Composite joints: further eyperimental results

Composite Joints - Further Experimental Results

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Summary

The design of composite structures requires a classification system for the joints. Thus, it is necessary to calculate stiffness, moment resistance and ultimate rotation of the joints. If plastic design methods shall be used, the available ultimate rotation of the joint has to be compared with the rotation at the joint, required by the structure and its loading. In this paper the ultimate rotation of composite joints is investigated and described by means of selected test results. Possibilities to increase the ultimate rotation of the joint, which may be necessary for the required or full moment redistribution in the structure, will be lined out.

Introduction

Composite joints consist of a number of components transferring forces between the connected members, such as the steelwork connection, which in turn consists of several components, the reinforced concrete slab and the column web panel. All these components provide a particular, in general non-linear, force-deformation behaviour, thereby influencing the behaviour of the joint and the whole structure. Besides the structural detailing of the joint components, the arrangement within the structure and corresponding parameters as the shear connection between the steel beam and the slab, the loading and the method of erection have to be considered in order to describe the moment-rotation behaviour of the joint [1]. Tests on substructures as well as tests on components and large scale tests with semi-continuous composite structures have been carried out. The results from such tests can be used to calibrate models to determine the characteristic joint properties (the initial stiffness, the moment resistance and the rotation capacity).

Tests with composite joints and structures

During the last years more than 30 tests on symmetrically loaded composite joints and four tests on semi-continuous composite floor beam structures over two spans were carried out at Kaiserslautern University. Some results will be shown and discussed in this paper.

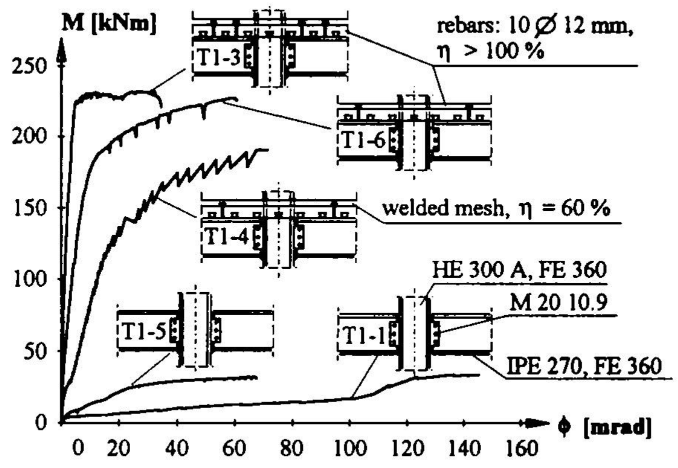

Fig. 1: Moment-rotation curves from tests with finplate connections [2]

In a series of tests with joints with finplate connections the influences from different components were investigated. Figure 1 shows achieved moment-rotation curves. In the tests on bare steel joints (T1-1 and T1-5) the influence of a contact plate between the lower beam flange and the column flange was investigated. The contact plate leads to a direct transfer of compression forces, thereby increasing stiffness and reducing ultimate rotation. In two other cruciform tests, the composite behaviour of the same steelwork connection, but with a continuous concrete flange was examined. In test T1-6 rebars with high ductility and full shear connection were used to increase stiffness and moment resistance of the joint. Partial shear connection however leads to a reduced stiffness and resistance, but increases the ultimate rotation as it can be seen from test T1-4, in which the same reinforcement ratio was provided as in test T1-6. This test indicates, that it might be possible to use welded mesh reinforcement in hogging bending areas, but only in combination with partial shear connection, which then provides the necessary ductility. In addition to these tests with cruciform specimens a large scale test with a composite floor beam over two spans with two point loads per span was carried out (T1-3). Full shear connection over the whole beam length was used. The joints at the interior support were the same as in test T1-6. Comparing the curves from tests T1-3 and T1-6 yields however opposite results. The differences are due to the acting width of the slab and the different arrangement of shear connectors. In test T1-6 the first shear connector was placed in the second rib of the steel sheet close to the joint, while in the beam test the first shear connector was located in the first rib.

In a further test series on composite joints with finplate connections the number of rebars was varied. Figure 2 shows the moment-rotation curves of these tests. A detailed description of these tests is given in [1, 3]. It was found that besides the stiffness and moment-resistance also the ultimate rotation is influenced by the reinforcement ratio, as it can be seen from the figure.

