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General Methods of Design of Composite Construction

Principes généraux pour le projet en construction mixte

Grundlagen zur Projektierung in Verbundbauweise

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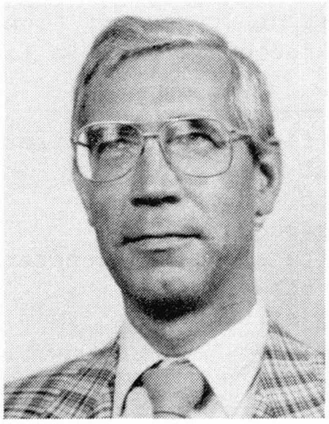

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SUMMARY

The European Community has initiated the preparation of a set of harmonised structural codes. Eurocode 4 is the code for the design of composite steel and concrete structures. The first draft of Part 1 for buildings was published in 1985 for public comment. Following this consultation phase, a revision was started. The revised document will be published by the European Committee for Standardisation as a European Prestandard (ENV). In this lecture an outline is given of the scope, the background and some principal features of the revision of Eurocode 4.

RÉSUMÉ

La Communauté européenne a lancé la préparation d'une série de règlements structuraux devant être harmonisés entre eux. Eurocode 4 représente les règles pour le dimensionnement des systèmes porteurs mixtes en acier et béton. Le premier avant-projet du fascicule 1 relatif aux bâtiments a été publié en 1985 en vue d'être soumis aux commentaires publics. Une révision a fait suite à cette phase consultative. Le document remanié correspondant sera publié par le Comité européen de normalisation pour servir de Norme européenne préliminaire (ENV). La présente contribution traite des objectifs, des données de base et de quelques unes des caractéristiques essentielles de la révision d'Eurocode 4.

ZUSAMMENFASSUNG

Die Europäische Gemeinschaft hat die Aufstellung harmonisierter Konstruktionsnormen in die Wege geleitet. Eurocode 4 behandelt den Entwurf von Stahl-Beton-Verbundtragwerken. Der erste Entwurf von Teil 1 (Hochbauten) ging 1985 in die Vernehmlassung. Die daraufhin überarbeitete Fassung wird vom Europäischen Normungsausschuss als Vornorm veröffentlicht werden. Der Aufsatz gibt einen Überblick hinsichtlich Umfang, Grundlagen und einiger Hauptmerkmale des revidierten Eurocode 4.

1. INTRODUCTION

1.1. Objectives of Eurocodes

The Commission of the European Communities (CEC) has initiated the preparation of a set of European Codes - the Eurocodes - for the design of buildings and civil engineering structures. These codes are intended to establish a set of common rules as an alternative to the differing rules in force in the various Member States.
The Commission's programme for aligning the regulations, laws and administrative provisions of the Member States concerning the safety, serviceability and durability of the different types of construction and materials provided initially for the eight Eurocodes listed in Fig. 1.

Eurocode 1 -	common unified rules for different types of construction and material
*Eurocode 2 -	for concrete structures
*Eurocode 3 -	for steel structures
*Eurocode 4 -	for composite steel and concrete structures
Eurocode 5 -	for timber structures
Eurocode 6 -	for masonry structures
Eurocode 7 -	for foundations
Eurocode 8 -	for structures in seismic zones

* These are relevant for composite structures.

Figure 1: List of the current draft Eurocodes

According to present considerations the Eurocodes are intended to serve as reference documents to be officially recognized by the authorities of the Member States for the following purpose:
- as a means to prove compliance of building and civil engineering works with the essential requirements as laid down in the Construction Product Directive,
- as a basis for specifying contracts for the execution of private and public construction works and related engineering services, and
- as a framework for drawing up European standards for construction products and special construction procedures (and guidelines for technical approval).

The benefits for the Construction Industry expected from the introduction of Eurocodes are:

(1) The Eurocodes will permit an engineer designing in one country to have documents which he can use on a familiar basis to design a structure in another. This will enable the design between engineers of all European countries to be harmonised, with potential benefits to all. It must be remembered that a beam will not know which country or rules were used in design and will carry the same load regardless!
(2) At the present time Codes are by their very nature becoming more complex; by introducing Codes on an international basis, facilities are present to enable more effort to be applied in generating the simplest form of the rules. This will, it is hoped, reduce the chances of misinterpretation and lead to faster design times.

(3) The cost of research is such that very few countries could carry the whole burden single handed; even if this were possible, the benefits of combining the work of many institutions on an international basis must be evident. The increase in the number of test results available for calibration means that many of the areas missed in one country can now be covered by the breadth of the test work.

1.2. Organisation

The organisation set up by the commission for the preparation of Eurocodes is given in Fig. 2. The actual drafting is contracted to a team of experts (drafting panel - DP).

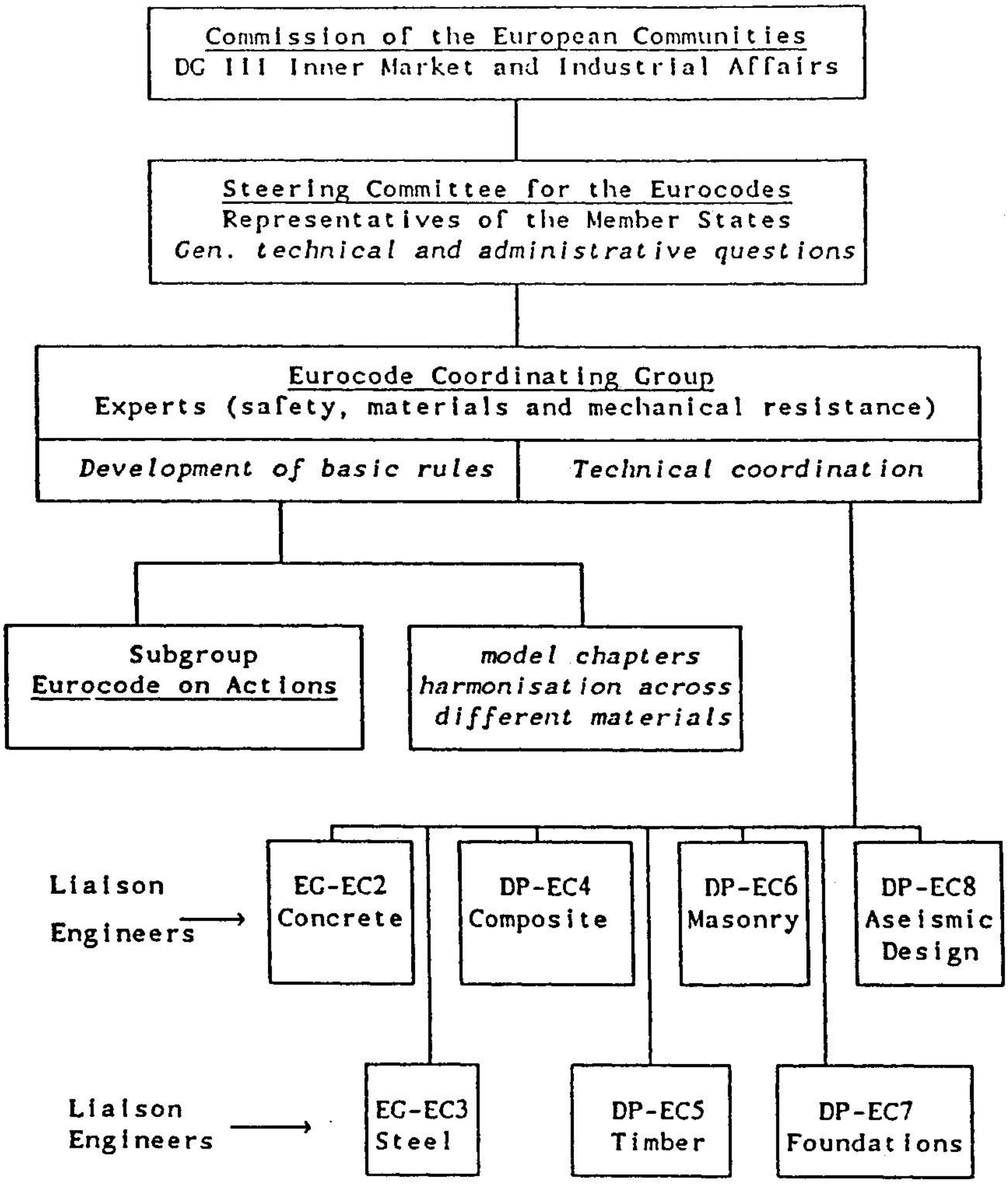


Figure 2: Preparation of the Structural Eurocodes

The interests of each country are being represented by liaison engineers appointed by each Government to work with the drafting panel for each Eurocode. In most countries the liaison engineer is supported by a national committee entrusted to provide technical support and to ensure that the final document is "user friendly". They are ideally composed of working engineers from many sections of industry, with academic specialists providing the necessary backup on formulae, etc.
The resulting comments and suggestions are assembled by the liaison engineers, passed to the drafting panel and discussed in meetings of the drafting panel with the liaison engineers. The Eurocodes produced to date do not have a clear legal status. Therefore the Commission has decided that in 1990 the future work on Eurocodes will be transferred to CEN. (European Committee for Standardisation). The members of CEN are the national standardisation organisations of the EEC Member States and the EFTA countries. The European Committee for Iron and Steel Standardisation, ECISS, which is associated with CEN, is already working in the field of steel products.

CEN prepares European Standards (ENs), which generally have to be introduced as national standards without modifications within six months of acceptance by members, and conflicting national standards have to be withdrawn.

CEN may also prepare European Prestandards (ENVs) which are intended to be used in a preliminary phase for experimental application. The method in which these are applied in Member States will depend both on conditions laid down by the Commission and on the regulating systems in place nationally.

ENVs are prepared by CEN's Technical Committees. The time allowed for national commenting on drafts is normally six months, except where ISO-standards are transferred to a CEN-standard, when three months is allowed.

The CEN-status binds Member States to a standstill on national standards work in the fields where mandates for ENs are given. To date CEN has been engaged in producing *product standards* for the construction and building industries and has not yet been involved in producing *structural design codes*.

Due to the special character of the Eurocodes the Commission has requested CEN to set up a Technical Committee "Structural Eurocodes" which within CEN will be solely responsible for all structural design codes. (Fig. 3).

According to an agreement between the Commission and CEN, the Eurocodes finalised so far will be issued as European Prestandards, ENVs, without technical modifications and these will then be transformed into European Standards, ENs, taking account of the experience gained during their experimental use and of the comments received.

The experts hitherto involved in the drafting and editing work will continue their work in Project Teams associated with the Subcommittees.

The work of TC "Structural Eurocodes" is covered by mandates given by the Commission to CEN, which include, inter alia, particular conditions, e.g. the cooperation between CEN and the Commission, the responsibilities of the regulatory authorities, the compatibility between the principles laid down in the Eurocodes and the work of the technical committees preparing related product standards.

