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EC 3: A Eurocode for Economical Steel Structures

EC 3: Un Eurocode pour des structures en acier économiques

EC 3: Ein Eurocode für wirtschaftliche Stahlbauten

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SUMMARY

In this contribution an outline is given of the design methods for connections in Eurocode 3. The design approach forming the basis for the presentation of the rules in EC 3 is discussed. Indications are given how to take advantage of the possible application of partial strength connections and semi-rigid frame design. Finally the evaluation procedure used to determine the resistances of bolts and welds is presented.

RESUME

La présentation traite des méthodes de calcul des assemblages dans l'Eurocode 3. Cette approche du projet est à base de la présentation des règles de calcul dans l'Eurocode 3. Des indications sont données sur les avantages possibles de l'application du projet d'assemblages à résistance partielle et des cadres semi-rigides. Les procédures d'évaluation en vue de déterminer la résistance des boulons et des soudures sont enfin présentées.

ZUSAMMENFASSUNG

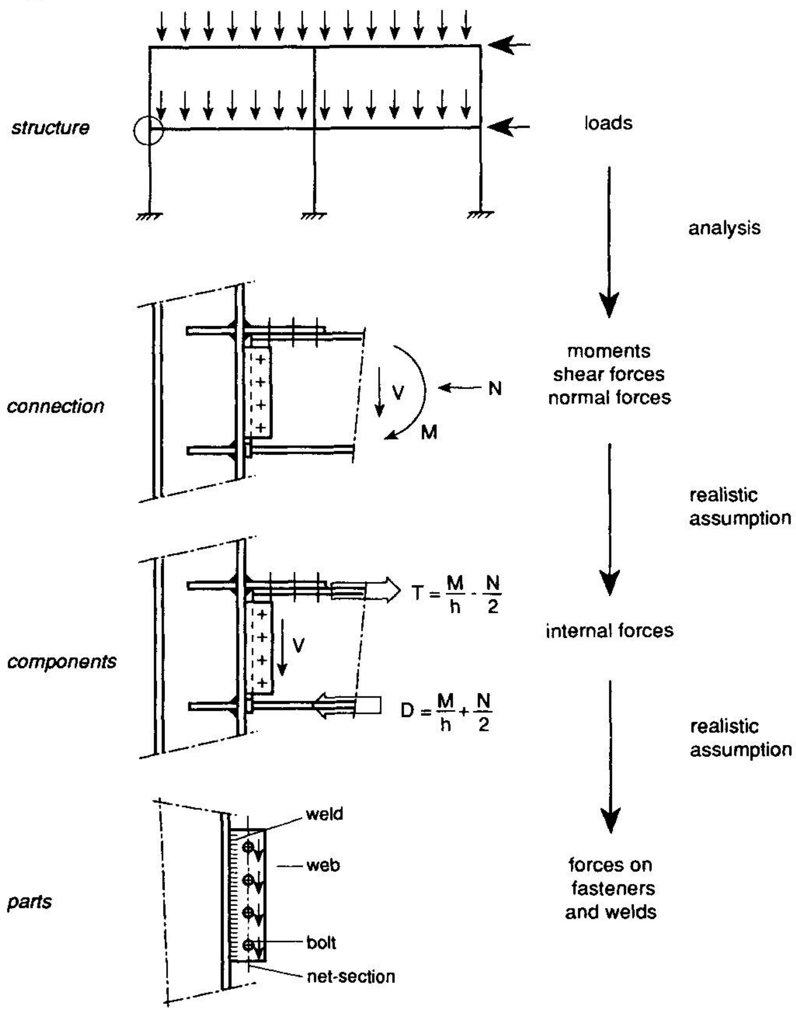
Im vorliegenden Beitrag wird ein Abriss der Bemessungsverfahren für Verbindungen in EC 3 gegeben. Das seinen Regeln zugrundeliegende Bemessungskonzept wird erörtert, und es werden Anhaltspunkte gegeben, wie mit Vorteil Verbindungen auf anteilige Widerstände ausgelegt und halbsteife Rahmen eingesetzt werden können. Schliesslich wird das Nachweisverfahren für Schrauben- und Schweissverbindungen vorgestellt.

1. INTRODUCTION

In structural steelwork the joints between the members play an important role. Evidently the properties of the joints influence the response of the structure to actions. The relevant structural properties are strength, stiffness and deformation capacity. But also from economical points of view the joints are very important. The number and simplicity of the joints influence greatly the time, required for designing and drawing.
Production of connections, cutting, drilling and welding of main members, plates, cleats and stiffeners. consumes much of the work in the fabrication shop. The ease with which the site connections can be made is a key factor for erection. So the selection, design and detailing of the connections significantly influences the total costs of a steel structure.
It is for that reason that in Eurocode 3 relatively much attention is spent to the design of connections. In this paper an outline is given of the design methods in Eurocode 3 for connections. Emphasis will be given to those rules which are expected to be of prime importance for the economy of steel structures.

