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Limitations on the use of partial shear connection in composite beams with steel T-sections and uniformly spaced rib shear connectors

Martin Classen

RWTH Aachen, Institute of Structural Concrete, Mies-van-der-Rohe-Straße 1, 52074 Aachen, Germany

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ABSTRACT

Design codes like EC4 allow the bending resistance of composite beams with partial shear connection (PSC) to be assessed by plastic analysis procedures, based on equilibrium of forces across the composite section (equilibrium method). However, these plastic design models are limited to connectors, usually shear studs, with sufficient deformation capacity (at least 6 mm) and composite beams, where the steel sections have either equal flanges or only slightly differing areas of top and bottom flange. Beyond that, EC 4 defines a minimum degree of shear connection to be provided, that was derived for composite members with double flange steel section and stud connectors through systematic FE analysis. For composite beams with steel T-section (without top flange) and uniformly spaced rib shear connectors, limitations on the use of partial shear connection have not yet been investigated. Due to significant discrepancies in terms of steel section geometry and ductility of the connectors a simple adoption of regulations from EC4 is not possible and presumably unsafe. To resolve that problem, the paper presents the results of numerical simulations, based on which limitations on the use of partial shear connection have been derived for beams with steel T-sections and uniformly spaced connectors.

1. Introduction

In steel-concrete composite girders, rib shear connectors can be used to transfer shear forces between the concrete slab and the steel section. While headed stud connectors are usually welded upright onto the upper flange of steel beams, rib shear connectors are burned immediately into the webs of steel members allowing single flange-steel beams (steel T-beams) to be used (Fig. 1b)). Here, the internal lever arm and the distance between centroidal axis of the steel section (C.A.) and plastic neutral axis of the composite section (P.N.A.) is significantly larger than compared to conventional composite members with double-flange steel sections and headed stud connectors. In consequence, composite members with single flange steel beams have significantly increased bending resistance and bending stiffness, when having the same depth and deploying the same amount of steel (Fig. 1c). Intense research in the recent years based on experimental and theoretical studies led to the development of design principles for the shear resistance of rib shear connectors with puzzle- or clothoid-shaped geometry (Fig. 1a) and either steel or concrete pry-out failure. Design approaches for steel failure have been proposed by Lorenc [3] and Feldmann [4], while models for the concrete pry-out failure were developed by seidl [5] and Classen [6], for instance. These models were recently consolidated into a general technical approval [7] for rib shear connectors, as their application is currently not embodied in international standards. Furthermore, there is an initiative promoted by well-known researchers aiming to incorporate the design of rib shear connectors into the next generation of EC4.

Nevertheless, the application of composite members with single flange steel beams (T-sections) and rib shear connectors in buildings is still subject to some open questions. Especially the use of beams with partial shear connection has been overlooked, for many years [8].

2. Theory of partial shear connection

2.1. Plastic design of composite beams with full and partial shear connection

In general, codes like EC4 [9] allow the bending resistance of steel composite beams to be assessed by plastic design strategies. These plastic analysis procedures are purely based on equilibrium of forces across a section (equilibrium method). Here, all components of the composite section (steel part, concrete part and shear connectors) are assumed to have unlimited ductility and to reach and maintain their plastic or yield strength. Hence, each fiber of the steel section is assumed to be stressed to the yield strength f_y, concrete fibers in the compression zone to be stressed to 0.85 f_c (Fig. 2) and the shear connectors to reach their shear resistance P_R.

Distinction is made between composite beams with either full (FSC) or partial shear connection (PSC) [10]. The degree of shear connection η is an index of the connection''s shear resistance. For beams with full shear connection ($\eta = 1$), the addition of more shear connectors does

E-mail address: mclassen@imb.rwth-aachen.de.

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Nomenclature

A_a	area of the steel section
A_c	area of the concrete section
D	ductility parameter to describe the slope of the descending branch of the concrete compressive stresses after Sargin [1]
K_c	factor in the concrete damaged plasticity model (CDP) for describing the shape of the yield surface by the ratio of the second invariants of the stress vector of the tension and compression median [2]
L_e	span, span length (distance in sagging bending)
M_{Ed}, M_{Ed} (x)	acting bending moment (at the position x of the beam axis)
$M_{pl,a,R}$	plastic bending resistance of the steel section
$M_{pl,R}$	plastic bending resistance of the composite section calculated though equilibrium method acc. to EC4
$M_{pl,R,FSC}$	plastic bending resistance of the composite section with full shear connection (FSC) calculated though equilibrium method acc. to EC4
$M_{pl,R,PSC}$	plastic bending resistance of the composite beam with partial shear connection (PSC) calculated though equilibrium method acc. to EC4
$M_{ultimate}$	flexural strength of a composite beam
$M_{ultimate,FE}$	flexural strength of a composite beam simulated through FE analysis
P_{max}	shear strength of shear connectors
P_R	shear resistance of shear connectors ($P_R = 0.9\ P_{max}$)
f_c	concrete compressive strength
f_{b0}/f_{c0}	ratio of biaxial and uniaxial compressive strengths of the concrete
f_y	yield strength of the steel
n	number of provided shear connectors in the steel-concrete interface
n_f	number of required shear connectors for full shear connection (FSC)
x	position of a cross section along the beam axis
Ψ	dilatation angle of the concrete defined as the ratio of volume strain to shear strain [2]
$\delta_{beam,max}$	maximum slip in the interface of the composite beam at ultimate load
$\delta_{beam,max,FE}$	maximum slip in the interface of the composite beam at ultimate load calculated through FE analysis
δ_u	deformation capacity of the connectors (in EC4 also called ductility)
ε	eccentricity parameter of the CDP model [2]
η	degree of shear connection
η_{min}	minimum degree of shear connection

