Figure 6. Moment rotation response with the effect of endplate contribution*

connection and which is the cause of the movement of the pivot point. This suggests that in certain circumstances care must be exercised in adopting an approach in which flexibilities are individually calculated and assumed to determine overall response.

## 5. Conclusions

All joint details responded in a ductile manner with no unexpected behaviour. The tests have shown that partial depth endplates can be used to simulate a pinned joint but some tube wall deformation will be present. Flush endplates may also be suitable for simple construction but are likely to fall outside the EC3 classification boundaries for assumed pin jointed frames.

## 6. Acknowledgements

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## 7. References


1. Manufacturer's literature, Lindapter Hollo-bolt specification. UK.
2. Sadri, S-M. & Wang, H-T. 1995. Advanced technique for joining of hollow steel structures. *Proc. Nordic Steel Construction Conference* : Vol.-II, 757-764: Swedish Institute of Steel Construction.
3. Flowdrill, 1993. Flowdrill technical guide. Flowdrill B.V. Holland.
4. Sherman, D. R., 1993. 'Framed connections to HSS columns'. *Proc. Fifth International Symposium Tubular Structures*: pp 241-248. London: E & FN Spon.
5. Banks, G., 1993. 'Flowdrilling for tubular structures'. *Proc. Fifth International Symposium Tubular Structures*: pp 117-124. London: E & FN Spon.
6. Ballerini, M., Bozzo, E., Occhi, F. & Piazza, M., 1993. 'Flowdrill connections in RHS elements: Experimental results and design evaluations'. *Proc. XIV Congresso C. T. A.*: pp 87-98.
7. France, J. E., Davison, J. B. & Kirby, P. A. 1995. Behaviour of flowdrilled connections in mult-storey frames with tubular columns. *Proc. Nordic Steel Construction Conference* : Vol.-II, 773-780: Swedish Institute of Steel Construction.