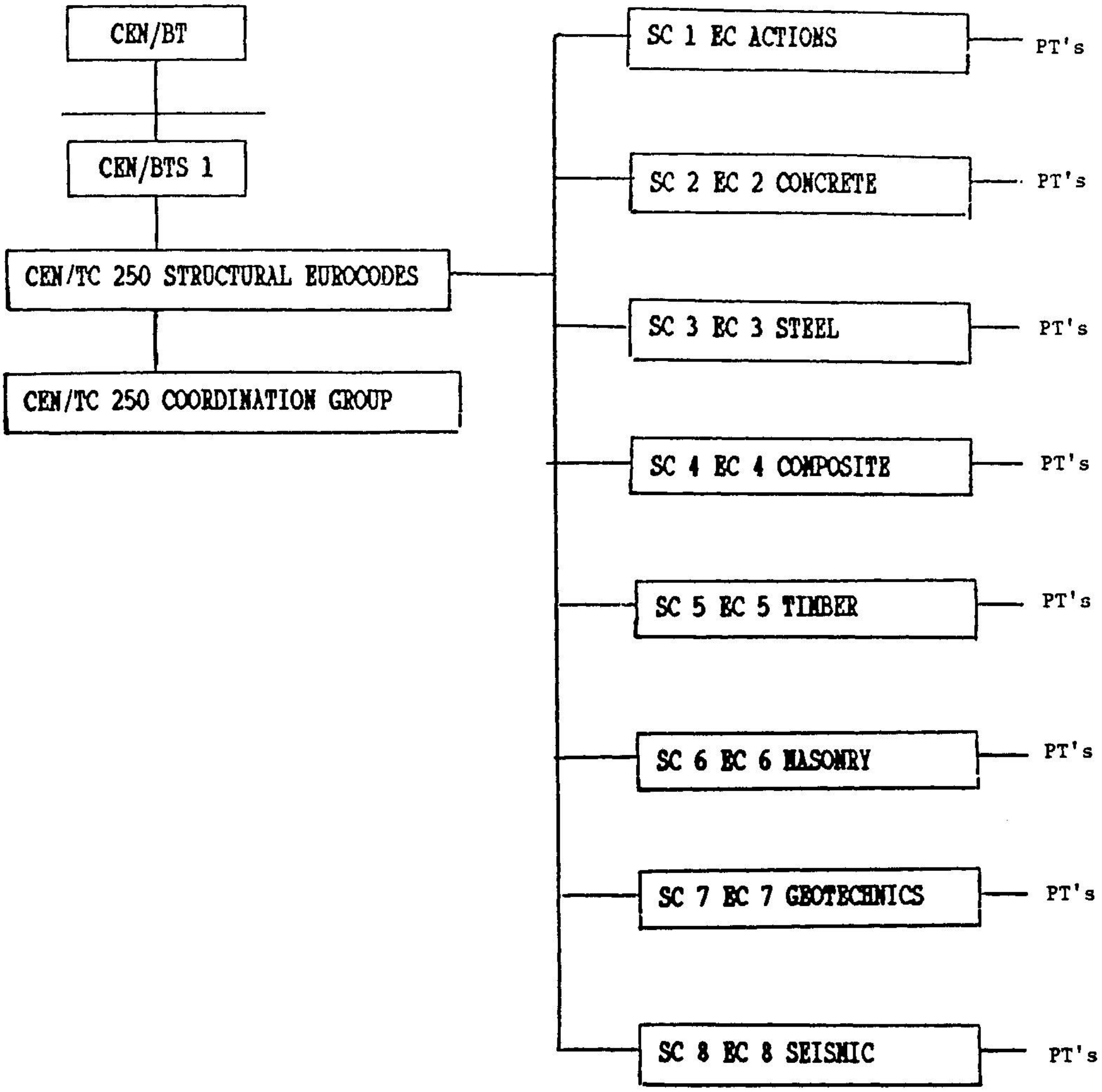


Figure 3: Proposed Structure for the TC "Structural Eurocodes

<u>1.3 Relation of Eurocode 4 to other Eurocodes</u>

A summary of the Eurocodes currently in preparation is given in Fig. 4.
Eurocode 4 is the code for the design of composite steel and concrete structures. It refers to Eurocode 2 and Eurocode 3 for specific aspects of the concrete and steel parts of a composite structure and it is consistent with Eurocode 2 and Eurocode 3. So Eurocode 4 shall in all cases be used in conjunction with Eurocodes 2 and 3. Design of composite structures under seismic loading is covered in Eurocode 8. The loads applied to the structure and the imposed deformations (direct and indirect actions) are given in a separate Eurocode for actions.
Eurocode 1 is not intended as an operational document. It provides the general philosophy and fundamental considerations from which unique solutions have been developed for practical use in the material dependent Eurocodes and will be used as a base document by those preparing future draft Eurocodes.

EUROCODE 1: Unified rules for different materials and ways of construction

EUROCODE ON ACTIONS

Chapter 1 General rules
2 Density of materials
3 Selfweight
.
.
6 Imposed loads on floors and roads
7 Snow and ice loads
8 Wind loads
.
.
12 Traffic loads on road bridges
13 Traffic loads on railway bridges
.
.
20 Fire action

EUROCODES - DESIGN RULES (resistance)

EC 2 Concrete structures
EC 3 Steel structures
EC 4 Composite structures

EC 5 Timber structures
EC 6 Masonry structures
EC 7 Foundatons

EUROCODE 8 : Design in seismic regions

Figure 4: Summary of Eurocodes currently available or in preparation

1.4 Present status

At this moment it is foreseen that Eurocode 4 will consist of three parts:

Part 1 - General Rules and Rules for Buildings
Part 2 - Bridges
Part 10 - Fire resistance

It is possible that these parts will be supplemented by further parts which will complement or adapt these parts for particular aspects of special types of buildings and for other civil engineering works (f.e. off-shore structures and maritime structures).
The first draft of Part 1 of Eurocode 4 was completed in English in October 1984, as a 150-page document of 12 chapters. It had to be consistent with the August 1983 draft of Eurocode 2 (concrete structures) and the July 1983 draft of Eurocode 3 (steel structures).
The drafting panel was a small group of 4 experts:

- Prof. R.P. Johnson, University of Warwick (Coventry)
- Ir. Gl.H. Mathieu, Association francaise du Béton (Paris)
- Prof. K. Roik, Ruhr-Universität (Bochum)
- Prof. J.W.B. Stark, IBBC-TNO (Delft) and Eindhoven University of Technology

and with Dr. D. Anderson (Coventry) acting as secretary.

This first draft of Part 1 of Eurocode 4 was prepared on the basis of a principal source document, the Recommendations for Composite Structures drafted by a Joint committee of CEB, ECCS, FIP and IABSE and published in 1981. Other sources were documents of CEB and ECCS and those national codes in which the limit state concept had already been applied. Eurocode 4 was first published by the Commission in 1985 for comment by Member States and interested international technical and scientific organisations.
Following this consultation phase a revision of the draft was started in cooperation with both the Eurocode Coordination Group which is responsible for the harmonised presentation and editing of those rules that are materially independent and the *liaison engineers* from the Member States through whom national comments are channelled and who are assisting in the processes of correcting, improving, compromising on and agreeing with draft clauses.
In view of the necessary consistency of Eurocode 4 with the revisions of Part 1 of Eurocodes 2 and 3 the progress was strongly dependent of the completion of Eurocodes 2 and 3. The final drafts of these documents became available recently. It is now foreseen that the final English draft of Eurocode 4: Part 1 is completed by June 1991, and background documents by the end of 1991.

The first complete draft of Part 10 of Eurocode 4 became available in April 1990 and was presented at a seminar on Structural Fire Design on June 26/27 in Luxembourg. This part of Eurocode 4 was prepared by the following experts:

- J.B. Schleich, ARBED-recherches (Luxembourg)
- J. Kruppa, CTICM (France)
- P. Schaumann, Consulting Engineer H.R.A. (Germany)
- L. Twilt, IBBC-TNO (Netherlands)

1.5 Further developments

Due to the transfer of work on Eurocodes to CEN the consultation procedure will change. Therefore its difficult to indicate the time scale of future developments.
The completion of Part 1 will set first priority. It is important that Eurocode 4: Part 1 will be accepted by CEN for publication as an ENV as soon as possible because many modern buildings in steel are designed as composite. For these type of buildings experimental application of ENV-EC3 is only possible if also ENV-EC4 is available.

Technical work on Eurocode 4: Part 2 for bridges could commence as soon as the Project Team has been set up, but no significant drafting should be done until first drafts of Parts 2 of Eurocode 2 and Eurocode 3 are available.
Assuming that drafting begins early in 1991, then December 1992 is the earliest possible completion date, assuming that the Code is short, is based on existing national codes, and consists mainly of Principles. If it is decided that the Code should be detailed and comprehensive, with Application Rules based on the latest research (e.g. like BS5400 and some DIN codes), drafting will take several years, due to the intervals for checking that national delegations will request, and the longer time taken to complete Parts 2 of Eurocode 2 and Eurocode 3.

2. TYPES OF COMPOSITE CONSTRUCTION

The use of composite construction in buildings has recently shown a considerable increase in various parts of Europe and especially in the UK. The merits of this construction type as speed of construction savings in weight and materials, quality, flexibility and fire resistance are now widely recognised by designers.
Traditionally, composite construction means utilizing the compressive resistance of concrete slabs in conjunction with steel beams, to increase the strength and stiffness of the beams.
More recently, profiled steel sheeting has been designed both to act as permanent formwork for the concrete slab, and to behave compositely with the slab for in-service loading. The composite slab can be connected to the steel beam by shear connectors to act as the upper flange of a composite beam. A variety of shear connectors may be used to develop the composite action of the beam and slab.
Another application of composite construction is in composite columns. This may be concrete encased steel columns or concrete filled hollow sections.
In this lecture a number of application will be shown. This presentation is restricted to building construction. For composite construction in bridges reference is made to the lectures of Dr. Lebet and Mr. Cremer.

3. SCOPE OF EUROCODE 4 - PART 1

Part 1 of Eurocode 4 gives a general basis for the design of composite structures and members for buildings and civil engineering works. In addition, Part 1 gives for composite slabs, beams, columns and frames detailed rules which are mainly applicable to ordinary buildings subjected to predominant static loading. This scope is similar to that in the 1985 Draft for National Comment, except that subjects mainly for bridges (fatigue, prestressing and precambering of beams) have been excluded. When fatigue and vibration have been completed for Part 2, an annex on these subjects for buildings with non predominant static loading may be appropriate for Part 1.
Propped and unpropped construction and lightweight concrete are included.
Particular aspects of totally or paretically encased beams are not covered and the scope does not include, piles for foundation or composite plates.
The revisions of Part 1 as it stands now is structured as illustrated in Fig. 5. This gives a global impression of the contents.
Resistance to fire is covered in a separate Part 10.

CONTENTS OF EUROCODE 4 - PART 1

MAIN DOCUMENT

CHAPTER 1. Introduction
(Scope, units, symbols)
2. Basis of design
(General rules concerning limit state design, actions, combination of actions, safety factors)
3. Materials
(Properties of concrete, reinforcing steel, structural steel, profiled steel sheeting, shear connectors)
4. Ultimate limit states
(Cross-sectional behaviour, moment distribution in beams and frames, design of beams, columns and connections)
5. Serviceability limit states
(Deflection of beams, cracking of concrete)
6. Shear connection in beams for buildings
(Full and partial shear connection, resistance of shear connectors, detailing, transverse reinforcement)
7. Composite slabs with profiled steel sheeting
8. Floors with precast concrete slabs
9. Execution
10. Design assisted by testing

ANNEXES

ANNEX A Reference standards
B Lateral torsional buckling
C Resistance of doubly symmetric composite cross-sections in combined compression and bending
D Composite columns with mono-symmetric cross-section
E Sway frames
F Partial connection theory for composite slabs

COMPLEMENTARY PARTS (Provisional Guides)

ANNEX G Checklist of the required information on the testing procedure
H Evaluation of test results on composite slabs
W Welding of stud shear connectors

Figure 5: Structure of the draft Eurocode 4

Depending on the character of the individual clauses, distinction is made between Principles and Application Rules.
The principles comprise:
- general statements and definitions for which there is no alternative, as well as
- requirements and analytical models for which no alternative is permitted unless specifically stated.

The Application Rules are generally recognised rules which follow the Principles and satisfy their requirements.
It is permissible to use alternative design rules different from the Application Rules given in the Eurocode, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the resistance, serviceability and durability achieved by the structure if designed using the present Eurocode.
Although this distinction is essentially sound and clear, it is not always easy to determine the category.

4. PRINCIPLES OF COMPOSITE ACTION

4.1 Introduction

The essence of composite action is that two (or more) members are connected at the interface between the two by some form of longitudinal shear connection in order to prevent slip. The members may be of the same material or of different materials. Due to the composite action the member will be stiffer and stronger then the sum of the individual members.