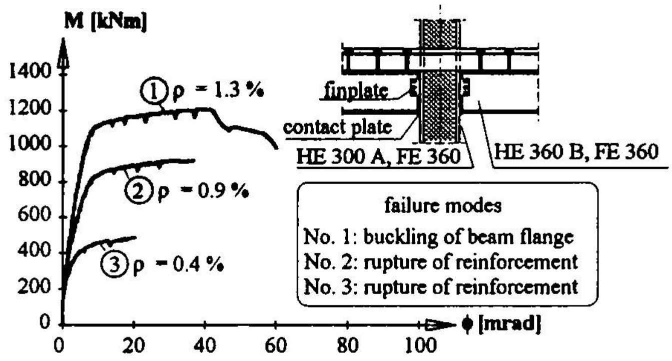

Fig. 2: Influence of reinforcement ratio on joint behaviour

Besides such tests on beam-to-column joints, beam-to-beam connections were also investigated. The considered boltless steel connection (Fig. 3) is an example for interconnected floor beams with the main beam underneath. Moment resistance was achieved by reinforcement in the slab and a contact plate between the lower steel flanges. In this test series the degree of shear connection and the arrangement of shear connectors were varied. A detailed description of these tests is given in [4, 5]. The tests show, that in comparison with full shear connection and uniformly distributed shear connectors along the whole beam length, partial shear connection (test S2-3) as well as a certain distance between the first shear connector and the joint (test S2-4) reduce the stiffness, but increase the ultimate rotation of the joint.

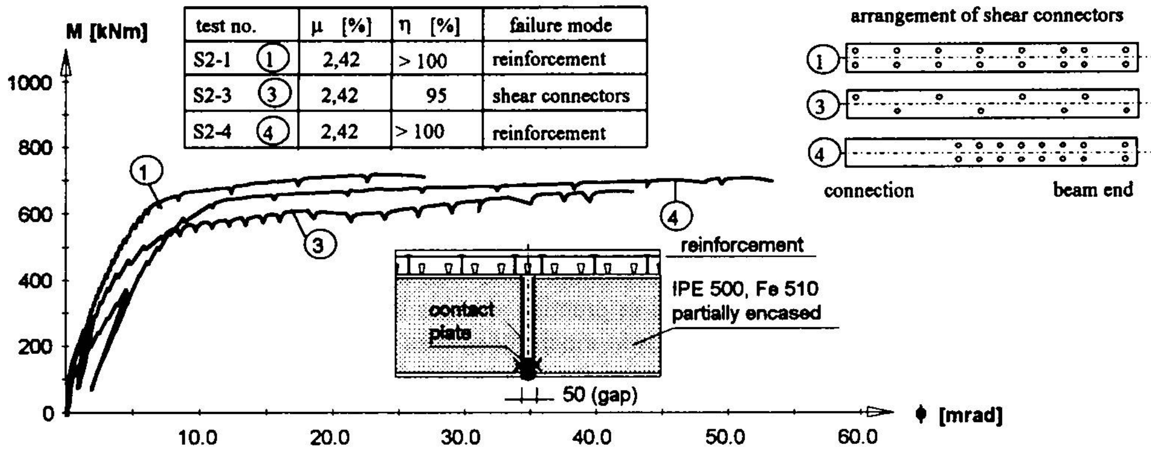

Fig. 3: Experimental moment-rotation curves

Figure 4 shows test results from a composite floor beam structure semi-continuous over two spans, which consisted of two simply supported steel beams and a continuous reinforced composite slab. Each beam was loaded at four points per span. At the interior support a boltless connection as in figure 3 was used. Continuity and moment resistance in negative bending were again achieved by ductile rebars in the slab and a contact plate between the lower steel flanges. Welded mesh provided some additional anti-crack reinforcement. Full shear connection along the whole beam length was provided by uniformly distributed shear connectors. The beams were propped during casting.

