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Eurocode 4: Composite Steel and Concrete Structures

Eurocode 4: constructions mixtes acier-béton

Eurocode 4: Bemessungsnorm für Stahl- und Betonverbundkonstruktionen

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SUMMARY

The first draft of Eurocode 4 was written in 1984, to be consistent with the available drafts of Eurocodes 2 and 3. An account is given of its scope, principal contents and main innovations, and its relationship to Eurocodes 2 and 3 and to the Model Code for composite structures that was prepared between 1972 and 1981.

RÉSUMÉ

En 1984, la version préliminaire de l'Eurocode 4 fut écrite conformément aux versions préliminaires des Eurocodes 2 et 3. L'exposé donne un aperçu du contenu, les innovations essentielles, les relations avec les Eurocodes 2 et 3, les relations avec le Code Modèle des Constructions Mixtes, mis au point entre 1972 et 1981.

ZUSAMMENFASSUNG

Der erste Entwurf des EC 4 wurde 1984, im Einklang mit den bereits vorhandenen Entwürfen EC 2 und EC 3, aufgesetzt. Er enthält eine Darstellung der Anwendungsbereiche, der Hauptinhalte und der Hauptneuerungen und zeigt die Zusammenhänge zu EC 2 und EC 3 und zu den zwischen 1972 und 1981 erarbeiteten Modellrichtlinien für Stahlverbundkonstruktionen.

1. INTRODUCTION

1.1 Background

The first draft of Eurocode 4 was completed in English in October 1984, as a 150-page document of 12 chapters. It will be published by the Commission of the European Communities (CEC) within the next few months in English, French, German, and Dutch. It is less self-contained than other Eurocodes, as it had to be consistent with the August 1983 draft of Eurocode 2 for concrete structures (EC2), and the July 1983 draft of Eurocode 3 for steel structures (EC3), and it can be used only in conjunction with them.

The work on EC4 exposed a few difficulties in using EC2 and EC3, which will be considered by their drafting panels at the same time as the comments on them now being prepared in the ten Member States.

Eurocode 4 is based on the Recommendations for Composite Structures of 1981 [1], referred to here as the Model Code; but there are substantial differences of scope and presentation, because much of the Model Code was written in the mid-1970's before EC2 and EC3. Three of the members of the drafting panel for EC4 worked on the Model Code, one of these also on EC3; and the fourth contributed to EC1 and EC2. The technical content of EC4 is thus firmly based on the expertise of the much larger groups that drafted these preceding documents.

Eurocode 4 is the first one to take full advantage of the work of the Coordinating Committee for Eurocodes, which was set up in 1982 with Mr. G. Breitschaft as Chairman. Working from draft Eurocode 1 ("Common unified rules for different types of construction and material"), this group produced in 1983 a model Preface and model Chapters 1 and 2, for use in Eurocodes 2 to 8. Appropriate versions of them were incorporated in EC2 and EC3 at a late stage of drafting. This established the mutual consistency that was essential for the preparation of EC4.

1.2 Scope of EC4

The Code is concerned with composite floor slabs, beams and columns, and with framed structures that have composite beams or columns or both. The scope is thus wider than that of the Model Code, in respect of frames. Profiled steel sheet is not included in EC3, so reference had to be made also to ECCS recommendations. Propped and unpropped construction and lightweight concrete are included.

The scope does not include encased composite beams, piles for foundations, cable-stayed or box-girder bridges, or composite plates. The range of plate-girder bridge decks for which EC4 can be used is likely to be limited by omissions from EC3 (e.g., longitudinally stiffened members), rather than from EC4.

2. THE PREFACE

This provides background information on the harmonisation process, and the presentation, calibration, and future use of EC4.

2.1 Harmonisation

As load specifications are not harmonised between Member States, no partial safety factors (γ_F) nor coefficients for representative values (ψ) are given in EC4. Each State will decide its own. The values that were envisaged during drafting are given in the Preface, for discussion only.

Harmonisation of materials specifications is more advanced, so partial safety factors for materials (γ_M) are given as a basis for trial designs. They are enclosed in boxes to indicate that that they are not an official proposal from CEC. For consistency, all the values in EC4, except those for shear connectors, are copied from EC2 or EC3, even though it is doubtful whether the values of γ_M at the ultimate limit state for structural steel (1.0) and reinforcing steel (1.15) are in harmony.