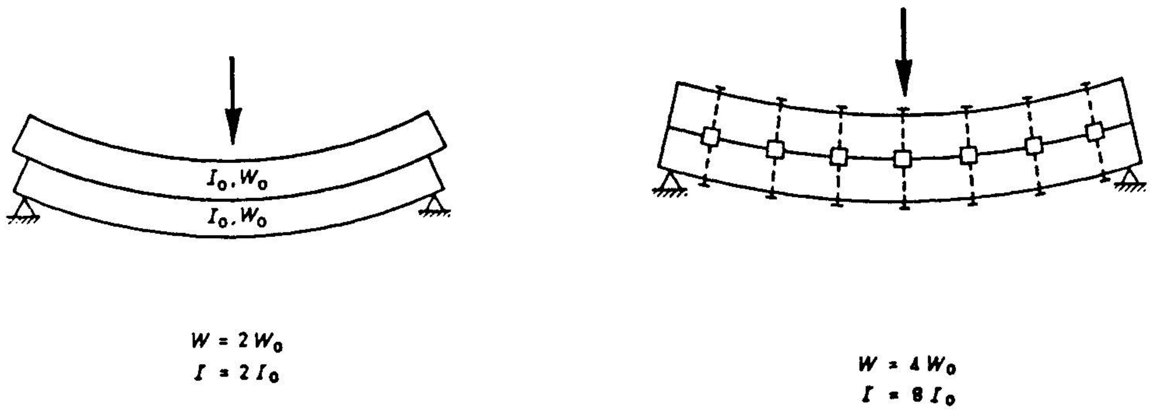
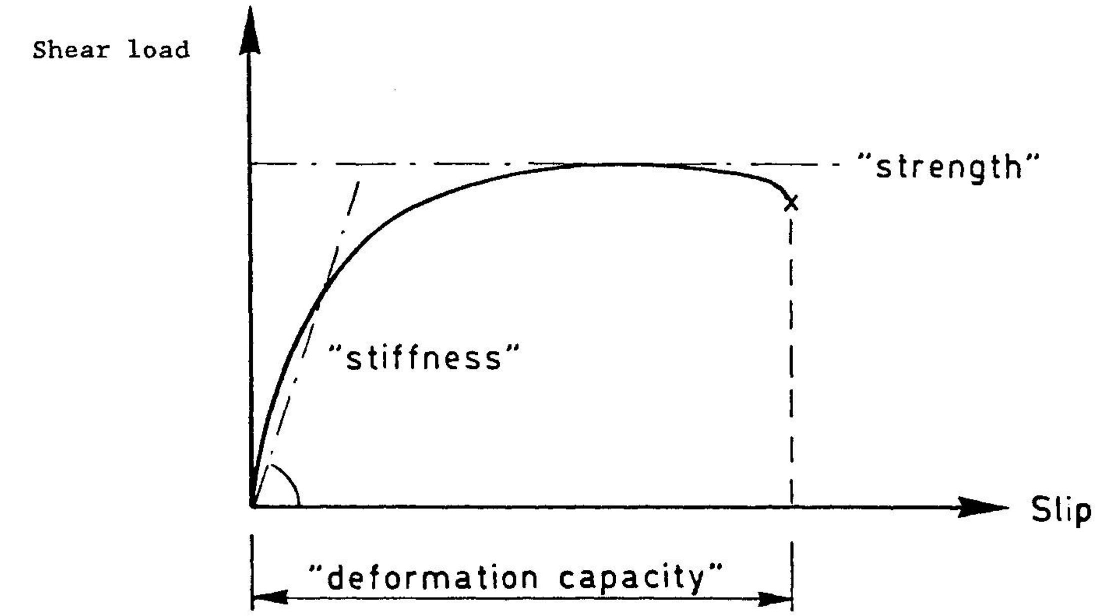


Figure 6: Illustration of composite action

In this short course the composite action between concrete and steel members is considered. In different national codes and recommendations for composite construction the nomenclature used for the properties of shear connections are not uniform. In Eurocode 4 the following definitions, given in Fig. 7, are used as related to the basic properties of a structural element being strength, stiffness and deformation capacity.



Strength	full shear connection partial shear connection
Deformation capacity	ductile connectors non-ductile connectors
Stiffness	complete interaction (rigid connectors) incomplete interaction (flexible connectors)

Figure 7: Definitions for properties of shear connection

In Eurocode 4 full shear connection and under certain restrictions also partial shear connection is allowed for beams and slabs. A full shear connection is formed when the shear connection is so strong that the ultimate load is determined by the maximum moments of resistance. The maximum load is reached when the optimum stress distribution occurs in the critical cross-sections (Fig. 8, cross-section II). The application of more shear connectors cannot result in a larger maximum load, as the maximum moments are normative.
However, when fewer shear connectors are used, this will result in a smaller ultimate load, dependent on the number of shear connectors applied (Fig. 8, cross-section III). The shear connection is then defined as a partial shear connection. The limit is reached when no shear connectors at all are used. In that case the contribution of the concrete flange can usually be neglected.
Then the ultimate load equals the ultimate load of the steel beam.

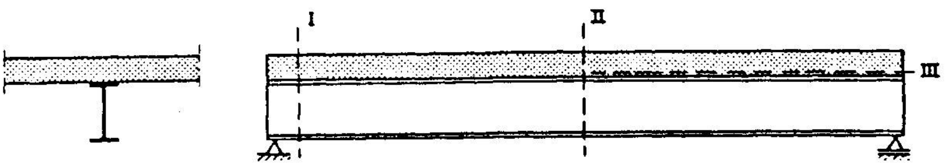


Figure 8: Cross-sections that may be critical for failure

Fig. 9 qualitatively shows the relation between the ultimate load and the strength of the shear connection, where 100 % corresponds with the shear resistance in case of a full shear connection.

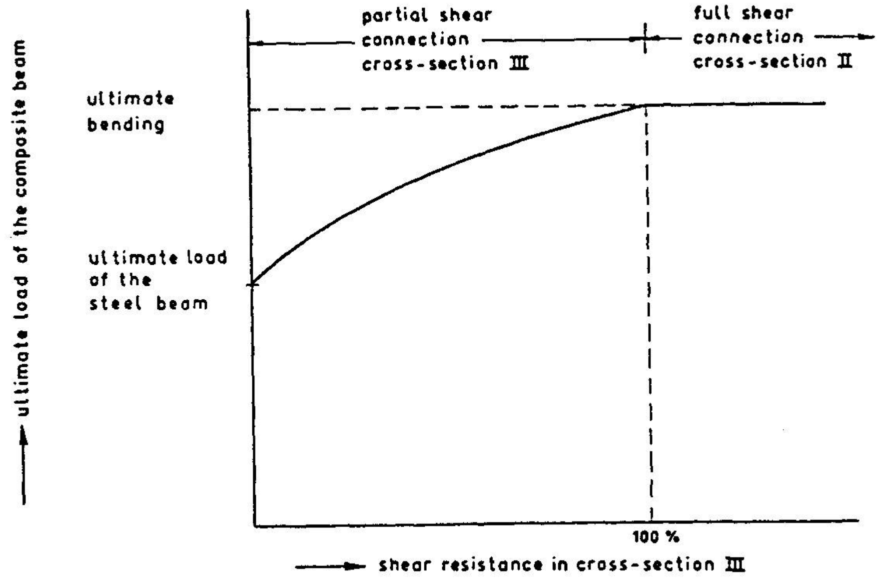


Figure 9: Qualitative relation between the ultimate load and the longitudinal shear resistance

The concepts full and partial *shear connection* are related to the *strength* of the longitudinal shear connection. The concepts complete and partial *interaction* only relate to the *stiffness* of the connection between the concrete slab and the steel beam.
When slip between the steel and the concrete is completely prevented by the connection, the interaction is said to be complete. However, most shear connectors have to undergo some deformation before they can supply any force. In that case the interaction is essentially partial. This difference is illustrated in Fig. 10. Fig. 11 shows the influence of the longitudinal slip on the form of a M-κ diagram.

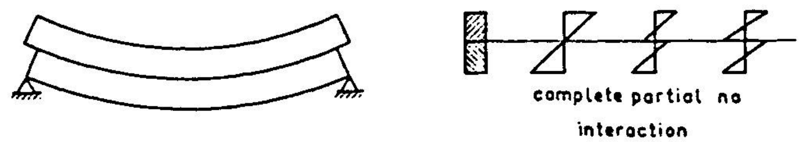


Figure 10: Two beams showing different degrees of interaction

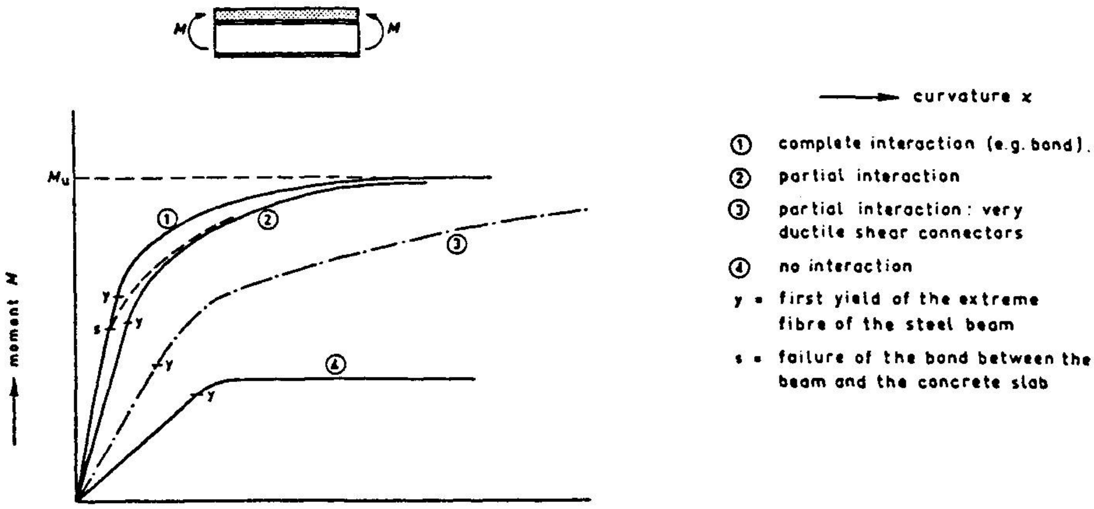


Figure 11: Influence of slip on a M-κ diagram

4.2 Motivation for the application of partial shear connection

The application of partial shear connection is of interest in those structures in which cooperation between the steel section and the concrete slab need not be fully exploited to get sufficient resistance. This may occur in the following cases.

a. When the concrete slab is not propped, the dimensions of the steel beam may be determined by the load during laying of the concrete. In that case it is not economical to determine the number of stud connectors from the full plastic moment of resistance of the composite cross-section, because the composite beams will then be too strong for the load applied after the concrete has hardened.
b. According to the deflection limitations the stiffness of the composite beam can be critical for the dimensions of the beam.
c. For economical and technical reasons a designer may choose a larger steel section with fewer shear connectors instead of a minimum steel section with a relatively large number of shear connectors.

A partial shear connection is an important option for the economic use of composite beams in buildings. It enables the designer to use a smaller number of shear connectors in cases where it is not feasible or necessary to provide as many shear connectors as required for a full shear connection. This gives the designer the possibility to choose between a light steel beam with relatively more shear connectors and a heavier steel beam with fewer shear connectors. Apart from this economical comparison, a partial shear connection will always be advantageous when an oversize steel beam must be selected from the available rolled beam size, or when deflection controls and strength requirements are met by less than full composite action. A partial shear connection may even be a "must" when a profiled steel sheet is used as formwork for the concrete slab. The size and spacing of the ribs can dictate the maximum number of connectors that can be placed.

4.3 Qualitative description of the behaviour of beams with partial shear connection

Beams with partial shear connection will fail as a result of failure of the shear connection. The moments of resistance of the critical cross-sections have to be determined in order to determine the ultimate load.
The ultimate load depends on:

a. the number of shear connectors, which determines the resistance of the shear connection;
b. the type of shear connectors, which determines the deformation characteristics of the shear connection.

