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Composite Beams

Poutres mixtes

Verbundträger

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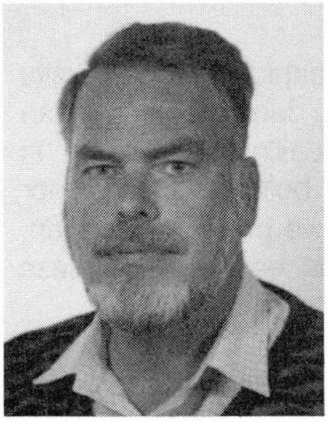

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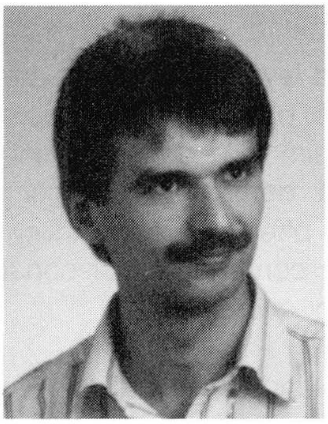

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SUMMARY

Compact composite beams are introduced concerning the various possibilities of construction, the behaviour under vertical loads (single- and continous span beams) and the checking of the ultimate limit state. The fundamental behaviour of connectors is illustrated and design rules for different kinds of construction are presented.

RÉSUMÉ

Les poutres mixtes compactes sont présentées du point de vue de leur possibilité d'utilisation dans la construction, de leur comportement sous l'effet de charges (poutre à une ou à plusieurs travées) ainsi que de leur résistance à l'état limite. Le comportement fondamental des moyens d'assemblage est expliqué ainsi que les règles de calcul pour différentes sortes de construction.

ZUSAMMENFASSUNG

Es wird eine Einführung in kompakte Verbundträger bezüglich der Konstruktionsmöglichkeiten des Tragverhaltens (Einfeld- und Durchlaufträger) und der Nachweise im Grenzlastzustand gegeben. Die grundsätzliche Wirkungsweise der Verbindungsmittel wird erläutert, und es werden Bemessungsregeln für verschiedene Konstruktionsarten vorgestellt.

1. INTRODUCTION

In buildings and bridges it is common to find concrete slabs supported by steel beams. Nowadays these two parts are interconnected by means of mechanical shear connectors: the steel sections then act compositely with one or two flanges of reinforced concrete, mainly subjected to bending. Such composite beams are the most important parts of composite floor construction for commercial buildings. Fig. 1 shows different typical examples of composite beam layout. They stand for a modern approach for a wide range of commercial buildings, which offer the designer and his client following advantages:

- speed and simplicity of construction;
- lighter construction than a traditional concrete building;
- less on site construction.

Different shapes and layouts of beams are in use: rolled and fabricated sections, in situ concrete as well as prefabricated concrete slab elements. They facilitate long span structures with the aim to accomodate large services without increasing floor depth. In addition there is a demand for larger column-free spans in buildings, either for open-planning or to offer greater flexibility in office layout. Today we prefer large span, rectangular grids for composite floor constructions, and it becomes feasible to increase the spans to 12, 15 or even 20 m.

Generally speaking, composite buildings comprising steel frames and composite floors combine greater structural economy with a faster speed of construction than normal bare steel or concrete structures would do.

2. GENERAL ON CROSS-SECTION AND BEAM BEHAVIOUR

Due to the short time available for this lecture, we shall restrict the large variety of forms of composite beams to the use of rolled steel sections alone and shall say nothing about haunched tapered or fabricated beams, nothing about large rectangular web openings, trusses, open web steel joists and stub girders.

It is European practice today, to achieve the composite action mainly by means of headed studs, but also with other connectors, which in general are welded to the structural steel and embedded in the concrete slab. In case of single span beams it is obvious, that sagging bending moments due to the applied vertical loads cause tensile stresses in the steel section and compression forces in the concrete slab: material properties and stresses are assigned to each other in an optimal way. Due to these facts, composite beams are able to sustain heavy loadings, while spans are large, depths are low, size of steel section can be reduced and stiffnesses are great.

Fig. 2 shows several beam cross sections exemplarly:

- case a, the normal construction with steel sections and concrete flange. The wet concrete can be casted on timber shuttering. On other construction sites, thin precast

Fig. 1 Composite floor constructions for office buildings

concrete elements are used, which serve as permanent shuttering for the in situ concrete. Alternatively, large precast concrete deck units are in use, mainly for car parks.

- case b, the use of metal decking, a very interesting and economical way to speed up construction. The profiled steel sheets act as permanent shuttering. If they have further embossments/indentations and/or end anchorage means, they are able to represent the bottom reinforcing bars after the concrete has hardened (composite slab action).

- case c, fully encased steel sections, which have been used sometimes. They are characterized by their high fire resistance ratio (concrete cover at least 50 mm), but the structural steel is invisible and it is impossible, to fasten brackets, struts, cantilevers and other production means directly to the steel flange - a clear disadvantage!

- case d, during the last 10 years developped partially encased composite beams, the chambers of which are filled with reinforced concrete. This type of construction is nowadays very often used in Germany for commercial and industrial buildings in order to enhance the fire resistance ratio without additional measures, but to leave the lower steel flange unprotected as it is.

Composite beams, which belong to case b or d, are such, that any scaffolding and timber shuttering are unnecessary. This speeds up construction and saves money.

And last but not least: the fully developed connection technique of normal steel constructions is available to carry out connections to steel columns and frames and to make the best use of it. Fig. 3 shows several connection details to give the impression, how simple such connections are, even in case of composite beams.

The design of such composite members using rolled I-section steel beams interconnected with concrete forms the subject of this lecture.

3. CONSTRUCTION METHODS

Before we proceed to the determination of cross section strengths, however, we should consider two different construction methods. Because of the benefits in structural performance, it would seem preferable to ensure the composite action at all times. This would result in all loads, including the dead weigth of the structure, being resisted by the composite beam section. For this to be achieved, it is necessary to support the steel beam until the concrete has hardened. Such support is known as "propping". The number of temporary supports need not be high. These props are usually left in place until the concrete slab has developed an adequate strength.

Fig. 4 shows the effects of different construction methods - propped or unpropped - in principle, and this in comparison with a bare steel beam without any composite action. The drawing represents bending moments at midspan over midspan deflections. M_G

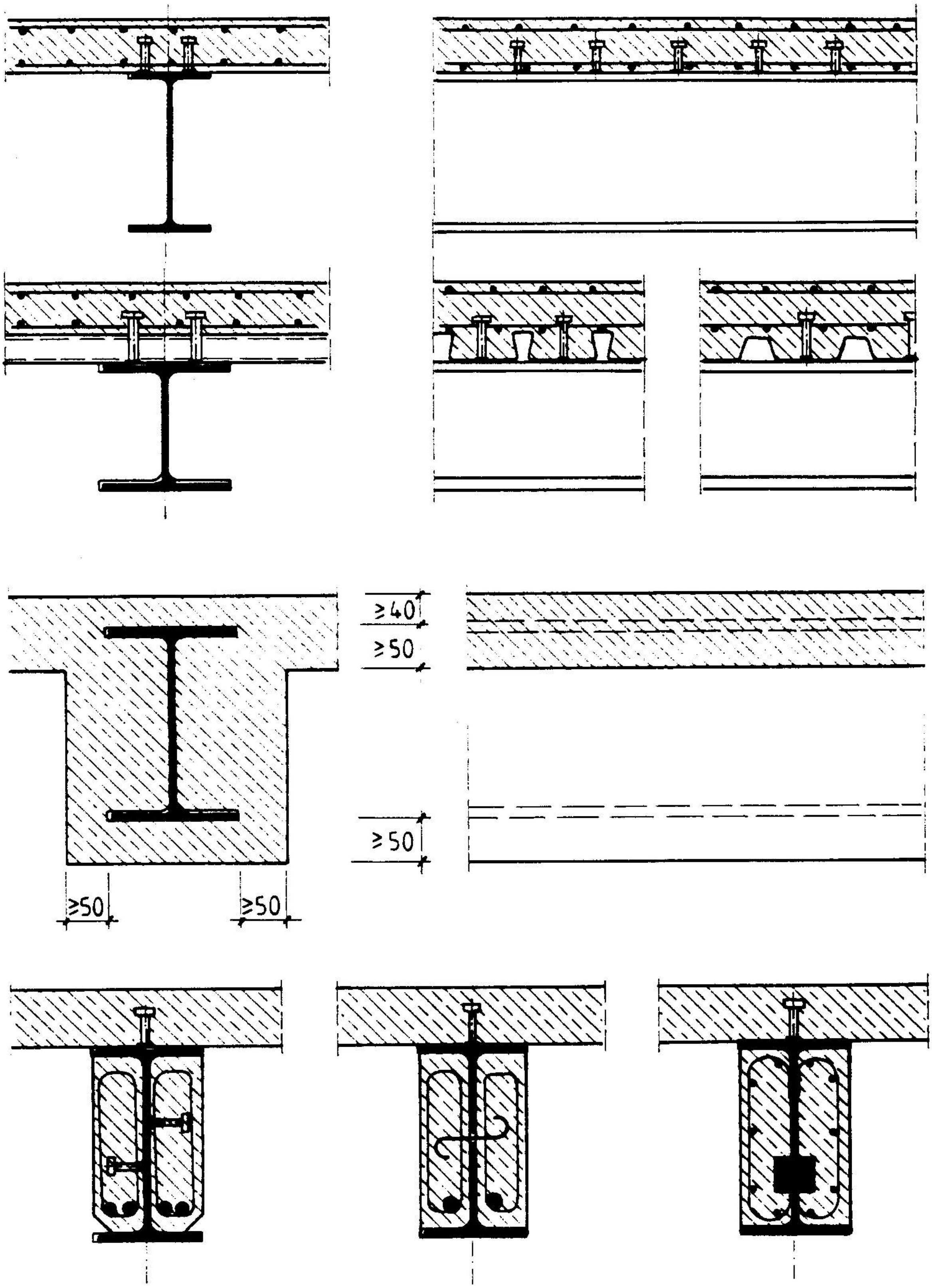


Fig. 2 Composite beam crosss-sections

denotes the bending due to the dead weigth of the structure.

