Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 65 (1992)

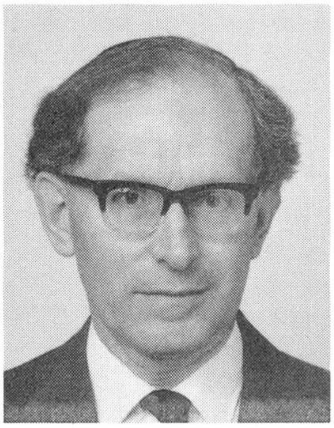
Rubrik: Eurocode 4: Composite steel and concrete structures

EC 4: Composite Structures of Steel and Concrete

EC 4: Structures mixtes en acier et béton

EC 4: Verbundtragwerke aus Stahl und Beton

R.P. JOHNSON
Prof. of Civil Eng.
University of Warwick
Coventry, UK



SUMMARY

The history and current plans for the three Parts of Eurocode 4 are explained, and also the relationships between them and Eurocodes 2 and 3. The scope and main features of the original «Eurocode 4», now Part 1.1, are outlined, including the treatment of materials, products, and their testing; the need for simplification and where it has been achieved; the reasons for differences from Eurocodes 2 and 3; the choice of partial safety factors; and statistical calibration based on test data.

RESUME

L'historique et l'organisation des trois parties de l'Eurocode 4 sont expliqués ainsi que leurs liens avec les Eurocodes 2 et 3. On présente l'étendue des activités et les caractéristiques principales du premier «Eurocode 4», maintenant la partie 1.1, ainsi que le traitement des matériaux, les produits et les essais à effectuer, la simplification nécessaire dans certains cas; les différences entre l'Eurocode 4 et les Eurocodes 2 et 3; le choix des coefficients partiels de résistance; et la calibration statistique basée sur des essais.

ZUSAMMENFASSUNG

Der Werdegang und die laufende Planung der drei Teile von EC 4 werden erklärt, ebenso ihre Beziehung untereinander und zu EC 2 und 3. Der Beitrag skizziert Umfang und Hauptmerkmale des ursprünglichen «EC 4» (jetzt Teil 1.1), einschliesslich der Abhandlung von Werkstoffen, Produkten und Prüfverfahren. Dabei zeigt er erforderliche und teilweise erreichte Vereinfachungen auf, bespricht die Wahl des Widerstandteilbeiwerts und erörtert die statistische Kalibrierung aufgrund von Versuchsergebnissen.

1. BACKGROUND AND PLANNING

Eurocode 4, "Design of composite steel and concrete structures", will eventually be a Euronorm numbered EN 1994. It is expected to consist of three documents, each drafted by its own project team:

- Part 1.1, General rules and rules for buildings
- Part 1.2, Structural fire design
- Part 2, Bridges

The histories and future plans for these Parts are as follows.

Part 1.1. This was first drafted in 1983/4, based on a Model Code [1] prepared between 1971 and 1980 by the IABSE/CEB/ECCS/FIP Joint Committee for Composite Structures, chaired by Dr. D. Sfintesco. It was published as "Eurocode 4" [2] and studied for 18 months (1985-87) in the twelve Member States of the European Economic Community. They did trial designs and submitted many comments and proposals for revision. A commentary on this draft was prepared [3] and papers were presented at an IABSE-ECCS Symposium [4, 5]. Since then, much research has been done on problems evident from this draft, and papers have been presented at several conferences, especially the IABSE Symposium at Brussels in 1990 [6].

Eurocodes 2 and 3 have been substantially revised and extended since 1984, and the current draft Eurocode on Actions has been written since then. Eurocode 4 has to be consistent with these three codes, so the project team that has been revising it since 1987 has made many changes and additions, to take account of the national comments and of the progress of other codes, of research, and of practice.

Eurocode 4: Part 1.1 was issued as a prENV document, under the CEN system, in 1992. If it is accepted by CEN Committee TC 250/SC4, it will be translated into French and German and published as ENV 1994: Part 1.1, for trial use over a period of three years. It will be the first Part of any Eurocode to follow this route, as Parts 1.1 of Eurocodes 2 and 3 were approved by the European Commission, before the transfer of responsibility to CEN.

Design manuals (commentaries) on Part 1.1 are being published [7, 8].

Part 1.2. Structural fire design. Until 1992, this code was numbered Eurocode 4: Part 10. Drafting began in 1987, in parallel with work on Parts 10 of Eurocodes 2, 3, 5, and 6. It was issued for national comment in 1990, [9] and is now being revised. It should be issued as a prENV in 1993, for approval by CEN before publication as ENV 1994: Part 1.2.

Part 2. Bridges. It was the intention of the Sfintesco Committee, and of the Commission of the European Communities, that the original "Eurocode 4" should be applicable to bridges, as well as to structures for buildings. The scope of the 1985 draft [2] states that it is " ... concerned with buildings and civil engineering structures. The basic principles apply to all types of composite structures or elements ..... However, they do not cover all aspects relevant to special structures, such as ..... certain types of bridges ....."

The 1985 draft could not cover all aspects of the design of composite highway or railway bridges. It became evident that it would not be feasible to design simple bridges to Part 1.1 and more complex ones using supplementary rules to be given in Part 2. But it is still stated in the latest Part 1.1 that it " .... gives a general basis for the design of composite structures and members for buildings and civil engineering works" (i.e., including bridges).

This led to the addition of the words "for buildings" to the titles of some of the chapters and sections of Part 1.1, to make clear that they are intended to be replaced, rather than supplemented, by material in Part 2.

It is also stated in Parts 1.1 of Eurocodes 2 and 3 that they give "a general basis for the design of ..... civil engineering works ...", but few of the titles of their chapters or sections carry the exclusion "... for buildings".

It is not yet known in what way the Parts 2 of Eurocodes 2 and 3 will be related to their Parts 1.1. At one extreme, they could be comprehensive stand-alone documents. At the other, they could be limited to supplementary rules for specific types of structure (e.g., box girders).

This uncertainty, and lack of funding, have caused the drafting of Eurocode 4: Part 2 to be delayed until substantial progress has been made on Parts 2 of Eurocodes 2 and 3. It has been proposed to CEN that work should begin in January 1993, with a target date, for approval by CEN for publication as an ENV 1994: Part 2, about a year after that stage has been reached by Parts 2 of Eurocodes 2 and 3.

There is little further reference to Eurocode 4: Part 2 in this paper, or in the other five papers on Eurocode 4. Four of them are on Part 1.1 (i.e., the original "Eurocode 4"), and one is on Part 1.2, Fire. In the present paper, all cross-references to chapters or clauses are to the prENV draft of Eurocode 4: Part 1.1, unless noted otherwise.

2. SCOPE OF EUROCODE 4: PART 1.1

Eurocode 4 applies to composite structures and members "... made of structural steel and reinforced or prestressed concrete connected together to resist loads", but prestressed structures are not included in Part 1.1. It is "... only concerned with requirements for resistance, serviceability, and durability ...", but excluding seismic design and resistance to fire, and to actions liable to result in fatigue.

Execution (known as "construction" in the U.K.) is covered only to the extent that it is necessary to indicate the quality of construction materials and workmanship on site needed to comply with the assumptions of the design rules. This subject is elaborated in another paper.

Detailed application rules, mainly applicable to ordinary buildings, are given for composite slabs, beams, columns, and frames. These may be constructed using the full range of materials covered by Parts 1.1 of Eurocodes 2 and 3, except structural steel of Grade Fe E 460 (yield strength 460 N/mm^2), and concretes with cylinder strength less than 20 N/mm^2. Rules are given for the use of lightweight concrete, pending the completion of Eurocode 2: Part 1C.

The scope is wider than that of any equivalent national code known to the Project Team, but there are many aspects of current practice for which no application rules are given. These include:

- certain new or developing types of shear connector,
- use of large holes in the webs of beams, and stub girder construction,
- base plates beneath composite columns,
- framed structures with "semi-rigid" connections; i.e. connections that are neither "rigid" nor "nominally pinned", using terms defined in Eurocode 3,
- members where the structural steel component has cross-sections with no axis of symmetry parallel to the plane of its web.

Application rules for these situations are not yet well established, and their provision at too early a stage of development can stifle innovation. Other subjects (e.g., sway frames) are omitted because they are rarely used in composite structures, and would require complex design rules.

Beams and columns consisting of steel I or H sections with concrete-encased webs are included, as are concrete-filled and fully-encased steel sections used as columns. No application rules are given for steel beams that are either fully encased or have encased flanges. The former are widely used in bridges; and also in seismic areas, such as Japan, where they are designed by superposition of the resistances of the structural steel and reinforced concrete components.

Fully-encased beams without shear connectors are not included in Part 1.1 because:

- it is not known to what extent other rules (e.g., for moment-shear interaction and redistribution of moments) would be applicable;
- no satisfactory model has been found for resistance to longitudinal shear.

The applicability of codes may be limited by the unstated assumptions commonly made during drafting; for example, that members in frames are orthogonal and concrete slabs are horizontal. In Eurocode 4 it is assumed (e.g. in the rules for redistribution of moments) that the steel member of a composite beam is below the concrete slab; but this is not stated. The design of unconventional structures requires wider knowledge than can be found in a code.

3. RELATIONSHIP TO PARTS 1.1 OF EUROCODES 2 AND 3

Eurocode 4 is unusual in not being the principal source of information for design in any structural material. Why does it not consist solely of one chapter, "Shear connection" and the instruction "Follow Eurocodes 2 and 3"?

Design would then be found to be more complex than it is either for structural steel or for reinforced concrete. For example, cracking, creep, and shrinkage of concrete and slip at the steel-concrete interface create uncertainties about stress levels in slender steelwork that may influence buckling loads. The need for economy and simplicity has led to design methods that sometimes differ in detail from those for structural steel or for concrete. But the differences are not so extensive that Eurocode 4 could be self-contained. That could treble its length.

The policy has been to cross-refer to Eurocodes 2 and 3 wherever practicable. For example, the three pages on vertical shear in beams give only the main requirements and the few modifications needed to the 14 pages that the subject requires in Eurocode 3.

There are two exceptions to this practice. Chapters 1 (Introduction) and Chapter 2 (Basis of design) are as far as possible identical with those in Eurocodes 2 and 3, to ensure harmonisation. Also, information from Eurocodes 2 and 3 that is both concise and frequently needed is repeated in Eurocode 4. This applies mainly to the properties of materials, given in Chapter 3.

There are many differences of sequence and presentation between Parts 1.1 of Eurocodes 2 and 3, and a few inconsistencies of technical content. Eurocode 4, as applied to buildings, is based on the concept of the initial erection of a steel frame, perhaps including precast composite members. So Eurocode 4 relates closely to steel construction, and in presentation and content, it is more like Eurocode 3 than Eurocode 2; but in one respect its sequence follows the latter: "ultimate limit states" precede "serviceability limit states", as this is the usual sequence in design. Many aspects of a composite structure are covered in both Eurocodes 2 and 3 (e.g., lateral instability of a multi-storey frame). The policy for Eurocode 4 has been to use as a basis the more appropriate of the two methods, and modify it as little as possible.

4. MATERIALS, PRODUCTS, AND TESTING

The Structural Eurocodes are intended for use with a full set of international standards for materials, products, and their testing. These will either be Euronorms from CEN, or ISO Standards. For steel components and for concrete, Eurocode 4 gives basic information in Chapter 3, but otherwise refers to Euronorms or to Eurocodes 2 and 3, both of which include Provisional Guides, for use until the relevant CEN or ISO standards are available.

For shear connectors, there are four distinct situations.

(a) For connectors that consist of steel blocks, bars, sections, or reinforcement, attached by welding, the materials and welding should be in accordance with Eurocodes 2 or 3, as appropriate.

(b) For welded stud connectors, international standards are needed both for tests on the material and for the automatic welding process. Provisional guidance is given in clauses 3.5.2 and 9.4.3 of Eurocode 4.

(c) For friction-grip bolts, the relevant clauses of Eurocode 3 are supplemented, in Section 6.5.

(d) New types of connector continue to be developed. Those based on the use of shot-fired steel pins, for example, are not covered by any of items (a) to (c), above. New types of connector should be the subject of European technical approvals, and should comply with the Principles of Eurocode 4 (e.g. in Section 3.5 and clause 9.4.3).

Design assisted by testing is treated in general terms in Chapter 8 of Eurocode 3, supplemented by the Provisional Guide in Annex Y. These are applicable also to the two types of product for which design to Eurocode 4 is closely related to results of tests: shear connectors, and profiled steel sheeting used in composite floor slabs.