Fig. 12 shows the difference in behaviour of ductile shear connectors and absolutely rigid non-ductile shear connectors. With regard to the stress distribution at failure, the compressive force in the concrete and the resulting tensile force in the steel respectively, have to be equal to the total shear force that can be transferred by the shear connectors longitudinally (= shear resistance). In principle various stress distributions may occur.
When ductile shear connectors are applied, slip may occur at the interface between the steel beam and the concrete slab. Once the ultimate load of a shear connector is reached, the load remains constant with further slip. Then stress distributions occur in which the neutral axes in the concrete slab and the steel beam no longer coincide. On the basis of Kist's hypothesis (2nd law of Prager), a stress distribution will occur at failure that leads to the maximum moment of resistance corresponding to the applied number of shear connectors (provided the deformation capacity is sufficient). This moment of resistance can be determined on the basis of equilibrium.
When absolutely rigid non-ductile shear connectors are applied, fracture will occur when the ultimate load of a shear connector is reached without any slip

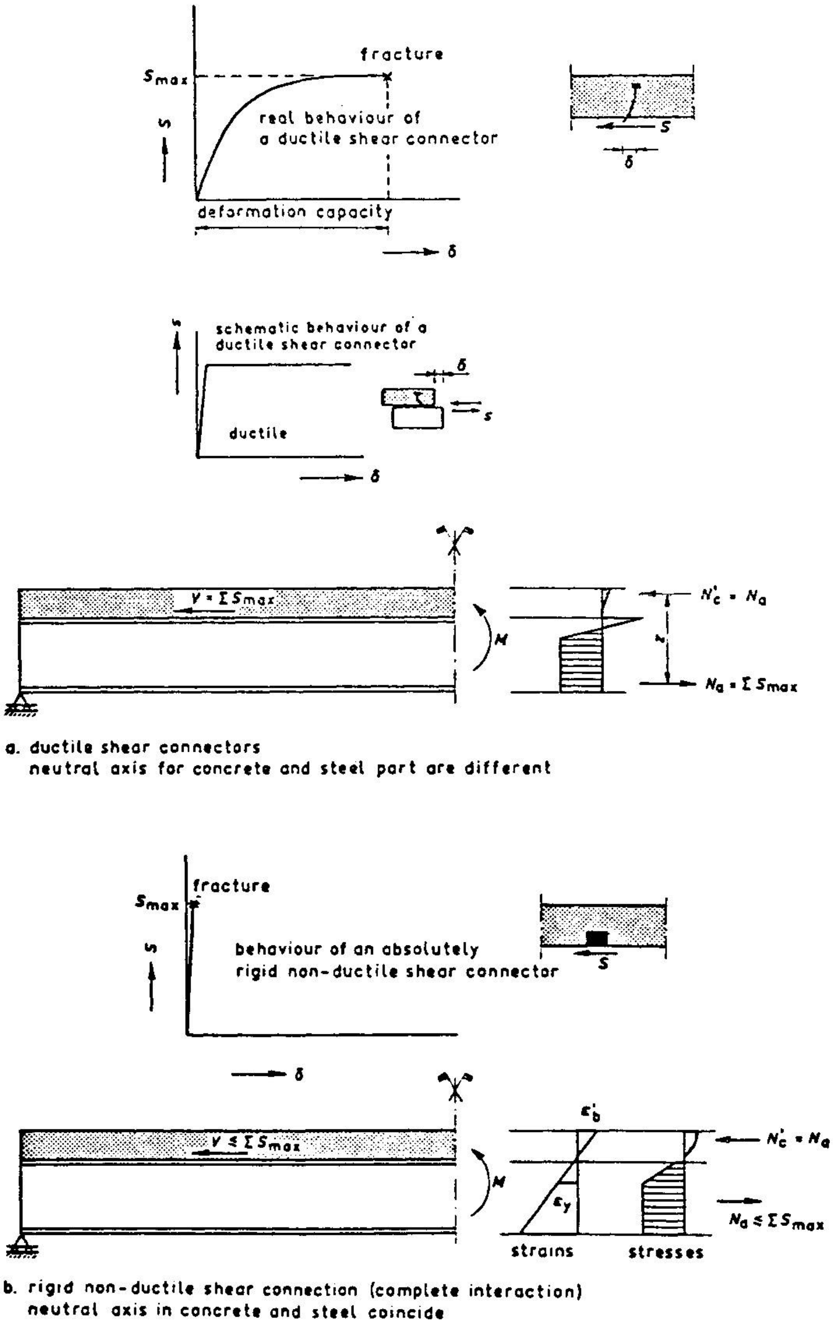


Figure 12: The influence of the deformation capacity of shear connectors on the strain and the stress distribution in case of a partial shear connection

and subsequently the shear resistance will suddenly drop to zero. So in theory no slip is possible before failure, which means that the strain distribution and consequently the stress distribution are fixed. The neutral axes in the steel beam and the concrete slab coincide. As soon as, under an increasing load, the longitudinal shear force on the heaviest loaded shear connector becomes equal to its shear resistance, the ultimate load is reached. Unless the distribution of the shear connectors coincides with the distribution of the longitudinal shear force, the total longitudinal shear resistance at failure is not equal to the sum of the shear resistances of the shear connectors. In reality the so-called rigid shear connectors that are applied in practice (e.g. block connectors), do

have some deformation capacity, so that some redistribution of the forces on the shear connectors may occur.
Fig. 13 qualitatively shows the relation between the moment of resistance and the resulting tensile force in the steel beam at failure. So, these lines indicate the moments of resistance for various beams, each with a different number of shear connectors. The tensile force N_{au}, which will occur at failure, is determined by the number of shear connectors. For some specific cases the stress distribution over the cross-section at failure is shown in Fig. 13. In case of rigid non-ductile shear connectors, failure of the shear connection has been taken as a criterion. When very few shear connectors are applied the moment of resistance will be smaller than the plastic moment M_{pl} of the steel beam. After failure of the shear connection, large deformations occur due to a sudden drop in stiffness. However, after this sudden increase of deformation, it will theoretically be possible to raise the bending moment to M_{pl} (dashed line in Fig. 13). This region has no practical use, because application of a composite structure in that region does not offer any advantages.

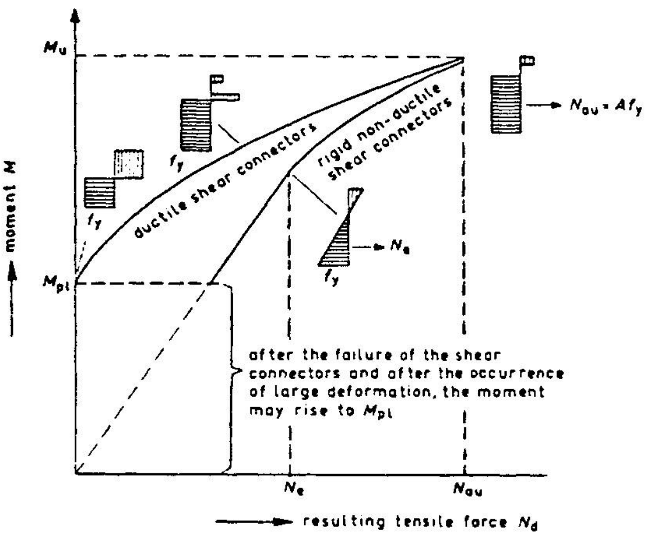
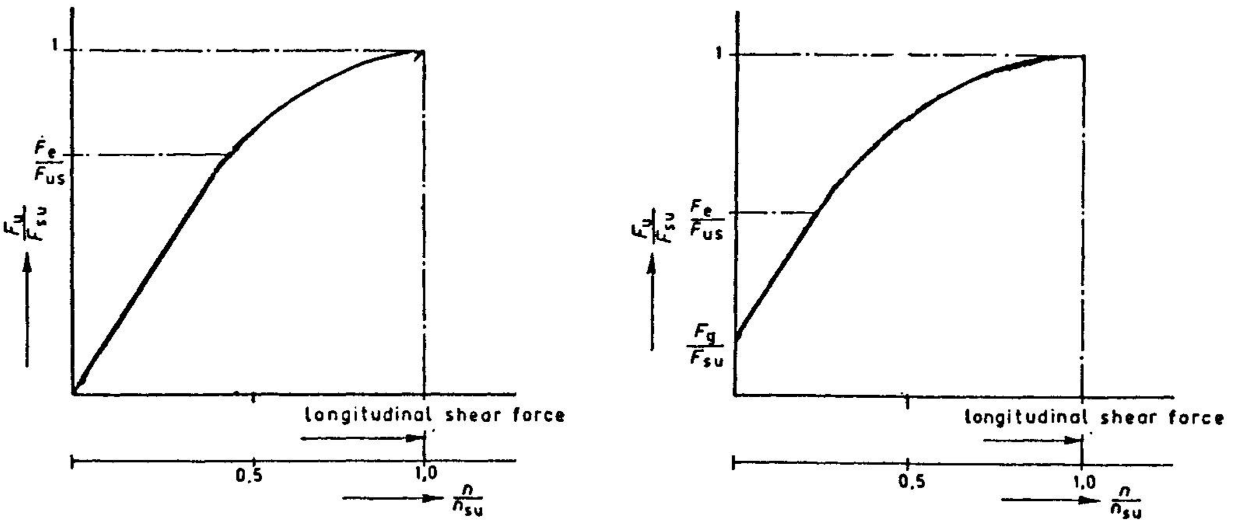


Figure 13: Qualitative relation between the moment of resistance and the tensile force in the steel beam (= longitudinal shear force) at failure

The relation shown in Fig. 13 for rigid non-ductile shear connectors, is influenced by preloading of the steel beam and by internal stresses, for example caused by shrinkage and creep of the concrete. This means that in case of a *partial* shear connection and application of very *rigid* non-ductile shear connectors, the ultimate load is indeed influenced by these factors. This contrary to beams with full shear connection and beams with partial shear connection with ductile shear connectors. This is caused by the fact that the ultimate load is determined by a part of the structure that does not meet with the requirements of ideal plastic behaviour of the materials (the rigid connection with limited deformation capacity).
As an example the effect of preloading of the steel beam is illustrated in Fig. 14. The design curves for rigid non-ductile shear connectors will be different for propped and unpropped construction, which is not the case when ductile shear connectors are used.



a. propped b. unpropped

Figure 14: Relation between the load and the longitudinal shear force i.e. the degree of shear connection, when rigid non-ductile shear connectors are applied.

4.4 Design rules

4.1.1 Beams with ductile shear connectors

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. (Fig. 15).
Based on these assumptions the stress distribution at failure will be as shown in Fig. 15.

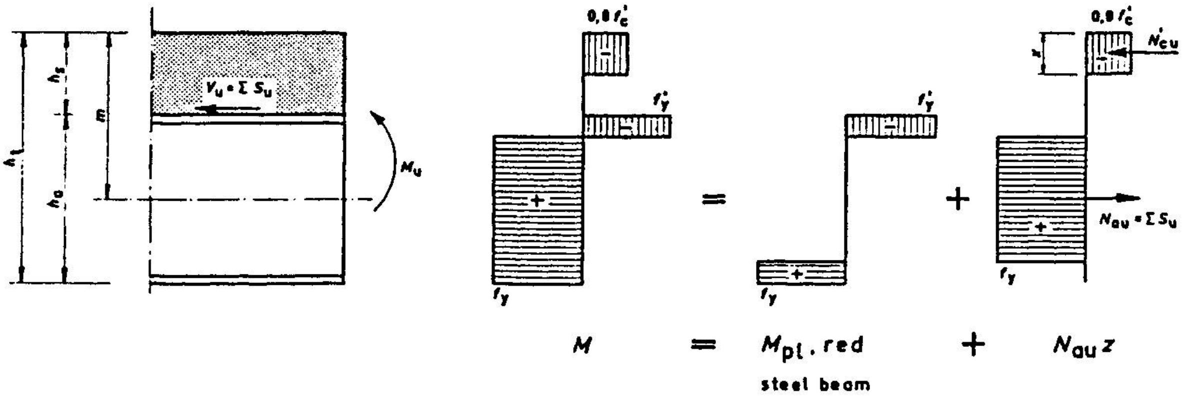


Figure 15: Stress distribution if the shear connection is partial

When a hot-rolled I section is used, the calculation may be simplified by splitting the moment of resistance into two parts as shown in Fig. 15. Both parts of the moment can be calculated as follows:

- The part of the moment supplied by the normal forces in the steel beam and the concrete slab is:

$$M_{ul} = N_{au}\, z = N_{au}\,(m - 0.5\,x)$$

Where: $N_{au} = N'_{cu} = \Sigma S_u$

$$x = \frac{\Sigma S_u}{0.8\ b_e\ f'_c}$$

Therefore:

$$M_{u1} = \Sigma S_u\ (m - 0.5\ x)$$

- The part of the moment additionally supplied by the steel section (= reduced plastic moment) can be approximated for standard rolled European I or H sections by the following interaction formula taken from the Dutch Steel Standard (The interaction formula given in Eurocode 3 is slightly different):

$$M_{u2} = M_{pl,red} = 1.18\ M_{pl}\ (1 - \frac{N}{A\ f_y})$$

Where: $N = N_{au} = \Sigma S_u$

Therefore:

$$M_{u2} = 1.18\ M_{pl}\ (1 - \frac{\Sigma S_u}{A\ f_y})$$

When the number of shear connectors and the design resistance of a shear connector are known (and so ΣS_u), the moment of resistance can be determined with the equation:

$$M_u = M_{u1} + M_{u2}$$

$$M_u = 1.18\ M_{pl}\ (1 - \frac{\Sigma S_u}{A\ f_y}) + \Sigma S_u\ (m - 0.5\ x) \qquad (4.1)$$

Fig. 16 shows the relation between the ultimate load and the degree of shear connection according to equation (4.1) by the curve DAB.

