Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 65 (1992)

Artikel: Serviceability, shear connection and composite slabs
Autor: Stark, J.W.B.
DOI: https://doi.org/10.5169/seals-50053

EC 4: Serviceability, Shear Connection and Composite Slabs

EC 4: Aptitude au service, connecteurs et planchers mixtes

EC 4: Gebrauchstauglichkeit, Scherverbindungen und Verbundplatten

J.W.B. STARK
Prof.
Technical University
Eindhoven, The Netherlands

SUMMARY

In this paper an overview is given of Eurocode 4, and includes subjects on the control of deflections and cracking, design rules for shear connection in beams, design of composite slabs with profiled steel sheeting and design assisted by testing. Special attention is given to modifications of the rules in the 1985 draft of Eurocode 4.

RESUME

L'article donne une aperçu de l'Eurocode 4 traitant du contrôle des flèches et des fissures, des règles de projet pour les assemblages à cisaillement dans les poutres, le projet de planchers mixtes avec des tôles métalliques profilées, et du projet de calcul basé sur des essais. Une attention particulière est portée aux modifications des règles présentées dans le projet 1985 de l'Eurocode 4.

ZUSAMMENFASSUNG

Dieser Beitrag gibt einen Rückblick über verschiedene Kapitel des Eurocode 4. Dazu gehören der Nachweis von Durchbiegung und Rissweite, Bemessungsregeln für Schwerverbindungen in Trägern, für Verbundplatten auf Trapezblechen und die experimentell gestützte Bemessung. Besondere Aufmerksamkeit gilt den Änderungen gegenüber dem Entwurf von 1985.

1. SERVICEABILITY LIMIT STATES

1.1 General

In chapter 5 application rules are given only for the control of deflections and of cracking of concrete in beams. Serviceability limit states for composite slabs, precast concrete slabs, and friction grip bolts used as shear connectors are covered in the relevant chapters.
Analysis of the structure, and of sections, for the serviceability limit state are avoided wherever possible. Where analysis is required, creep may be allowed for by using an "effective" modulus for concrete. In most building structures only a single modular ratio is needed. The value may be taken equal $E_{cm}/2$, where E_{cm} is the modulus for short-term loading. If specified for a particular project and in any case for buildings intended for storage, two values E_c' should be used: E_{cm} for short term effects and $E_{cm}/3$ for long term effects.
The effects of shrinkage of concrete on deflections need only be taken into account for simply supported beams in buildings when the span-to-depth ratio exceeds 20 and the predicted free shrinkage strain exceeds 400×10^{-6}.

1.2 Deflections

The recommended limiting values for deflections are the same as in EC3 for steel structures.

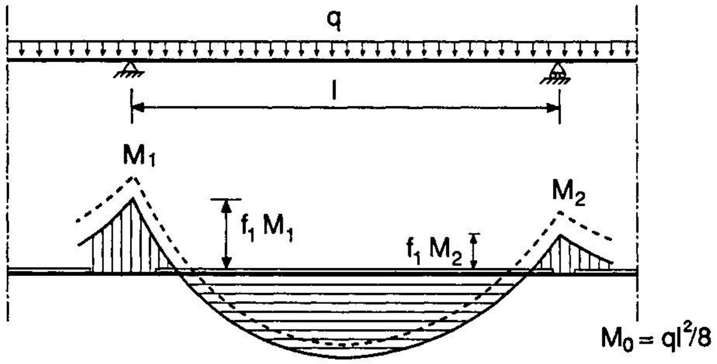

Figure 1 Simplified method for control of deflection

The influence of the cracking of concrete on deflections in continuous beams is allowed for by two alternative simplified methods.

a. Over a length of 15% of the span on each side of a support the flexural stiffness E_aI_2 of the cracked section (ignoring the concrete) is used and for the rest of the span the flexural stiffness E_aI_1 of the uncracked section.

b. Reduction by a factor f_1 of the negative moments, as calculated with a constant "uncracked" flexural stiffness E_aI_1 over the full length (fig. 1). The reduction factor may conservatively be taken as 0.6 or within some limitations for span and loads as $(I_1/I_2)^{-0.35} \geq 0.6$.
The mid-span deflection may then be calculated from the simple formula:

$$\delta = \delta_0 \; [\; 1 - C_1 \; (f_1M_1 + f_1M_2)/M_0 \;]$$

where: $C_1 = 0.6$ for uniform load
$C_1 = 0.5$ for a central point load
δ_0 and M_0 are the deflection and the mid-span moment of the equivalent simply supported beam.