2. DESIGN APPROACH

There are so many different structural solutions, even for the same type of connection that it would be impracticable to cover each separately in detail in the Code. Therefore in 6.1 of EC3 a procedure is given that essentially can be applied to all type of connections and leads to a check of individual fasteners and other parts of a connection. This procedure is illustrated in figure 1.

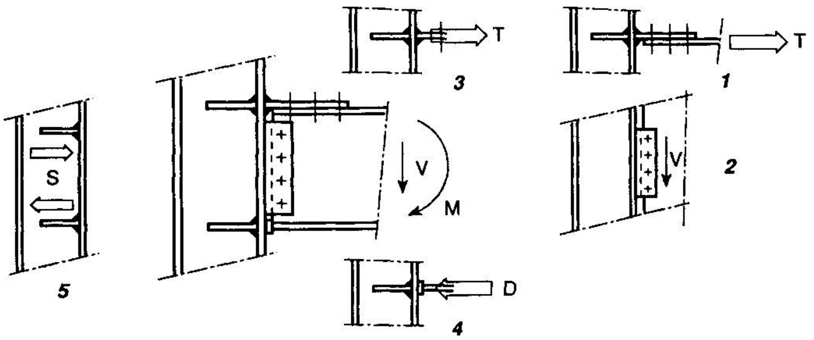


The first step is to schematise the structure. In this phase the connections must be classified as covered in 6.4 of EC3. This subject is discussed in more detail in section 3 of this paper. Then the forces and moments applied to the connections shall be determined by elastic or plastic global analysis. The next step is to determine the distribution of forces within the connection. It is not necessary, and often not feasible to determine the real internal distribution of forces. It is sufficient to assume a realistic distribution, provided that (clause 6.1.4.):

(a) the internal forces are in equilibrium with the applied loading,
(b) each element is capable of resisting the forces
(c) the deformations implied by this distribution are within the deformation capacity of the fasteners or welds and of the connected parts.

This is the most difficult part of the procedure, because this will, of necessity, entail the making of certain simplifying assumptions about the way in which the connection "works".
Account should be taken of the distribution of forces in the elements that are connected.
In addition, the assumed distribution shall be realistic with regard to relative stiffenesses within the joint. The internal forces will seek to follow the path with the greatest rigidity.
It is most important to ensure that the analysis is consisted throughout the connection.

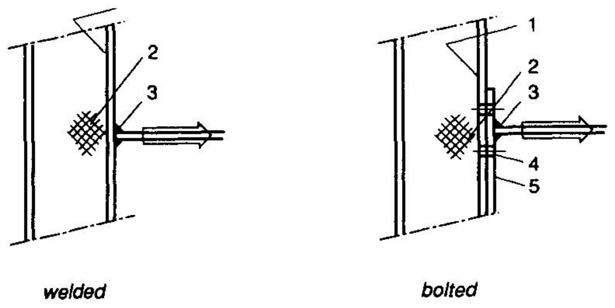
To cover the large variety of different forms of connections it is useful to use the concept of a set of basic force transfers of the type found in the component parts of many forms of connection. The basic forms are shown in figure 2.



1 Axial force
2 Shear force
3 Introduction tensile force
4 Introduction compression force
5 Shear panel

Figure 2
Basic force transfers in connections

For each of these basic force transfers a number of failure modes are possible. Each of these shall be checked and the weakest link shall be able to resist the applied load.
This is illustrated in figure 3 for the introduction of a tension force in an unreinforced web. The rules for checking the welds and the bolts (criteria 3 and 4) are covered in chapter 6.6 and 6.5 of EC3. The other criteria are covered in Annex J for beam to column connections, but as explained above parts of the method can also be applied to other forms of connections. The procedure can in principle also be used to check the other two structural properties of a connection being stiffness and deformation capacity.



1. Excessive deformation column flange
2. Yielding/tearing column web
3. Failure weld
4. Failure bolts
5. Yielding end plate (c.q. L or T)

Figure 3
Possible failure modes for introduction of tension force (case 3 in figure 2)

In Annex J, besides traditional welded stiffeners, other often more economical means to reinforce or stiffen connections are treated. These are supplementary web plates and backing plates as shown in figure 4.

