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EC 2: Design for Ultimate Limit States

EC 2: Vérification aux états-limites ultimes

EC 2: Bemessung in den Grenzzuständen der Tragfähigkeit

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SUMMARY

This paper outlines the provisions given in Eurocode 2 for the design of elements for the ultimate limit state. The subjects covered are: design for bending with or without axial force, shear, torsion, punching shear and the effects of structural deformations (buckling). For each mode of behaviour the main features of the methods given in the code are described.

RESUME

Cet article présente les recommandations de l'Eurocode 2 pour la vérification des éléments structuraux aux états-limites ultimes, en particulier les états-limites ultimes pour les sollicitations d'effort normal et de flexion, pour les sollicitations d'effort tranchant, de la torsion, du poinçonnement ainsi que les états-limites atteints par déformation structurale (flambement). Pour chacun des états-limites ultimes, les principes les plus importants dans l'Eurocode 2 sont énoncés.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt die Festlegungen in Eurocode 2 für die Bemessung von Bauteilen in den Grenzzuständen der Tragfähigkeit. Im einzelnen werden behandelt: Bemessung für Biegung mit oder ohne Längskraft; Schub, Torsion, Durchstanzen sowie die Auswirkungen von Tragwerksverformungen (Knicken). Für jeden dieser Grenzzustände werden die wesentlichen Nachweisverfahren in EC 2 beschrieben.

1. INTRODUCTION

The ultimate limit states are treated in chapter 4.3 of the Code. This chapter does not, of course, stand alone, but draws particularly on material in Chapter 2 (Partial Safety Factors and Analysis) and chapter 4.2 (Design Material Properties). Satisfactory design for the ultimate limit state also depends upon applying the provisions of Chapter 5, Detailing Provisions.

The basic principles and methods proposed for the treatment of the ultimate limit states follow closely those given in the CEB Model Code of 1978. The CEB proposals have, however, been amended in detail for various reasons. Firstly, simplicity. The CEB Code, being a Model Code, can afford to develop more complex and rigorous methods than can be done in an operational code. Furthermore, operational codes cannot afford to include too many alternative methods of design. Secondly, development of knowledge. In some areas new research has allowed improvement to the CEB proposals. EC2 has attempted to take account of the latest developments wherever possible.

The organisation of the chapter is as follows:

- Bending and longitudinal force	4.3.1
- Shear	4.3.2
- Torsion	4.3.3
- Punching	4.3.4
- Buckling	4.3.5

Each of these subject areas will be covered briefly in the following sections.

2. BENDING AND LONGITUDINAL FORCE

This section follows very closely the proposals in the CEB Model Code. The design stress-strain curves for ordinary reinforcement and concrete are shown in Figure 1(a) and (b). It should be noted that, for both, possible alternatives are suggested. Figure 1(c) indicates the assumptions relating to the strain distribution at ultimate for reinforced concrete. For prestressed sections, allowance has to be made in assessing the steel strain for the prestrain in the tendons. The indicative (boxed) values given in EC2 for the partial safety factors on the steel and concrete strengths are, respectively, 1.15 and 1.5.

3. SHEAR

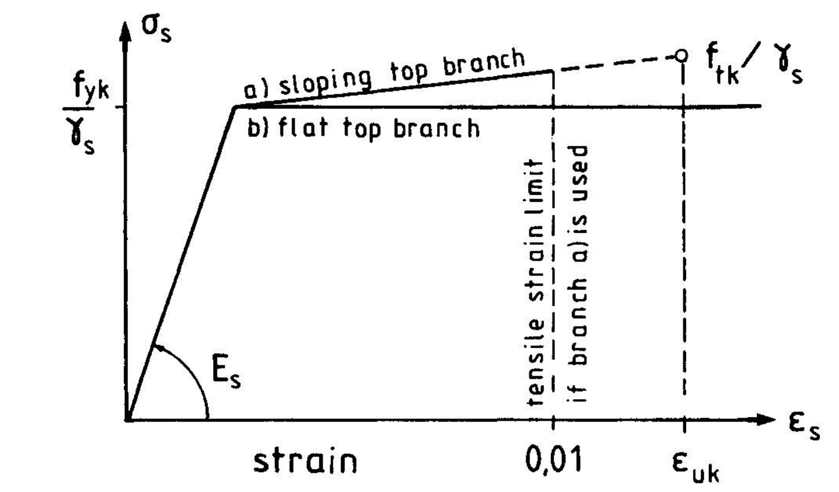
There are three basic values defined for shear resistance. These are:

