

# Composite connections: experimental results

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## Composite Connections - Experimental Results

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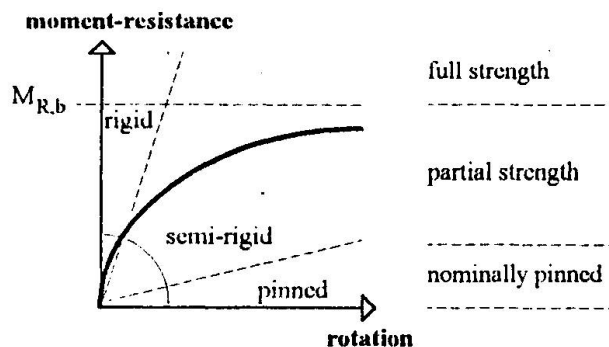
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### Summary

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.

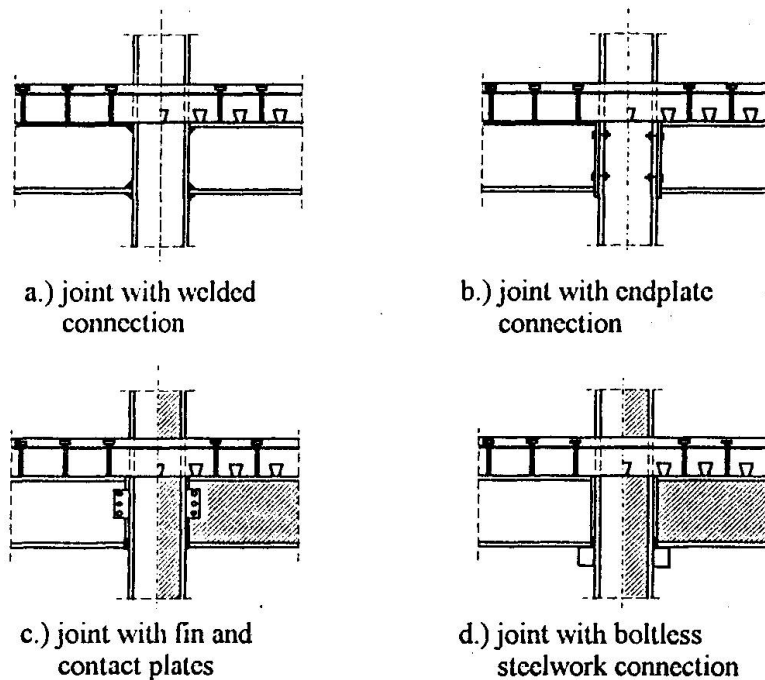
The intention is to develop design rules which make allowance for the whole range between nominally pinned and rigid or full strength connections. As a first step the behaviour of so called semi-rigid or partial strength joints has been investigated. In a second step the influence of such joints on the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

### 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

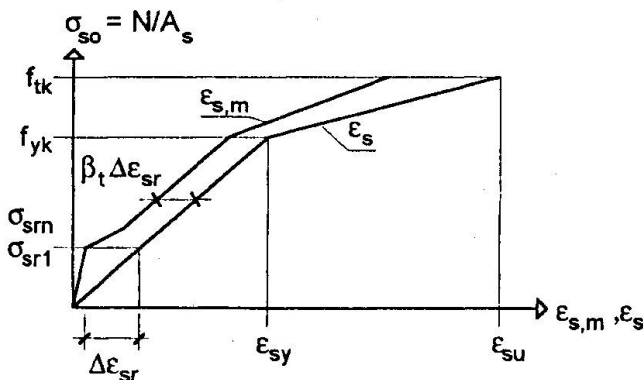


Fig. 3 Simplified stress-strain relationship of embedded reinforcing steel

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



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.