For both types of product, extensive supplementary requirements are given in Eurocode 4

(Chapter 10 and Annexes E and F). These include details of test specimens and procedures, interpretation and recording of results, and calculation of values for use in design.

5. SIGNIFICANT FEATURES OF COMPOSITE CONSTRUCTION

Reference was made in Section 3 (above) of the need for a Eurocode 4 to supplement Eurocodes 2 and 3. The main characteristics of composite construction that influenced the content of Eurocode 4 are now summarised. Some have equivalents within the scope of Eurocode 2 and/or Eurocode 3, but they are more significant in a composite structure.

(a) The use of unpropped construction is general, both for beams and for composite slabs. It has a potential influence on all aspects of subsequent response to loading, and is referred to in clause 2.2.1.2(2), "design situations", and in many other places.

(b) A continuous beam of uniform section may have a resistance to hogging bending as low as one-third of its resistance to sagging bending. The use of slab reinforcement to strengthen the hogging region is limited to material of high ductility, because of the large extensions caused by the flexibility of common types of steel beam-to-column connections, and the economic need for the use of redistribution of moments in design. Reinforcement worsens the Class of the steel section, with adverse consequences for design.

(c) Shear connectors apply concentrated forces to the concrete slab, that must be resisted without local failure. The ductility required of a shear connection can be large, and the effects of its flexibility are complex and not always negligible.

(d) It is necessary to ensure that a composite slab fails in a ductile manner. This depends on the type of profiled sheeting used, and influenced the specification of the tests needed to determine resistance to longitudinal shear.

(e) The Class (relevant to local buckling) of a rolled section forming a steel beam is the same in hogging and sagging bending. A typical composite beam may be Class 3 in hogging bending, but is usually Class 1 at midspan.

(f) For steel beams, design methods are needed to ensure that non-distortional lateral buckling does not occur. It cannot occur in composite beams, which have to be checked for distortional lateral buckling, using different methods.

(g) Uniform change of temperature and shrinkage of concrete can both, in theory, alter the curvature, as well as the length, of a composite beam; and the relevant calculations are complex.

(h) Significant economy can be achieved in the design of some concrete-filled steel tubes, as columns, by taking account of triaxial stress effects.

(i) There are many potential combinations of situations. For example, in any framed structure that has composite members, the possibilities include:

- composite beams, propped or unpropped, or steel or reinforced or prestressed concrete,
- concrete, normal density or lightweight,
- steel beam section in Class 1, 2, 3, or 4,
- shear connection partial or full,
- shear connectors ductile or not,
- web of steel beam encased or not,
- profiled sheeting spans longitudinally, transversely, or is absent,
- beam-to-column connections nominally pinned, semi-rigid, or rigid,
- frame braced or unbraced,
- columns steel, concrete, or composite.

There are over 15000 combinations from this list, and most of them are practicable.

The preceding list illustrates the need to specify simple design methods wherever possible. This has been done in Eurocode 4: Part 1.1:

- in the treatment of creep of concrete by the use of modular ratios;
- by enabling the local effects of temperature and shrinkage of concrete in buildings to be neglected, in almost all situations;

- by simplification of the rectangular stress block for concrete in compression;
- by the use of a polygonal interaction diagram for the resistances of a cross-section of a column;
- by allowing global analyses that neglect shrinkage and cracking of concrete, shear lag, and in some structures, the effects of unpropped construction and creep of concrete;
- by defining situations where no check on lateral buckling need be made and providing simplified methods, where checks are needed.

One of the targets originally proposed for Eurocodes was that there should be a unique design procedure for each problem. Eurocodes were not to be collections of different methods customary in various countries. This objective has been achieved. It was also hoped that designers of the same structure, in different offices, would have to use the same methods. This has not been achieved; and should not be, because simplicity, so often needed for composite structures, usually implies some loss of economy. The conflict between these aims is better resolved by the designers of particular structures, than in a code with wide applicability.

Another target was that the Eurocodes should be suitable for contractual use, under the European Public Works Directive, No. 89440. This led to the presentation of some clauses in the form "Unless differently specified, ...".

6. PARTIAL SAFETY FACTORS, AND CALIBRATION

All γ factors in Eurocode 4: Part 1.1 are "boxed". This is to enable the authorities in member countries of CEN to assign other values, if they so decide. All the values for actions given in Eurocode 4 are as in Eurocodes 2 and 3. They will be supplemented by, and aligned with, values to be given in Eurocode 1 in due course.

Since Eurocode 4 copied the values γ_M given in Eurocodes 2 and 3, most of its calibrations were, in effect, checks on the coefficients used in the design formulae. There was also much reliance on existing practice, because the extent of reliable test data is far less than for structural members of steel or reinforced concrete.

Statistical calibrations have been done for the plastic moments of resistance of composite beams in sagging bending, for both full and partial shear connection, and in hogging bending [10]. The results appear to confirm the values γ_M given in Part 1.1. They also show the sensitivities of probabilities of failure to reductions in γ_a (for structural steel) below the current value, 1.1. Reference is made in another paper to the calibration work for shear connectors.

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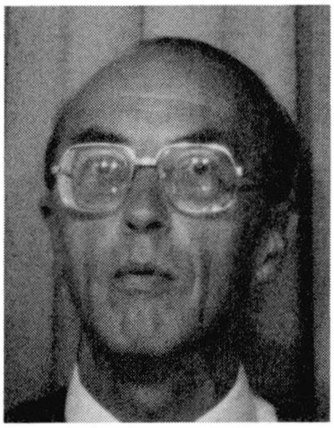
EC 4: Relationship to Eurocodes 1, 2 and 3

EC 4: Lien avec les Eurocodes 1, 2 et 3

EC 4: Beziehung zu den Eurocode 1, 2 und 3

Henri MATHIEU
Insp. Gén. des Ponts
et Chaussées e.r.
Bagneux, France



Henri Mathieu, born 1925, received his diploma in Ecole Polytechnique and Ecole Nationale des Ponts et Chaussées in Paris. Firstly bridge designer, then responsible for bridge execution, he became head of central bridge division in France, and finally general inspector for bridge related activities. His interests lie with problems associated with structural reliability.

SUMMARY

Eurocode 4 Part 1.1 has many relations with Eurocodes 2 and 3. These relations are numerous and very complex, both technically and editorially. For chapter 1 the main problems were editorial. In chapter 2 the specific aspects of composite structures resulted in some rules additional to the model clauses, and others adjusting the reliability format. Chapter 8 was established in accordance with a draft Part 1 B of Eurocode 2. Annexe A dealing with reference documents is very provisional.

RESUME

L'Eurocode 4 (Partie 1.1) est principalement lié aux Eurocodes 2 et 3. Ces relations sont nombreuses et très complexes, techniquement aussi bien que rédactionnellement. Pour le chapitre 1 les principaux problèmes étaient rédactionnels. Dans le chapitre 2 les aspects spécifiques des structures mixtes ont donné lieu à des règles additionnelles aux clauses modèles, et à d'autres règles ajustant le format de fiabilité. Le chapitre 8 fut établi pour être en accord avec un projet de Partie 1 B de l'Eurocode 2. L'annexe A traitant des documents de référence est très provisoire.

ZUSAMMENFASSUNG

Eurocode 4, Teil 1.1 ist hauptsächlich mit EC 2 und EC 3 verknüpft, wobei die Bezüge technisch als auch editorisch zahlreich und kompliziert sind. Im Kapitel 1 waren sie vor allem editorischer Natur. Im Kapitel 2 verlangten die Besonderheiten von Verbundtragwerken einige zusätzliche Regeln im Vergleich zu den Musterparagraphen und Anpassungen der Zuverlässigkeitskriterien. Kapitel 8 wurde in Übereinstimmung mit dem Entwurf von EC 2, Teil 1 B aufgestellt. Der Anhang A über Referenzdokumente ist noch rudimentär.

1. RELATIONS WITH EUROCODES 1, 2, 3

The obvious need of consistency throughout the set of Eurocodes has been particularly important and critical for EC 4. Its critical character resulted from the fact that EC 2 and 3 (Parts 1.1) were not totally consistent together on a series of details and remained under revision up to the end of 1991. These difficulties were a supplementary reason for publishing the Eurocodes first as ENV.

Since the beginning of the work on EC 4, EC 1 has been deeply modified in its scope (see the reports on EC 1). This had however no consequence on EC 4 Part 1.1 because the reliability format has not been modified. At present the only references made by EC 4 to EC 1 are for the representative values of actions and have a general character, and in clause 2.2.5 for an application rule on simple load arrangements which maybe will be finally transferred to EC 1.

The relations with EC 2 and 3 are much more complex. Only the main or most typical examples are given below.

There were first some minor numerical discrepancies which, although identified rather soon, could not been avoided at the ENV stage :
- the modulus of elasticity of steels E_s was 210 GPa in EC 3 (the most precise value) and 200 GPa in EC 2 (a simple value chosen for a property that is not identical for all steels) ; 210 GPa was chosen for EC 4
- the thermal expansion coefficients were also different for the various steels and for the concrete ; simple values were chosen for these minor coefficients in EC 4.

More difficult was the fact that EC 2 and 3 did not refer to the same degree of plastification in bending nor to identical types (and terminology) of structural analyses (see the report on chapter 4).

It has still to be mentioned that the Chapter 8 of EC 4 on floors with precast concrete slabs had to refer not to Part 1.1 of EC 2, but to Part 1B which was still under discussion at the end of 1991.

2. CHAPTER 1 OF EUROCODE 4

As for most of the ECn Part 1.1, this introductory chapter is based on a model text established by the past Coordination Group. Only what is specific for EC 4 is mentioned below.

The clause 1.1.2 is very specific and mainly deals with two problems :
- what are the status of the various Annexes ? This question will become very important only at the EN stage. It was however already carefully considered, first to clarify it as a guidance for the experimental uses of EC 4 as ENV, and also to provide a serious

basis of discussion for the transformation of EC 4 into an EN. This revision will obviously have to take also into account the consequences of further events
- what is not covered by the present version of EC 4 ? This is important for a complete understanding of the content and for contractual uses of this EC. The contractual importance will increase at the EN stage (application of the European Directive 89 440 on contracts for public construction works).

The clause 1.4.2 which supplements the common clause 1.4.1 by the special terms used in EC 4 Part 1.1 is rather developed for two reasons :
- there should be adopted, commonly after selection, a series of terms coming separately from EC 2 and 3. As mentioned in an ENV Note, a better consistency across EC 2 and 3 has still to be achieved on the denomination of the various types of analyses
- a composite structure is more complex than a concrete or steel structure and even than both together, as well because it needs supplementary elements and concepts (relating especially to shear connection and connection of members), as because its construction process (propped or not) usually has various consequences on the design.

We strongly recommend to use carefully the Eurocode terminology in applications and discussions, in order to avoid misunderstandings. The clause 1.6 finally, dealing with symbols, had to establish a consistent set of symbols starting from EC 2 and 3. This harmonization was not a very difficult task, but the result shall be considered as intended only for the applications of EC 4 Part 1.1 because the limited contents of the alphabets do not make it possible to envisage a complete and unique set of symbols for all ECs. For example the subscript p is used in EC 4 Part 1.1 for profiled steel sheeting while in EC 2 it is used for prestressing steel.

3. CHAPTER 2 OF EUROCODE 4

For this chapter also, which deals with basis of design and is based on a model text, only what is specific for EC 4 is mentioned below.

Clause 2.2.1.1 mentions limit states relating to the shear connection, and clause 2.2.1.2 requires the identification and consideration of specific transient situations during the construction process.

Clauses 2.2.2.1 and 2.2.2.2 require to take into account the action of the shrinkage of the concrete and classify its effects (with the effects of temperature differences, if relevant), as primary and secondary, having different consequences in the verifications.

At this occasion it can be mentioned that the shrinkage has generally small effects on the design of composite structures for buildings. On the other hand its magnitude depends on many parameters, some of which are not precisely known at the time of the design and (this is operationally worse) are different from one member to another. Finally its effects are blurred by the effects of temperature differences which are still more imprecisely known. For this reason, and as confirmed by a long and wide practice in several countries, simplified rules including modular ratios have been included in Chapter 3 (clauses 3.1.3 and 3.1.4.2) for the shrinkage and also, for the same reasons, for the consideration of the creep, to be used freely in common cases. It is hoped that the text is flexible enough to be considered as an acceptable compromise between very various national opinions in this respect.