In continuous beams, unpropped during construction, it is most likely that at the supports the bottom flange of the steel beam will yield under service loads. Yielding is not considered as a limit state but the effect on the deflection should be allowed for. A simplified method is given in which the effect is taken into account by reducing the moments M_1 and M_2 with an additional factor f_2. This factor is 0.5 when the yield stress is allready reached, due to the dead-weight of the concrete and 0.7 when yielding is caused by loads applied after the concrete has hardened. The values are based on a parameter study reported in background document [1].

Deflections increase due to the effects of slip at the interface between steel and concrete. These effects may be ignored when a composite beam is designed for full shear connection. In unpropped construction only the influence of slip may also be ignored when not less shear connectors are used than half the number for full shear connection, except when the connectors are placed in ribs of height exceeding 80mm running transverse to the beam.
For other cases a simplified method is given. The deflections may be determined from:

$$\delta/\delta_c = 1 + C_c\ (1-N/N_f)\ (\delta_a/\delta_c-1)$$

where: δ_a is the deflection of the steel beam acting alone
δ_c is the deflection of the composite beam without slip
N/N_f is the degree of shear connection
C_c is a coefficient, taken as 0.3 for unpropped construction and 0.5 for propped construction.

The two cases, considered for C_c, take into account that the forces in the shear connectors at serviceability are higher in propped construction.

2.3 <u>Cracking</u>

The extent to which crack widths need to be controlled in negative moment regions of continuous composite beams in buildings depends on their environment. Where cracks have no influence on durability, it is not required that their width be controlled. In such cases it is sufficient to provide a nominal reinforcement specified as 0.2% for unpropped construction and 0.4% for propped construction.
For regions where some control is needed, a simple rule for minimum reinforcement is given.

$$p = A_s/A_c = k_c k\ f_{ct}/\sigma_{st}$$

where: k_c is a coefficient to take into account the stress distribution ($k_c \approx 0.7$)
k is a coefficient to take into account decrease in tensile strength ($k \approx 0.8$)
σ_{st} is the maximum permitted stress in the reinforcement.

A typical value of p is 0.4 to 0.6% which is well in excess of the nominal reinforcement.

More comprehensive rules for limiting crack widths to 0.5 mm and 0.3 are given. It was not possible to refer to the rules in Eurocode 2, because crack widths in composite beams are influenced by the stiffness of the structural steel member.

2. SHEAR CONNECTION IN BEAMS

2.1 Full and partial shear connections

In Eurocode 4 the use of full and partial shear connection is allowed for. A full shear connection is formed when the shear connection is so strong that the ultimate load is determined by the maximum bending resistances of the critical cross-sections. For beams with all critical cross-sections in Class 1 and 2 the design longitudinal shear V_l to be resisted by shear connectors for full shear connection, follows from equilibrium with the forces used for the calculation of the plastic moments of resistance of the cross-section. This method may also be used if cross-sections at supports are initially Class 3 but are treated as Class 2 by the use of an effective web.

The first contribution to the development of a theory for the ultimate strength of beams with partial shear connection was presented by Slutter and Driscoll in 1965. They suggested that the resistance of the cross-section of the beam can be determined on the basis of a rigid-plastic stress distribution (rectangular stress blocks) for normal forces in the slab and the beam equal to the total resistance of the shear connectors in the relevant shear span.

This method, known as the "plastic" or "equilibrium" method is adopted in Eurocode 4 but with the provision that the connectors have sufficient deformation capacity. This method leads to a design curve as given by ABC in figure 2. A more conservative but simpler approach is the "interpolation" method where a linear interpolation between points A and C is used. This method also exploits the deformation capacity of the connectors because it is essentially a simplification of the plastic method. If the shear connectors are classified as "non-ductile" the longitudinal shear has to be determined from stress distributions at the critical cross-sections based on full continuity (no slip) at the interface between steel and concrete.

If the stress-strain diagrams for steel and concrete are known, the relation between M_{Sd} and N/N_f in principle can be calculated by the elasto-plastic method. The calculation of the elasto-plastic branch ($M_{Sd} > M_{el}$) of the curve is too elaborate for use in practice. Therefore in EC4 this part is approximated by a straight line (EC in figure 2). To establish point E an elastic analysis of the section is needed.