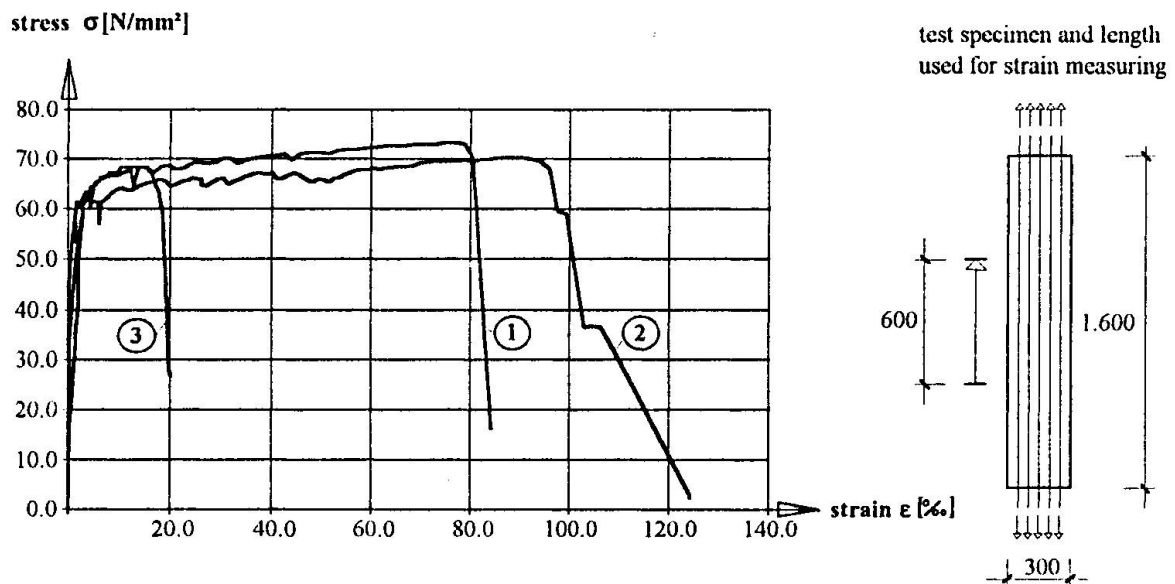


Fig. 4 Stress-strain curves of tensile tests with reinforced concrete

Test No.	reinforcement ratio	diameter of rebars	ultimate strain $\epsilon_{su}$	ratio $f_u/f_t$
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.