The uncertainties that are represented in Eurocode 1 by γ_d, and in the British Bridge Code (BS 5400) by γ_{f_3}, are split into two parts, represented by γ_{Rd} and γ_{Sd}. These are merged with the partial coefficients γ_m and γ_f as follows:

$$\gamma_M = \gamma_m \gamma_{Rd} \quad , \quad \gamma_F = \gamma_f \gamma_{Sd} \ ,$$

to give γ_M and γ_F, which are the values given in the Preface or (in boxes) in the clauses.

2.2 Presentation

This was based on that used in EC3, except that Ultimate Limit State precedes Serviceability Limit State, as it does in EC2.

Principles are distinguished from Rules for Application by a vertical line in the margin, as in EC3, as this was thought to be simpler than the separate clause numbers used in EC2.

The division of the text into Principles and Rules gave particular difficulty, which has led to a review by the Steering Committee for Eurocodes of the definitions of Principles and of Rules given in Model Chapter 1.

Careful thought was given to the extent to which material relevant to EC4 but given in EC2 or EC3 (or, for much of the Preface and Chapter 2, in both) should appear again. It was decided to repeat data that were concise, important, and frequently used (e.g., the list of the Grades of concrete and associated values of modulus of elasticity); but otherwise to give cross references to clauses in EC2 or EC3, sometimes supplemented by a summary of their content.

3. CHAPTERS 1 TO 3 (INTRODUCTION, BASIS FOR DESIGN, AND MATERIALS)

3.1 Symbols

Reference is made mainly to ISO document IS 3898 [2]. Minor deviations are usually for consistency with EC2 or EC3 or with established practice for profiled steel sheeting. Innovation was necessary for the subscripts meaning "steel". In EC4, "a" is used for structural steel, "s" for reinforcing steel, and "ps" for prestressing steel; except that the yield strength of structural steel is f_y, not f_a.

The fact that symbols gave little difficulty is encouraging evidence of the recent progress towards a European consensus on notation.

3.2 Terminology

The writer had difficulty in breaking the habits of half a lifetime in learning to use "strength" only for materials, and "resistance" for cross sections or members; and in adopting the comprehensive word "action", with associated "effects". Having done so, he urges fellow English-thinkers to persevere with "Eurospeak", not to reject it. We owe it to our colleagues from the other Member States, who so often have to use or translate from our language, to remove some of its ambiguities; and also to allot precise and unfamiliar meanings to certain words, to match existing words in other languages.

3.3 Classification of actions

Unlike EC2, a distinction is made in EC4 between prestressing by tendons and by imposed deformation of the structure: the latter is named "precamber". Definitions are also given in EC4 for the "primary" and the "secondary" effects of shrinkage of concrete, prestressing, and non-uniform temperature.

3.4 Properties of concrete

For analysis of composite structures, it is often appropriate to use simpler (i.e., less accurate) values than are given in EC2. Two approximate values for the long-term free shrinkage of normal-density concrete are given in EC4; and three different levels of approximation for modular ratios - the simplest being to use $n = 15$ for all purposes.

As in EC2, properties of concrete are related to the 28-day cylinder strength, f_c. When considering data related to cube strengths, f_{cu}, it was assumed that for normal-density concrete, $f_c = 0.80\, f_{cu}$. The ratio can in fact be as low as 0.67 or as high as 0.85. Values related to compressive strength are given in Ref. [3].

4. ULTIMATE LIMIT STATE

4.1 Beams and frames

A definition is given of *critical cross sections*, at which design action effects in bending and shear should be compared with resistances. Design longitudinal shear between adjacent pairs of such sections has to be compared with shear resistance.

No attempt is made to define procedures appropriate for a particular type of structure, such as "buildings" or "bridges". Design methods are related to the slenderness of the steel flanges and webs at the sections considered. As in EC3, there are four classes: 1, plastic; 2, compact; 3, semi-compact; and 4, slender.

The limiting slendernesses for class 3 are as in EC3, but those for classes 1 and 2 are lower by between 10% and 25%. This is to take account of two differences from all-steel members:

- the limit to plastic curvature set by crushing of concrete, and
- the greater redistribution of moments from internal supports to midspan that often must occur before the sagging moment of resistance is developed.

For the analysis of structural systems, two methods are given in detail: elastic analysis, for all classes, and plastic hinge analysis, for members or frames where all the critical sections are in class 1. Elastic-plastic analysis is also allowed, but no details are given.

Frames with some composite members are classified according to the type of the joints between beams and columns. Methods of analysis are given, using references to EC2 or EC3 as appropriate, for frames where the joints are "simple" or "rigid", as in steel construction, or "monolithic", as in concrete construction. Semi-rigid joints are not excluded, but no methods of analysis are given.