Under service conditions, the different construction methods lead to different deflections, force distributions and stress states. But when the composite beams of same cross section are loaded up to failure, they fail at the same bending moment. Their strength is independent of the method of construction, and this bending strength can be calculated easily based on rectangular stress block, as demonstrated later.

There is another reason for different strain and stress distributions and deflections under service conditions: the longterm behaviour of concrete. Both, creep and shrinkage of the concrete part yields larger strains and stresses in the steel section under service conditions. At the ultimate limit state, however, strains due to loadings are much larger than the strains due to creep and shrinkage, and the latter can be neglected.

4. FAILURE MODES FOR SIMPLY-SUPPORTED COMPOSITE BEAMS

In order to develop a design method for composite beams, it is necessary to consider first the various possible failure modes. Simply-supported beams, with the upper surface of the concrete in compression, are considered here first. This form of construction is widely used for buildings and for similar structures, in which horizontal forces are resisted by stiff cores or some other form of bracing.

Tests and other studies show that many modes of failure are possible in composite beams, even when simply-supported (see fig. 5):

- failure of the composite member in flexure by the development of a plastic hinge in the sagging moment region (I);

- failure in vertical shear (II);

- failure in combined bending and vertical shear in hogging moment regions in case of continuous spans (III);

- reduction or complete loss of composite action, leading to collapse of the member in flexure, due to excessive slip or complete failure of the shear connection (IV);

- local shear failure in the slab in regions of high stress around shear connectors (V);

- horizontal shear failure in the adjacent concrete flanges (VI).

According to these failure modes, the ultimate strength of composite beams has to be determined. Additionally, the resistance to

- local buckling of web and flanges,

- lateral torsional buckling,

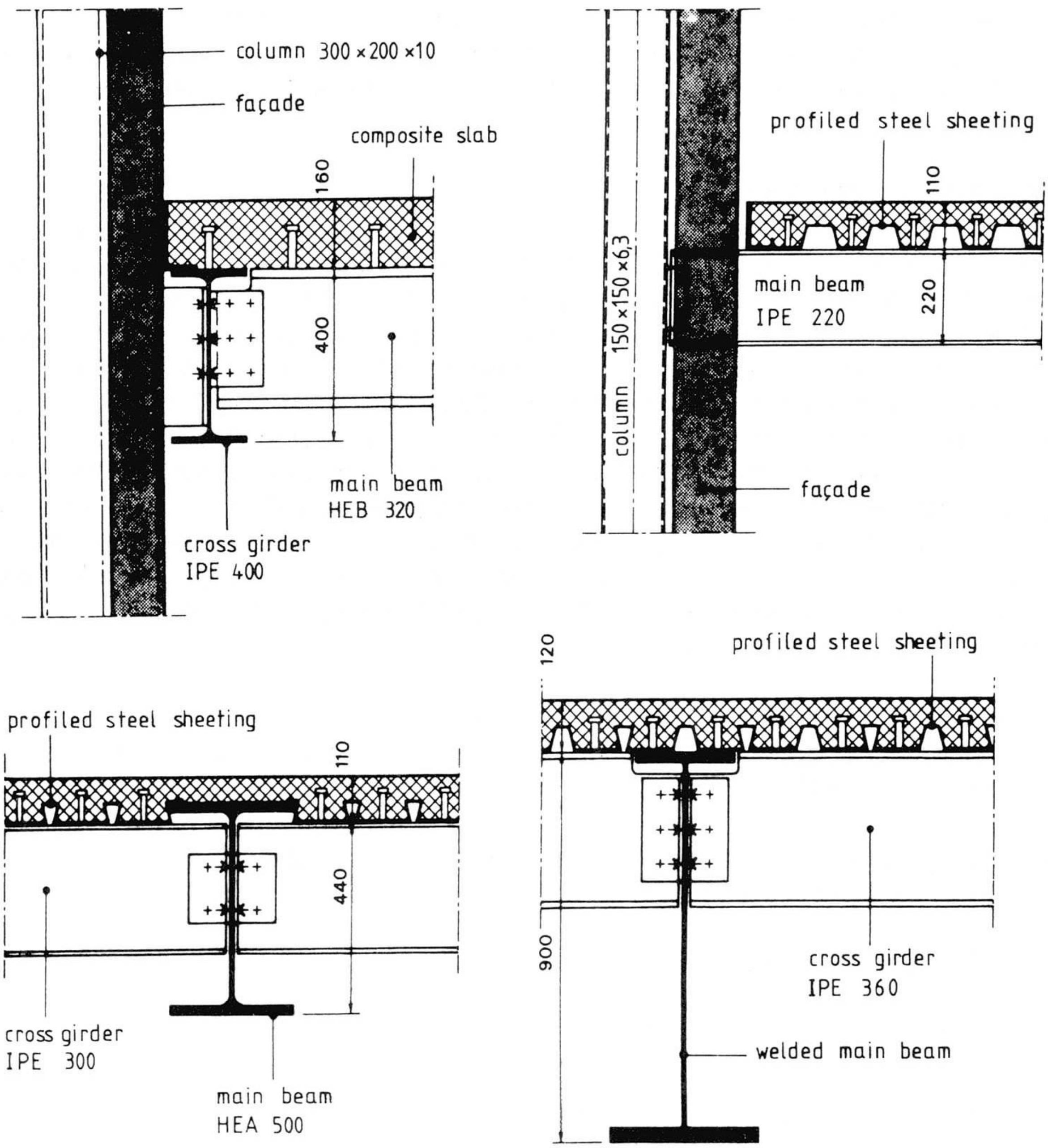


Fig. 3 Connection details

both with due regard to the compressed part of the considered steel section has to be investigated, and this in case of simply supported beams, where the neutral axis lies within the web, and in case of continuous span beams in hogging moment regions.

5. CROSS-SECTION STRENGTH

In this chapter the cross section strength has to be determined. It is assumed that calculation of forces is based on complete interaction without slip at the steel-concrete interface and that strength design is based on complete shear connection. Both, horizontal shear design as well as partial shear connection are concerned later. The determination of the ultimate moment of resistance in positive bending then can be based upon a linear strain distribution over the whole cross section. But the use of the stress-strain curves for concrete as given in Eurocode 2 leads to complex calculations when used for the bending strength of composite cross section. For the sake of simplicity therefore a simplified method using rectangular stress blocks for both steel and concrete is given in Eurocode 4, compare figure 6. The concrete design strength under compression should be reduced to 0,85, and the following partial safety factors for the different materials should be used generally:

- concrete: $\gamma_c = 1{,}5$
- structural steel: $\gamma_a = 1{,}0$ (under consideration)
- connectors with ductile behaviour: $\gamma_v = 1{,}25$

The bending moment resistance for design purposes can be calculated as follows, if the plastic neutral axis lies in the concrete slab:

$$M_{pl,Rd} = R_a \cdot (h/2 + h_a + h_c - z_{pl}) \tag{1}$$

where $$R_a = A_a \cdot f_y/\gamma_a \tag{2}$$

and $$z_{pl} = R_a/(b_{eff} \cdot 0{,}85 \cdot f_{ck}/\gamma_c) \leq h_c \tag{3}$$

In addition by eq. (3) due regard is taken of the application of metal decking with ribs transverse to the composite beam axis: the concrete compression force is only resisted by the concrete part above the profiled steel ribs.

Similar formulae can be given for other cross sections, where the neutral axis does not lie within the concrete cover above the ribs, but below in the steel flange or within the steel web. If the upper part of steel web is stressed under compression, at least compact steel sections (class 1 or 2, see below) should be used to prevent the compressed part from buckling.