For the static equilibrium EC 3 has been recently modified : the associated $GAMMA_F$ factors have been proportionally increased in order to make it possible to include a resistance as a complement in the limit state equation. The Project Team has considered this modification as useful and has introduced it in EC 4 (clauses 2.3.2.3 and 2.3.3.1). It shall however be recognized that no set of constant GAMMA, factors can be fully appropriate for the treatment of all static equilibria which can be very various.

The most difficult problems were met for the format of $GAMMA_M$ factors, their conditions of use and their numerical values. This first results from the fact that, for sound technical reasons, there are substantial discrepancies between the corresponding rules given in EC 2 and 3.

In the most general case a material factor $GAMMA_M$ given in ECs may be subdivided into several partial factors, as

$$GAMMA_M = GAMMA_{Rd}\ ETA\ GAMMA_m \quad \text{where}$$

- $GAMMA_{Rd}$ relates to model uncertainty of a resistance R and in practice, in EC 2, covers also geometrical uncertainties

- ETA is a conversion factor (essentially for concrete) covering the difference between the strength measured on standardized specimens and the strength in the structure

- $GAMMA_m$ relates to the scattering of the strength of standardized specimens.

It can be seen that normally a design resistance should be written

$$R_d = (1/GAMMA_{Rd})\ R\ (f_k/ETA\ GAMMA_m)$$

For a steel element ETA is generally equal to 1, $GAMMA_{Rd}$ and $GAMMA_m$ should not be very different from 1 and R is proportional to f_k, which makes strong simplifications fully acceptable.

For a composite element three strengths f_{ck}, f_{sk} and f_y generally intervene in R which generally is not even a linear function of them because of the shift of the neutral axis when the forces and moments applied to a cross section vary. Consequently strong simplifications such as practiced in EC 3 are not possible in EC 4.

Further difficulties resulted from the fact that the values of $GAMMA_c$ (for concrete), $GAMMA_s$ (for reinforcing steels) and $GAMMA_a$ (for structural steel) given in EC 2 and 3 are not directly comparable together for many reasons, e.g. :

- $GAMMA_c$ given in EC 2 refers to rupture (at 28 days), refers to a partial plastification and includes a subtantial part taking into account an imprecision on the location of reinforcing bars

- $GAMMA_a$ given in EC 3 refers to the yield, refers to a total plastification and takes on various values depending on the risk of local buckling (and if relevant on the presence of bolt or rivet holes).

The values given in these ECs are also widely pragmatic.

Besides the various strengths intervene very differently in various resistances (e.g. in columns and in hogging and sagging resistances in beams). For this reason "true" values of GAMMA factors cannot exist in a code : only values acceptable within limited fields of application can be provided.

It shall be clear that the choice of the numerical values was not, at the time EC 4 Part 1 was in preparation, the main aim of the document, because at the ENV stage all GAMMA values in ECs had to be boxed which means that they are only indicative and that the decisions should be taken at the national level.

At the same time the drafting panel was not ignoring that some $GAMMA_M$ values given in EC 2 and 3 remained subjected to contests which probably will lead to further discussions. Considering this situation, the EC 4 panel did not want to make it still more confused by giving in EC 4 new values, and kept in EC 4 the $GAMMA_M$ values provided in EC 2 and 3, having only checked by referring to the pratice, that they could be considered as sufficiently safe. This judgement was later confirmed by some theoretical researches which demonstrated, in some particular cases, that in spite of the differences between steel and composite structures, the level of safety resulting from EC 4 was numerically consistent with that of EC 3.

It can still be mentioned that the $GAMMA_M$ factors to be applied to material properties other than strengths have been specified more explicitly in EC 4 than in other Eurocodes.
Finally a partial factor very important for composite structures is the $GAMMA_M$ factor to be used for shear connection.

In Eurocode 3, the partial safety factor $GAMMA_M$ for resistance of steel connections is generally given as 1.25, though there are a few exceptions. The various expressions for bolts, welds, etc., that emerged from statistical calibration were scaled up or down to enable a single value of $GAMMA_M$ to be used, for the convenience of designers.

In Eurocode 4 Part 1.1, the design resistance of a welded stud shear connector is similarly found by dividing the characteristic resistance by a factor $GAMMA_v$, also taken as 1.25 :

$$P_{Rd} = P_k/GAMMA_v$$

This is done even though P_k is in some cases proportional to the ultimate tensile strength of the material of the stud, and in others to the cylinder strength of the concrete. Thus, the factors $GAMMA_a$ and $GAMMA_c$ for concrete and steel are not used. This procedure is an exception to that described above. It reflects the complex interaction between deformation of steel and concrete that determines the resistance of a stud.

Where the influences of steel and concrete are more distinct (e.g. for block-and-hoop connectors), the usual partial safety factors $GAMMA_a$, $GAMMA_c$, and $GAMMA_s$ are used.

4. CHAPTER 8 OF EUROCODE 4

This chapter, which deals with floors with precast concrete slabs for buildings, had to be drafted at a time where Part 1B of EC 2 was far from its finalization and before the revision of ENV 206 was undertaken. It should therefore be revised before the publication of EC 4 as EN. It should be also expected that in any case its content might have to be supplemented by some specifications particular to any particular project.

The content of this chapter is solely intended for slabs to be used as or in top members of composite beams for buildings, while Part 1B of EC 2 has a more general scope. It is therefore mainly devoted to the consideration of joints between such slabs and to the consequences of the shear connection.

The numerical values of the $GAMMA_M$ factors applicable for prefabricated concrete slabs and their conditions of applicability were at this time under discussion within TC 250 SC 2. It was not considered to be appropriate to deal separately with this problem in EC 4. These values will in any case be "boxed" in EC 4 as in EC 2.

5. ANNEX A TO EUROCODE 4

This Annex, at the present stage indicative and very provisional, was intended to define the relations of EC 4 Part 1.1 with other documents which include standards but possibly also other documents (e.g. European approvals for shear connectors).

As for other Eurocodes a general difficulty resulted from the fact that most documents likely to be referred to are not available at the present time, some of them being in preparation but not yet finalized so that their full compatibility with EC 4 cannot yet be certified. The situation in this respect will widely evolve during the ENV period, and it has to be coped with by the relevant authorities through the National Application Documents, themselves having a transitory and evolving character.

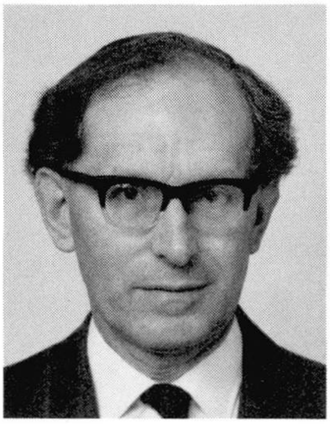
EC 4: Composite Beams, Connections, and Frames at Ultimate Limit States

EC 4: Etats-limites des poutres, assemblages et cadres mixtes

EC 4: Grenzzustände der Tragfähigkeit von Verbundträgern, Verbindungen und Rahmen

R.P. JOHNSON
Prof. of Civil Eng.
University of Warwick
Coventry, UK

David ANDERSON
Senior Lecturer
University of Warwick
Coventry, UK



SUMMARY

An outline is given of the design methods of Eurocode 4: Part 1.1 for simple and continuous beams, and for composite frames and connections for buildings. Subjects include the classification and resistances of cross-sections of beams, global analysis of beams and braced frames, and lateral-torsional buckling of beams. It is explained why no application rules are given for unbraced or sway frames or for the use of semi-rigid connections.

RESUME

On présente les méthodes de dimensionnement de l'Eurocode 4: partie 1.1 pour les poutres sur appuis simples et continus, et pour les cadres et les assemblages mixtes dans les bâtiments. Les sujets suivants sont abordés: la classification et la résistance des sections de poutre, l'analyse globale des poutres et des cadres contreventés et le déversement latéral des poutres. On explique pourquoi les règles se rapportant aux cadres non contreventés et aux assemblages semi-rigides ont été omises.

ZUSAMMENFASSUNG

Der Beitrag skizziert die Bemessungsmethoden im EC 4, Teil 1.1 für einfache und durchlaufende Träger, Verbundrahmen und Verbindungen in Hochbauten. Behandelt werden die Klassifizierung und Ermittlung der Widerstandsgrössen von Profilen, die Gesamtnachweise an Trägern und ausgefachten Rahmen sowie das Biegedrillknicken. Es wird begründet, warum keine Anwendungsregeln für unausgefachte oder verschiebliche Rahmen und für nachgiebige Verbindungen gegeben werden.

1. GENERAL

The subject of this paper is the content of Chapter 4, "Ultimate limit states" (except for columns) and Annex B, "Lateral-torsional buckling" of Eurocode 4: Part 1.1. It does not include shear connection or the use of composite or precast concrete floor slabs, as these are covered in other papers.

The Sections of Chapter 4 are arranged in order of increasing complexity. This enables simply-supported beams to be designed using Sections 4.1 to 4.4 only. For beams that are continuous over simple supports and do not take part in frame action, Sections 4.5 to 4.7 are also relevant. Section 4.8 is self-contained for no-sway columns subjected to known end loads and bending moments. The remaining Sections, 4.9 and 4.10, are on frames and connections respectively.

In Eurocode 3, three factors γ_M are defined for structural steel: γ_{M0} when buckling is not relevant; γ_{M1}, when it is; and γ_{M2}, for the resistance of a net section at bolt holes.

For net steel sections, Eurocode 4 follows Eurocode 3; and for γ_{M0} and γ_{M1} it preserves the distinction made in Eurocode 3, even though the boxed values for both factors are the same, 1.1. This is to enable any country that so wishes to assign different values to γ_{M0} and γ_{M1}.

The *symbols* for γ_{M0} and γ_{M1} used in Eurocode 4 are different: γ_a and γ_{Rd}, respectively. This is done because the factors are used in a different way, as explained in other papers on Eurocode 4, and in clauses 2.2.3.2 and 4.1.1(5) of Part 1.1.

2. CROSS-SECTIONS OF COMPOSITE BEAMS

Specific reference is made only to beams where the concrete slab lies above a steel rolled or fabricated section that is symmetrical about the plane of bending, though other situations are not excluded. No information is given on torsional properties, because few composite beams in buildings have significant torsional stiffness.

Provision is made for the encasement of steel webs in concrete, and for the increase in resistance to local, shear, and lateral buckling that encasement provides; but no provision is made for full encasement, nor for the influence of encasement on resistance to bending or longitudinal shear.

Where plastic analysis of cross-sections is used, as is usual, welded steel mesh is normally excluded from the effective section, unless it has been shown to have sufficient ductility *when built into a concrete slab*, to ensure that it will not fracture.

The effective widths of concrete flanges given in Eurocode 4 are greater than those in Eurocode 2 (typically, span/4 rather than span/5). This is because underestimates are unsafe for the design of shear connection, and although overestimates are in theory unsafe for the prediction of resistance to bending, the error is small. Furthermore, both inelastic behaviour of reinforcement and flexural cracking of concrete tend to increase the effective width of a concrete flange.

Cracking of concrete has a complex influence on the flexural stiffness of a composite section. This has been simplified by using only two stiffnesses in Eurocode 4, denoted E_aI_1 and E_aI_2. These are for the "uncracked" and the "cracked reinforced" sections, respectively.

The classification system for composite beams, based on the slendernesses of steel elements in compression, is as similar as possible to the system defined in Eurocode 3: Part 1.1. The definitions of classes 1 to 4 and the slenderness ratios at class boundaries are the same, following work by Kemp [1]. He showed that the adverse influence of crushing of concrete on rotation capacity of sections is offset by some less obvious advantages of composite members over steel members, and that the rules in Eurocode 4 are supported by the test data on continuous composite members.

Composite beams are usually in Class 1 at midspan. The Class at an internal support is strongly influenced by the area of longitudinal reinforcement in the slab. A small increase can move the steel web from Class 1 to Class 3 [2]. Design of Class 3 beams has to allow for the method of construction (propped or not) and cannot use partial shear connection, which is necessary for most beams where profiled steel sheeting is used.

The anomaly caused by the abrupt change in design procedures at the Class 2/3 boundary, (particularly the change from plastic to elastic section analysis) is avoided in Eurocode 4 by allowing the replacement of a web in Class 3 by an effective web in Class 2. This has a region near its centre that is assumed not to contribute to resistance to bending. The effective web is an extension of the idea of an effective width that has long been used in the elastic analysis of steel sections in Class 4. The stress distributions at the design ultimate hogging moment for a Class 3 section with and without "hole" in its web are compared in Figure 1, assuming that propped construction is used, f_y = 355 N/mm², f_{sk} = 460 N/mm², γ_a = 1.10, γ_s = 1.15. Without the hole, $M_{p\ell.Rd}$ would be 307 kNm.