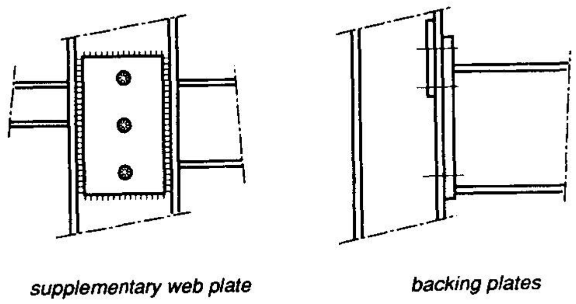


Figure 4
Alternative solutions to reinforce connections

3. CLASSIFICATION OF CONNECTIONS

The section 6.4 on classification of connections in Eurocode 3 has given the most controversial reactions. This is probably caused by the first impression that existing standard practice for modeling of connection behaviour would not be allowed anymore. This is certainly not the case. In the past, when "working stress" design was normally used, the connection design was based on rather simple though not necessarily economical assumptions.
The connections were assumed to behave either as hinges (simple construction) or as infinitely stiff (rigid construction). The forces on the connections then followed from an elastic analysis of the structure. The parts of the connections such as end plates and angles, welds and bolts, could subsequently be dimensioned. Even now this design procedure seems to be used in the majority of cases. It was not the intention to abandon this procedure in EC3 but to give the designer, as a matter of choice, an alternative to use connections having properties in between the two extreme cases.

This is important because the introduction of limit state design, including practical rules for plastic design, requires a more realistic treatment of the connections. When using these methods, the designer is confronted directly with the fact that for a better insight into topics such as the stability of columns and frames and for a minimum cost design of members and connections, understanding of the behaviour of connections is essential.
Another factor is that modern computer programs, now available to the majority of designers, allow a more sophisticated treatment of connections without an appreciable increase in calculation costs. Finally, also the use of automatic NC drilling and sawing equipment in the fabricators shop influences the cost relationship between various forms of connections, leading to a need to minimise the number of welded stiffeners and to use other means to reinforce or stiffen connections.
Now two situations will be discussed where a designer may decide to alter the traditional way of modeling beam-to-column connections.
An important factor for this choice is whether the frame is braced or unbraced. Let's consider now both cases.

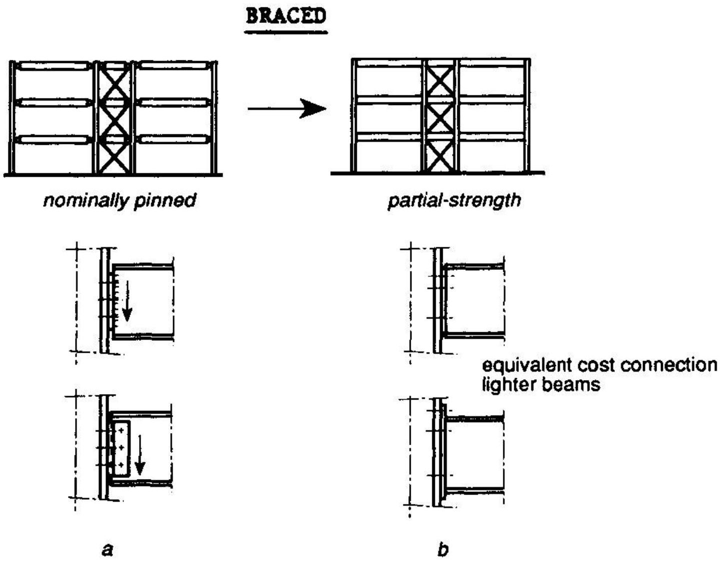


Figure 5
Connections in braced frames

In a braced frame it is possible to design the connections as hinged. Stiffness is not required for stability. Two possible forms of so-called nominally pinned connections are shown in figure 5 (a). The connections with a short-end plate or web cleats are designed only for transfer of shear force. By using detailing rules based on experience and test evidence the designer takes care that the flexibility and deformation capacity is sufficient. In 6.9.6.2(2) of EC3 this procedure is explicitely allowed for. In figure 5 (b) two forms of connections are shown which are not much more expensive to fabricate. Although these connections are normally not full-strength they offer some end restraint to the beam, so giving more load carrying capacity and less deflection. Therefore often a smaller beam section will do. Apart from the material savings this also will reduce construction depth. An additional advantage may be that the frame has allready some stiffness during erection before the bracing system has become effective. By using plastic design and assuming partial-strength connections the design is even simpler than the traditional elastic design.