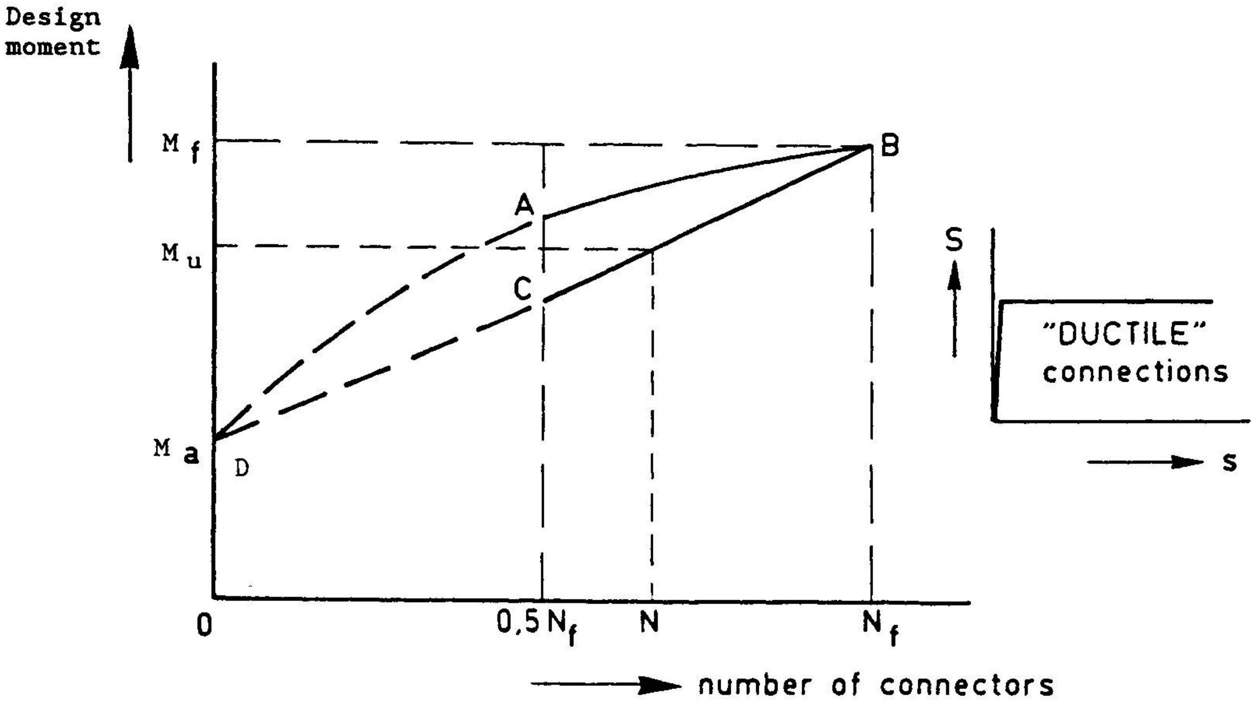


Figure 16: Relationship between design moment and number of shear connectors for ductile connectors

Simplified method

Assume that the height x of the compression zone in the concrete slab is known. A safe value is obtained if x is always taken as the value determined for full shear connection, because when ΣS_u decreases, N'_{cu} also decreases and so does the height of the compression zone of the concrete. Thus the lever arm is underestimated which is conservative.

Height of the compression zone:

$$x = \frac{A\ f_y}{0.8\ b_e\ f'_c} \leq h_s$$

The lever arm is:

$$z = m - 0.5\ x$$

So the moment of resistance M_u as described by equation (4.1) can be rearranged as follows:

$$M_u = 1.18\ M_{pl} + \Sigma S_u\ (z - \frac{1.18\ M_{pl}}{A\ f_y}) \tag{4.2}$$

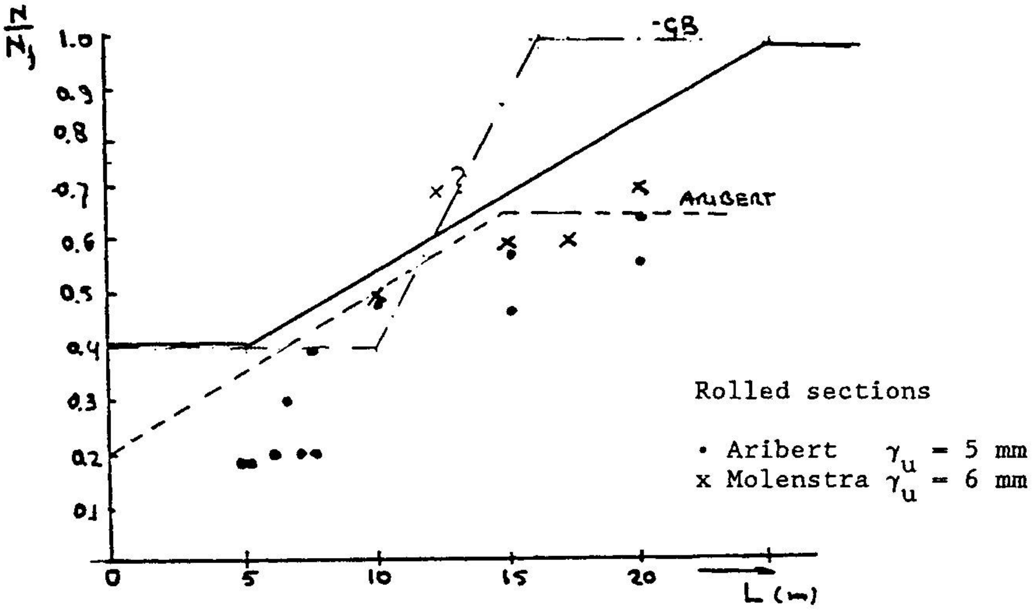
This expression represents a linear relation between M and ΣS_u. The relation between the ultimate load and the degree of shear connection for the calculated with this simplified method is indicated by the line DCB in Fig. 16.

As explained before the shear connectors must be able to deform sufficiently for the assumed full plastic stress distribution to develop.

It is self-evident that the required deformation capacity is larger for larger spans and for smaller degrees of shear connection.

In the 1985 Draft of Eurocode 4 the use of the method is restricted to beams with headed stud connectors with spans less than 20 m and degrees of shear connection not less than 50%.

For the revision of Eurocode 4 a more gradual boundary is proposed as shown in Fig. 17 and Fig. 18.



PROPOSED RELATION FOR EUROCODE 4

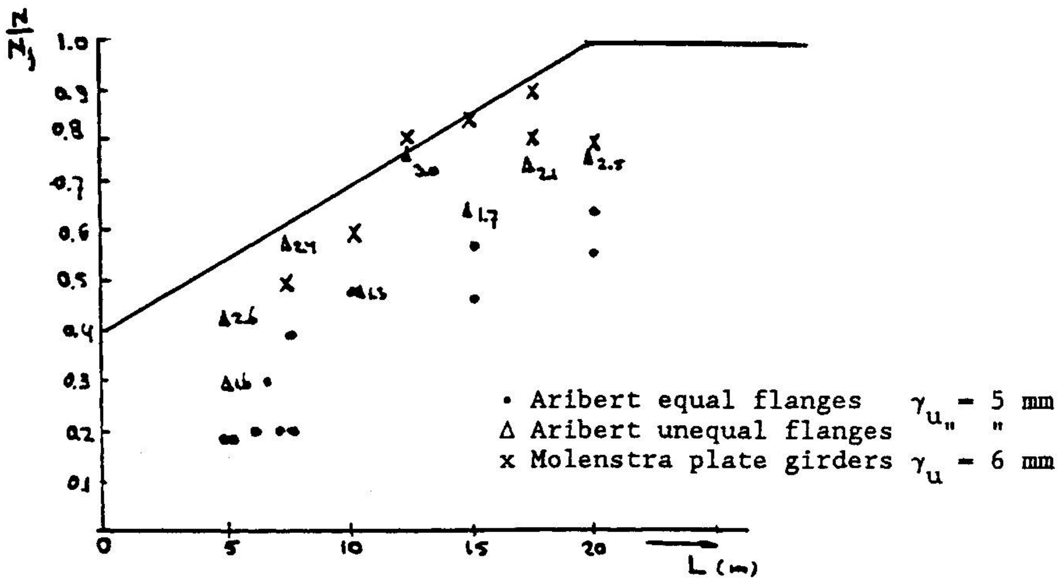
Rolled sections and sections with equal flanges

L ≤ 5m $\quad (\frac{N}{N_f})_{min} = 0.4$

5 ≤ L ≤ 25m $\quad (\frac{N}{N_f})_{min} = 0.25 + 0.03\ L$

L ≤ 25m $\quad (\frac{N}{N_f})_{min} = 1.0$

Figure 17: Proposed boundary for the use of partial shear connection is rolled sections



PROPOSED RELATION FOR EUROCODE 4

Plate girders with unequal flanges $\frac{Ab}{Au}$

L ≤ 20m $\quad (\frac{N}{N_f})_{min} = 0.4 + 0.03\ L$

L ≤ 20m $\quad (\frac{N_f}{N_f})_{min} = 1.0$

Figure 18: Proposed boundary for the use of partial shear connection in plate girders

4.4.2 Beams with rigid non-ductile shear connectors

Beams with partial shear connection and non-ductile shear connectors can be designed according to a partial interaction theory, based on the real properties of the shear connection. This design method will be rather laborious and not suitable for practical use. Therefore it is better to base the design of such beams on the assumption that no slip occurs between the concrete slab and the steel beam. This is a safe assumption.
If the stress-strain diagrams for steel and concrete are known, the relation between the ultimate moment M and the number of shear connectors for complete interaction can be calculated by the elasto-plastic method as is qualitatively shown in Fig. 19. The calculation of the elasto-plastic branch of the diagram is rather laborious. To simplify this calculation the curved part of the diagram can safely be approximated by a straight line. For this line the following equation can be derived:

$$N_{au} = \Sigma S_u = N_e + \frac{M_u - M_e}{M_{fsu} - M_e} (N_{fsu} - N_e) \qquad (4.3)$$

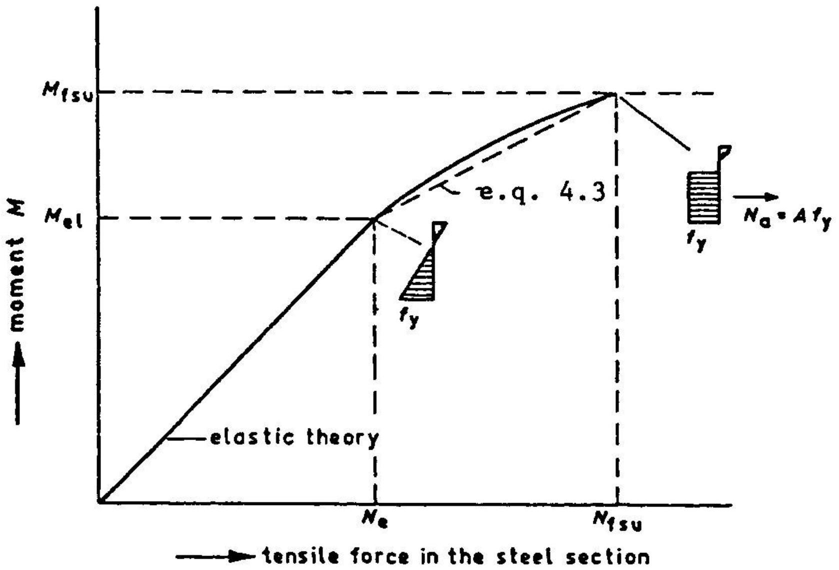


Figure 19: Qualitative relation between the moment and the tensile force in the steel section with complete interaction (absolutely rigid)

5. SAFETY CONCEPTS AND DESIGN ASSUMPTIONS

5.1 Safety format

All Eurocodes are written in a limit state design format. The limit state conditions are expressed as those of the ultimate limit state, thus:

$$\boxed{S_d \leq R_d}$$

The following partial safety factors are used.