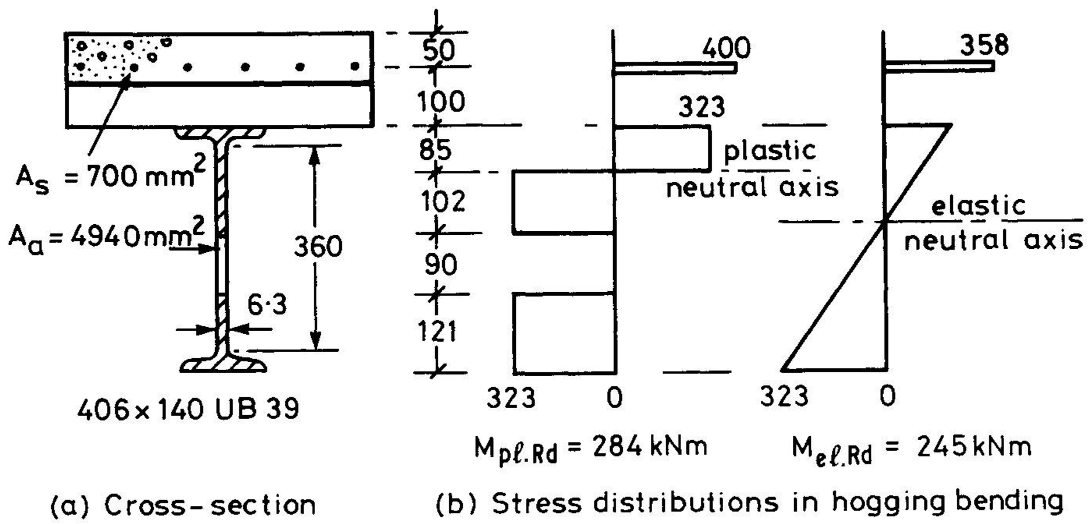


Fig. 1 Replacement of a Class 3 web by an effective web in Class 2

3. RESISTANCES OF CROSS-SECTIONS OF COMPOSITE BEAMS

3.1 Bending moment

For sagging bending of sections in Class 1 or 2, the rectangular stress block used for concrete (a compressive stress of 0.85 f_c) extends to the neutral axis. This is not in accordance with Eurocode 2, but is necessary to avoid complex calculations where (as commonly occurs) the neutral axis passes through a steel flange. Calibration has shown [3, 4] that the bending resistance thus calculated is satisfactory for beams with a concrete flange; but it can be unconservative for column sections, so a factor 0.9 is introduced in clause 4.8.3.13.

Account has been taken of the influence of crushing of concrete on the rotation capacity of a composite section by limiting the applicability of rigid-plastic global analysis where continuous beams have non-uniform spans, or carry concentrated loads.

Resistance to sagging bending is often determined by the degree of shear connection provided. In theory, this can also influence the classification of the section, but it rarely does so in practice.

3.2 Bending moment

The treatment of vertical shear is closely related to that in Eurocode 3, except that the resistance to buckling of slender webs is improved by the ability of shear connectors to anchor a tension field in the concrete slab, and also by encasement in concrete, if provided.

The curve of interaction between resistance to bending and to vertical shear is parabolic-rectangular, as in Eurocode 3, but intermediate points on the diagram are defined differently, for reasons explained elsewhere [2].

4. INTERNAL FORCES AND MOMENTS IN CONTINUOUS BEAMS

A distinction is made between "continuous beams", where the bending moments are not influenced by the properties of their connections and supports, and "beams in frames". Global analysis of the latter is discussed in Section 6, below.

Plastic global analysis is the simplest method for continuous beams in buildings, because no account need be taken of the method of construction or of creep or shrinkage of concrete. Where elastic analysis has to be used, redistribution of moments is allowed, depending on the classes of the cross-sections, the method of elastic analysis (i.e., "uncracked" or "cracked"), and whether midspan moments are being increased or decreased. The results of checks on these rules are available [5, 6].

Care has been taken to avoid giving any application rules in Eurocode 4 that are related to the position of a point of contraflexure in a beam. This is because these positions are different for each combination and arrangement of actions, and cannot easily be found where partial shear connection is used.

5. LATERAL-TORSIONAL BUCKLING IN BEAMS FOR BUILDINGS

A set of Principles of wide applicability is followed by Application Rules for the only relevant problem that commonly occurs in composite structures: lateral buckling of the bottom flange of a continuous beam, in a region of hogging moment. Under the "alternate-span" arrangement of variable load, such a region may extend over most of the length of an internal span.

Many designers doubt the need for such flanges to be braced. They dislike the complexity of methods based on the well-known elastic critical theory for non-distortional buckling, which has been shown to be over-conservative for beams where the steel top flange is restrained against rotation.

The three methods given in Eurocode 4 are based on the theory of distortional buckling and on tests [7, 8]. The simplest method (clause 4.6.2) defines the maximum depths of steel I and H sections for which no check on buckling is required. Its use has had to be qualified by many conditions, relating to the dimensions and loading of the structure. Graphs that simplify the checking of several of these conditions have been prepared [2].

The more general methods involve the calculation of a slenderness ratio $\bar{\lambda}_{LT}$ and the use of a reduction factor χ_{LT} taken from Eurocode 3. For many situations, a simplified expression for $\bar{\lambda}_{LT}$ can be used, based only on the properties of the steel section and the bending-moment distribution for the span concerned. The third method, the least conservative but the longest, involves calculation of the elastic critical moment at the internal support, and then $\bar{\lambda}_{LT}$.

6. INTERNAL FORCES AND MOMENTS IN FRAMES FOR BUILDINGS

A composite frame is defined as a structure in which some or all of the beams and columns are composite members. It is assumed that most or all of the remaining members are of structural steel. Eurocode 3 is therefore applicable during the construction phases. It can also be used as a basis for the composite stage. Eurocode 4 therefore mainly gives modifications and additions to Eurocode 3, necessary for the particular features of composite construction. Where the structural behaviour is essentially that of a concrete frame with only a few composite members, global analysis is to be in accordance with Eurocode 2.

As for steel structures, composite frames may be classified as sway or non-sway, braced or unbraced. For a frame to be considered as braced, it must be acceptable to assume that all horizontal loads are resisted by the bracing system. This requires the latter to provide a stiff response compared to other load paths. It has been usual to require the stiffness of the braced frame to be at least five times that of the unbraced structure [9]. This is expressed in Eurocode 4 as a reduction in horizontal displacements due to the inclusion of bracing. For a composite frame, the response will also be influenced by cracking of concrete and by creep. Neglect of cracking is permitted though, because this increases the unbraced stiffness and therefore increases the minimum stiffness required of the bracing.

A sway frame is one in which account needs to be taken of the additional internal moments and forces arising from horizontal displacements of the nodes (the 'P - Δ' effect). Sway frames are excluded from Section 4.9 because of their comparative rarity and because Section 4.8 on columns treats only non-sway members.

A frame that is classified as braced is treated as a non-sway frame. It is also possible for an unbraced frame to respond as a non-sway structure, but rules are not yet established to predict easily the appropriate portions of each span where the concrete should be taken as cracked. Unbraced non-sway frames are not excluded from the scope of Section 4.9, but no Application Rules are given for their analysis.

A further classification relates the method of global analysis to the anticipated behaviour of the connections, as shown in Table 4.8 of Eurocode 4. Some of the methods permitted by Eurocode 3 are not included in the Application Rules of Eurocode 4 because of their rarity in design practice.

Rules are given for rigid-plastic global analysis, but not for elastic-plastic analysis. Eurocode 3 distinguishes between two forms of the latter. "Elastic-perfectly plastic" analysis adopts the plastic hinge concept, and the rules for rigid-plastic analysis may therefore be used as a basis for the user to formulate rules for this approach. The second form, "elasto-plastic analysis", takes account of the spread of plastic zones and is therefore a specialised method for which it is inappropriate to give rules in a design standard.

The Application Rules include the use of partial-strength connections in frames analysed by rigid-plastic theory. A connection of this type has a design resistance moment less than that of the connected beam, so that a plastic hinge will tend to form in the connection. Tests on end-plate connections [10] with continuous slab reinforcement provided by mesh and by bars of diameter up to 12 mm have shown that fracture of reinforcement is the likely failure mode. It is important therefore to be sure that sufficient rotation capacity exists to develop the plastic collapse mechanism assumed in design.

For elastic global analysis, flexural stiffnesses for beams in braced frames may be taken as the uncracked values throughout each span, or a region each side of an internal support may be taken as cracked. Internal moments may be redistributed, usually by reducing support values. Limits to redistribution have been established by studies on continuous braced structures. Rules are not yet established for flexural stiffness and redistribution of moment in unbraced composite frames.

No Application Rules are given for elastic analysis of frames with semi-rigid connections. This is because methods to predict moment-rotation characteristics are not well-enough established for inclusion in Eurocode 4.

7. COMPOSITE CONNECTIONS IN BRACED FRAMES FOR BUILDINGS

Modifications and additions are needed to Eurocode 3, for connections in which reinforcement is intended to contribute to the resistance. A wide variety of composite connections can be envisaged and therefore only Principles are given in much of this Section.

For beam-to-column connections, Eurocode 3 has given rules for classification, in order that the appropriate method of global analysis can be determined. Eurocode 4 extends these to composite construction, for situations in which the slab reinforcement contributes to the tensile resistance of the connection. For this reason, Section 4.10 is restricted to braced frames.

To non-dimensionalise the classification, the properties of the connection are compared with those of the connected beam adjacent to the connection. The non-dimensional limits which define the classification have been adopted from Eurocode 3.
No detailed rules are given for the calculation of moment resistance, rotational stiffness and rotation capacity. Methods to predict these are not well-enough established to justify inclusion in Eurocode 4. At present, experimental evidence may therefore be required. Attention is drawn though to use of the rules in Eurocode 3, supplemented by consideration of the slab reinforcement. Limited studies [10] show reasonable agreement between resistance moments calculated on this basis and test results.

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EC 4: Columns, Slabs and Some Remarks on Execution

EC 4: Colonnes, dalles et remarques sur l'exécution

EC 4: Stützen, Platten und einige Bemerkungen über die Ausführung

K. ROIK
Prof. emer. Dr.
Ruhr-University
Bochum, Germany

H. BODE
Prof. Dr.
University Kaiserslautern
Kaiserslautern, Germany

SUMMARY

This paper deals with Eurocode No. 4, Part 1.1, chapters 4.8 and 9, and Annexe E. The design of composite columns assumes complete interaction between steel sections and concrete. This is the basis to establis slenderness ratios and to use buckling curves. In contrast, the design of composite slabs with ductile behaviour takes account of incomplete interaction and partial shear connection. This allows the inclusion of end anchorage facilities as well as additional reinforced bars.

RESUME

Cette contribution traite de l'Eurocode no 4, partie 1.1, chapitres 4.8 et 9 ainsi que l'annexe E. Le dimensionnement des colonnes mixtes suppose une interaction totale entre les sections d'acier et le béton. Ceci constitue la base pour définir des coefficients d'élancement et pour appliquer des courbes de contrainte de flambement. Par contre le dimensionnement des dalles mixtes avec un comportement ductile tient compte d'une interaction partielle et d'une connexion partielle. Ce qui permet de tenir compte des moyens d'ancrage ainsi que des armatures supplémentaires.

ZUSAMMENFASSUNG

Der Beitrag behandelt Eurocode Nr. 4, Teil 1.1, Kapitel 4.8 und 9 sowie Anhang E. Die Bemessung von Verbundstützen legt vollständiges Zusammenwirken zwischen Stahlprofilen und Beton zugrunde. Das liefert die Grundlage, Schlankheitsgrade zu definieren und Knickspannungskurven zu verwenden. Demgegenüber stellt die Bemessung von Verbunddecken mit duktilem Verhalten das unvollständige Zusammenwirken und den Teilverbund in Rechnung. Das ermöglicht es, Endverankerungsmassnahmen sowie Zusatzberechnungen zu berücksichtigen.