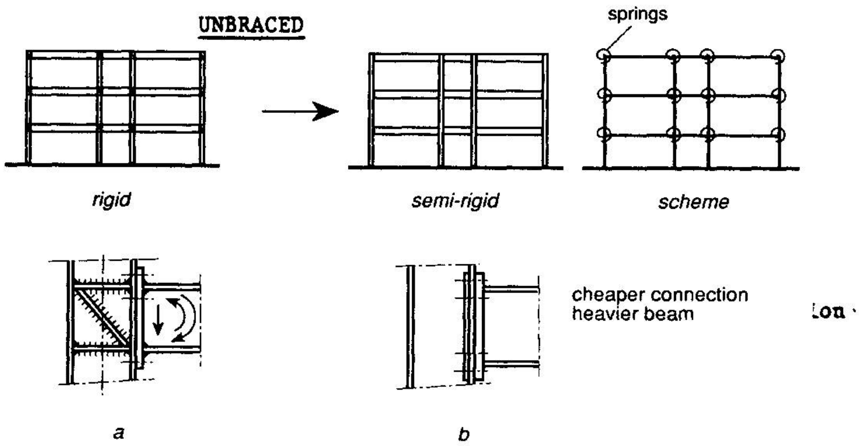


Figure 6
Connections in unbraced frames

If the frame is unbraced stiffness of the connections is essential for stability. Unreinforced connections are normally more flexible than the connected elements. However this is not unacceptable provided that the reduced stiffness is compensated by the choice of heavier members. The optimum is dependent of the relation between material cost and fabrication cost. For the analysis of the structure the connections are represented by fictitious structural elements at the ends of the members or by springs. In Annex J of EC3 rules are given to determine the stiffness of joints.

4. RESISTANCES OF BOLTS AND WELDS

In the drafting process it became clear from a comparison of rules in national standards that the various national rules for bolts and welds showed considerable differences. As an example the extreme values of the tensile strength of a simple element as a bolt differed by more than a factor 2. It was found that many of the rules in the national standards were based on engineering judgement more than a consistent evaluation of experimental evidence. This of course can not form the right basis for the determination of design rules in a harmonisation process. For that purpose an objective procedure was needed. Fortunately the Eurocode Coordination Group had developed a semi-probabilistic limit state verification to be used in level-I codes. In a level-I code the verification of the ultimate limit state is expressed by the condition that the design effect of loads and other actions on the structures will not exceed the design resistance.

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

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

$$\gamma_F \, S_k \leq \frac{R_k}{\gamma_M} \longleftarrow \text{evaluation of tests}$$

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 [1]. This method is successfully used for the determination of design rules for bolts and welds and beam to column connections.
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 <u>characteristic resistance</u> 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 <u>design values</u> 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 $\beta = 3.8$). For practical reasons the coefficients in the design functions derived from the statistical evaluation were modified slightly, to enable a single value of $\gamma_M = 1.25$ to be recommended for all cases.
The total collection of the results used for the evaluation amounted about 2000 for bolts and about 500 for welds. More details are given in [2], [3], [4]. An overview of the design capacity is given in Table 1.

Table 1 Design resistance for bolts $\gamma_{Mb} = 1.25$

<u>Shear resistance per shear plane:</u>

if the shear plane passes through the threaded portion of the bolt:

- for strength grades 4.6, 5.6 and 8.8:

$$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}}$$

- for strength grades 4.8, 5.8, and 10.9:

$$F_{v.Rd} = \frac{0{,}5\, f_{ub}\, A_s}{\gamma_{Mb}}$$

if the shear plane passes through the unthreaded portion of the bolt:

$$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A}{\gamma_{Mb}}$$

<u>Bearing resistance:</u> *)

$$F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u\, d\, t}{\gamma_{Mb}}$$

where α is the smallest of:

$$\frac{e_1}{3d_o} \,;\, \frac{p_1}{3d_o} - \frac{1}{4} \,;\, \frac{f_{ub}}{f_u} \text{ or } 1{,}0.$$

<u>Tension resistance:</u>

$$F_{t.Rd} = \frac{0{,}9\, f_{ub}\, A_s}{\gamma_{Mb}}$$

A is the gross cross-section area of bolt
A_s is the tensile stress area of bolt
d is the bolt diameter
d_o is the hole diameter

After presentation of the evaluation results to the Liaison Engineers and to members of ECCS-TC10 the question was raised whether the new formulae could adversely affect the economy of steel structures. Therefore it was decided to carry out calibration studies. The results according the proposed EC3 strength functions were calibrated against results according to national standards [5], [6].
It was found that EC3 was more liberal than most of the existing codes.
Just as an example two points are raised to show that the EC3 rules for bolts and welds allow for more economic steel structures.

- The end distances and the pitch may be chosen freely within certain limits. Of course the choice of small distances must result in lower bearing resistances. But still this possibility can be of great interest to avoid gusset plates and allow for compact design of joints.
- It is allowed to use bolts with the threaded portion in the shear plane. This allows the use of fully threaded bolts, leading to considerable reduction of bolt types to be kept in stock, improving efficiency and reducing the potential for error.

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