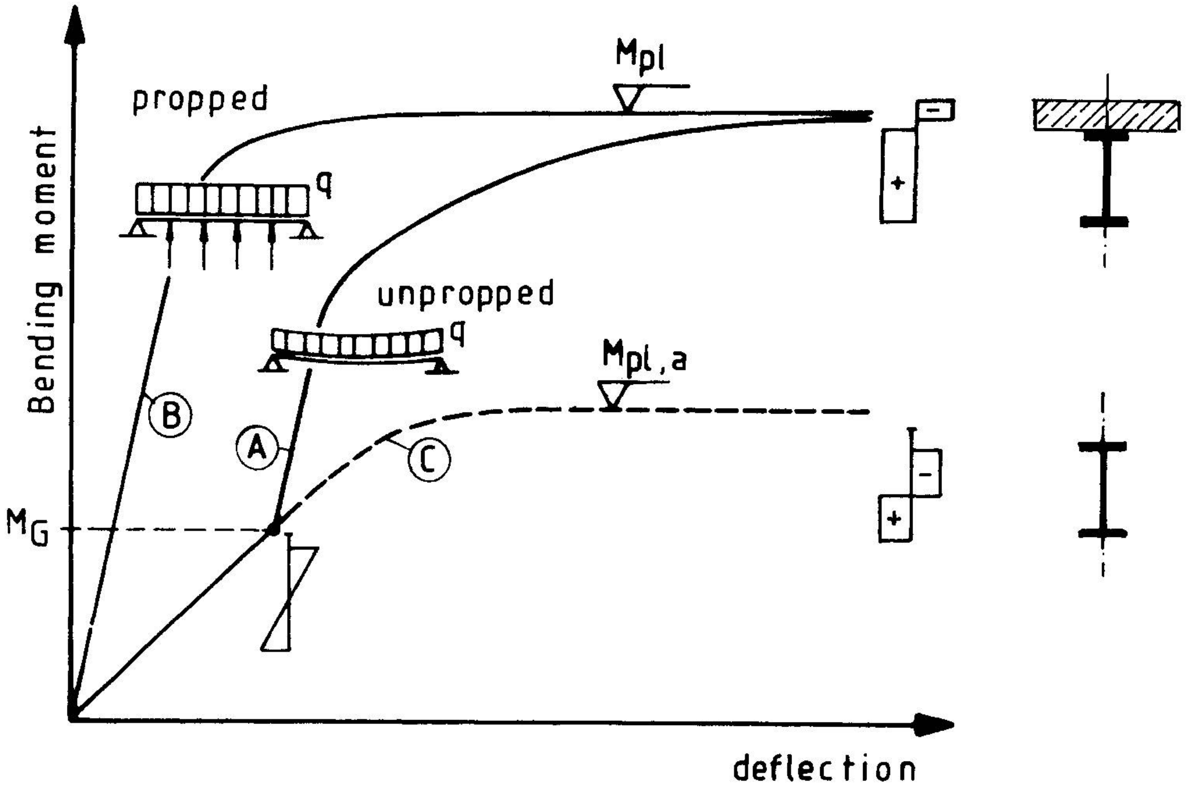


Fig. 4 Effects of propped and unpropped construction

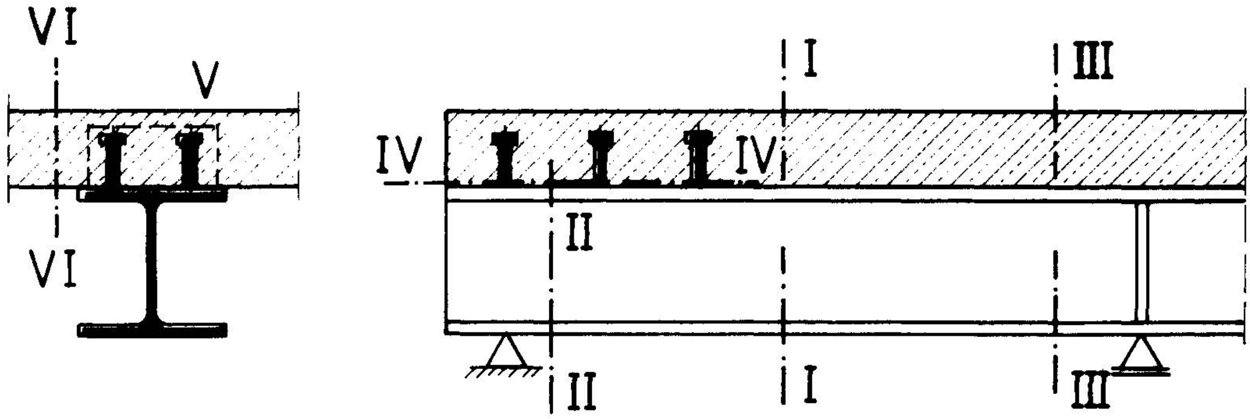


Fig. 5 Critical sections which have to be checked

Worked example (see fig. 7)

To verify the ultimate limit state for the given problem (see fig. 7), it shall be established, that at the midspan cross-section the design action effect does not exceed the bending resistance of the composite cross-section:

Design loadings:

$$\gamma_G = 1{,}35; \quad \gamma_Q = 1{,}50;$$
$$G_a = 0{,}907 \text{ kN/m}; \quad G_c = 18{,}0 \text{ kN/m}; \quad P = 22{,}5 \text{ kN/m};$$
$$Q = 1{,}35 \cdot (0{,}907 + 18{,}0) + 1{,}50 \cdot 22{,}5 = 59{,}27 \text{ kN/m};$$

Design bending moment at midspan:

$$M_{Sd} = 59{,}27 \cdot 13^2/8 = 1252 \text{ kNm}$$

Effective area of concrete flange (compare fig. 9):

$$L_e/b = 13/2{,}25 = 5{,}8; \quad \beta_1 = 0{,}84; \quad b_{eff} \geq 0{,}84 \cdot 4{,}50 = 3{,}78 \text{ m}$$

Partial material safety coefficients:

$$\gamma_c = 1{,}5; \quad \gamma_a = 1{,}0;$$

Design strength of steel and concrete:

$$f_y = 35{,}5 \text{ kN/cm}^2; \quad f_{ck} = 3{,}5 \text{ kN/cm}^2;$$

Position of the neutral axis assuming full interaction between steel and concrete:

$$z_{pl} = (116 \cdot 35{,}5/1{,}0)/(378 \cdot 0{,}85 \cdot 3{,}5/1{,}5) = 5{,}5 \text{ cm} < 10{,}9 \text{ cm}$$

The ultimate bending resistance may be determined by plastic theory using the full plastic bending moment:

$$M_{pl,Rd} = 116 \cdot 35{,}5/1{,}0 \cdot (25 + 16 - 5{,}5/2) = 1575 \text{ kNm} \geq M_{Sd}$$

These few calculation steps verify a sufficient bending resistance of the considered composite beam.

We should realize, that the effective width or area - concept has already been used in this short worked example. A typical form of composite construction consists of a slab connected to a series of parallel steel members. The structural system is therefore essentially a series of interconnected T-beams with wide, thin concrete flanges, as shown in fig. 1.

In such a system, the flange width may not be fully effective in resisting compression due to "shear lag". This phenomenon will be explained by reference to a simply-supported

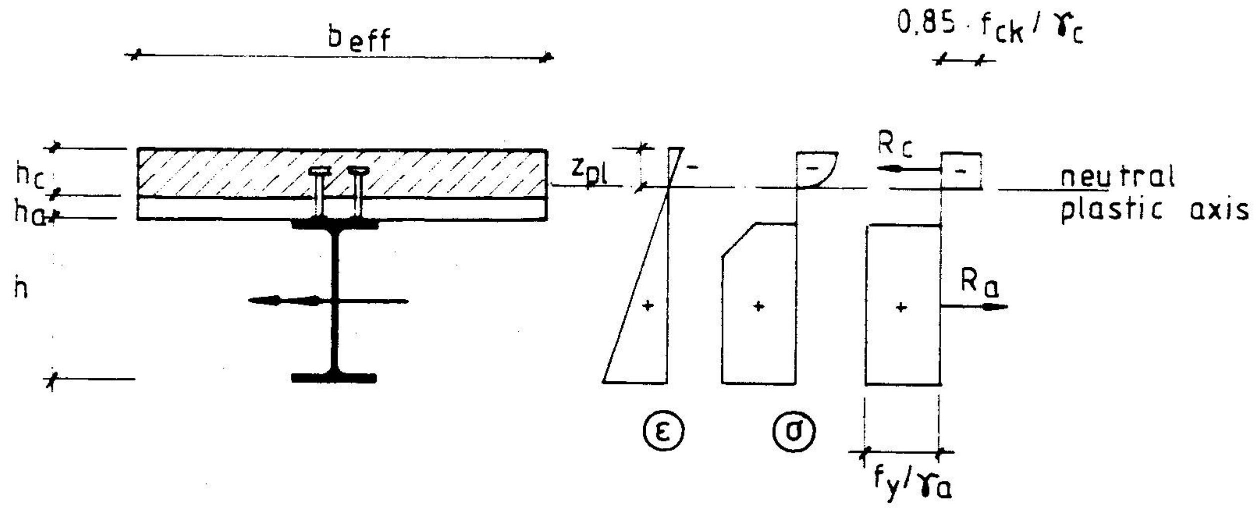


Fig. 6 Strain and stress distribution (sagging bending moment)

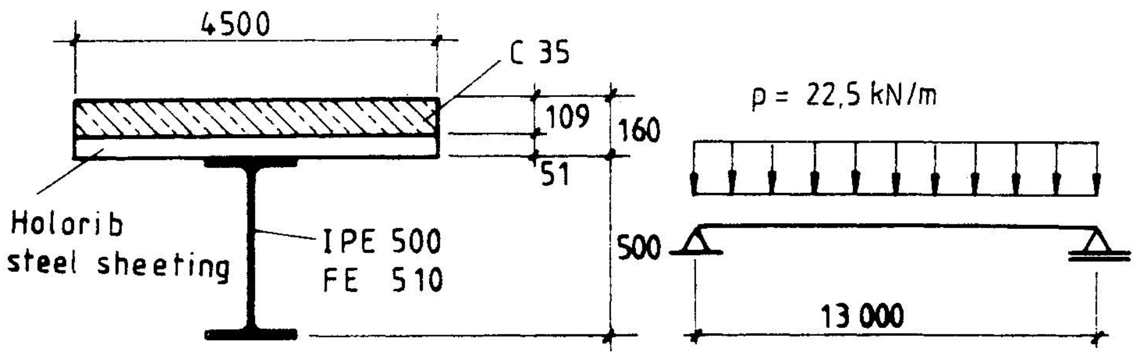


Fig. 7 Worked example - assumed problem

member, part of whose length is shown in plan in fig. 8.

The maximum force in the slab will be in the mid-span region, whilst the force at the ends will be zero. The change in longitudinal force is associated with shear in the plane of the slab. The resulting deformation shown in fig. 8, is inconsistent with simple bending theory, in which initially plane sections are assumed to remain plane thereafter. The edge regions of the slab are effectively less stiff, and a non-uniform distribution of longitudinal bending stress is obtained across the section. The simple theory will give the correct value of the maximum stress if the true flange widths b_1 and b_2 are replaced by an effective width of breadth $b_{eff} = b_{e1} + b_{e2}$, such that the area GHJK equals the area ACDEF (fig. 8).

An exact examination shows that the ratio b_e/b depends not only on the relative dimensions of the system, but also on the type of loading, the support conditions and the cross section considered.

In Eurocode 4, very simple formulae are given for the calculation of effective widths, though this may lead to some loss of economy. For simply-supported beams, EC4 proposes that the effective width b_e on each side of the steel web should be L/8, but not greater than half the distance to the next adjacent web. As an alternative the procedure outlined in fig. 9 can be used to calculate the effective-breadth ratios b_e/b, denoted by β. The breadth b_w as defined in fig. 9 may be ignored in the determination of b_e and b.

Fig. 10 demonstrates the determination of the bending strength under negative moments, while fig. 11 represents the vertical shear - bending moment interaction. With regard to vertical shear it is assumed, that this vertical shear is resisted by the structural steel section alone. The treatment of vertical shear therefore follows that used in the design of steel beams.