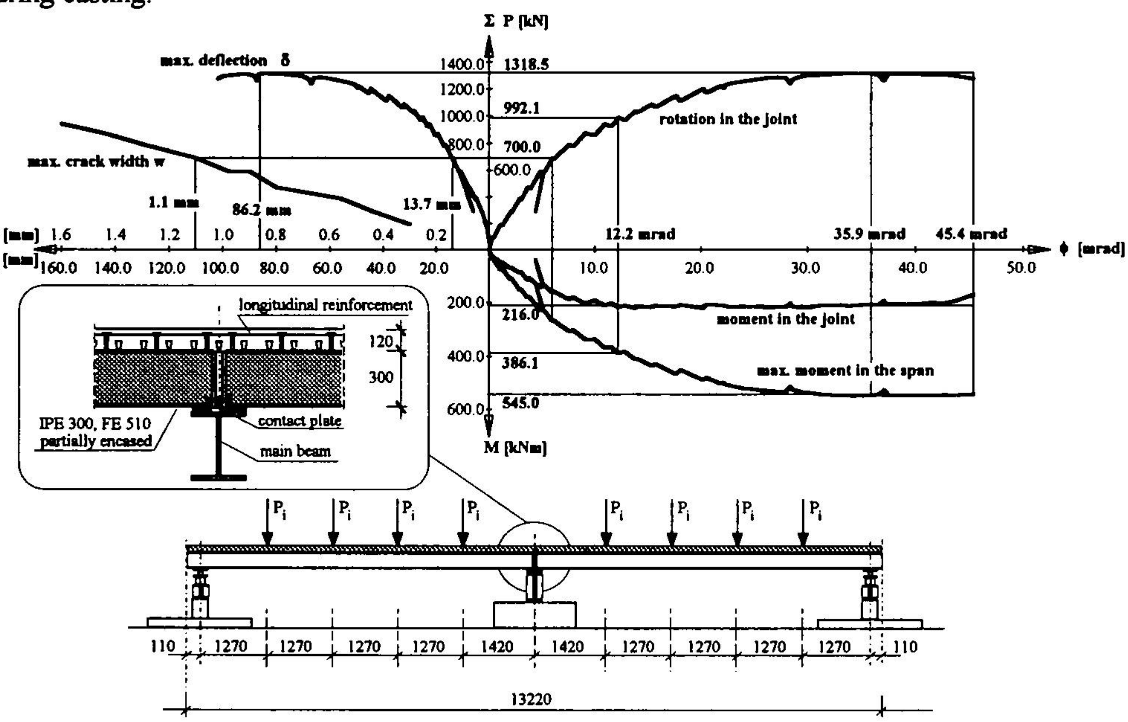

Fig. 4: Results from a test with a semi-continuous floor beam [5]

Figure No. 4 presents the full test information from this floor beam structure. The upper right diagram shows the applied load versus rotation in the joint at the interior support measured during the test. The diagram below represents the moment development at the joint and in the span. At a rotation of about 12 mrad the joint reaches its plastic moment resistance while only 2/3 of the plastic moment resistance at midspan is achieved. From this rotation up to failure the joint behaves plastically. A rotation of about 36 mrad yields the full plastic moment resistance at midspan. The joint provides a higher rotation capacity than necessary, and failure of the structure due to rupture of the reinforcement occurs at a rotation of about 45 mrad. Thus, this part of the diagram shows that for this investigated test specimen rigid plastic analysis can be applied to calculate the ultimate load. The left diagram contains the deflection at midspan and the crack width at the interior support. From the obtained ultimate test load the load-level corresponding to the serviceability limit state was recalculated being approximately 700 kN. Up to this load the joint still shows a linear elastic behaviour. The corresponding maximum deflection at midspan was measured to be 13.7 mm, which is within the limits required in practice. At this load the maximum crack width however was measured being 1.1 mm, which is much more than the corresponding upper limit according to EC 4 [6], even though a reinforcement ratio of 1.54 % was used in the test specimen.

Ultimate rotation of composite joints

In [8] a method is described for the calculation of the ultimate rotation of a joint in cases where failure occurs in the tension zone of a joint. This procedure takes into account the deformations in the slab and at the steel concrete interface. In order to provide better results, it was enlarged by an additional factor taking into account some plastic deformations in the compression zone of the joint [9]. Thus, the ultimate rotation of a composite joint can be calculated by:

$$\phi_u = \frac{\Delta_{u,s} + s + \Delta_a}{D + D_r}, \qquad (1)$$

where $\Delta_{u,s}$ is the elongation of the reinforced concrete slab, s is the slip at the steel concrete interface close to the joint, Δ_a is the plastic deformation including buckling effects in the compression zone of the joint, D equals the height of the steel beam and D_r is the distance between the upper layer of reinforcement and the steel beam.

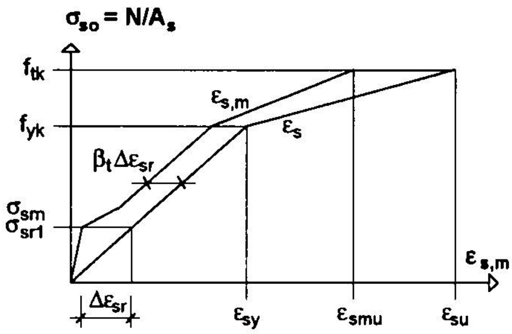

Fig. 5: Simplified stress-strain relationship of embedded reinforcing steel

The reinforced concrete slab is anchored to the steel beam by shear connectors, and its force-deformation behaviour differs from the behaviour of the reinforcement only. Figure 5 shows the simplified stress-strain relationship of embedded reinforcing steel together with the corresponding curve for reinforcement only as described in [10]. The embedded reinforcement curve provides a higher stiffness and a lower ductility than reinforcement alone (tension stiffening effect). Besides other parameters the properties of the reinforced concrete part depend mainly on the reinforcement ratio.