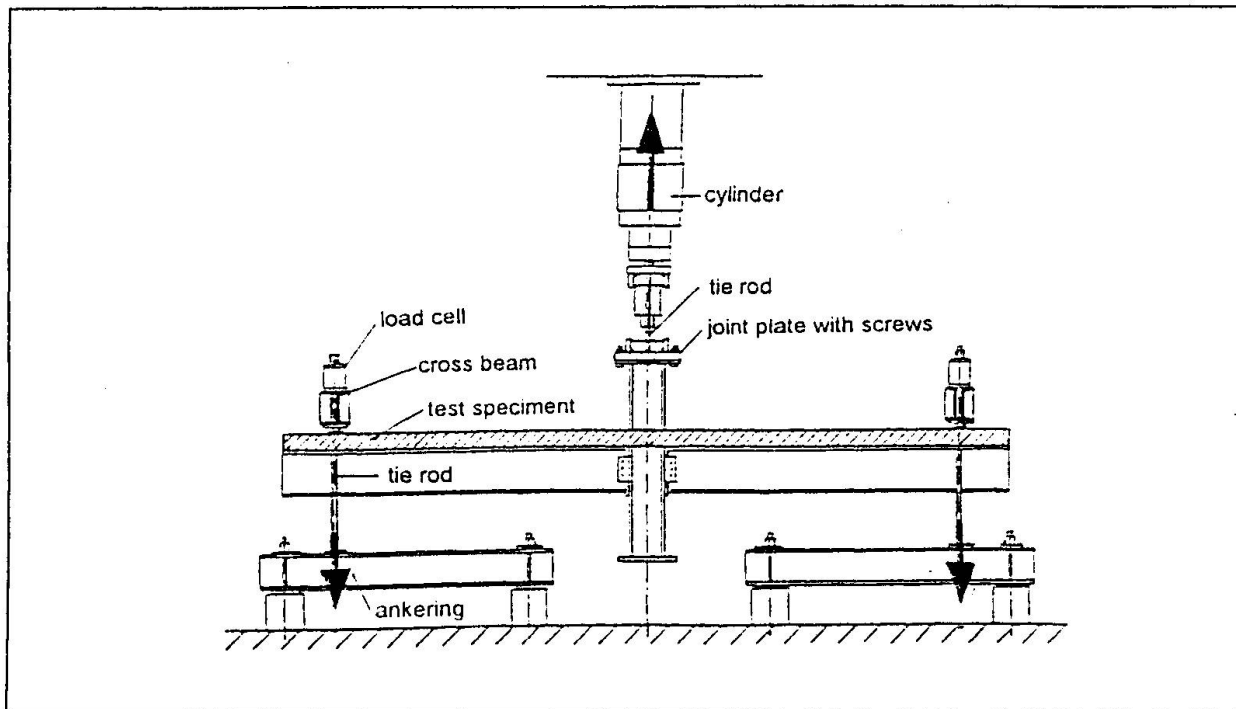


Fig. 5 Setup of the second test series

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. Afterwards 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 occurred, which can be seen too in figure 6.