The shear area of the steel member is defined as follows:

$$A_v = d \cdot t_w \qquad \text{welded I-sections} \qquad (4)$$
$$A_v = 1{,}04 \cdot h \cdot t_w \qquad \text{rolled I-sections} \qquad (5)$$

In the latter formula, parts of the steel flanges are also taken into account.

The design plastic shear resistance is given by

$$V_{pl,Rd} = A_v \cdot f_y/\gamma_a/\sqrt{3} \qquad (6)$$

In case of simply supported beams, the influence of vertical shear on the ultimate moment of resistance can be neglected, because cross sections under maximum bending and under maximum vertical shear do not coincide (see fig. 5).

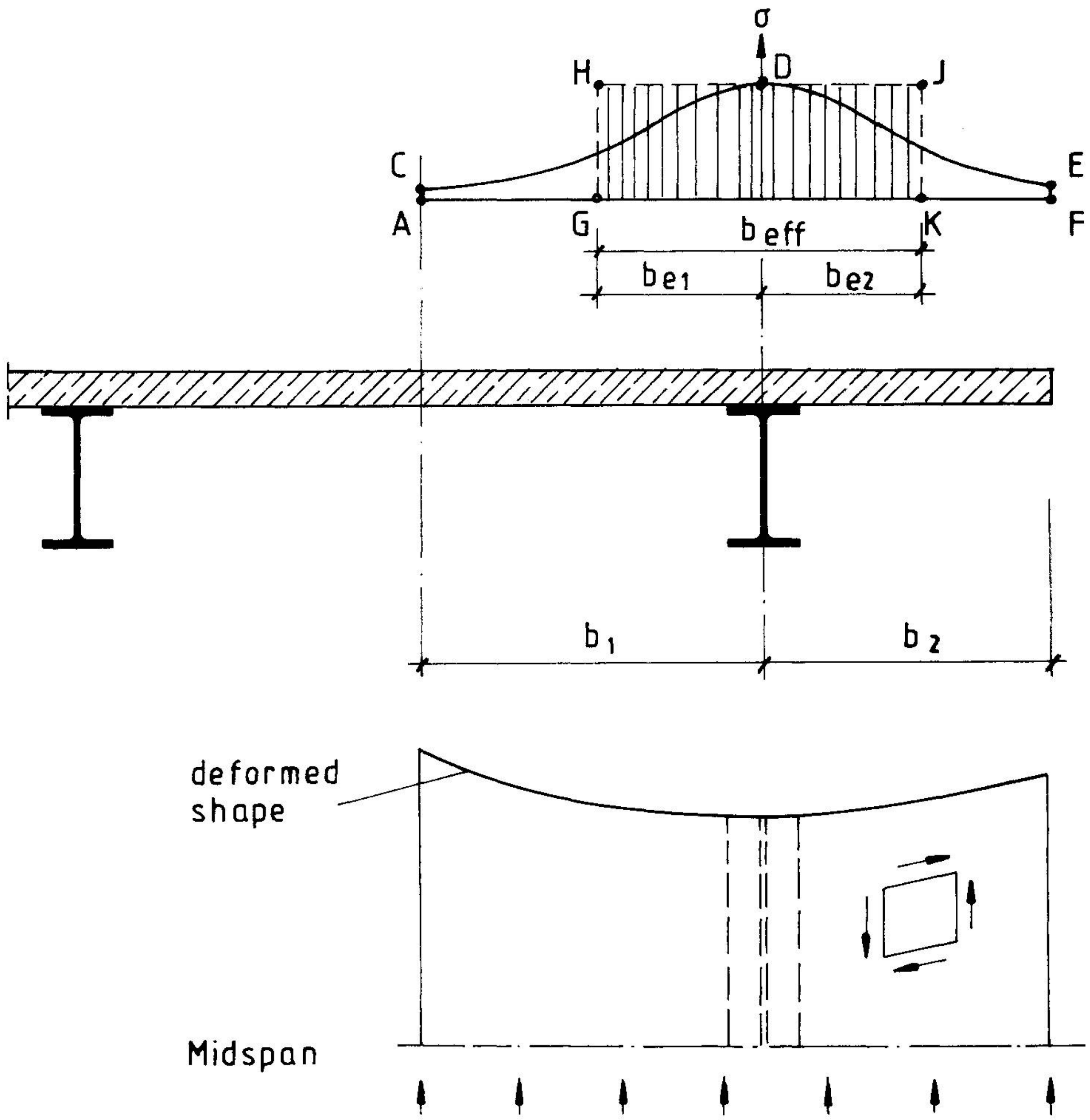


Fig. 8 Unequal bending stress distribution due to shear lag

6. CONTINUOUS SPAN COMPOSITE BEAMS

This chapter deals with continuous beams over simple supports. The following advantages in comparison with single span beams should be mentioned:

- greater load capacity due to the redistribution of bending moments;

- greater stiffness;

- smaller steel section to withstand the same loading.

On the other hand, we observe sometimes an increased complexity in design, particularly with regard to lateral torsional and local buckling problems in negative moment regions. But in most cases, where rolled sections, which belong to class 1 or 2 are used, these are not real problems.

Steel sections can be classified into class 1 to 4 depending upon the local buckling behaviour of steel flange and/or web in compression. In case of a simply supported composite beam, we have already used the plastic design method for the cross-section - see chapter 4 and the worked example. Sections of class 2, however, are admitted only if no rotation capacity is required, that means, if global plastic analysis design is not applied.

We distinguish (compare table 1) between:

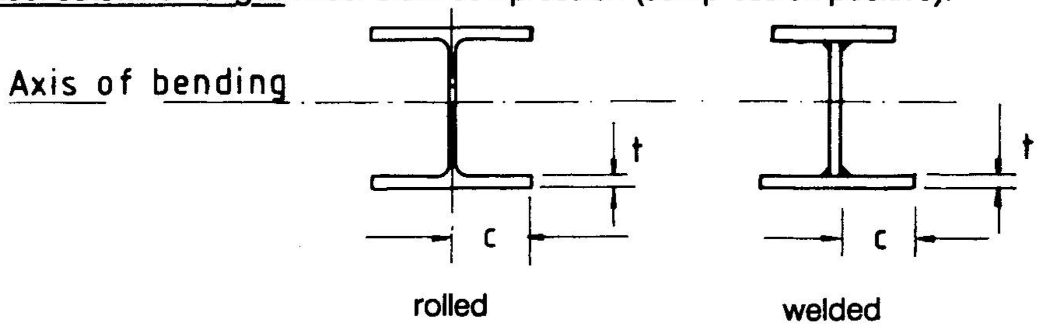
- class 1: plastic cross sections, which can form out a plastic hinge with sufficient rotation capacity for global plastic analysis.

- class 2: compact cross sections, which can develop the plastic moment of resistance, but which have limited rotation capacity.

The steel compression flange attached to the concrete flange and sufficiently interconnected with it by means of mechanical connectors may be assumed to be in class 1. For the other parts of steel sections under compression, the width to thickness ratio has to be checked in accordance with table 1 (partly table 4.2 of Eurocode 4).

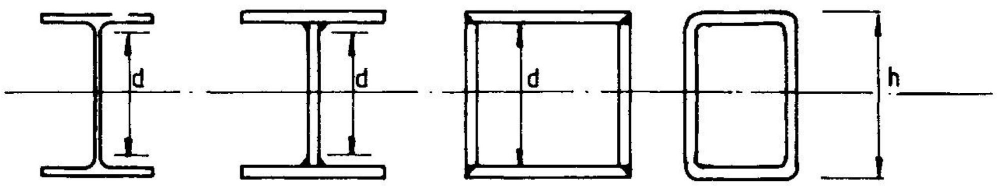
Fig. 12 demonstrates the elastic-plastic behaviour of a two span continuous composite beam under uniformly distributed loading including the simplifying assumption, that plastic strains and deformations are concentrated at discrete points and may be represented by so called "plastic hinges". According to this with class 1 members rigid-plastic global analysis may be applied. For the analysis to be valid, critical cross sections must be capable of developing and sustaining their plastic resistance moment until - under increasing load - sufficient zones have fully-yielded for a mechanism of plastic hinges to be present.

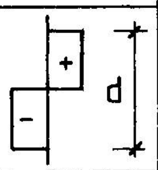
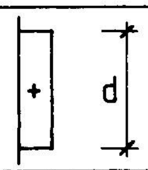
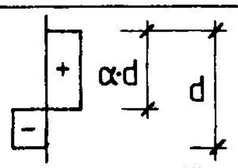
From fig. 12 and 13 these plastic hinges, which have formed out over the internal support and in the sagging moment regions, can be seen very clearly. The same applies to the mechanism of plastic hinges, which arose as result of redistribution of bending moment, steel outstand flanges in constant compression (compression positive):



class	type	web not encased	web encased
1	rolled	$c/t \leq 10\varepsilon$	$c/t \leq 10\varepsilon$
	welded	$c/t \leq 9\varepsilon$	$c/t \leq 9\varepsilon$
2	rolled	$c/t \leq 11\varepsilon$	$c/t \leq 15\varepsilon$
	welded	$c/t \leq 10\varepsilon$	$c/t \leq 14\varepsilon$

steel webs (compression positive):



class	bending	compression	combined bending and compression
stress distribution			
1	$d/t \leq 72\varepsilon$	$d/t \leq 33\varepsilon$	$d/t \leq 396\varepsilon/(13\alpha-1)$ for $\alpha > 0.5$ $d/t \leq 36\varepsilon/\alpha$ for $\alpha < 0.5$
2	$d/t \leq 83\varepsilon$	$d/t \leq 38\varepsilon$	$d/t \leq 456\varepsilon/(13\alpha-1)$ for $\alpha > 0.5$ $d/t \leq 41.5\varepsilon/\alpha$ for $\alpha < 0.5$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355
	ε	1.00	0.92	0.81