The ultimate average strain $\varepsilon_{s,m,u}$ of embedded reinforcement can be calculated as follows [10]:

$$\varepsilon_{s,m,u} = \varepsilon_{sy} - \beta_t \cdot \Delta\varepsilon + \delta \cdot \left(1 - \frac{\sigma_{sr1}}{f_{y,s}}\right) \cdot \left(\varepsilon_{su} - \varepsilon_{sy}\right), \qquad (2)$$

where β_t and δ are curve parameters. However, due to stress concentrations at the column flanges, this strain does not occur uniformly along the whole distance between the centreline of the column and the first shear connector. This location of the first shear connector influences the strain distribution in addition, as it can be seen from the test in figure 3. Thus, in case of full interaction the ultimate elongation of the reinforced concrete slab can be calculated being

$$\Delta_{u,s} = L \cdot \varepsilon_{s,m,u} \tag{2}$$

for $a \le D_r$, with the distance L between the centreline of the column and the first shear connector and the distance a between the joint and the first shear connector or

$$\Delta_{u,s} = \left(\frac{h_c}{2} + D_r\right) \cdot \varepsilon_{s,m,u} + \varepsilon_{sy} \cdot (a - D_r) \tag{3}$$

for $a > D_r$, and the depth h_c of the column section.

The improved method was used to calculate the ultimate rotations, measured in the tests outlined before and to compare them with test results, see table No. 1. The agreement is excellent, although the amount of reinforcement, the degree of shear connection, the arrangement of the shear connectors and the type of test specimen (cruciform, large scale beam test over two span) were varied in these tests.

Test	ρ_s	η	$\phi_{u,test}$ [mrad]	$\phi_{u,calc}$ [mrad]	$\phi_{u,calc} / \phi_{u,test}$	figure	failure mode
S2-1	2.42 %	> 100 %	27,6	28,0	1,01	3	reinforcement
S2-2	1.45 %	> 100 %	24,5	24,7	1,01	-	reinforcement
S2-3	2.42 %	≅ 95 %	42,7	41,1	0,96	3	shear connectors
S2-4	2.42 %	> 100 %	53,2	54,4	1,02	3	reinforcement
S5-2	1.54 %	> 100 %	43,6	43,8	0,99	4	reinforcement
T1-3	0,88 %	> 100 %	33,8	35,8	1,06	1	reinforcement
T1-6	0,88 %	> 100 %	60,0	59,7	0,99	1	reinforcement
No. 1	1.30 %	> 100 %	43,5	44,8	1,03	2	beginn of buckling
No. 2	0.90 %	> 100 %	37,4	38,2	1,02	2	reinforcement
No. 3	0.40 %	> 100 %	18,9	17,9	0,95	2	reinforcement

Table 1: Comparison between measured and calculated ultimate rotation

It should be mentioned however, that this method provides correct results only if failure of the reinforcement or the shear connectors occurs. This method yields an upper limit for the ultimate rotation in cases, where the bolts in the steelwork connection fail or if local instabilities in the compression zone of the joint occur.

Conclusions

Equation (1) contains the main contributions to calculate the ultimate rotation with high accuracy if the realistic behaviour of reinforced concrete in negative bending and appropriate slab lengths are taken into account. Slip at the steel concrete interface as well as deformations due to local instabilities can also contribute to the ultimate rotation.

The detailing of joints and the adjacent beam sections is very important and can reduce or enlarge the ultimate rotation. Shear connectors placed close to the joint reduce the free elongation length of the slab. Such an arrangement increases stiffness, but reduces the ultimate rotation.

Partial shear connection in combination with ductile shear connectors and profiled steel sheeting enlarges the ultimate rotation of the joint, even if reinforcement of normal ductility (for example prefabricated welded mesh) is used.

Rigid plastic analysis has been applied to analyse the ultimate load of the tested floor beam structure over two spans. Large rotations in the joints are however necessary to make use of the high beam resistance in sagging moment areas. The required ultimate rotation of the joint can clearly be reduced, if the load carrying capacity is reduced to only 90 %. In case of the tested beam, such a reduction reduces the required rotation to about 50 % and this would lead to a required rotation capacity of about 2.

The use of partial strength joints together with rigid plastic analysis reduces the beam length in hogging bending. Thus, the contribution of the beam in negative bending to the required rotation (cracking of concrete and yielding of steel) is further reduced, while the contribution of the joint has to be increased.

The required rotation in the joint however can be further reduced by the loading history and type of erection. Unpropped construction together with simple steelwork joints and continuity in hogging bending areas only after the concrete has hardened reduces the required rotation of the composite joint in addition.

Acknowledgement

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