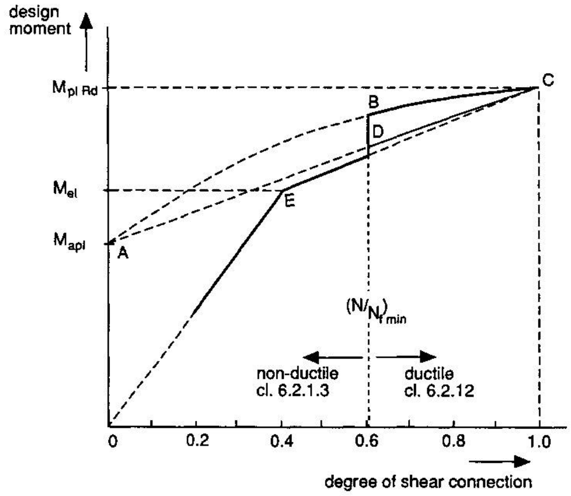

Figure 2 Design diagrams for partial shear connection

2.2 Deformation capacity of shear connectors

The basic requirement is that the shear connectors shall be able to maintain resistance to shear at slips not less than relied on in the design. It was decided to base the application rules on an available slip of 6 mm, and to accept as "ductile" those connectors that have a characteristic slip capacity exceeding 6 mm. This value is so chosen that the most commonly used headed stud connectors in solid slabs may be considered as ductile.
It has been shown by many tests that the required slip increases with the span L of the beam, and as the degree of shear connection N/N_f is reduced. Based on tests and numerical parameter studies combinations of span and the ratio N/N_f are defined such that the required slip did not exceed 6 mm. It was shown by the parameter calculations that the combinations are more stringent if the top flange of the steel beam is smaller than the bottom flange. Because this is not the most common case in buildings separate combinations were defined for equal flanges and unequal flanges.
In figure 3 the rule for equal flanges is shown.

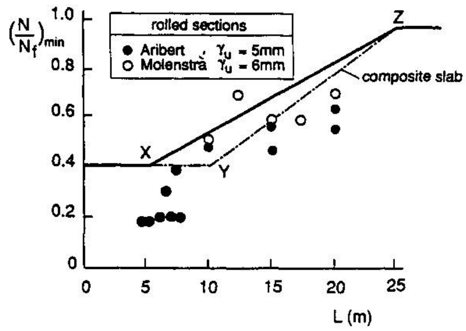

Figure 3 Minimum degree of shear connection for ductile connectors

Recent studies have shown that where certain types of profiled steel sheeting are used, available slips are much greater than 6 mm. In view of the economical importance the minimum (N/N_f) ratios for connectors to be treated as ductile were reduced for this category of applications as shown by the line YZ in figure 3. In this case the more conservative interpolation method (ABC in figure 2) should be used only.

It should be emphasized that no minimum degree of shear connection is specified in EC4, but the design method is more conservative for "non-ductile" connectors (see fig. 2), and simplified rules for checking deflections in service are valid only where $N/N_f \geq 0.4$.

2.3 Design resistance of shear connectors

Provision is made for headed studs; block, angle and friction-grip bolt connectors and for anchors and hoops using welded reinforcement.
New proprietary types of connector (e.g. cold-formed connectors using shot-fired pins and welded strips with holes) are not excluded but no application rules are given. It was considered that the provision now of application rules for such systems could inhibit development. Data on performance are available from manufacturers and these can be used to prove that the principles of chapter 6 are complied with.
The design rules for the resistance of welded stud shear connectors were based on statistical analysis according the procedure given in draft Annex Z of Eurocode 3 [2] [3].
The shear resistances are defined by formulae because of the wide range of parameters to be covered.

The design equations for headed studs in solid slabs are based on the simplified engineering model that a stud fails either in the steel alone or in the concrete alone. It is of course realised that especially in the area of interest in reality interaction occurs between the two assumed models of failure.
The two equations are:

steel: $P_{Rd} = 0.8\ f_u\ (\pi d^2/4)\ /\ \gamma_v$

concrete: $P_{Rd} = 0.29\ \alpha d^2 \sqrt{(f_{ck}\ E_{cm})}\ /\ \gamma_v$

Here, and elsewhere in chapter 6, coefficients as determined from the statistical evaluation were modified slightly, to enable the use of a single value of 1.25 for γ_M, denoted as γ_v.