The primary effects of shrinkage and of non-uniform temperature may be neglected, and the secondary effects have to be considered only in exposed structures, such as bridges, with cross sections in class 3 or class 4.

Deck or floor slabs have to be designed for local loads to EC2, but these loads can be ignored in the design of composite members.

In the analysis of structures, and of sections in classes 1 and 2, shear lag is allowed for by a simple but conservative rule for effective breadth of a flange. A more accurate but complex method for calculating effective breadth is provided for analysis of cross sections in classes 3 and 4.

4.1.1 Resistance of cross sections

Two idealisations are given in EC2 for the stress-strain curve for concrete in compression. They both lead to complex calculations when used for the bending resistance of a composite beam, in which the neutral axis is often within or close to the steel top flange. In EC4 a third, less conservative, method is given. This is to use a rectangular stress block at $0.85\ f_c/\gamma_M$ extending to the neutral axis, with a recommended value of 1.5 for γ_M.

Parametric studies have shown that for composite beams, the difference from the methods of EC2 is negligible; but for composite columns the bending resistance of a cross section is over-estimated. A correction factor of 0.90 is given in EC4 in the design method for columns that is based on Ref [4], to allow for this and other approximations.

In EC4, the preceding method is used for the calculation of the bending resistance M_{pc} of beam cross sections in class 1 or 2. The resistance to combined bending moment M and vertical shear V is as shown in Fig. 1, in which $R_f d_f$ is the resistance of the member when the steel web is excluded, and $V_{p\ell}$ is the resistance to shear found in accordance with EC3. For cross sections in classes 3 and 4, elastic analysis is used.

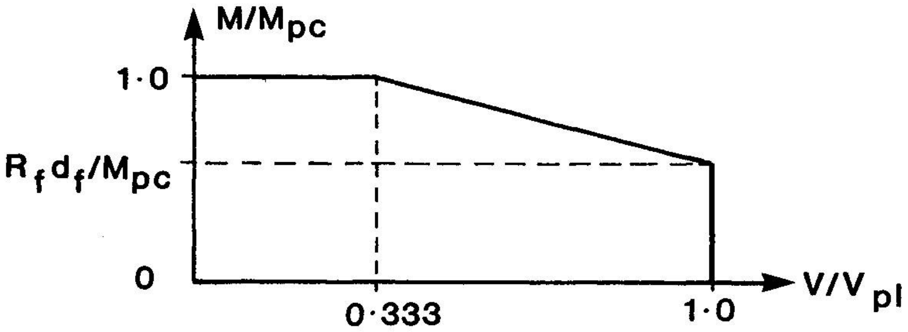

Fig. 1. Resistance of cross section in combined bending and shear

4.2 Composite columns

The cross sections considered in detail are fully or partly encased I or H sections, and concrete-filled tubes of rectangular or circular section. General recommendations are given for design of column lengths that are either in frames or loaded through bearings.

Two simplified design methods are given for column lengths of uniform doubly symmetric cross section, including those in unbraced frames or with biaxial bending, subjected to known axial load and end moments. Effective lengths are found by the methods of EC3.

The first method, developed at the University of Bochum [4], makes use of the European column curves, as given in EC3, and of an ingenious simple method of calculating for a cross section the interaction curve between axial load and bending moment, given in an Appendix to EC4.

The second method, developed at the University of Warwick [5], is quicker but of reduced scope, in that transverse loads applied within the column length and some other situations are excluded. For given slendernesses and bending moments, the axial resistance is calculated directly from simple formulae, that were deduced from parametric studies and checked against test data.

4.2.1 Effective flexural stiffness of cross sections

For analysis of frames, the uncracked flexural stiffness is used, with allowance for creep by means of an effective modulus, as for beams. For the design of a column length, the de-stabilising effects of creep are allowed for by a different effective modulus. It is taken as

$$E_{ce} = 600\, f_{ck}\,(1 - N_{Gd}/2N_d)$$

for slender columns loaded at low eccentricity, and otherwise as $600f_{ck}$, where

f_{ck} is the characteristic cylinder strength of the concrete,

N_d is the design axial load, and

N_{Gd} is the part of this load that is permanent.

4.2.2 Longitudinal shear in columns

A method is given for the design of shear connection in columns. None is needed if the mean shear stress at the steel-concrete interface is less than 0.6 N/mm² for encased columns and 0.4 N/mm² for filled tubes.