- V_{Rd1} - the design shear resistance of the member without shear reinforcement
- V_{Rd2} - the maximum design shear force that can be carried without crushing of the notional concrete compressive struts
- V_{Rd3} - the design shear force that can be carried by a member with shear reinforcement

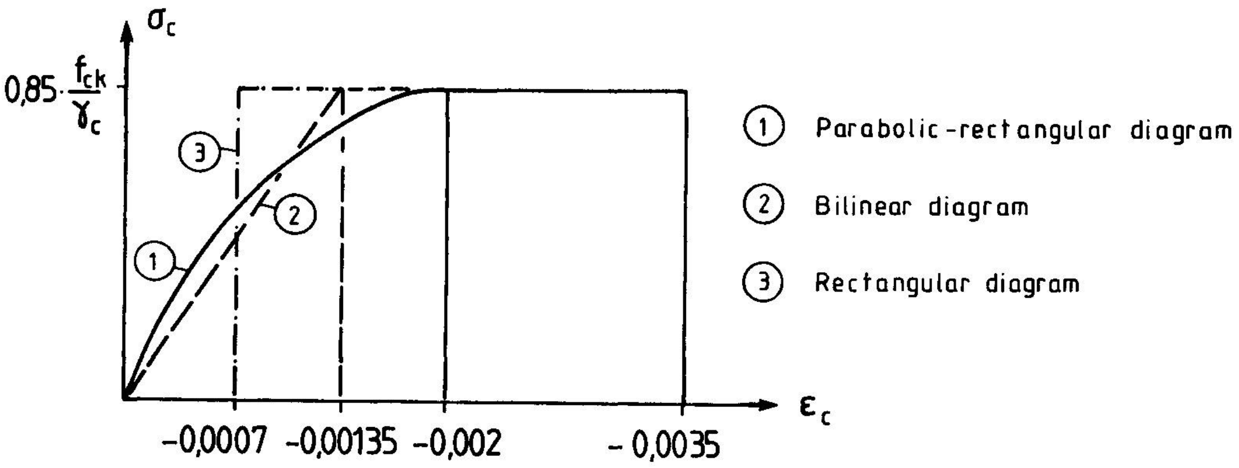
If the design shear, V_{Sd}, is less than V_{Rd1}, only a minimum amount of shear reinforcement need be provided. This minimum may generally be omitted in slabs and members of minor importance.

If V_{Sd} exceeds V_{Rd1}, but is less than V_{Rd2}, then shear reinforcement should be provided so that $V_{Rd3} = V_{Sd}$.

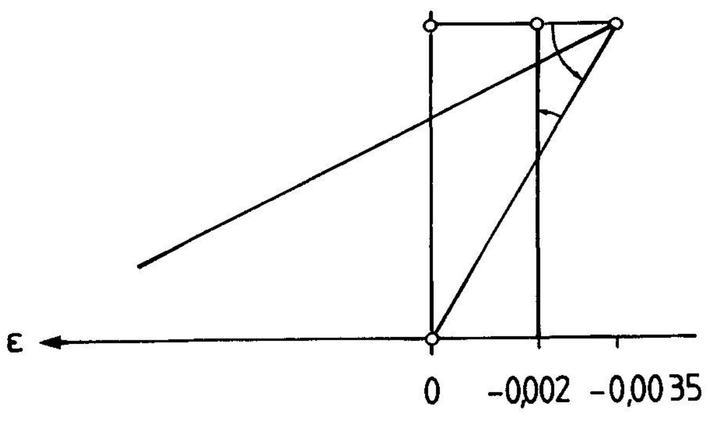
V_{Rd1} is calculated from an empirical relationship which gives the design stress as a function of the tensile strength of the concrete, the reinforcement ratio, the average longitudinal stress, and, for members less than 600 mm deep, the section depth. V_{Rd1} may also be adjusted to allow for enhanced strength close to supports. This relationship has been justified against a very large population of test data.