Table 1 Maximum width to thickness ratios (class 1 and 2)

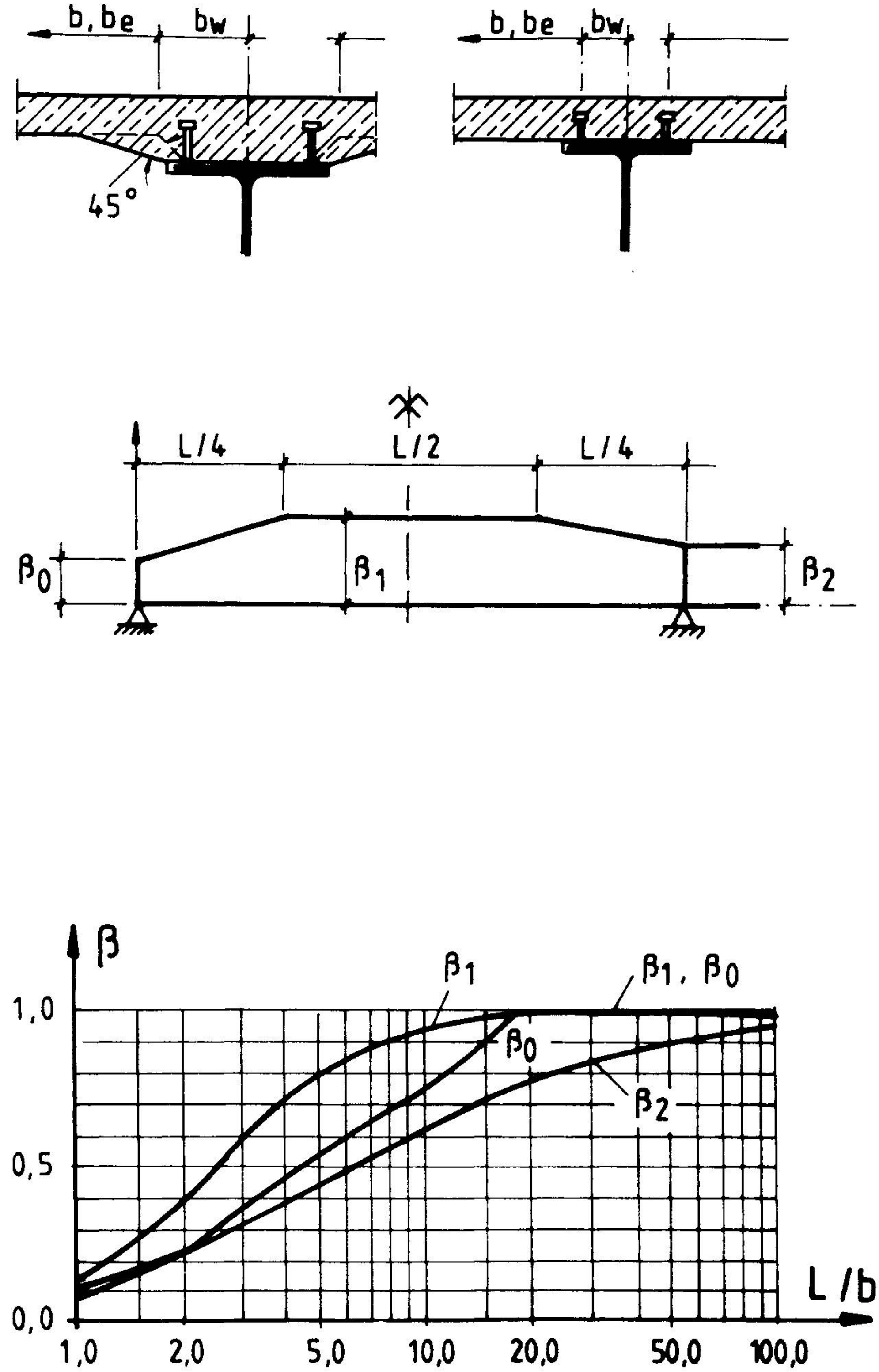


Fig. 9 Effective width - calculation aids

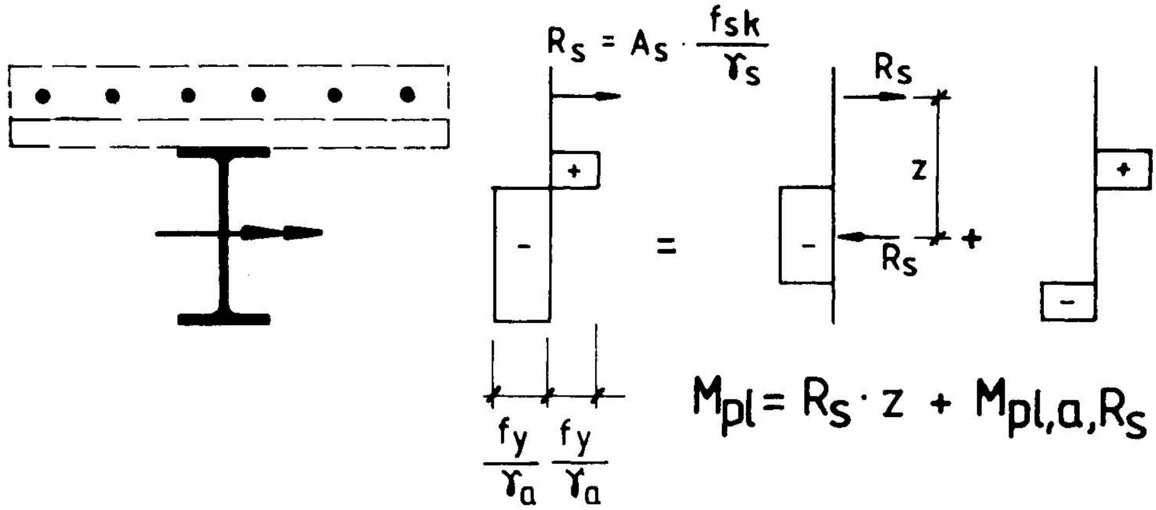


Fig. 10 Hogging bending moment

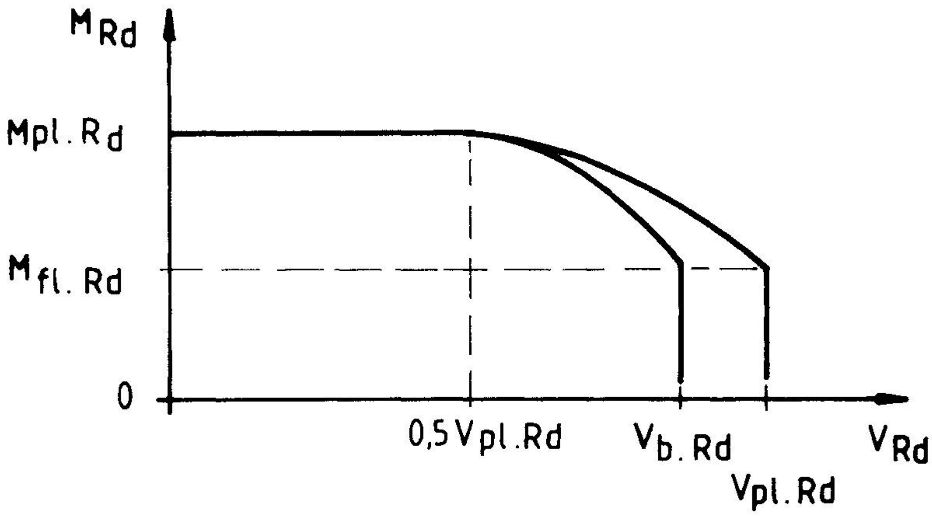


Fig. 11 Interaction for bending and vertical shear

which is achieved by rotation of the already-yielded regions (in this case: the first over the interior support, the last at midspan). To ensure that the resulting strains can be accomodated without any reduction in resistance below the plastic moment, the limitations as given in table 1 must be placed on the width to thickness-ratio of the steel section in class 1.

For an application of global plastic analysis to composite beams in buildings, further requirements have to be satisfied:

- adjacent spans should not differ in length by more than 50 % of the shorter span;

- end spans should not exceed 115 % of the length of the adjacent spans;

- in case of concentrated heavy loads, at any hinge location, where the concrete slab is in compression, not more than 15 % of the overall depth of the composite member should be in compression;

- the steel compression flange at a plastic hinge should be laterally restrained;

In addition, figure 13 considers the effective cross-sections. In the cross section over the internal support, the reinforcement has been neglected. If it shall be taken into account, however, only reinforcement of high ductility should be included in the effective section.

In any case a sufficient anti-crack reinforcement should be provided there, if the concrete slab is not interrupted by a joint.

7. CONNECTORS AND SHEAR CONNECTION

Mechanical connectors develop the composite action between steel beams and the concrete part. As this connection is provided mainly to resist longitudinal shear, it is referred to as the "shear connection". The following definitions should be used, which are related to the basic properties of structural elements such as strength, stiffness and deformation capacity.

With regard to strength we distinguish between complete and partial shear connection. The connection is considered to be complete, if the horizontal shear resistance is not decisive for the design carrying capacity of the composite beam, but the design bending strength. Partial shear connection is dealt with in the next chapter.

If we consider the composite action and how both parts of the composite beam act together, we can observe complete or incomplete interaction, which results in a more or less stiff composite beam. Such incomplete interaction arises when flexible connectors are used due to slip (relative displacements) at the steel-concrete interface.