1. COMPOSITE COLUMNS

1.1 General

Composite columns are composite members subjected mainly to compression and bending. The steel section and the uncracked concrete section usually have the same centroid. Typical types of cross sections are shown in Fig. 1:

- concrete encased sections (steel section completely covered by concrete - Fig. 1 a),
- concrete filled sections (concrete completely covered by steel - Fig. 1 d - f),
- partially encased sections (Fig. 1 b - c).

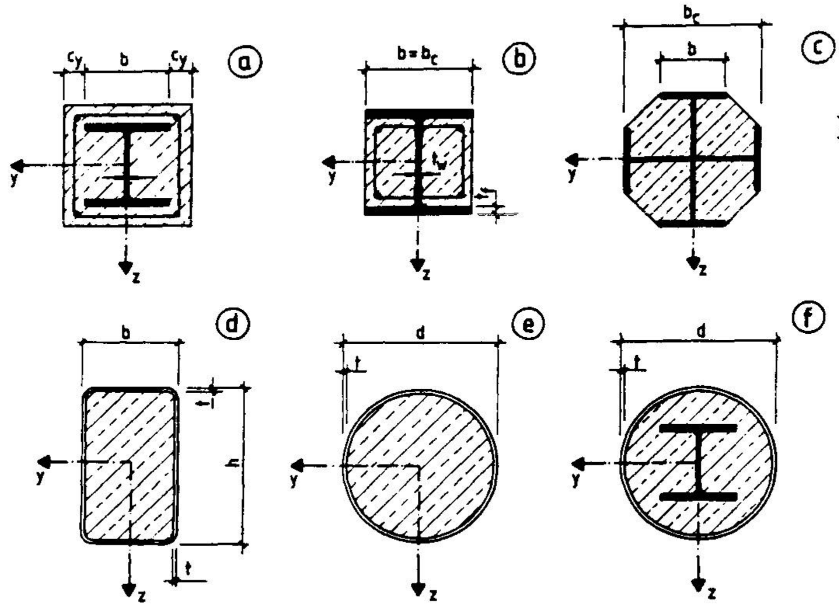


Fig. 1: Typical composite column cross sections

Composite columns have high load carrying capacities, while the outer dimensions are relatively small due to the structural steel sections, which provide a considerable amount of "reinforcement". In addition fire protection measures are not necessary or visible in most cases.

EC 4, clause 4.8 applies to isolated non-sway columns. These may be:

- compression members, which are integral parts of a non-sway frame, but which are isolated for design purposes, or
- real isolated compression members, that satisfy the classification "non-sway".

1.2 Ultimate limit state verifications

A composite column of any cross section, loaded by normal forces and bending moments, shall be checked at the ultimate limit state for:

- resistance to local buckling,
- introduction of loadings,
- resistance to shear (longitudinal and transverse),
- resistance of member (including lateral buckling).

Effects of local buckling may be neglected for steel sections fully encased and for other types of cross sections with limited width over thickness ratios.

Where internal forces and/or moments have to be distributed between the steel and concrete components, it must be ensured that within a specified introduction length, the individual components are loaded according to their capacity. A clearly defined load path shall be established without (excessive) slip at the interface.

The shear resistance shall be provided by bond stresses and friction at the interface or by mechanical shear connection, but again must be such that no significant slip occurs. This leads to the mechanical model of a homogeneous column with full interaction and no slip in the steel concrete interfaces.

To check the resistance of columns, two methods of design are given:

- a general method (4.8.2) including columns with non-symmetrical or non-uniform cross section over the column length,
- an attractive simplified method (4.8.3) for columns of double symmetrical and uniform cross section over the column length, but with a limited scope. Additional application rules for columns of mono-symmetrical section are given in Annex D.

The general method of design takes account of second order effects including imperfections and the non-linear material behaviour. It ensures that instability does not occur, and that the resistance of individual cross sections subjected to longitudinal force and bending is not exceeded.

Comprehensive numerical calculations are necessary to carry out such a non-linear design, which is possible only by means of a computer, and there is a large variety of composite column cross sections. The need to specify simple design methods has led to the simplified method (4.8.3) as an attractive alternative. The scope of it is limited, as it has been based on certain assumptions and adopts the European buckling curves originally established for bare steel columns, as basic design curves for composite columns.

Both design methods assume full composite action up to failure without (excessive) slip at the steel-concrete interface.

1.3 Simplified method of design

1.3.1 Resistance to axial loads

The steel contribution ratio $\delta = A_a \cdot f_{yd}/N_{pl,Rd}$ must satisfy the requirement

$$0.2 < \delta < 0.9 \qquad (1)$$

where A_a is the area of the structural steel section,
f_{yd} is its design yield strength, and
$N_{pl,Rd}$ is the design plastic resistance to compression, for the composite cross section.

If δ is less than 0.2 the column may be designed according to EC 2; if δ is larger than 0.9, design must be done on the basis of EC 3.

The plastic resistance to compression of an encased cross section should be calculated by adding the plastic resistance of its components:

$$N_{pl,Rd} = A_a \cdot f_y/\gamma_a + A_c \cdot (0.85 \cdot f_{ck}/\gamma_c) + A_s \cdot f_{sk}/\gamma_s. \qquad (2)$$

Significant economy can be achieved in designing stocky concrete-filled circular steel columns by taking account of triaxial effects due to the confinement of steel tube:

$$N_{pl,Rd} = A_a \cdot \eta_2 \cdot f_y/\gamma_a + A_c\,(f_{ck}/\gamma_c)[1 + \eta_1 \frac{t}{d} \cdot \frac{f_y}{f_{ck}}] + A_s \cdot f_{sk}/\gamma_s \qquad (3)$$

where $0 < \eta_1 < 4.90$ and $1.00 > \eta_2 > 0.75$,
A_c and A_s are the cross-sectional areas of the concrete and the reinforcement, and
f_{ck} and f_{sk} are their characteristic strengths, respectively.
γ_a, γ_c, and γ_s are the partial safety factors γ_M for structural steel, concrete, and reinforcement, respectively; and dimensions t and d are defined in Fig. 1.

These triaxial effects diminish with increasing load ecentricity or column slenderness $\bar{\lambda}$. If the ecentricity e exceeds the value d/10, or the relative slenderness $\bar{\lambda}$ exceeds the value 0.5, then the confinement is no longer effective, yielding

$$\eta_1 = 0 \text{ and } \eta_2 = 1.0.$$

A slender composite column has sufficient resistance if for both axes

$$N_{Sd} < \chi \cdot N_{pl,Rd}, \tag{4}$$

where the reduction coefficient χ depends on the relevant slenderness $\bar{\lambda}$ and the appropriate buckling curve in Eurocode 3: Part 1.1:

- curve a for concrete filled hollow profiles,
- curve b for partially and fully encased profiles with bending about the strong axis of the steel section,
- curve c for encased sections with bending about the weak axis.

Extra imperfections within the column length need not be considered as they are taken into account in this determination of column resistance.

The non-dimensional slenderness is given by

$$\bar{\lambda} = \sqrt{N_{pl,R}/N_{cr}} < 2.0 , \tag{5}$$

where $N_{pl,R}$ is the value of $N_{pl,Rd}$ when the γ_M-factors are taken as 1.0, and N_{cr} is the elastic critical load calculated from

$$N_{cr} = \pi^2(EI)_e/l^2. \tag{6}$$

where l is the buckling length.

$(EI)_e$ denotes an effective flexural stiffness of cross section, where a term $0.8 \cdot E_{cd} \cdot I_c$ is used for the concrete part. Particularly this term has been calibrated in such a manner, that ultimate load test results are in good agreement with calculated column resistances.

Additional application rules are given to reduce the effective elastic modulus of concrete in order to account for long-term loading.

1.3.2 Resistance to combined compression and uniaxial bending

The resistance of cross sections in combined compression and bending can be determined from the interaction diagram, Fig. 2. The curve can be represented by the further simplified

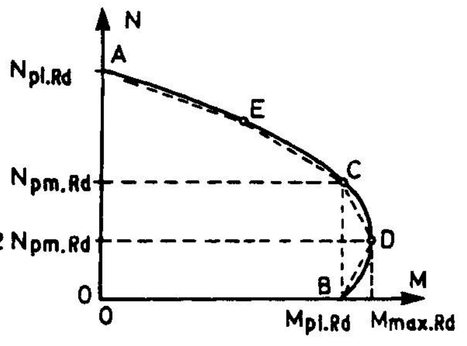


Fig. 2: Interaction curve, cross section

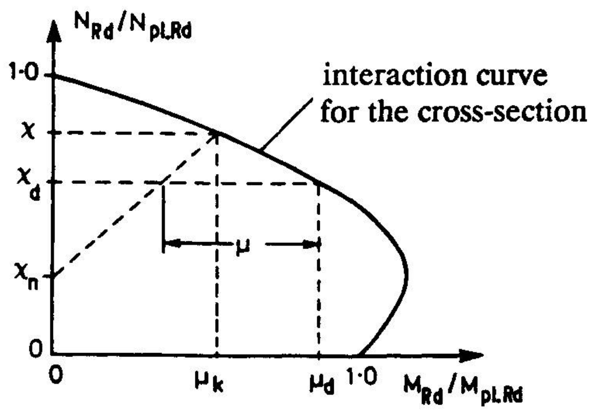


Fig. 3: Design procedure for columns

polygonal diagram (dashed line). Points A to D may be calculated assuming rectangular stress blocks, disregarding particular strain limitations. More information for the simple calculation of points A to D is given in Annex C.

The moment of resistance at point C is obviously identical to that at point B: $M_c = M_{pl,Rd}$. It can be shown that the axial force $N_{pm,Rd}$ equals the compressive resistance of the whole area of concrete, which can be calculated easily.

This interaction diagram for the resistance of cross sections may be used to check the column resistance too, see fig. 3.

First the resistance of the column under axial compression has to be determined as mentioned before. This resistance is defined by the reduction factor χ, which accounts for the influence of imperfections and slenderness. According to this factor χ the μ_k-value for the bending moment, which represents the moment due to imperfection, can be read off the interaction curve (or polygon). The influence of this imperfection moment is assumed to decrease linearly to the value χ_n. For the related design normal force $\chi_d = N_{Sd}/N_{pl,Rd}$ the moment factor μ represents the remaining moment resistance. It must then be shown that

$$M_{Sd} < 0.9 \cdot \mu \cdot M_{pl,Rd}, \qquad (7)$$

where M_{Sd} is the maximum design bending moment within the column length, calculated including second order effects if necessary (see below).

The value χ_n accounts for the fact that imperfections and bending moments do not always act together unfavourably. For columns with end moments, χ_n may be calculated from

$$\chi_n = \chi \cdot \frac{1-r}{4}, \text{ but } \chi_n < \chi_d \qquad (8)$$

where r = ratio of end moments ($-1 < r < +1$)

Columns generally shall be checked for second order effects. This influence may be neglected in case of isolated non-sway columns as long as:

- the normal force N_{Sd} is smaller than 10 % of the critical load N_{cr} or
- the relative slenderness $\bar{\lambda}$ does not exceed the value $\lambda_{crit} = 0.2(2 - r)$.

The length μ in Fig. 3 may be calculated from the equation

$$\mu = \mu_d - \mu_k (\chi_d - \chi_n)/(\chi - \chi_n). \qquad (9)$$

This equation can be further simplified by setting $\chi_n = 0$.

This simplified design method is based on a lot of international research reports on composite columns, including work done by Janss, Dowling, Johnson, Roik and their teams.

The background paper /2/ contains comparison calculations with 208 well documented tests. These calculations yielded an average variation coefficient $V_{Rt} = 0.07$ and statistically determined design values $r_d = 0.665$ related to mean values. Compared with the simplified design method of EC 4 the ratio lies between 0.97 and 1.25, with a mean value of 1.08 on the safe side.

1.3.3 Combined compression and biaxial bending

Due to the different slenderness, bending moments, and resistances of bending for two axes, in many cases a check for the biaxial behaviour is necessary. EC 4 contains a similar design method for this case, using values μ_y and μ_z for the two axes of bending.

2. EXECUTION

Minimum standards of workmanship required during execution are specified in chapter 9 to ensure that the design assumptions are satisfied and hence that the intended level of safety can be attained. But this chapter, which includes reference to Eurocode 2 and 3, is neither intended, nor extensive enough, for a contract document.
Paticularly the following topics are mentioned:

- Stability of the steelwork during erection,
- Early and sufficient fixing of profiled steel sheeting,
- Speed and sequence of erection, propped and unpropped construction,
- Welding of headed studs through metal decking to the supporting beam; welding conditions; checks and visual inspection,
- Use of friction grip bolting, anchors, hoops, and block connectors including corrosion protection in the interface.