Factors for loading	load combinations	ψ_o	
	accuracy of loading	γ_f	γ_F
Gap factor (for model uncertainties)	structural	γ_S	
	material	γ_{Rd}	γ_M
Factors for resistance	accuracy of strength		
	manufacturing tolerances	γ_M	

The format of the formula for the design resistance R_d should be consistent with Eurocode 2 and Eurocode 3. However in Eurocode 2 the partial safety factor γ_M for concrete and reinforcing steel are applied to properties of the materials while in Eurocode 3 the factor γ_M is applied to the resistance of cross-sections or members.

$$\text{Eurocode 2: } R_d = R\left[\frac{f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}\right]$$

$$\text{Eurocode 3: } R_d = \frac{1}{\gamma_M} R\left[f_y\right]$$

This leads to an inconsistency and a problem for Eurocode 4.
As it stands now the following format is proposed:

$$\text{Eurocode 4: } R_d = \frac{1}{\gamma_{Rd}} R\left[\frac{f_y}{\gamma_a}, \frac{f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}\right]$$

The partial safety factors for resistance γ_M are given as recommended values. They are enclosed in boxes to indicate that for the moment they are not an official proposal from the Commission.

5.2 Evaluation of test results for the determination of design resistances

The Eurocodes focus on a presentation in a Limit State Design (LRFD) format. The conversion of traditional working stress design methods into a limit state design format is not easy moreover when at the same time differences between practice in the various member states have to be leveled also.
It has been found that many of the rules in existing national standards were based on engineering judgement, more than on a consistent evaluation of experimental evidence.
This is clearly demonstrated in Fig. 20 where a non-dimensional comparison is given for the allowable tensile-force in structural bolts of different grades as derived from the national standards.
Amazingly enough the difference between the highest and the lowest value for a relative simple element as a bolt is more than a factor 2. This of course can not form the right basis for the determination of design rules in a harmonisation process. Fortunately the Eurocode Coordination Group has developed a semi-probabilistic limit state verification to be used in level-I codes as discussed in 5.1. In a level-I code the verification of the ultimate limit state

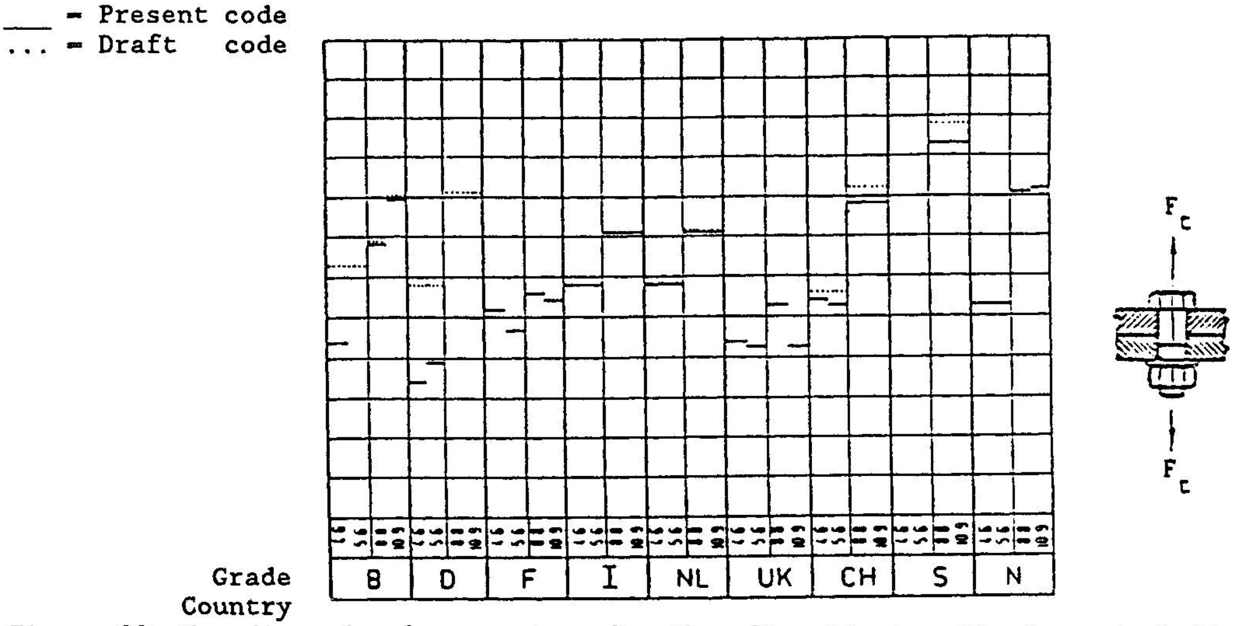


Figure 20: Non-dimensional comparison for the allowable tensile-force in bolts

is expressed by the condition that the design effect of loads and other actions on the structures will not exceed the design resistance.

$$\text{Effects of actions} \longrightarrow \boxed{S_d \leq R_d} \longleftarrow \text{Design resistance}$$

$$\gamma_f \, \gamma_{S_d} \, S_k \leq \frac{R_k}{\gamma_m \, \gamma_{R_d}}$$

$$\gamma_F \, S_k \leq \frac{R_k}{\gamma_M}$$

↑
evaluation of tests

Basis for each side of the expression are the characteristic values for action effects and resistances S_k and R_k respectively.
Also on both sides partial safety elements, so called γ factors are introduced to arrive at the required safety level. Based on the proposed verification procedure of the Coordination Group the Eurocode 3 Drafting Panel developed a procedure for the determination of characteristic values, design values and γ_M values for resistances from test results. This method is successfully used for the determination of design rules for bolts and welds, beam to column connections, column stability, lateral stability, local stability and others in Eurocode 3. It is also used for the determination of design rules for Eurocode 4.

EVALUATION PROCEDURE:
The evaluation procedure goes along the following lines.
Based on observation of actual behaviour in tests and on theoretical considerations, a "design model" is selected, leading to a strength function. The efficiency of the model is checked by comparing the theoretical results from the strength function with available results of tests.
The design model has to be adapted until the correlation of the theoretical values and the test data is sufficient.

The accepted strength function can then be used to derive an expression for the *characteristic resistance* R_k. The characteristic resistance is defined as having a 5% probability of not being exceeded for a level of confidence of the prediction of 75%. The procedure also includes a method to derive *design values* from the given data and hence to deduct γ_M- factors, to be applied to the relevant characteristic strength functions.
The value of γ_M is dependent on the required failure probability determined by the safety index β (for the ultimate limit state normally $\beta = 3.8$).
The evaluation procedure would be very simple if the test population can be regarded as a representative sample of the total population. This is normally not so. In normal circumstances the test specimen are not representative for variations of material strength and stiffness and for variations of geometrical properties. Therefore the evaluation is only used to determine the variation in the prediction of the design model. This variation is then combined with variations of other variables in the resistance function, which are based on preknowledge.
For an easy understanding the standard procedure will be presented as a number of discrete steps under ideal assumptions for the test population and data. These ideal assumptions are:

A. The strength function is a product function of independent variables.
B. A large number of test results is available.
C. All actual geometrical and material properties are measured.
D. All variables have a log-normal distribution. Adopting a log-normal distribution for all variables has the advantage that no negative values can occur for the geometrical and strength variables which is also physically correct.
E. The design function is expressed in the mean values of the variables.
F. There is no correlation between the variables of the strength function.

The *standard procedure* comprises the following steps (see Fig. 21):

<u>Step 1</u>: Develop a theoretical model for the strength of an element or a structural detail and derive a strength function.
The strength function should include all relevant basic variables which control the resistance. All the basic parameters should be measured for each test specimen i and be available for the evaluation.

<u>Step 2</u>: Compare the experimental and the theoretical values.

From the tests the experimental values r_{ei} are known. Using the relevant strength function and putting the actual properties into the formula, leads to the theoretical values r_{ti}.
The combinations of corresponding values (r_{ti}, r_{ei}) form points in a diagram (Fig. 22). If the strength function is exact and complete, all points (r_{ti}, r_{ei}) lie on the bisector of the angle between the axes of the diagram and the correlation coefficient ρ is 1.0.
In general the points (r_{ti}, r_{ei}) will scatter.

Step 1 — Design model $r = g_R(X)$

Step 2 — Compare theory <--> test $r_t \leftrightarrow r_e$

Step 3 — Check correlation $\rho \geq 0.9$

Step 4 — Mean value correction $\bar{b}$
Mean value corrected strength
$r_m = \bar{b}\ r_t$ → $\bar{b}$

Step 5 — Variation of strength function V_δ → V_δ

Step 6 — From preknowledge the coefficients of variation for the basic variables are determined V_{Xi} → V_{Xi}

Step 7 — Characteristic strength

$$V_r = \sqrt{\sum_{i=1}^{J} V_{Xi}^2 + V_\delta^2}$$

$$\sigma_{\ell nr} = \sqrt{\ell n\ (V_r^2 + 1)} \approx V_r$$

$$r_k = r_m\ (\underline{X}_m)\ \exp\ (-k_s\ \sigma_{\ell nr} - 0.5\ \sigma_{\ell nr}^2)$$

→ $R_k = r_k/r_m$

Step 8 — Design strength

$$r_d = r_m\ (\underline{X}_m)\ \exp\ (-0.8\ \beta\ \sigma_{\ell nr} - 0.5\ \sigma_{\ell nr}^2)$$

$$\gamma_M = r_k/r_d = \exp\ ((0.8\ \beta - 1.64)\ \sigma_{\ell nr})$$

→ $R_d = r_d/r_m$
→ γ_M

Figure 21: Standard procedure

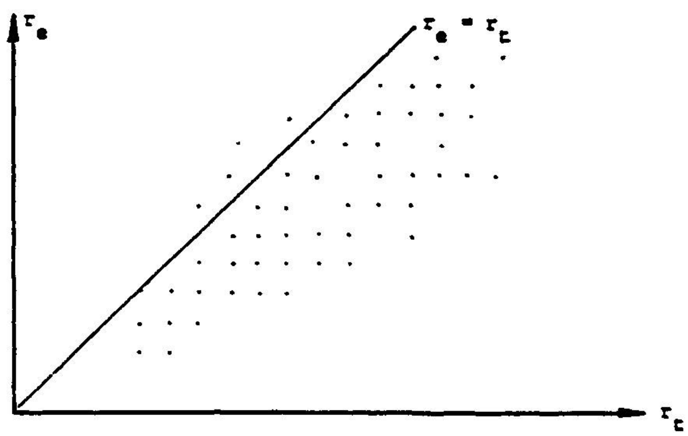


Figure 22: r_e - r_t diagram

Step 3: Check whether the correlation between the experimental and the theoretical values is sufficient.

a. Determine the mean values $\bar{r}_e$ and $\bar{r}_t$ of the experimental values r_{ei} and the theoretical values r_{ti} respectively and their standard deviations s_{re} and s_{rt}.
b. Determine the correlation coefficient ρ. If ρ is not less than 0.9 the correlation can be considered to be sufficient.

Step 4: Determine the mean value correction $\bar{b}$.

For each specimen i, the comparison of the theoretical value r_{ti} with the corresponding experimental value r_{ei} renders a correction term b_i. In the r_e - r_t diagram the mean value correction $\bar{b}$ is the direction coefficient of a straight line going through the origin of the diagram which represents the mean value of the test results via a correction of the theoretical values (Fig. 23).

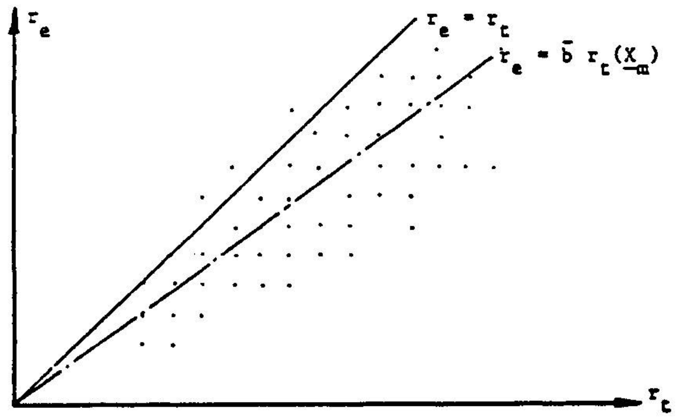


Figure 23: r_e - r_t diagram with the mean value correction line

Step 5: Determine the coefficient of variation V_δ of the observed error terms.

For each test comparison of the theoretical results, inclusive the mean value correction, and the relevant experimental result gives the error term δ_i. From all the error terms the value of the variation coefficient V_δ can be calculated. This variation coefficient only represents the accuracy of the strength function and not the influence of scatter of the variables in the strength function. This is clear because in the strength function the actual measured properties were used.