not yield an increase in bending resistance, as the sum of shear resistances from all connectors (located in-between two successive critical sections) suffices for transfer of the plastic tensile or compression strength in either the steel $A_a\ f_y$ or the concrete section 0.85 $A_c\ f_c$. However, partial shear connection ($\eta < 1$) applies, when fewer dowels are arranged than required for full shear connection. For ductile shear connectors, the degree of shear connection is the ratio of the number of provided shear connectors n and the number of shear connectors n_f required for full shear connection [9]. Fig. 2 illustrates the diagram of partial shear connection resulting from plastic design (equilibrium method) with the calculated bending resistances plotted over the degrees of shear connection. Here, the cases (I), (II) and (III) reaching from no shear connection (I) to full shear connection (III) are highlighted. For beams with no shear connection (I), the plastic bending resistance $M_{pl,R}$ of the composite section is equal to the plastic bending resistance of the pure steel section $M_{pl,a,R}$. The optimum exploitation of steel and concrete strength coming along with the highest bending resistance $M_{pl,R} = M_{pl,R,FSC}$ is reached for beams with full shear connection (III). The corresponding strain profile has one plastic neutral axis (P.N.A.) with its position depending on the tensile or compression strengths of steel beam and concrete slab (Fig. 2). In beams with partial shear connection (II) there are always two P.N.A.s (Fig. 2) with a significant delta strain in the steel-concrete interface. Here, the bending resistance $M_{pl,R} = M_{pl,R,PSC}$ of the composite beam is limited by the strength of the shear connection.

It is interesting to note, that the plastic bending resistances according to equilibrium method only depend on the cross-section geometry, material strengths and degree of partial shear connection, while parameters of the structural system (like beam span, loading configuration) as well as the compatibility of slip and deformation capacity of the shear connectors are not considered.

2.2. Limitations on the use of partial shear connection

2.2.1. General

As outlined above, in plastic design strategies only equilibrium of forces is considered, while the strains and deformations of the materials and connectors are completely disregarded. However, in real composite beams with partial shear connection, significant relative deformations (slip) occur between steel beam and concrete slab [11,12]. There are cases, where ignoring these slip deformations may yield unsafe design results.

Fig. 3(a) shows a composite beam with variable degree of shear connection (cases 1, 2, 3) exposed to a uniformly distributed load. A close-up of the beam end region illustrates the occurring slip in the steel-concrete interface. Fig. 3(b) shows the relations between bending moment – beam slip for different degrees of partial shear connection (cases 1 to 3) and Fig. 3(c) the deformation characteristic of the shear connector. The following facts can be observed:

- For high degrees of shear connection (case 1) the beam end slip δ_{beam} is smaller than the deformation capacity of the connectors δ_u (Fig. 3b). In such cases, the flexural strength $M_{ultimate}$ of the composite beam is (slightly) larger than the bending resistance $M_{pl,R}$ calculated though plastic design procedures. Here, plastic design procedures yield safe design results.
- For small degrees of shear connection (case 3), the beam slip in the interface δ_{beam} exceeds the deformation capacity δ_u of the shear connectors (Fig. 3b). In regions with large beam slip (near the beam end) the connectors fail due to excessive slip, before connectors in regions with small beam slip (near midspan) reach their shear resistance. Hence, the required ductility of the beam is higher than that provided by the connectors and the flexural strength $M_{ultimate}$ is smaller than the bending resistance $M_{pl,R}$ calculated though plastic design procedures. Obviously, in such cases, the assumption of plastic connector behavior with unlimited ductility is not applicable and plastic design procedures yield unsafe results.
- Between cases 1 and 3, there is one specific case (2), where the deformation capacity of the connectors δ_u just matches the occurring beam slip δ_{beam} (Fig. 3b). Here, the bending resistance $M_{pl,R}$ calculated though plastic design is identical to the flexural strength $M_{ultimate}$ of the composite beam. This specific degree of shear connection is defined to be a lower limit for the use of partial shear connection in plastic design procedures. It is called "minimum degree of shear connection" η_{min}.

As a result, in design codes, the use of plastic design procedures (Chapter 2.1) for composite beams is limited to cases where the degree of shear connection provided is higher than η_{min}. In the next chapter, methods to derive η_{min} will be explained.