The behaviour of a stud connector in a rib of profiled steel sheeting is much more complex than in a solid slab. It is influenced by the following parameters:
- the direction of the ribs relative to the beam
- the breadth b_0 and depth h_p of the ribs
- the diameter d and height h of the stud
- the number N_r of the studs in one rib and their spacing
- the eccentricity and the direction of the shear when the studs are placed off centre.

At the moment no reliable theoretical model is available covering a sufficient wide range of parameters. Therefore the empirical reduction factors, as proposed by Grant, Fisher and Slutter (1977), are still used as a basis for the rules. These reduction factors are applied to the design resistances of studs in solid slabs.
However, the evaluation of all the available tests revealed that the reduction factors as originally proposed do not give safe results over the whole range of possible applications. Therefore the coefficient 0.85 in the Grant formula was reduced to 0.7 and limitations are given for the rib height, the rib breadth and the number of connectors per rib.

$$k_t = 0.7\ (b_0/h_p)\ [(h/h_p)-1]/\sqrt{N_r} \leq 1.0$$

with: $h_p \leq 85$ mm
$b_0 \geq h_p$
$N_r \leq 2$

2.4 <u>Transverse reinforcement</u>

The design rules for transverse reinforcement in the flanges of T-beams have been aligned with those given in Eurocode 2 for reinforced concrete T-beams.
Account has been taken of profiled steel sheeting acting as transverse reinforcement. Sheeting may be assumed to be fully effective if the sheeting is running transverse to the beam and is continuous across the steel flange. Where the steel sheet is discontinuous but anchored by through-deck welding the sheeting may also be assumed to contribute to a given extent.
Sheeting discontinous and not anchored and sheeting running parallel to the beam is assumed to make no contribution to the requirement for transverse reinforcement.

3. COMPOSITE SLABS

As indicated in the title of Chapter 7 this section of the Code is only applicable for building structures. A further restriction is that the loads should be predominantly static. The reason is that at the moment no application rules are available to determine the effect of repetitive or dynamic loads on the composite action.
Propped and unpropped construction are covered. Additional reinforcement in the sagging moment regions, including any provided for fire resistance, may be taken into account for the flexural resistance.
No application rules are given for diaphragm action of composite slabs, although the use is allowed for. For diaphragm action of the steel sheeting, before the concrete has hardened, reference is made to Eurocode 3, Part 1.3.
To achieve composite behaviour, that is that the profiled steel sheets combine structurally with the concrete, horizontal shear must be transmitted at the interface between the sheet and the concrete.
Pure bond is not considered effective for this purpose. Accepted means to achieve composite behaviour are mechanical interlock and exclusively for re-entrant shapes also frictional interlock. These means may be combined with some forms of end-anchorage.

Characteristic for composite slabs are the two consecutive different structural states. First, the temporary state of construction where the steel sheeting resists the applied loads and, secondly the final state where composite action is effective.
Normally sheeting is first used as a construction platform. This means that it supports workmen, their tools and other material commonly found on construction sites. Design loads for the construction phase are 1.5 kN/m^2 on any 3 meters by 3 meters area and 0.75 kN/m^2 on the remaining area. Also the sheeting should be able to resist a local load of 1 kN on a square area of side 300 mm.
Next the sheeting is used as shuttering. This means that it supports the weight of the wet concrete, reinforcement and concreting gang. If the central deflection in this phase exceeds L/250 or 20 mm the effect of ponding should be allowed for in design. For the verification of the profiled steel sheeting reference is made to Eurocode 3, Part 1.3.

The verification of the sheeting in the construction phase may be based upon calculated properties or testing. Normally the decking manufacturer will provide values in the form of allowable live load tables. Due to the conservative nature of the design rules these values will almost ever be based on testing.
Verifications at the ultimate limit state and the serviceability limit state are required for the composite slabs after composite behaviour has commenced and any props have been removed. The following methods of analysis may be used:
- Linear elastic;
- Linear elastic with moment redistribution, where the bending moments at internal supports may be reduced by maximum 30%;
- Consider the slab as a series of simply-supported spans. A nominal reinforcement over the internal supports should then be provided;
- Plastic hinge analysis may be used for the ultimate limit state provided the span is less than 3.0 meters and the reinforcement over the supports has high ductility (Class H). The background for this rule is given in [1];