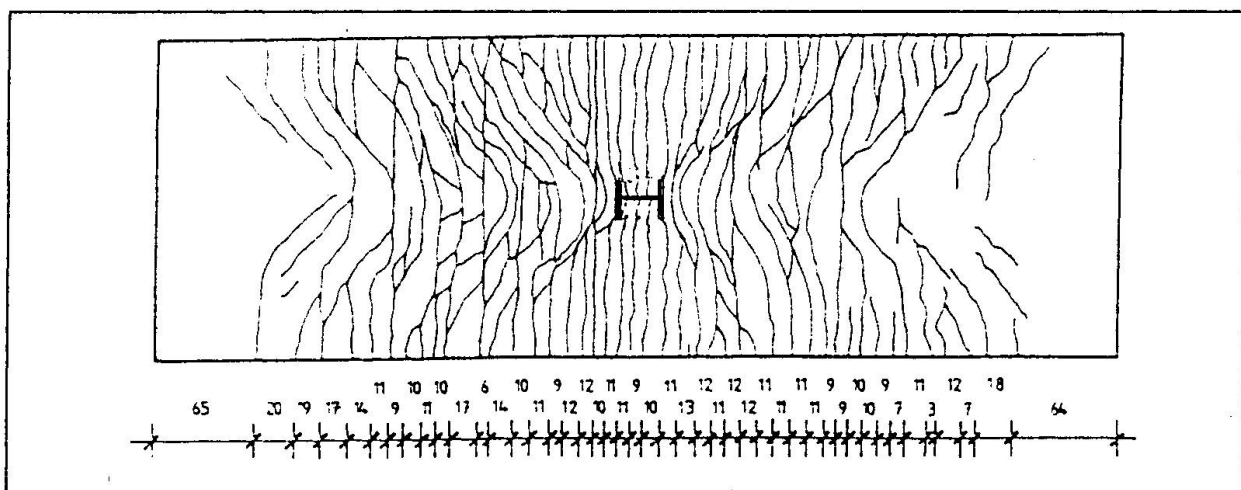


Fig. 6 Crack pattern at the end of the test with a reinforcement ratio of 1.3 %

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-

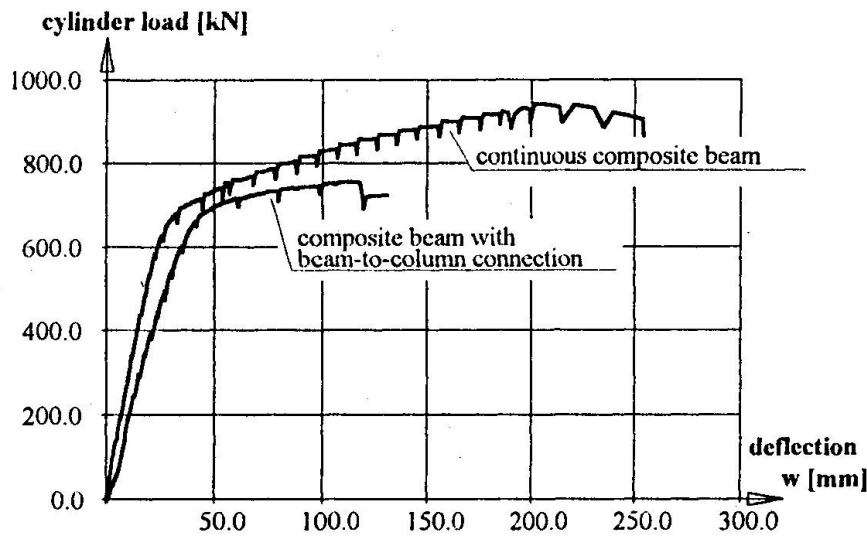


Fig. 7 Load-deformation curves of composite beams with connection and continuous (reinforcement ratio 0.9 %)

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  $\rho = 0.4\%$  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

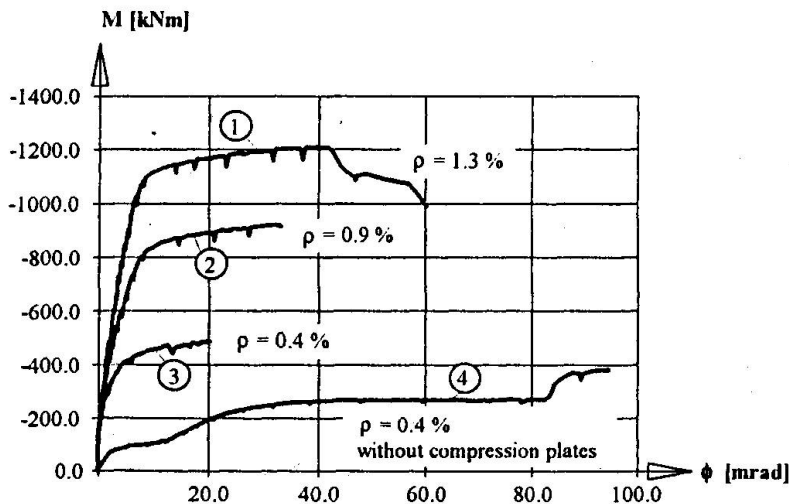


Fig. 8 Finplate connections - moment-rotation curves

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.

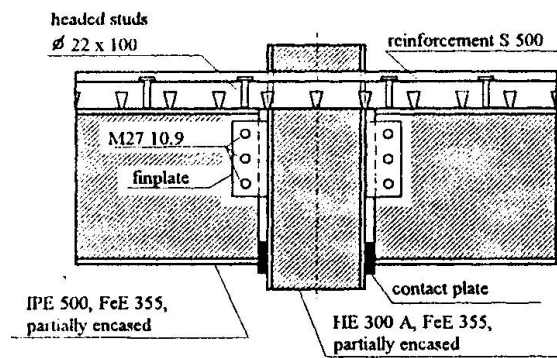
Further test series are reported in the following, where the cooperation of the components 'reinforced concrete flange' and 'steel connection' is considered in detail with regard to the shear connection along the composite beams, together with further variations. In all these further tests, a 12 cm thick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