Eurocode 4 concerns complete shear connection and - under certain restrictions - also partial shear connection. In the last case the deformation capacity of the connection has

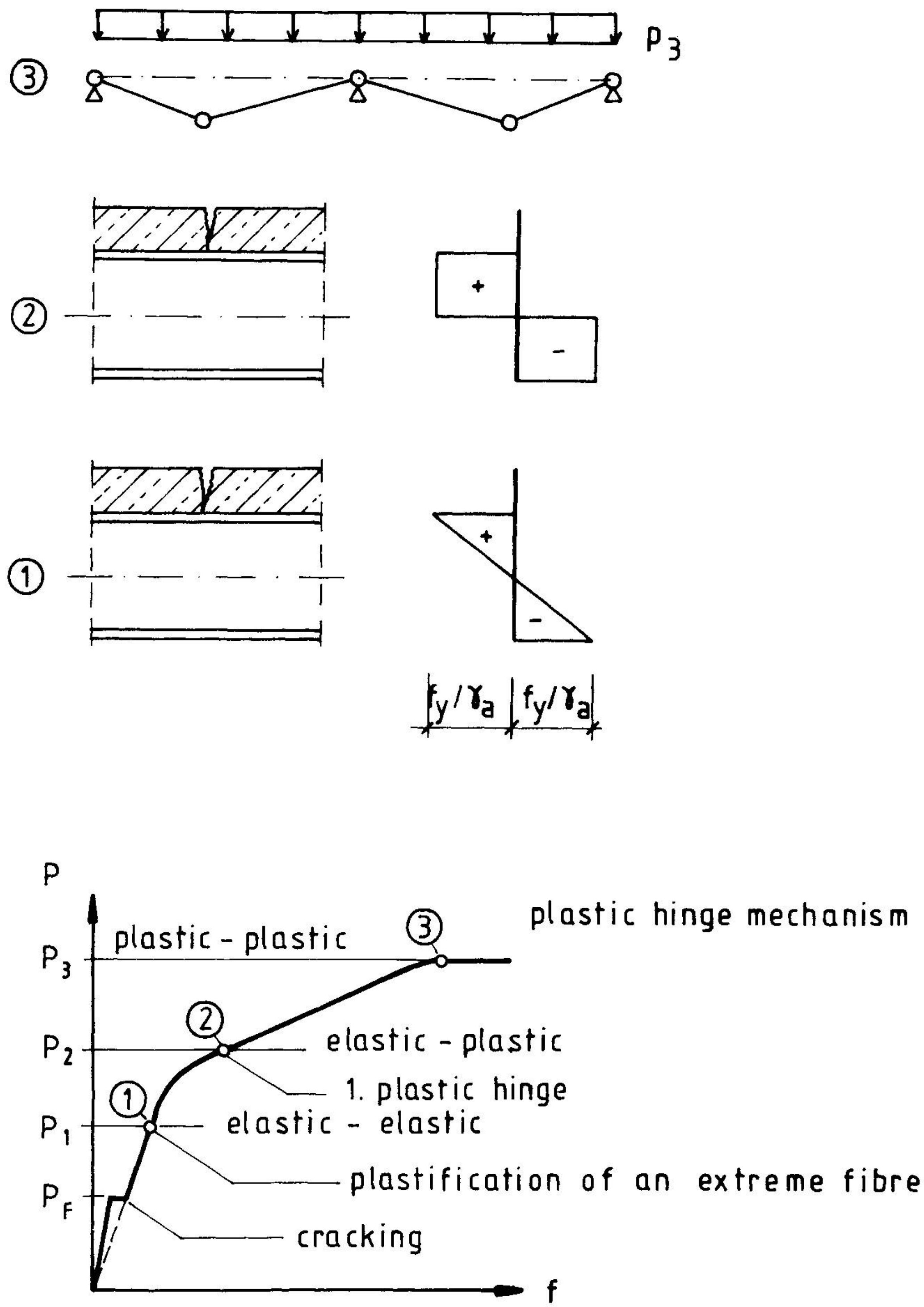


Fig. 12 Development of plastic hinge mechanism

to be taken into account.

Fig. 14 shows several types of shear connectors. Basicly they have to fulfill a couple of requirements, for example:

- to transfer direct shear at their base;

- to create a tensile link into the concrete;

- to enable an economical production.

Nowadays in the industrialized countries the most common one is the headed stud connector. It can be welded semi-automatically (see fig. 15) to the upper flange, and this either directly in the shop or through thin galvanized steel sheeting on site.

On construction sites, where metal decking is used and sufficient power supply is not available or ensured, alternatively shot fired connectors are used sometimes, but not yet in all countries.

The advantge is, that instead of a complex throughwelding technology modified cartridge guns can be used.

For precast concrete decks, high strength friction grip bolting has been used sometimes, particularly in Germany for car parks.

Behaviour and strength of headed studs and other connectors are examined by means of shear or push tests. These tests yield load-slip curves for headed studs as given in figure 16. The behaviour is characterized by great stiffness at lower loading and large deformations at higher loads up to failure. This ductile behaviour makes shear force redistribution possible and makes allowance for partial shear connection with slip at the steel-concrete interface. Besides, this figure 16 demonstrates the difference between behaviour and strength of headed studs in solid slabs and embedded in concrete, which has been casted on profiled steel sheeting.

Design rules are given for stud connectors in beams for which plastic theory has been used to calculate the cross-section resistance. In case of complete shear connection the total longitudinal shear V_l between critical cross-sections should be resisted, where V_l is the smaller value of the axial capacity of the concrete slab and the steel beam. P_{RD} is the design shear resistance of each connector, that is the characteristic value devided by the appropriate material safety factor $\gamma_V = 1{,}25$.

The number of connectors is determined by

$$N_f = V_l / P_{Rd} \tag{7}$$

where $$P_{Rd} = 0{,}9 \cdot f_u \cdot (\pi d^2/4)/\gamma_V \tag{8}$$

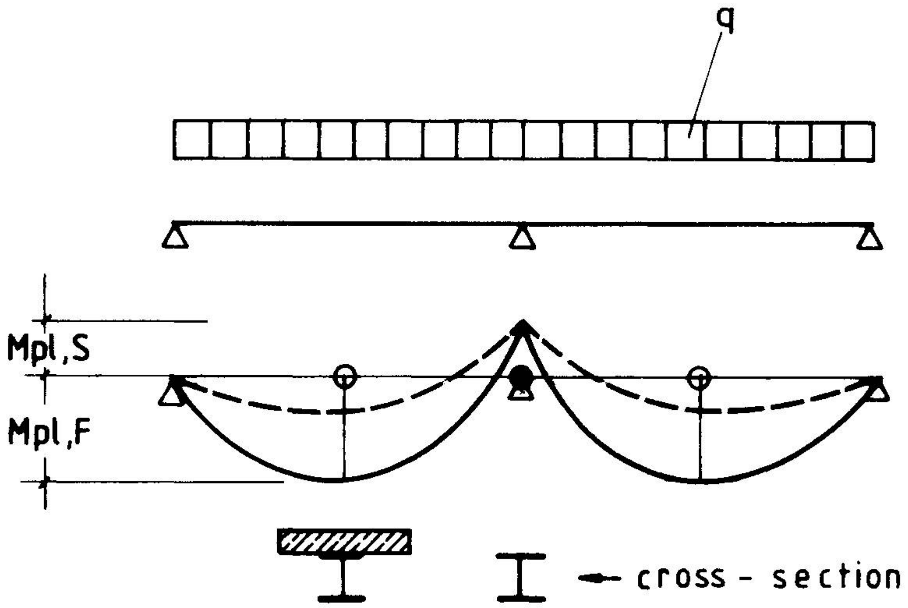


Fig. 13 Rigid-plastic global analysis

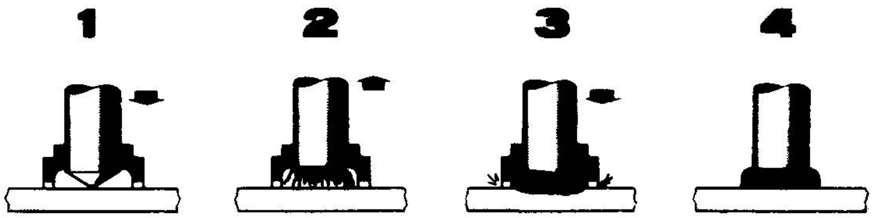


Fig. 15 Welding-process of headed studs

$$\leq 0{,}29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}} / \gamma_v \qquad (9)$$

d	is the diameter of the shank of the stud;
f_u	is the specified ultimate tensile strength of the material of the stud;
f_{ck}	is the characteristic cylinder strength of the concrete at the age considered;
E_{cm}	is the mean value of the secant modulus of the concrete
α	$= 0{,}2 \cdot [(h/d) + 1]$ for $3 \leq h/d \leq 4$;
α	$= 1$ for $h/d > 4$;
h	is the overall heigth of the stud.

P_{Rd} based on equations (8) to (9) is valid for headed studs embedded in solid concrete slabs and equals the following values:

$$P_{Rd} = 110\ \text{kN}/\gamma_v \quad \text{for } \emptyset\ 19\ \text{mm} \ (3/4")$$
$$= 130\ \text{kN}/\gamma_v \quad \text{for } \emptyset\ 22\ \text{mm} \ (7/8")$$

We have already seen, that the strength is noteworthy lower, if profiled steel sheeting is used (see fig. 16). The correct reduction is under consideration till now, but the following formulae have been proposed for example as P_{RD} for sheeting with ribs transverse to the supporting beam:

- with studs placed in holes in the steel sheeting:

$$P_{Rd} = (0{,}25\, f_u\, d^2/\alpha \cdot \beta \cdot \alpha_n)/\gamma_v \qquad (10)$$

where
- $\alpha = 0{,}8\,(h_a/b_t)^2 + 0{,}6$
- b_t = is the effective width at the top of the rib
- β = 1,0 for sheets with trapezoidal shape; 1,1 for sheets with reentrant shape.
- $\alpha_n = 1{,}0$ if $N_r = 1$;
  $= 0{,}3 + 0{,}15\,(h - ha)/d \leq 1$ if $N_r = 2$.

- with studs welded through the sheeting, P_{Rd} is the smaller value obtained from equation (8) and equation (9) multiplied with the following reduction factor:

$$(0{,}3/\sqrt{N_r}) \cdot (b_o/h_a) \cdot (h/h_a - 1) \leq 1.0 \qquad (11)$$

where
- b_o is the effective width of one rib;
- N_r is the number of studs in one rib at a beam intersection, not to exceed 2 in computations.