For the determination of the bending resistance of cross-sections rectangular stress-blocks for both steel and concrete are used. Tests have shown that cross-sections of embossed and indented sheets are not always fully effective. The reduction is dependent on the dimensions and the shape of the embossments or indentations. No exact design rules are available to determine the reduction and therefore as a safe approximation the width of the deformed parts are neglected unless more accurate information is available from tests.
If parts of the steel sheeting with large b/t ratios are in compression, they may be not fully effective due to local buckling. This is only relevant if for sagging bending the neutral axis is in the steel sheet and for hogging bending if the contribution of a continuous sheet is taken into account. The restraining effect of the concrete is taken into account by allowing arbitrarily to use an effective width twice the value for a Class 1 unrestrained web.
If in sagging bending the plastic neutral axis is above the steel sheet ("under-reinforced" section) the calculation of the bending resistance is rather simple. All commonly used sheets ($h_p \leq 60$ mm) in combination with a concrete slab of minimum thickness are "under-reinforced".
For deeper sheets, used for long spans, the plastic neutral axis may be in the steel sheet. The calculation of the bending resistance is than elaborate. Therefore a simplified method is provided in the Code. The background of the formulae is given in [4].

For the determination of the design resistance against longitudinal shear EC4 presently includes two alternative design methods, i.c. the empirical method ("m-k" method) and the partial connection method. Both methods are based on testing. No reliable theoretical method is as yet available.
The empirical method is developed by Porter and Ekberg in the United States in the 1960's. This is presently the most commonly used method, included in many national codes and forming the basis for almost all design information by decking manufacturers. In this method the design shear resistance is determined from a semi-empirical relation using two factors (m and k) obtained from at least six tests of simply supported slabs.
The second method, included in Annex E as an alternative method, attempts to incorporate composite slab design into the EC4 design method for composite beams. It is only applicable to slabs having a ductile load-slip behaviour. This method was first proposed by Stark in 1978 [5] and further developed by Bode. It is now used in Germany, where national rules do not allow use of composite slabs exhibiting brittle behaviour. In this method the shear load capacity τ_u is derived from tests on slabs with various spans, so chosen that the test information is representative for the whole range of degree of shear connection in practice. The value τ_u being determined and using essentially the same methods as for composite beams, a design diagram giving M_{Rd} as a function of the shear span L_x, can be calculated.
The method can be extended to cover also slabs with additional reinforcement and end-anchorages. This subject is covered in more detail in the contribution by Roik and Bode.

The rules for verification of the vertical shear resistance and punching shear resistance are consistent with the relevant rules in Eurocode 2.

For the crack width control of hogging moment regions in continuous beams reference is made to Eurocode 2. The nominal reinforcement, required if the slab is designed as a series of simply supported slabs, should have a cross-sectional area of not less than 0.2% of the area of the concrete on top of the steel sheet for unpropped construction and 0.4% for propped construction.

4. DESIGN ASSISTED BY TESTING

Design assisted by testing is treated in general terms in Chapter 8 of EC3, supplemented by the Provisional Guide in Annex Y. These are applicable also to the two types of products for which design is closely related to results of tests: shear connectors and composite slabs.
For both types of products, extensive supplementary requirements are given in Eurocode 4 (Chapter 10 and Annexes E and F). These include details of test specimens and procedures, recording and interpretation of results, and calculation of values for use in design.
For various types of shear connectors EC4 gives rules for the design resistance. For others not covered in the Code a standard push test is given. From push tests the failure load, the mode of failure and the load-deformation performance are obtained. Recent research has shown that the test specimen defined in the 1985 Draft can give over-conservative results, due mainly to splitting of the slabs. Therefore the specimen now defined in EC4 has larger concrete blocks and more transverse reinforcement. The evaluation now also includes the design slip capacity δ_u relevant for the partial shear connection method.

REFERENCES

1. Stark, J.W.B., Van Hove, B.W.E.M.,
The midspan deflection of composite steel-and-concrete beams under static loading at serviceability limit state
TNO-report BI-90-033, march 1990
2. Stark, J.W.B., Van Hove, B.W.E.M.,
Statistical analysis of push-out tests on stud connectors in composite steel and concrete structures
TNO-report BI-91-163, september 1991
3. Stark, J.W.B.,
General methods of Design of Composite Construction,
IABSE report vol. 61, november 1990
4. Stark, J.W.B., Brekelmans, J.W.P.M.,
Plastic Design of Continuous Composite Slabs
J. Construct. Steel Research 15 (1990) 23-43
5. Stark, J.W.B.,
Design of Composite Floors with Profiled Steel Sheet,
4th Specially Conference on Cold Formed Steel Structures;
Department of Civil Engineering, Missouri-Rolla, 1978

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