5. SERVICEABILITY LIMIT STATES

5.1 Analysis of the structure, and stresses

The number of structures for which re-analysis is needed for the serviceabiliy limit state has been reduced to a minimum. When it is necessary, all cross sections in class 1 or 2 are deemed to be in class 3, and the methods of analysis, both for the structure and for sections, are essentially as for the ultimate limit state.

No limiting stresses are specified, but approximate and conservative methods are given, for allowing for the influence of any local yielding of steel at internal supports of continuous members on deflections and the widths of cracks.

5.2 Limit states

The limits for deflections in buildings are as in EC3. For bridges, guidance is given on when deflections should be calculated, but no limits are given.

For cracking of concrete, the intention is to use the limits and methods given in EC2; but some revision and extension of these is necessary.

Guidance is given on the avoidance of excessive vertical and horizontal vibration in footbridges. For vibration of floors in buildings, reference is made to EC3.

6. CHAPTERS 6 TO 12

6.1 Chapter 6, Shear connection in beams

Provision is made for the use of studs with or without heads; bar, tee, channel and horseshoe connectors; anchors made from round bars; and friction-grip bolts; and also for the use of headed studs with profiled steel sheeting. Design is for the ultimate limit state. Between critical sections in classes 1 or 2, connectors may be uniformly spaced, and partial shear connection down to 50% is allowed in simple and in continuous beams. Spacing related to the design shear flow may be used when the critical sections are in any class.

The resistance in shear of a headed stud is influenced by the size of the weld collar [6]; but the existing design resistances, given in the Model Code and essentially reproduced in EC4, are based mainly on test data where no details of the collars were reported. Current practice of not requiring studs to have collars has been followed in EC4; but alternative higher design resistances have been given for studs that have collars of at least a specified size, for use when other conditions relating to the strength of the concrete and the stud material are satisfied.

The rules for transverse reinforcement near shear connectors have been aligned with those in EC2, with the result that one of the two design equations is more conservative than in the Model Code, even though the latter is supported by the evidence from tests on composite members.

6.2 Chapter 7, Composite floors with profiled steel sheet

This chapter covers simply-supported and continuous floors spanning parallel to the ribs of the sheeting, with predominantly static loading, slabs at least 90 mm thick, and sheeting of depth not exceeding 80 mm. The design methods are similar to those of BS 5950: Part 4 [7], which were based on the well-known work of ECCS Committee 11, which in turn used work from Professor Badoux' group at Lausanne and from Lehigh University.

The strength of the composite floor slab in longitudinal shear is given by a semi-empirical equation including factors m and k which have to be obtained from tests (which are specified in Chapter 12). Details are given of design for vertical and punching shear, and of serviceability checks.

6.3 Chapters 8 to 12

Chapter 8 covers reinforced or precast slabs used either as permanent formwork, or as complete floors spanning between steel beams and composite with them. It is generally as in EC2 and the Model Code.

Chapter 9, Fatigue, refers to EC3, which gives data on fatigue resistance of shear connectors.

Chapter 10, Prestressing and precambering of beams, is based on the Model Code. It gives details of checks at the serviceability limit state.

Chapter 11, Workmanship and construction, is also based on the Model Code, but with new material on profiled steel sheeting and on through-sheet welding of studs.
Chapter 12, Testing, includes the push test for studs as given in the Model Code and in a British Code (CP117) of 1965. The slabs are small and are reinforced with mild steel bars that are not fully anchored. Subsequent research has shown [6] that although this test may be expected to give relevant results for small connectors set in strong concrete, it is over-conservative for other situations, in which splitting would occur in the narrow test slabs, but not in an unhaunched T-beam. There is need for a better test to be standardised; but there are so many relevant variables that its calibration would be a costly process.

The probability-based methods for deducing design resistance from the results of tests are consistent with those in EC3, but differ from the existing practices for push tests and for shear tests on composite slabs.

7. CLOSURE

It is hoped that the preceding outline of the scope and principal features of Eurocode 4 will be of interest, at least to those who use it for trial designs and calibration studies, and make comments on it.

As noted earlier the main division in EC4 is between design methods applicable only to members with critical sections in class 1 or class 2, and more restrictive methods of general applicability. The limiting slendernesses given for class 2 are thus the most significant numbers in the code. They are based on rather limited evidence.

There is need for research on these limits, for if they could be relaxed, the cost of composite structures for buildings could in many instances be reduced.

7.1 Acknowledgement

The writer is deeply grateful to the other members of the Drafting Panel for EC4, Mr. H. Mathieu, Professor K. Roik, and Mr. J. Stark, for their substantial contributions to the work.

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