Ductile connectors as for instance headed studs may be uniformly spaced along the beam length between critical cross sections. This is possible due to less or more slip, which can be seen from fig. 16 and which causes redistribution of horizontal shear flow. In addition - and this even in case of rigid connectors - the real shear flow differs from the

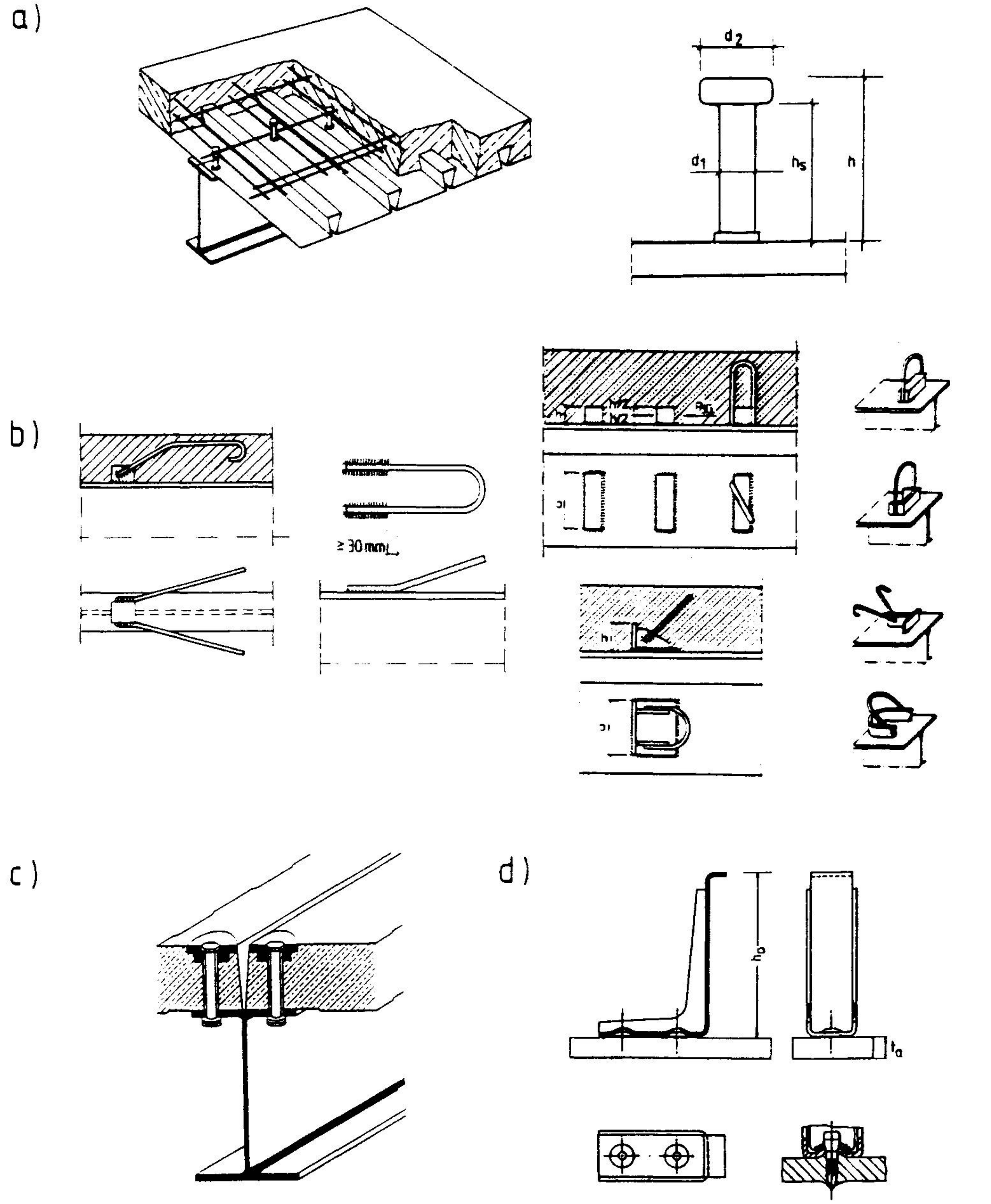


Fig. 14 Some types of connectors

calculated distribution based on theory of elasticity in those regions, where plastic strains are present. Figure 17 shows these effects for a single span composite beam, when loaded up to its load capacity, and this for rigid as well as for flexible connection.

In case of heavy concentrated loads, however, the connectors should be spaced broadly in accordance with the elastically calculated shear flow along the beam length.

Headed studs are welded to the top flange either directly in the workshop or on site through the metal decking, if there is any. About these procedures there are different opinions. In Germany for example we are welding 7/8" headed studs in the shop and use holed profiled steel sheeting. The British colleagues on the other hand are very successful in welding 3/4" studs through metal decking on site.

8. PARTIAL SHEAR CONNECTION DESIGN

The design of the shear connection forms the subject of another lecture. It is assumed that there will be a sufficient number of connectors for the strength of the beam to be not less than that given by plastic analysis. The resulting method of design is known as "complete shear connection design". However, this may result in a large number of shear connectors, particularly if the dimension of the cross-section or the material properties are governed by factors other than the strength of the composite beam. The most economical design may then be that, in which the number of shear connectors is just sufficient to provide the required flexural strength. Such "partial shear connection design" my be unavoidable when a slab is constructed with permanent formwork of profiled sheeting. The number of shear connectors attached to the steel beam may then be limited by the restriction of only being able to place them in the troughs of the sheeting profile. Partial shear connection design may only be applied to beams not exceeding 20 m in span and subject to predominantly static loading. For other types of beams the method has to be improved (the work has not yet been finished).

Figure 18 presents partial shear connection theory, if the degree of shear connection is less than 100 % and if the number of connectors N is less than N_f. N_f shear connectors are necessary to develop the ful plastic bending moment $M_{pl,Rd}$.

If flexible connectors, for example headed studs are used, which have enough deformation capacity, then the design may be based on partial interaction with slip at the steel-concrete interface. In such a design the upper curve 3 as well as the straight line 2 can be used to determine the reduced bending strength depending on the actual degree of shear connection, see fig. 18.

In addition, in simply supported composite beams, no account need usually be taken of local buckling in the steel section. The compression flange is attached to the concrete slab by shear connectors, and the depth of the web in compression is usually small. In case of partial interaction however, the depth of the compressed part of the web is extended. There remains at least theoretically the possibility, that local buckling could occur in the web of a deep plate girder or in flange with a wide outstand beyond the shear connectors. It is assumed in this lecture that the proportions are such that plastic analysis