3. ALTERNATIVE DESIGN METHOD FOR COMPOSITE SLABS

3.1 General

The partial shear connection design method as given in Annex E should be used for composite slabs with ductile behaviour only. This alternative to the m+k-method may be used to account also for contributions from additional end anchorage means or longitudinal reinforcing bars.

Figure 4 illustrates the ductile behaviour of a particular composite slab. Presented are test loading and end slip plotted against the midspan deflection. In this special test a metal decking with re-entrant shape and additional embossments has been used. Ductility means that significant slip occurs at the steel-concrete interface, before the maximum

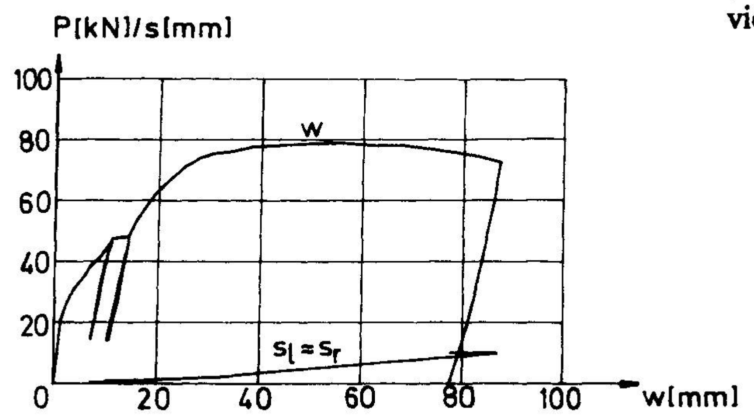
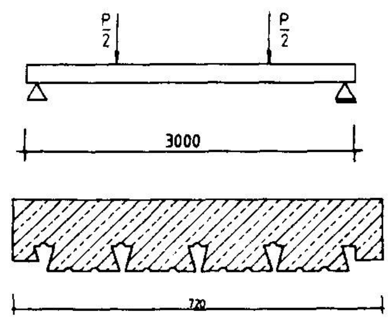


Fig. 4: Test results; particular composite slab with ductile behaviour



test load has been reached. In designing such composite slabs it may be assumed - and should be verified by tests - that sufficient slip can occur for moments of resistance at critical cross sections to be calculated from plastic theory, based on partial shear, and therefore with a second plastic neutral axis in the profiled sheeting. This design method leads to a unified design of composite beams and slabs with ductile shear connection.

3.2 Determination of design shear strength

Slab tests (see EC 4, chapter 10.3) only are to be carried out in order to determine the design value of the horizontal shear strength $\tau_{u.Rd}$. This is the only parameter which has to be evaluated from tests.

Fig. 5 shows a particular partial connection diagram for the test evaluation, which incorporates the actual geometry with measured dimensions and strengths of the considered test specimen.

At the end of a test, at failure, a bending moment M_{test} is acting on the critical cross section under the point load. The degree of shear connection η_{test}, which can be read off the diagram, yields the horizontal shear strength between the end of the metal decking and the load position:

$$\tau_u = \frac{\eta_{test} \cdot N_{cf}}{b(L_s + L_o)} = \frac{N_c}{b(L_s + L_o)} \tag{10}$$

where N_c is the compressive force in the concrete slab,
N_{cf} is the value of N_c for full shear connection,
b is the breadths of the concrete slab, and
L_s and L_o are defined in Fig. 5.

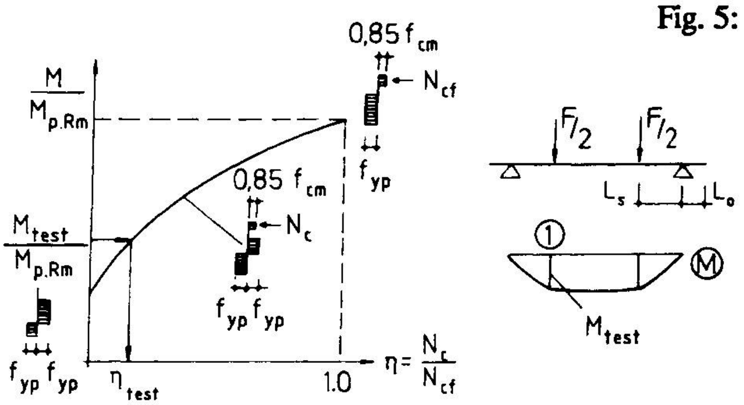


Fig. 5: Determination of the degree of shear connection from M_{test}

The force N_c is limited due to the incomplete shear connection, and thus it reduces the bending resistance. At the end of each test series the derived τ_u-values provide the basis to determine the characteristic value $\tau_{u.Rk}$ as the minimum value from all tests of this series minus 10 %. The design shear strength $\tau_{u.Rd}$ equals this characteristic value, divided by $\gamma_v = 1.25$.

3.3 Verification at the ultimate limit slate

The partial connection diagram - now calculated with design values - represents the boundary curve for the bending moment resistance M_{Rd} of the slab in Fig. 6:

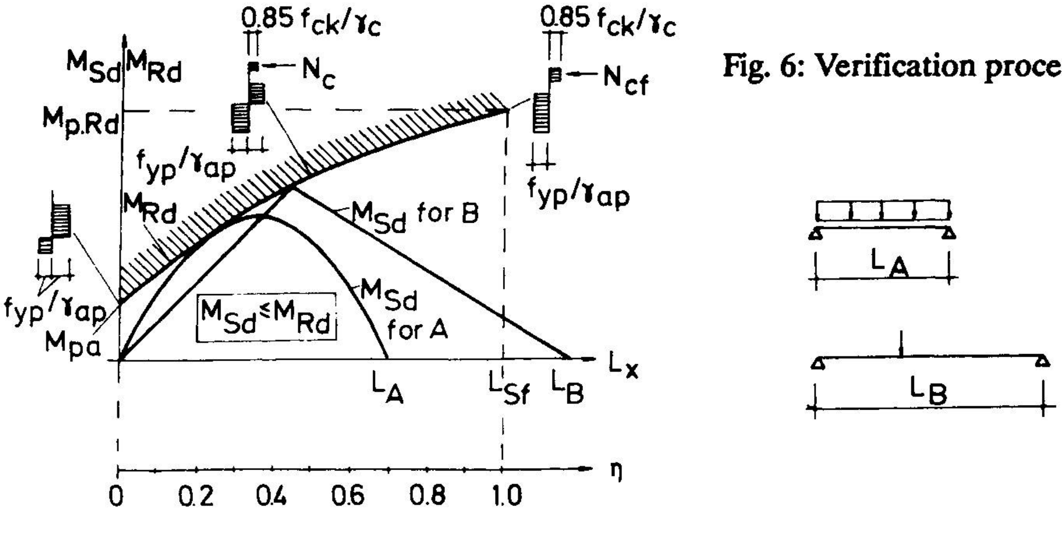


Fig. 6: Verification procedure

The compression force N_c at L_x can be determined from $N_c = b \cdot L_x \cdot \tau_{u.Rd}$, (11)

while the length L_{sf} is given by

$$L_{sf} = N_{cf}/(b \cdot \tau_{u.Rd}) \qquad (12)$$

and denotes the clear distinction between full and partial shear connection. At any cross section the design bending moment M_{Sd} due to loading and span should not exceed the design resistance M_{Rd}.

In case of additional end anchorage, account may be taken by adding the end anchorage design strength V_{ld} as follows:

$$N_c = b \cdot L_x \cdot \tau_{u.Rd} + V_{ld} \qquad (13)$$

This results in a shift of the basic partial interaction diagram in the L_x-direction over a distance of $- V_{ld}/(b \cdot \tau_{u.Rd})$. It should be noted, however, that end anchorage does not only increase the strength, but also enhances the total slab behaviour up to failure, particularly with respect to ductility. As an example Fig. 7 abows the different behaviour in composite slab tests, where a special trapezoidal sheeting has been used without and with end anchorage (3 and 5 throughwelded studs), respectively.

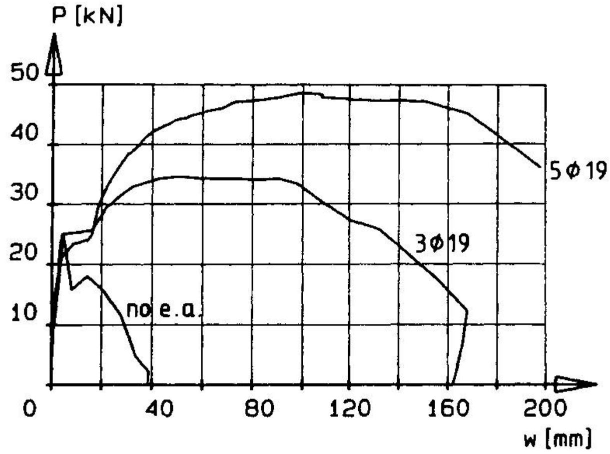
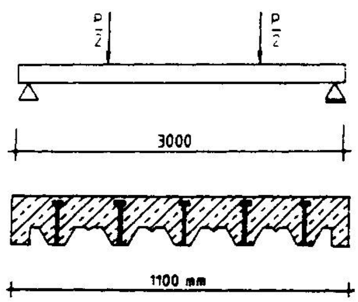


Fig. 7.: Enhancement of behaviour and load carrying capacity due to end anchorage



If additional bottom reinforcement shall be taken into account, the verification should follow the same procedure. But the partial interaction diagram should be modified by adding the bending strength of the reinforced concrete part, which leads to a larger compression force N_c simultaneously:

$$N_c = b \cdot L_x \cdot \tau_{u.Rd} + N_{as}, \qquad (14)$$

where N_{as} is the design strength of fully anchored bottom reinforcement.

The validity of the partial connection method for composite slabs with end anchorage or/and additional reinforcement should be proved by further tests.

From the today's point of view the following methods of end anchorage are of main interest:

- through welded headed studs
- bent rib anchors in case of metal decking with re-entrant shape.

3.4 Conclusion

It is likely that other methods of anchorage and new profiled sheeting will enter the market. Annex E will not prevent further developments, but will actually give some helpful support. Additionally, Annex E pushes the development of new products to slabs with ductile shear connection behaviour, mainly depending on the type of profiled sheeting used.

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.

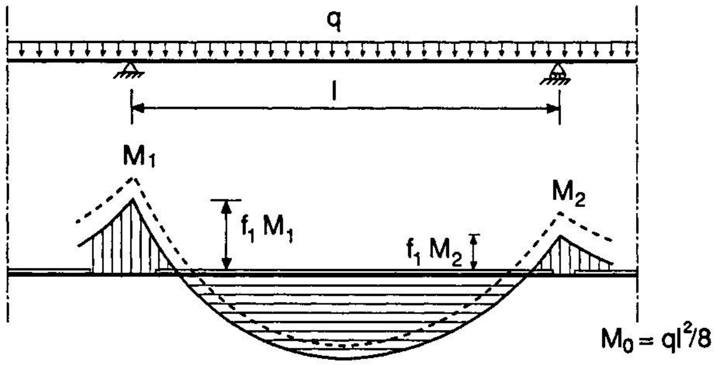


Figure 1 Simplified method for control of deflection

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:

$$\delta = \delta_0 \left[1 - C_1 \left(f_1 M_1 + f_1 M_2 \right) / M_0 \right]$$

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.

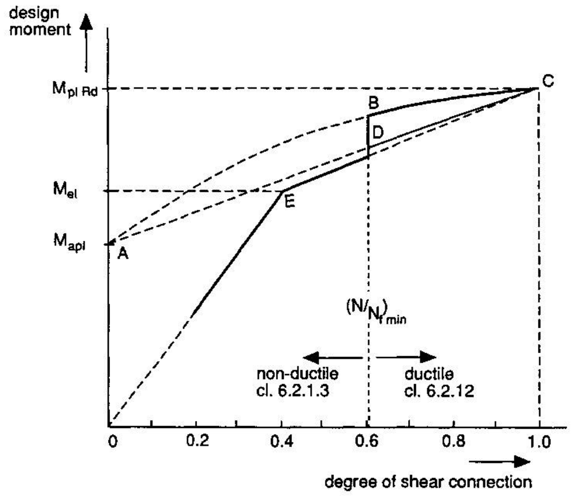


Figure 2 Design diagrams for partial shear connection

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.