Step 6: Determine the coefficient of variation of the basic variables in the strength function (V_{xi}).

The coefficient of variation of all the basic variables may only be determined from the test-data if it may be assumed that the test population is fully representative for the variation in the actual situation. This is normally not the case, so the coefficients of variation have to be determined from preknowledge.

Step 7: Determine the characteristic value of the strength (Fig. 24).

The characteristic value of the strength can be determined from the strength value, inclusive the mean value correction, and the combined variation coefficient V_r.

$$V_r = \sqrt{\sum_{i=1}^{J} V_{Xi}^2 + V_\delta^2}$$

In Fig. 25 this is represented by the line $r_e = r_k$.

characteristic value of the strength

$$r_k = r_m(\underline{X}_m) \exp(-k_s \sigma_{\ell nr} - 0.5\, \sigma^2_{\ell nr})$$

where: $\sigma_{\ell nr} = \sqrt{\ell n \ (V_r^2 + 1)} = V_r$

$$V_r = \sqrt{\sum_{i=1}^{J} V_{Xi}^2 + V_\delta^2}$$

$k_s = 1.64$ <--- 5% fractile

partial safety factor γ_M

$\uparrow$

$\gamma_M = r_k / r_d$

design value of the strength

$$r_d = r_m(\underline{X}_m) \exp(-k_d \sigma_{\ell nr} - 0.5\, \sigma^2_{\ell nr})$$

where: $k_d = \alpha_R\, \beta = 0.8 \times 3.8 = 3.04$

Figure 24: Characteristic value of the strength

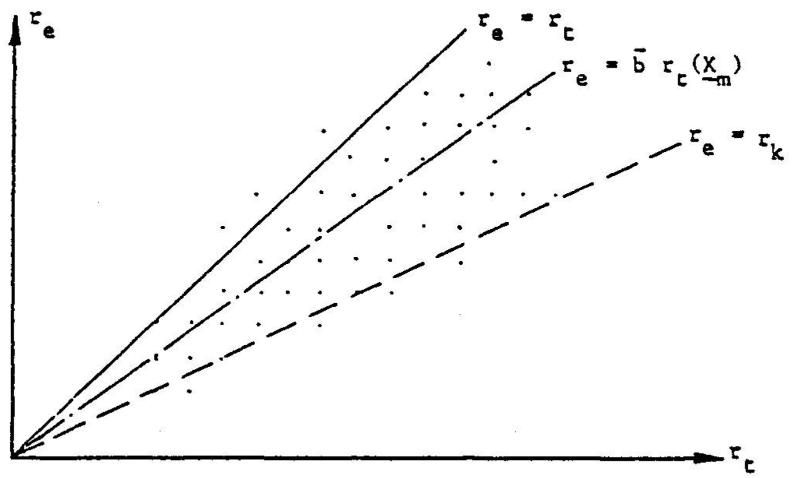


Figure 25: r_e- r_t diagram with the characteristic line $r_e = r_k$

Step 8: Determine the design value of the strength and the partial safety factor γ_M (Fig. 24).

When the 5%-fractile of the strength function is determined, it is possible to extend the evaluation to obtain the design function r_d related to a given safety index β by replacing the fractile coefficient k_s for the 5%-fractile by k_d for the design fractile.
The value of k_d can be taken as $\alpha_R \beta$. The sensitivity factor α_R on the resistance side (and α_S on the loading side) has to be determined under the assumption that the linearization of the ultimate limit state in the design point does not show large variations of the safety index β. Comparative studies with $\alpha_R = 0.8$ (and $\alpha_S = 0.7$) lead to an acceptable safety index $\beta = 3.8$.

The characteristic value determined in step 7 divided by the design value determined in step 8 gives the partial safety factor γ_M.

For the step-wise procedure, as discussed before, a number of ideal assumptions have been made. The experience in the Eurocode project has shown that often one or more of the assumptions are not valid, so that the method had to be adapted. One point is that the strength functions in design codes usually contain basic variables which are defined as characteristic values or nominal values instead of mean values. For example the material strength is normally expressed in terms of the characteristic strength.
Another problem is that in some tests it is difficult to measure the actual material properties as is the case in tests on bolts. The procedure is also adjusted for that case. It should be noted that this adjustment leads to a conservative result if the test population is fully or nearly representative for the variation of the variables in the actual situation.

EXAMPLES OF EVALUATIONS FOR EUROCODE 3

As an illustration some examples of the results of the evaluations will now be discussed briefly.
At first the design rules for bolted and welded connections.
The table in Fig. 26 gives an overview of the failure mechanisms for bolted connections which are considered to derive theoretical models and design functions.

	No.	Failure mechanism	
BOLT FAILURE	1	Tensile failure in the thread of bolts	F_t
	2	Shear failure in the shank of bolts	F_v
	3	Shear failure in the thread of bolts	F_v
	4	Combined shear and tensile failure in the thread of bolts	F_v, F_t
	5	Combined shear and tensile failure in the shank of bolts	F_v, F_t
PLATE FAILURE	6	Plate bearing failure (hole elongation and shear)	F_b
	7	Net section failure of plates	F_n
	8	Net section failure of angles connected at one side with one bolt	F_a
	9	Net section failure of angles connected at one side with two bolts	F_a
	10	Net section failure of angles connected at one side with three or more bolts	F_a

Figure 26: Failure mechanisms considered for statistical analysis

The original design functions given in the 1984 Draft of Eurocode 3 are given in Fig. 27. To validate these strength functions and to determine suitable values for the model factors a re-evaluation of available test information is carried

out. Test results from different sources are systematically stored in a data base at the University of Aachen. The total collection of test results on bolts amounts at the present more than 1900.
For practical purposes it was felt that one uniform value of γ_M was preferred. Therefore in some cases a small part of the safety element had to be hidden into the strength function.

Tensile capacity		$F_{tk} = f_{ub} A_s$
Shear capacity	thread	$F_{vk} = 0.7\ f_{ub} A_s$
	shank	$F_{vk} = 0.7\ f_{ub} A$
Bearing capacity		$F_{bk} = \alpha\ f_u\ d_n t$
Tensile capacity of parts		$R_{tk} = A\ f_y$ or $R_{tk} = A_n\ f_u$

Figure 27: Strength functions for bolted connections in the 1984 Draft of Eurocode 3

In Fig. 28 the results for bolts in tension are given. The original design function appeared to give not the required safety so that the design function was changed.
Fig. 29 gives the results for the design shear resistance if the shear plane is through the threaded position of the bolt.
This is a nice example of the possibility to improve the strength function by considering subsets of the test population (Fig. 30). This allows to make clear what parameters influence the scatter. The scatter can be reduced by correcting the strength function, such that additional parameters, not sufficiently contained in the strength function, are now taken into account.

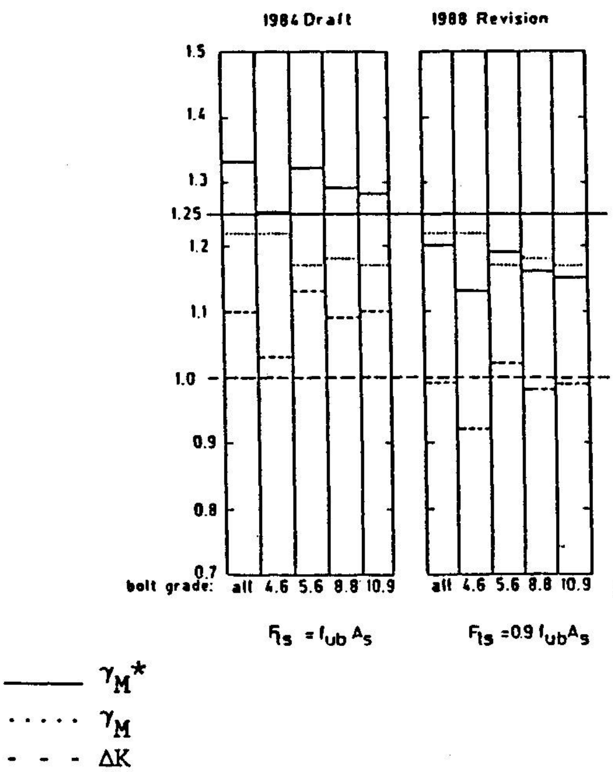


Figure 28: Tension

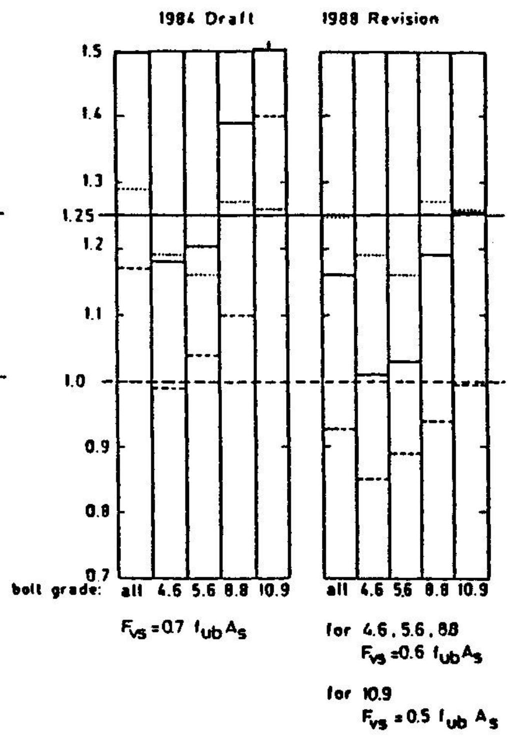


Figure 29: Shear

In this case the additional parameter is the influence of the bolt grade.

To arrive at a relative consistent safety the design strength for 10.9 bolts had to be further reduced than for the other grades (Fig. 29).

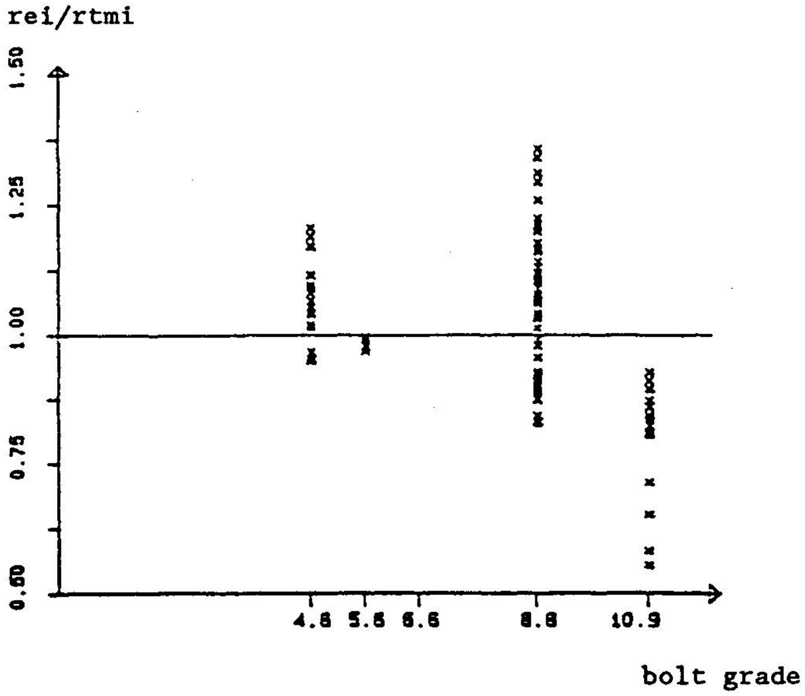


Figure 30: Sensitivity diagram for bolt grade

The same procedure is used for the stability formulae in Eurocode 3. As just one example Fig. 12 gives the result for buckling curve b. The Δk value is very close to 1. This indicates that the strength function is good.
The required γ_M value is approximately 1.1. There is still discussion whether it is preferable for practical purposes to use $\gamma_M = 1.0$ and to adapt the strength function accordingly.

CALIBRATION AGAINST NATIONAL STANDARDS
After presentation of the evaluation results to the Liaison Engineers and to members of ECCS Technical Committees the question was raised whether the new formulae could adversely affect the economy of steel structures.
Therefore it was decided to carry out a calibration study. The results according the proposed Eurocode 3 strength functions were calibrated against results according to national standards. Fig. 32 shows the results for bolts loaded in tension. A value lower than 1 indicates that Eurocode 3 gives a higher resistance.