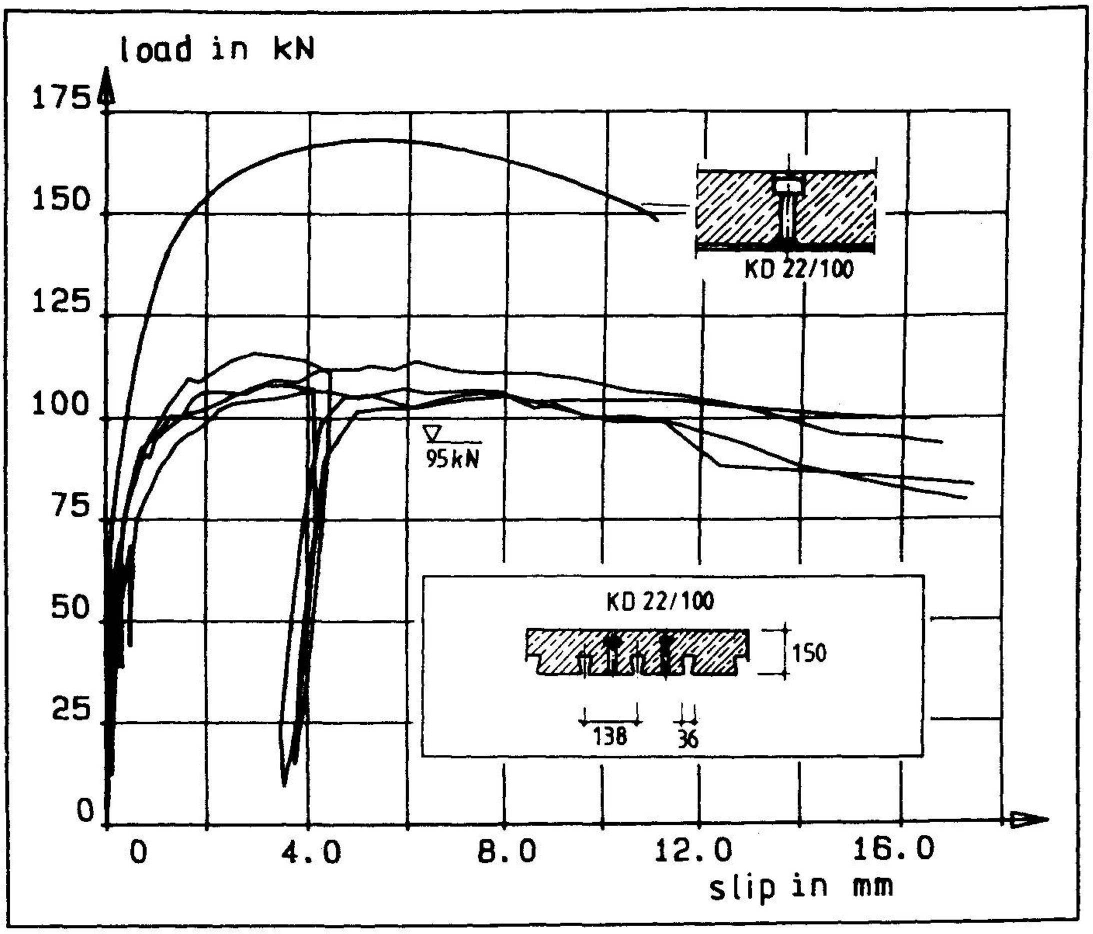


Fig. 16 Load - slip curves of stud shear connectors

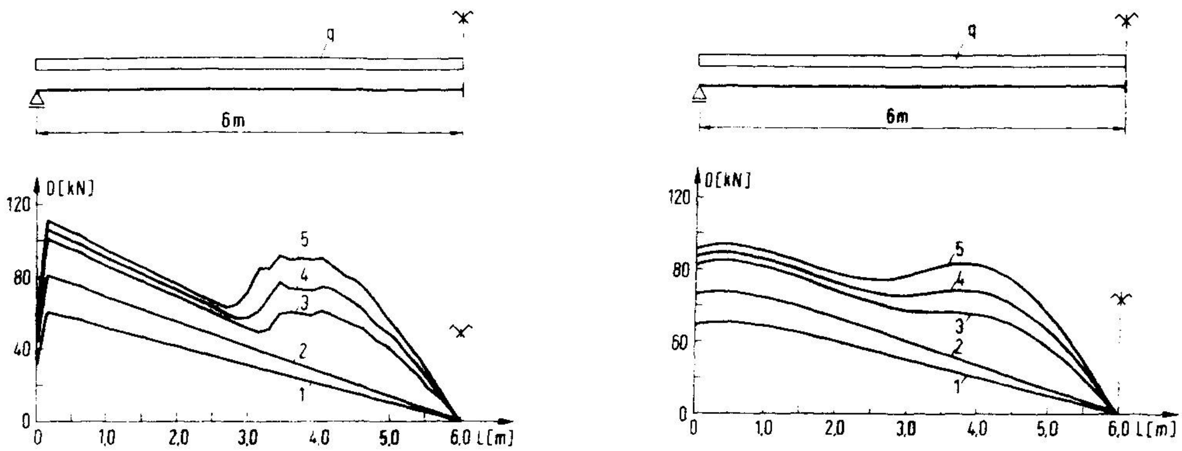


Fig. 17 Shear forces of rigid and elastic assumed connectors

can be applied to the cross-section of the composite beam. With other words: The calculation of the ultimate moment of resistance is therefore an application of the rectangular stress block diagram, and the steel section should belong to class 1 or 2 (see table 1).

9. PARTIALLY ENCASED BEAMS

In this last chapter, it should be reported on a very interesting development of the last 10 years: partially encased beams. In addition to steel section and concrete slab, they consist of reinforced concrete that encases the web between the two steel flanges, see fig. 19.

This concrete that encases the web shall be reinforced, mechanically connected to the steel section, and capable of preventing buckling of any part of the compression flange towards the web. The chamber concrete should be reinforced by longitudinal bars and stirrups, and should be attached to the web by stud connectors, welded bars, or bars through holes.

The most important feature of such a partially encased composite beam is its high fire resistance rating without any further fire protection measure. In addition the steel web is prevented from crippling and local buckling, and the resistance of the steel beam against lateral-torsional buckling is noteworthyly enhanced. And after all the greater stiffness of the total composite beam under bending and vertical shear should be pointed out, due to which deflections are reduced.

The concrete parts are casted either in a workshop or on site before the erection, and this enables a very quick construction with prefabricated composite members. Regarding the fire resistance, the concrete cover is preventing the inner parts of cross-section - steel and concrete - from warming up too fast.

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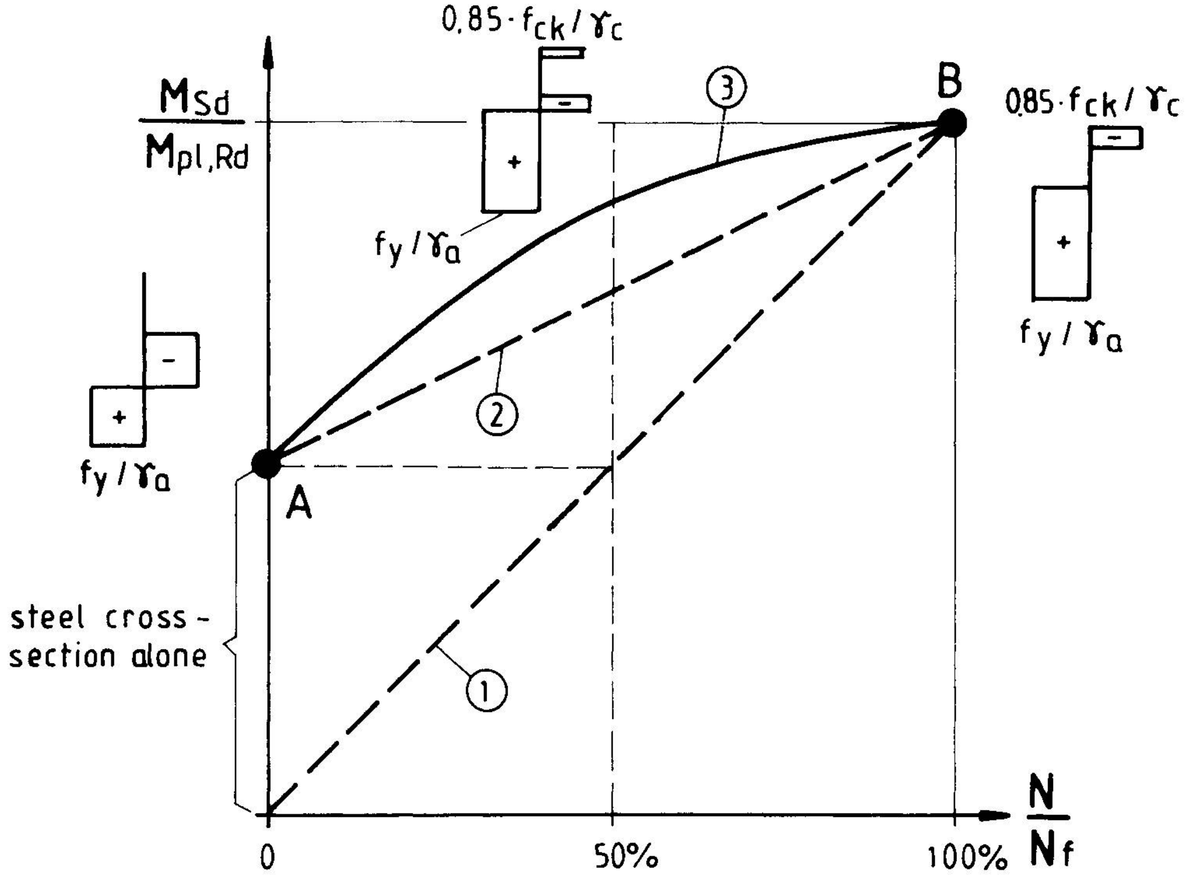

Fig. 18 Partial interaction

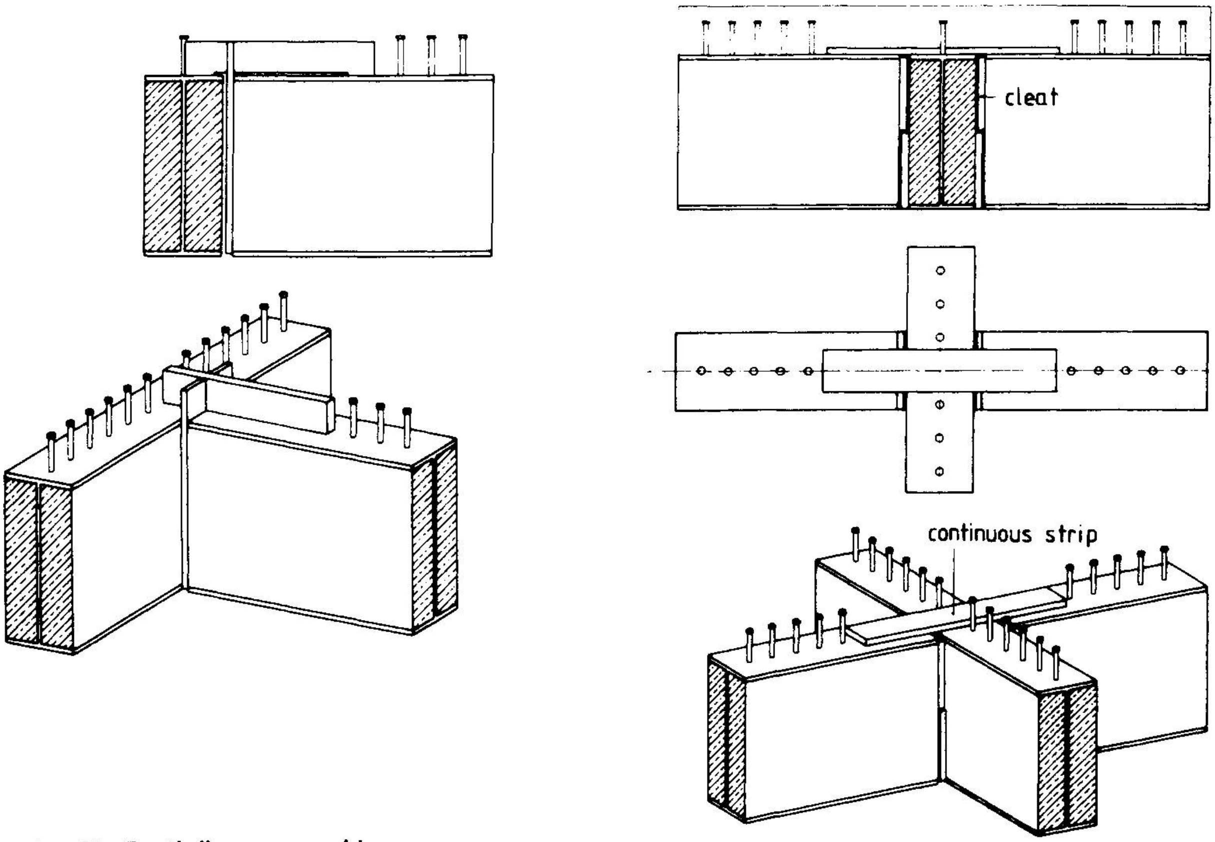

Fig. 19 Partially encased beams

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