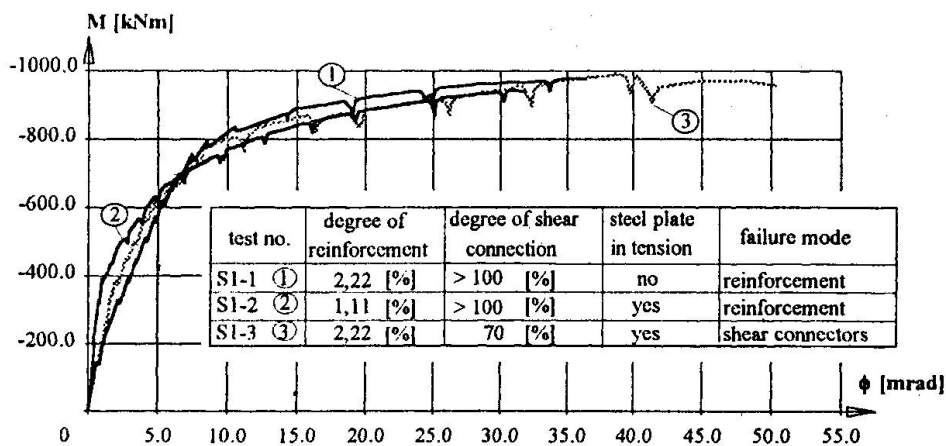


Fig. 10 Tests on composite joints with finplate connection

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 rein-





forcement, the rest by the bolts within the finplate. In this test reinforcement rupture occurred 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.

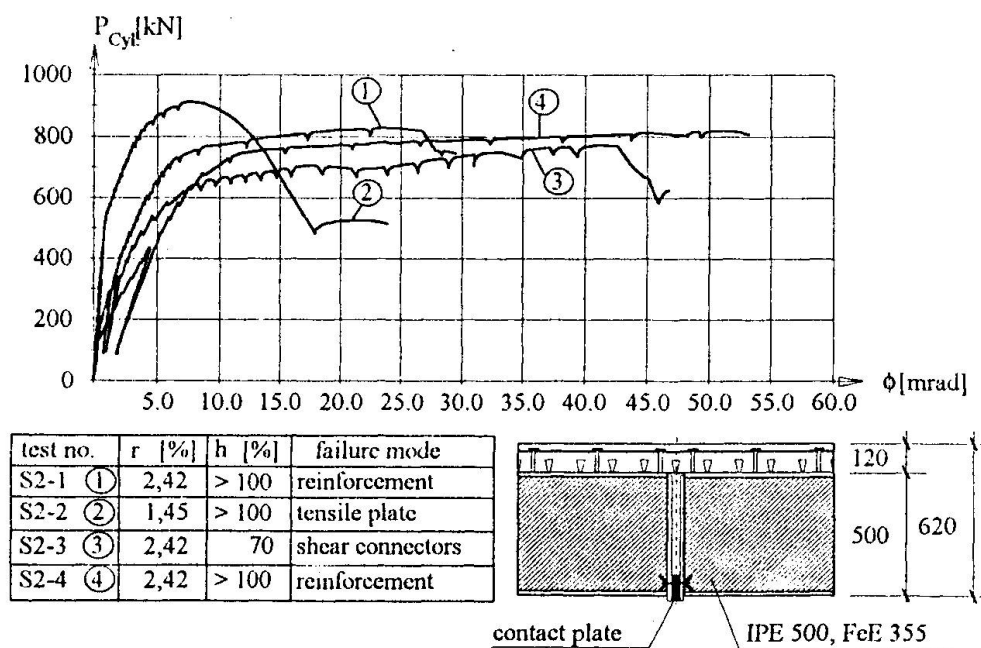


Fig. 11 Fifth test series: type of connection and experimental load-rotation curves

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 occurred 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 extent the moment rotation behaviour of a joint depends on the structural detailing.

## 5. Summary and Conclusions

From these tests and further research 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 extent, 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 strength, the behaviour of semi-continuous composite beams approaches 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 necessary, 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 simultaneously, 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 Béton, 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] Tschemmerneegg, 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 Zwangseinwirkung. 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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