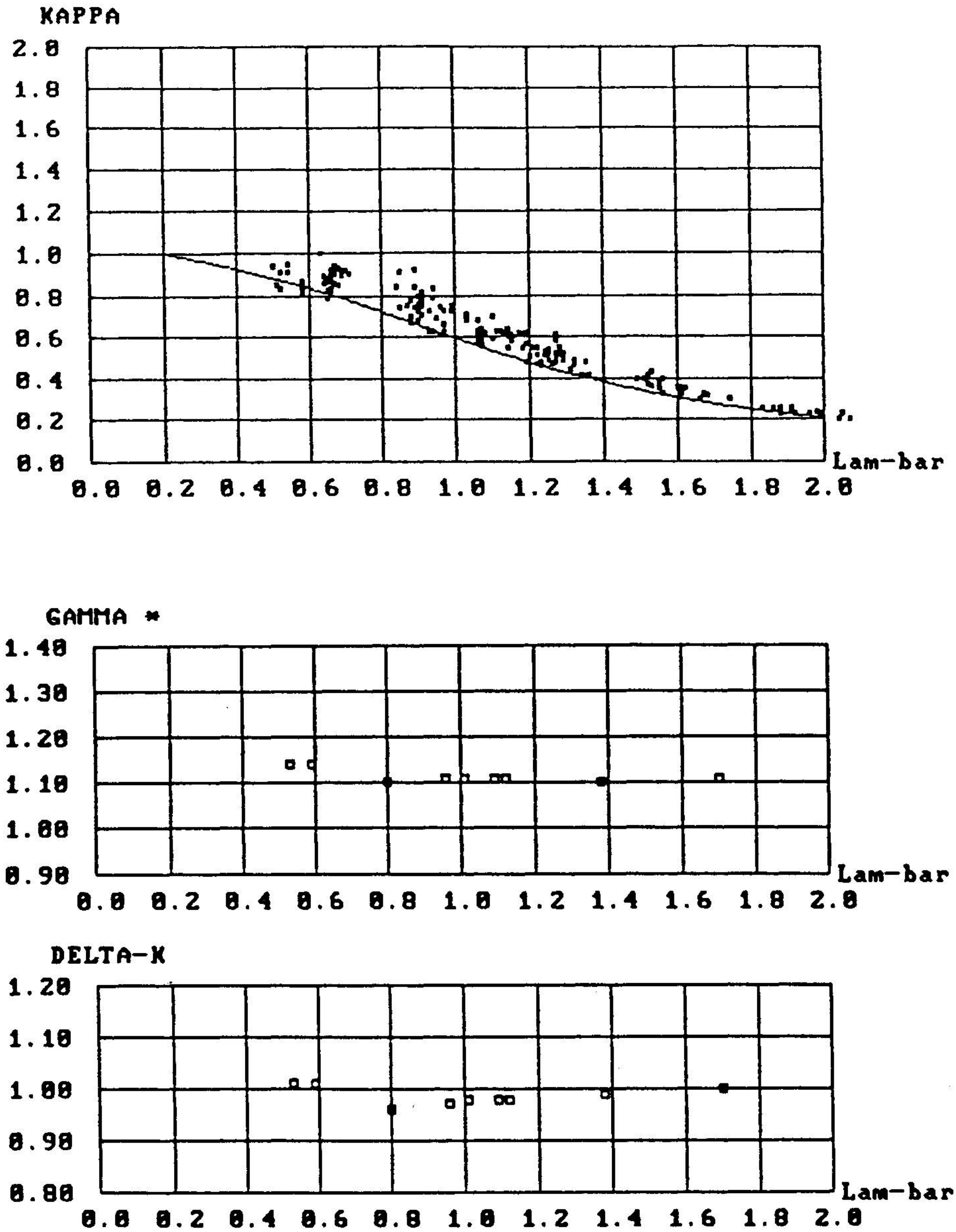
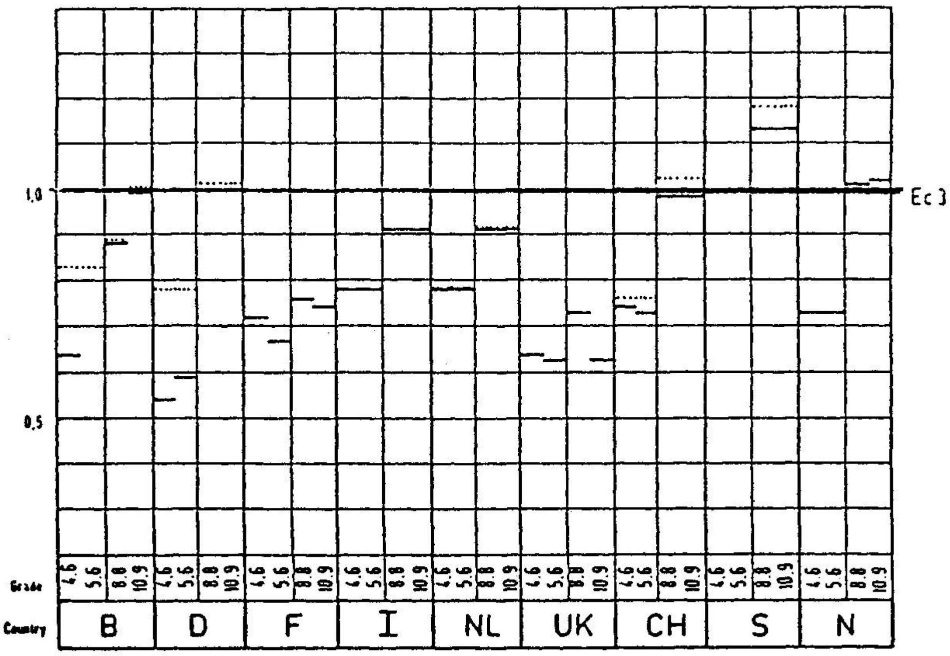
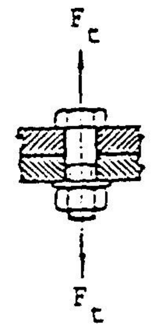


Figure 31: Buckling curve b



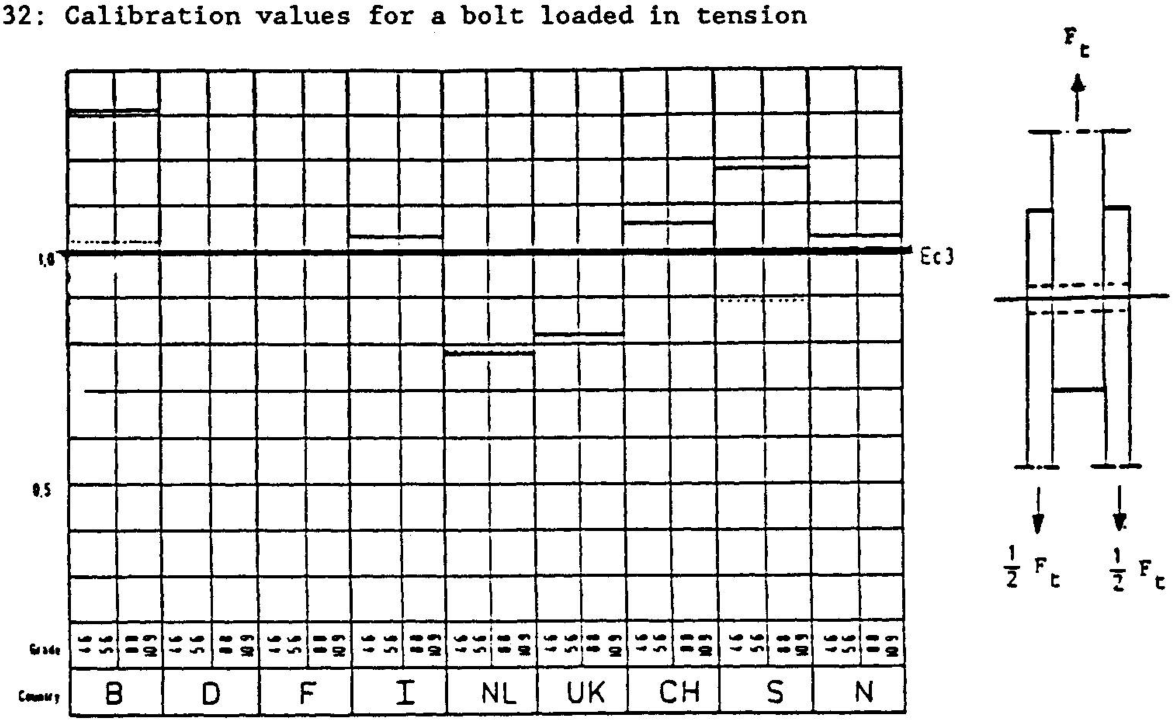


grade	MELP	MELD
4.6	0.70	0.79
5.6	0.67	0.79
8.8	0.91	1.00
10.9	0.89	1.02

____ = Present code Mean European Level Present codes MELP

.... = Draft code Mean European Level Draft codes MELD

Figure 32: Calibration values for a bolt loaded in tension



____ = Present code Mean European Level Present codes MELP = 1.03

.... = Draft code Mean European Level Draft codes MELD = 0.92

Figure 33: Calibration values for a lap joint with one bolt - case I

An indicative value for the average situation is the factor MELP and MELD which gives the mean value according to present and draft codes respectively. It is clear that Eurocode 3 is on the average more liberal than the existing codes.
Fig. 33 shows the results for a bolted joint loaded in shear.
For this case the results of the national standards are more close to the Eurocode 3 results. It is interesting that the Belgian and Swedish draft codes have lower values than the present codes and come now closer to the Eurocode 3 values.

At last attention is drawn to the fact that the method does not allow to take into account quantitatively the important aspect of deformation capacity. To illustrate this, results of load tests on composite slabs are used (Fig. 34). The load-deformation behaviour is very much dependent on the efficiency of the shear connection. Of course the behaviour given for deep embossments is to be preferred. For the minor embossments and surely for the plain sheet, if at all acceptable, a larger safety factor would be required. However in the evaluation procedure as described the design resistance and the γ_M will <u>not</u> be different for ductile and non-ductile behaviour. This is still a gap in the harmonisation process that should be solved in the near future.

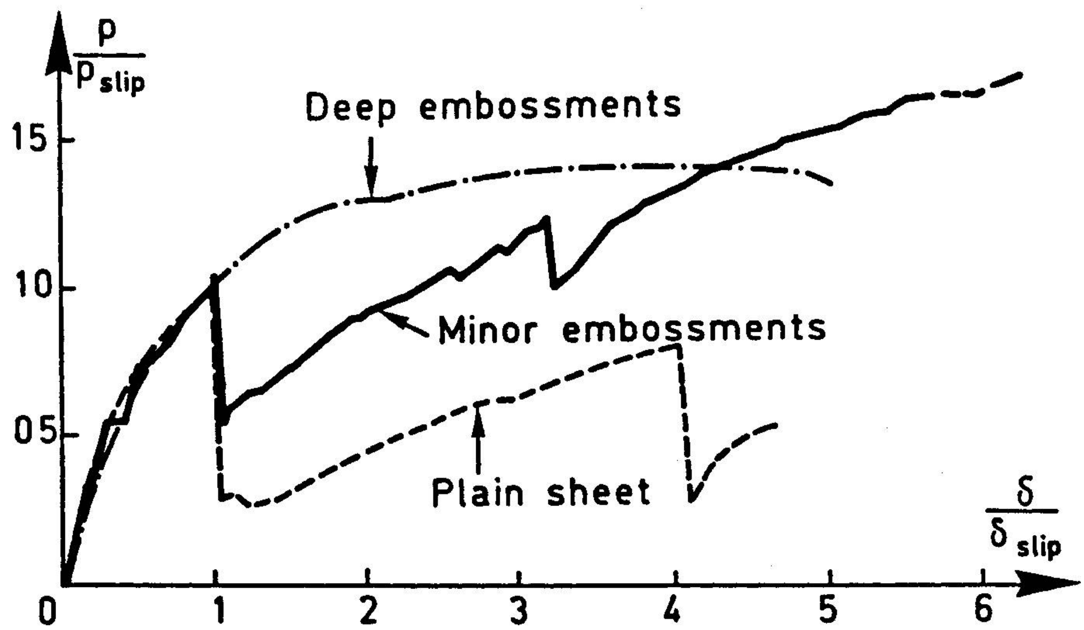


Figure 34: Load-deformation curves for composite slabs