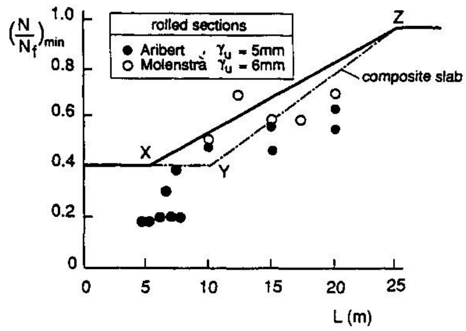


Figure 3 Minimum degree of shear connection for ductile connectors

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.

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EC 4: Structural Fire Design

EC 4: Calcul de la résistance au feu

EC 4: Brandbemessung

J.B. SCHLEICH
Mgr Struct. Steel Res.
ARBED Recherches
Luxembourg, Luxembourg

SUMMARY

The scope of Eurocode 4 Part 10, now Part 1.2, is to give principles and rules to carry out a structural fire design for composite steel and concrete structures. Three different levels of structural fire design are described, covering member analysis and analysis of fire performance of total building frameworks. The last procedure may be used for the standard fire and for general time-temperature regimes. Concerning the relationship with Parts 10 of EC 2, concrete structures and of EC 3, steel structures, it should be emphasized that Part 10 of EC 4 is in full compliance with the thermo-mechanical material properties at elevated temperatures for concrete and steel. Strainhardening of steel may be activated for composite structures as local instability is less critical than for purely steel structures.

RESUME

L'Eurocode 4: partie 10, maintenant la partie 1.2, traite les méthodes de calcul pour la résistance au feu des structures mixtes acier-béton. On décrit les trois types de calcul pour l'analyse des éléments et de l'ossature du bâtiment. Le troisième type peut être utilisé pour un incendie normalisé et des courbes température-temps générales. Les parties 10 des Eurocodes 2 et 3 sont compatibles avec la partie 10 de l'Eurocode 4 quant aux caractéristiques thermo-mécaniques de l'acier et du béton à température élevée. On explique comment on peut profiter de l'écrouissage de l'acier puisque le flambement local est moins critique pour les structures mixtes que pour les structures en acier.

ZUSAMMENFASSUNG

Eurocode 4, Teil 10 (nun Teil 1.2) umfasst Prinzipien und Regeln zur Brandbemessung für Verbundtragwerke aus Stahl und Beton. Es werden drei unterschiedliche Niveaus für die Brandbemessung von Tragelementen und die Ermittlung der Feuerbeständigkeit ganzer Geschossrahmenbauten beschrieben. Letztere kann für Standardbrandkurven oder allgemeine Temperatur-Zeitverläufe verwendet werden. Die Werkstoffeigenschaften von Beton und Stahl bei höheren Temperaturen stimmen mit denen der Teile 10 von EC 2 (Betontragwerke) und EC 3 (Stahltragwerke) überein. Allerdings darf in Verbundtragwerken die Verfestigung des Stahls berücksichtigt werden, da örtliche Instabilität weniger kritisch ist als in reinen Stahltragwerken.

1. INTRODUCTION

Eurocode 4: Part 10, Structural Fire Design, was issued for national comments in 1990 [1], and is now being revised [2]. It should be issued as a pr ENV in 1993, for approval by CEN before publication as ENV 1994: Part 1.2.

This Part 1.2 of Eurocode 4 deals with the design of composite steel and concrete structures for the accidental situation of fire exposure and shall be used in conjunction with Part 1.1 of Eurocode 4 and Part 10 of Eurocode 1. This Part 1.2 only identifies differences or supplements to the design for normal conditions of use.

Typical composite cross-section types [3, 4] for slabs, beams and columns, partially developed in view of fire resistance requirements, are given in Fig. 1.

2. BASIC PRINCIPLES

Shear connection. For all composite cross-sections longitudinal shear connection between steel and concrete shall be assured according to the principles of Part 1.1 of EC4. Nevertheless there shall be no shear connection between the steel components directly heated of a composite cross-section and the encased concrete.

Performance requirements. For structural fire design the main failure criterion to be fulfilled is CRITERION "R" which corresponds to the requirement that structures shall maintain their load bearing function during the relevant fire exposure. It shall be verified that $R_{f,d} \geqslant E_{f,d}$, which means that the design load bearing resistance of the structure exposed to a fire shall be larger than the design effect of actions during the required fire exposure time. Structural failure will correspond to the loss of equilibrium and may be due to rupture of sections, buckling, plastic hing formation or structural collapse mechanism [5, 6]. Composite structural elements easily comply with criterion "R", due to an adequate concept of the cross-section. Therefore, it is normally not required to apply additional insulation.

Partial safety factors. For design values of the thermal and mechanical properties of steel and concrete, a partial safety factor of $\gamma_{M,f} = 1{,}0$ shall be adopted when considering the accidental situation of fire exposure.

Assessment methods. The structural system adopted shall reflect the performance of the entire structure exposed to plan any fire [6]. This structural GENERAL ANALYSIS shall take into account the relevant failure mode in fire exposure, the temperature dependent material properties and stiffnesses and effects of thermal expansions. The design effect of actions $E_{f,d}$ shall be based on the fundamental factored load combination in the fire situation given in § 5.3.1 of Eurocode 1: Part 10 by f.i. 1,0G + 0,5W + 0,3Q.
As an alternative to the general analysis, an ANALYSIS OF PARTS OF THE STRUCTURE or a MEMBER ANALYSIS may be performed (see Table I). In this case, the design effect of actions in the fire situation $E_{f,d}$ may be deduced from E_d, the design effect of actions resulting from the fundamental factored load combination for normal conditions of use given in Eurocode 1, by $E_{f,d} = E_d/\gamma_F = 0{,}7E_d$. This alternative foreseen in § 5.3.2 of Eurocode 1: Part 10, considers a weighted mean partial safety factor for actions included in E_d of $\gamma_F = (1{,}35 + 1{,}5)/2 \cong 1{,}43$.

3. MECHANICAL AND MATERIAL PROPERTIES [1, 2, 5]

Strength and deformation properties. The strength and deformation properties of structural steel at elevated temperatures are - for heating rates between 2 and 50°C/min - characterized by a set of stress-strain relationships with a linear-elliptical shape for strains up to $\varepsilon_{a,\theta} \leq 2$ %. According to Fig. 2 this material law may be extended by the strain-hardening option for temperatures below 400°C, provided local instability is prevented and the ratio between the tensile strength at high termpature $f_{at,\theta}$ and the yield point at normal conditions of use $f_{ay,20°C}$ is limited to 1.25.
The strength and deformation properties of uniaxially stressed concrete at elevated temperatures are characterized by a set of stress-strain relationships as specified in Fig. 3. Whereas the main parameters are the compressive strength $f_{c,\theta}$ and the corresponding strain $\varepsilon_{c1,\theta}$, a descending branch should be adopted when a general calculation model is used (see Table II).

Thermal properties. The thermal conductivity λ and the specific heat C of steel and concrete are given in Fig. 4 as a function of temperature.

Fig.1/a: Typical cross-sections of composite slabs with profiled steel sheets.

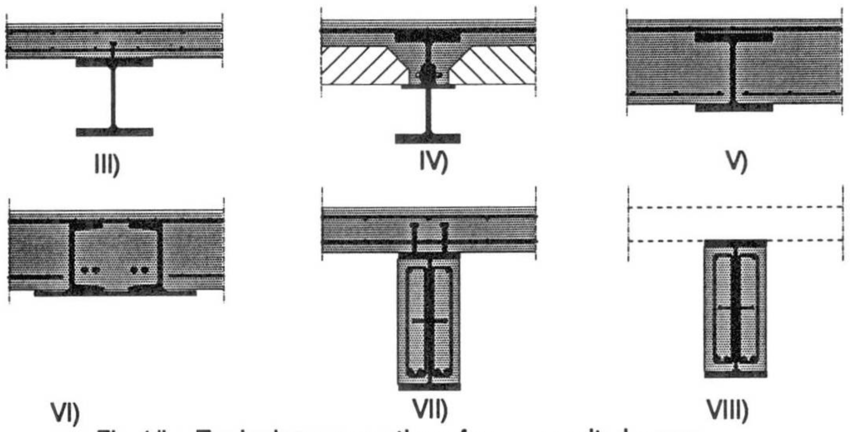


Fig.1/b: Typical cross-sections for composite beams.

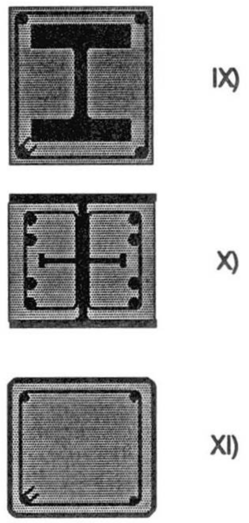


Fig.1/c: Typical cross-sections for composite columns.

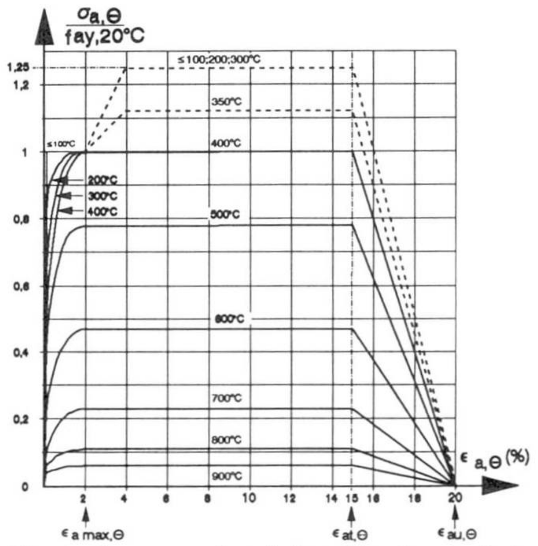


Fig. 2 : Stress-strain relationships for steel at high rising temperatures.

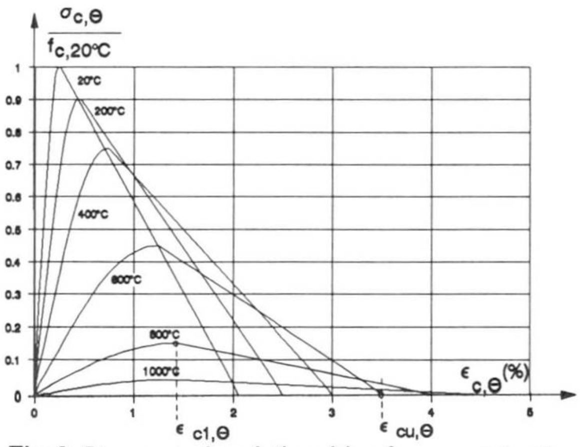


Fig.3: Stress-strain relationships for concrete at high rising temperatures.

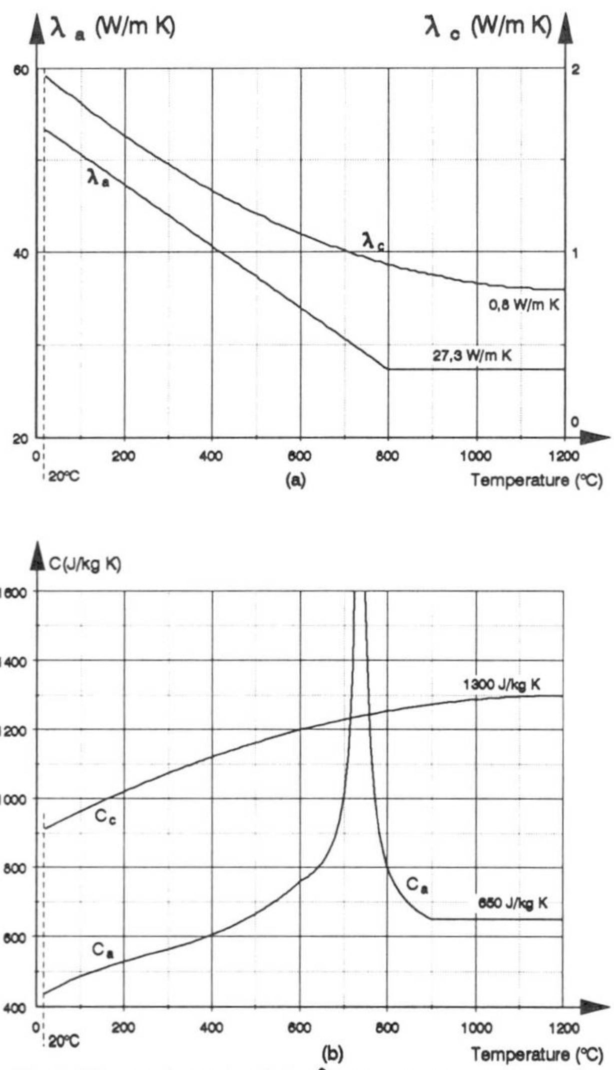
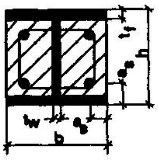


Fig.4: Thermal conductivity λ and specific heat C of steel (a) and concrete (c) given as a function of temperature.



Line		R30	R60	R90	R120
		Fire Resistance Class			
1	Minimum cross-sectional dimensions for load level $\eta_f = 0.3$				
1.1	minimum dimensions h and b (mm)	160	260	300	300
1.2	minimum axis distance of reinforcing bars a_s (mm)	40	40	50	60
1.3	minimum ratio of web/flange-thickness t_w / t_f	0.6	0.5	0.5	0.7
2	Minimum cross-sectional dimensions for load level $\eta_f = 0.5$				
2.1	minimum dimensions h and b (mm)	200	300	300	-
2.2	minimum axis distance of reinforcing bars a_s (mm)	35	40	50	-
2.3	minimum ratio of web/flange-thickness t_w / t_f	0.6	0.6	0.7	-
3	Minimum cross-sectional dimensions for load level $\eta_f = 0.7$				
3.1	minimum dimensions h and b (mm)	250	300	-	-
3.2	minimum axis distance of reinforcing bars a_s (mm)	30	40	-	-
3.3	minimum ratio of web/flange-thickness t_w / t_f	0.6	0.7	-	-

Fig. 5 : Minimum cross sectional dimensions, minimum axis distance and minimum ratio of web/flange-thickness t_w / t_f of I-sections concreted between the flanges (LEVEL 1).

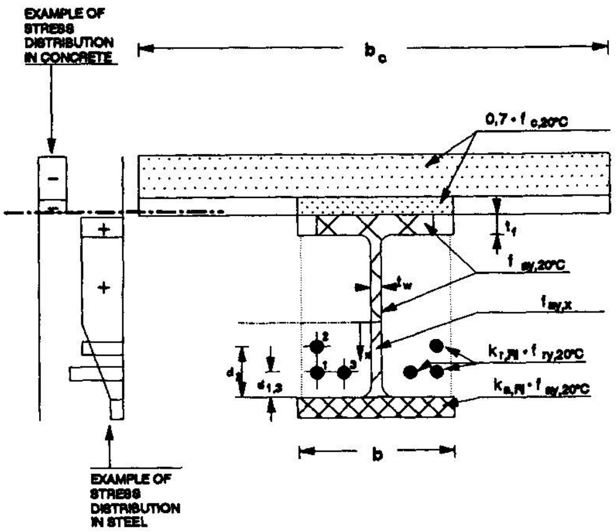


Fig. 6 : Evaluation of the positive plastic bending moment resistance of a composite beam, for different ISO-fire classes R30 to R180 (LEVEL 2).

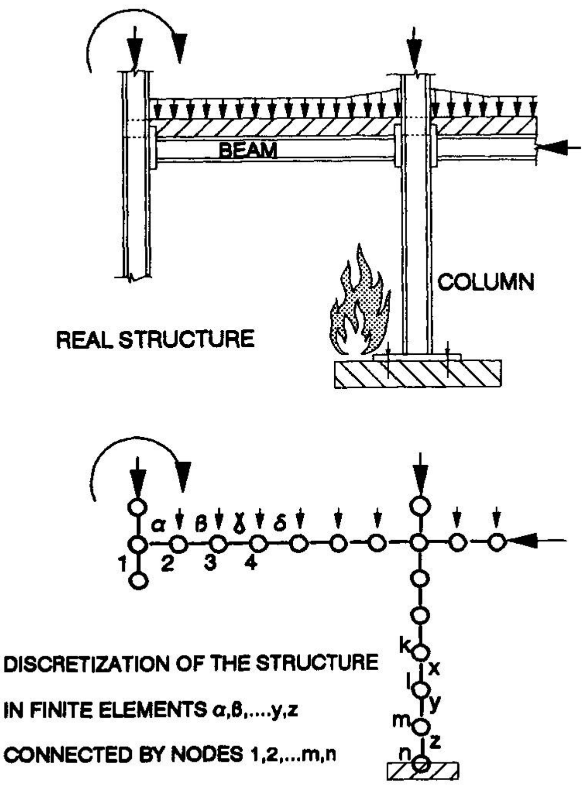


Fig.7 : Real structure with discretization in finite elements.

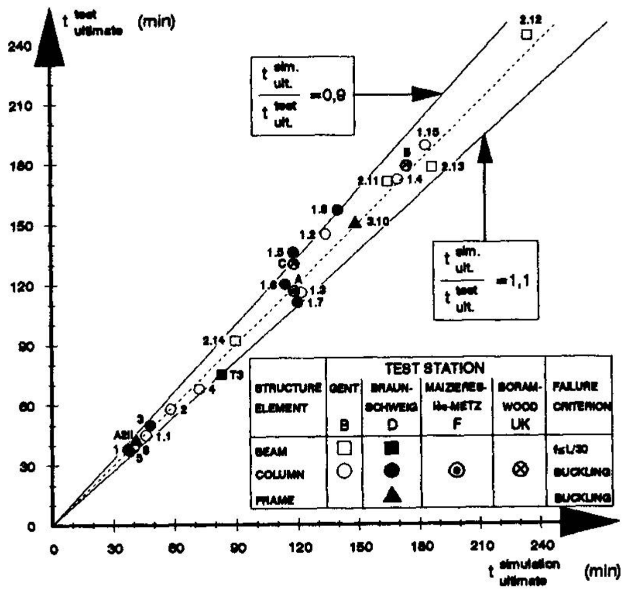


Fig. 8 : Fire resistance times measured and calculated by numerical model for columns, beams or frames of any cross section types (bare steel, protected steel, composite), [9,10,11,12].

4. STRUCTURAL FIRE DESIGN MODELS

Design shall be performed using any of the three fire design models, Level 1, 2 or 3, given in Table II. The relationship between these models and the three assessment methods related to the adopted structural system are shown in Table I.

Tabulated data / Level 1. Application of tabulated data is confined to individual member analysis. The structural member is considered as directly exposed to fire over its full length, and the thermal action corresponds to ISO-fire conditions. Extrapolation outside the range of experimental evidence is not allowed. This fire design model, as shown f.i. in Fig. 5, allows to determine the maximum load level η_f for fire design. This permits the evaluation of the design load bearing resistance of the structural member in the fire situation by $R_{f,d} = \eta_f \cdot R_d$, where R_d represents the design load bearing resistance at 20°C according to EC4: Part 1.1. It shall be verified that $R_{f,d} \geq E_{f,d} = 0,7 \cdot E_d$.

Simple calculation models / Level 2. Application of simple calculation models is normally confined to member analysis, but may be used for the analysis of parts of the structure (f.i. continuous beam or continuous column). The structural member is considered as exposed to ISO-fire conditions. Extrapolation outside the range of experimental evidence is not allowed. These fire design models [7 ,8], as shown in Fig. 6, permit the direct evaluation of the design load bearing resistance of the structural member in the fire situation $R_{f,d}$.

It shall, of course be verified that $R_{f,d} \geq E_{f,d} = 0,7\ E_d$.

General calculation models / Level 3. Application of general calculation models deals with the response to fire of structural members, of parts of the structure or entire structures. They permit the assessment of the interaction between parts of the structure which are directly exposed to any fire and those which are not exposed. Extrapolation outside the range of experimental evidence is allowed. These fire design models [9, 10, 11, 12] are based on a complete description of the physical processes involved (see Fig. 7). Their use is subject to a validation consisting in a verification of the numerical simulation results on basis of the corresponding test results (see Fig. 8). General calculation models permit a detailed investigation of the structural behaviour during and even after fire [13, 14]. They always lead to the design fire resistance time $t_{f,d}$ to be compared to the required fire resistance time $t_{f,r}$.

5. SOME SIGNIFICANT COMPARISONS

Members analysis by Level 1, 2 and 3 Design Models. As described in Table II the structural fire design model with a higher level, needs a larger calculation amount but corresponds also to a higher design accuracy. Therefore in order to obtain similar safety margins with these different design models, the corresponding design load bearing resistances for a given ISO-fire class should fulfill the following relation:

$$(R_{f,d})^{LEVEL\ 1} < (R_{f,d})^{LEVEL\ 2} < (R_{f,d})^{LEVEL\ 3}$$

Some representative calculation results for a partially encased beam connected to a slab and a partially encased profile column are shown in Table III.

Level 3 Design Model applied to different structural systems. Only general calculation models allow to determine the influence of the adopted structural systems. The general analysis of the entire structure leads of course to the largest calculation amount, but brings also the highest design accuracy. A general analysis permits the ACTIVATION OF HIDDEN LOAD BEARING RESISTANCES and leads to a HIGHER FIRE RESISTANCE TIME than that obtained with the member analysis of the weakest structural member of the structure. Some representative calculation results performed on an unprotected steel beam connected to a concrete slab are shown in Table IV [12, 15]. The permanent beam deflection after a local natural fire application, obtained when considering this heated beam as a part of the entire structure, is such that this composite beam may even not need to be replaced.

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TABLE I

STRUCTURAL FIRE DESIGN MODELS	ASSESSMENT METHODS $R_{f,d} \geq E_{f,d}$		
	MEMBER ANALYSIS	ANALYSIS WITH PARTS OF STRUCTURE	GENERAL ANALYSIS
LEVEL 1 TABULATED DATA	DESIGN EFFECT OF ACTIONS IN THE FIRE SITUATION GIVEN BY		
LEVEL 2 SIMPLE CALCULATION MODELS	$E_{f,d} = 0{,}7 \cdot E_d$		
LEVEL 3 GENERAL CALCULATION MODELS	EUROCODE1/PART10 / §5.3.2		LOAD COMBINATION IN THE FIRE SITUATION EUROCODE1/ PART10 / §5.3.1

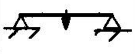
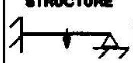
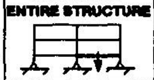
TABLE II

STRUCTURAL FIRE DESIGN MODELS	AIM	DESIGN RESULTS	DESIGN ACCURACY	CALCULATION AMOUNT
LEVEL 1 TABULATED DATA	INVESTIGATION OF MAIN PARAMETERS	ISO FIRE CLASSIFICATION	LOW	SMALL
LEVEL 2 SIMPLE CALCULATION MODELS	CALCULATION OF DESIGN LOAD BEARING RESISTANCE	$R_{f,d} \geq E_{f,d}$ FOR ISO FIRE	MEDIUM	MEDIUM
LEVEL 3 GENERAL CALCULATION MODELS	INVESTIGATION OF STRUCTURAL BEHAVIOUR DURING AND AFTER FIRE	$t_{f,d} \geq t_{f,r}$ FOR ANY FIRE	HIGH	LARGE

TABLE III

ANALYSED MEMBER	DESIGN LOAD BEARING RESISTANCE FOR ISO R90			
	TYPE	LEVEL 1	LEVEL 2	LEVEL 3
PARTIALLY ENCASED BEAM CONNECTED TO SLAB	BENDING MOMENT RESISTANCE	[1]	[8]	[12]
HE 900 AA / Fe 510 4φ28 / S500 SLAB 500 cm x 20 cm / C30	$M_{f,d}^{R90}$ in kNm	3548	3586	5105
PARTIALLY ENCASED PROFILE COLUMN	AXIAL LOAD BUCKLING RESISTANCE	[1]	[7]	[12]
HP 360x410x176 / Fe 510 8φ20 / S500 CONCRETE / C40	$N_{f,d}^{R90}$ in kN FOR $L_{cr} = 3$ m	3792	3851	4664

TABLE IV

COMPOSITE BEAM SPAN 15,5m	LEVEL 3/GENERAL CALCULATION MODEL [12, 15]		
IPE 600 / Fe 510 SLAB 240 cm x 15 cm / C35	MEMBER ANALYSIS 	ANALYSIS WITH PARTS OF THE STRUCTURE 	GENERAL ANALYSIS OF ENTIRE STRUCTURE 
PERMANENT BEAM DEFLECTION AFTER LOCAL NATURAL FIRE	24 cm	14 cm	10 cm