(a) Design stress-strain curves for reinforcement



(b) Concrete design stress-strain curves



(c) Ultimate strain profiles

Fig. 1 Assumptions for design for flexure

Two methods are given for assessing V_{Rd2} and V_{Rd3}. Both are based on the assumption of a notional truss within the beam where the tension members are formed by the flexural tension reinforcement and the shear reinforcement, while the compressive forces are carried by the concrete in the compression zone and by notional struts within the concrete (see Figure 2).

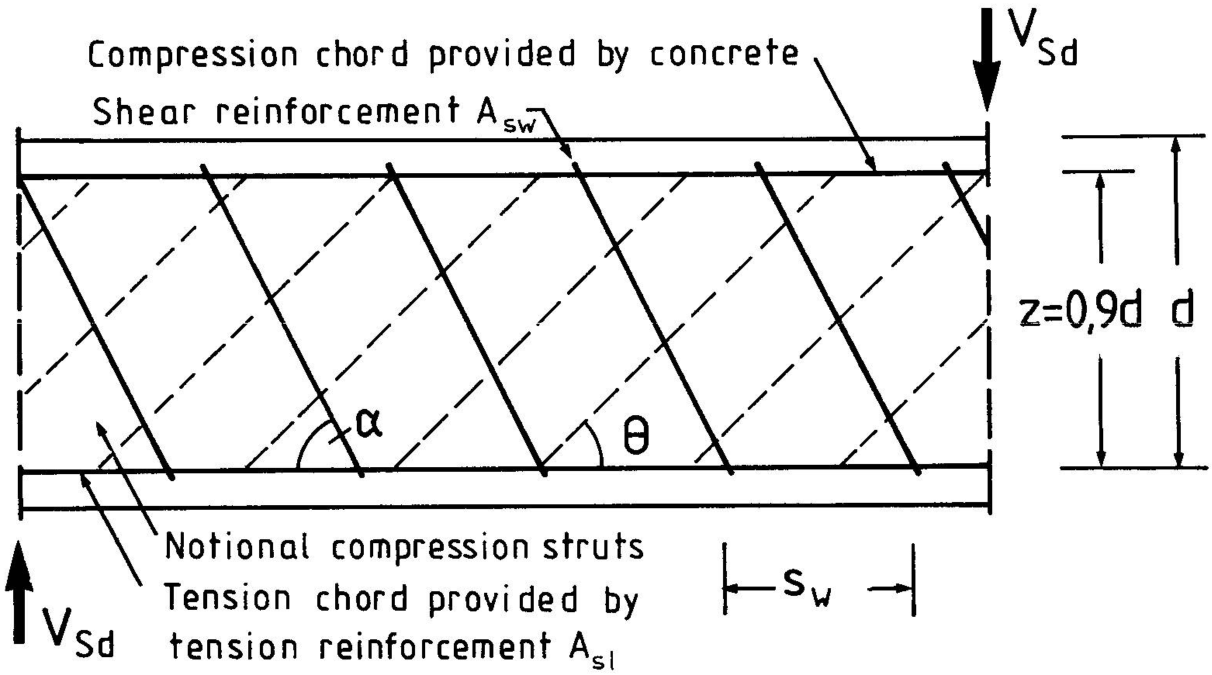


Fig. 2 Assumptions for the calculation of shear reinforcement

In the 'Standard Method', the struts are assumed to be aligned at an angle, θ, of 45° to the axis of the beam and reinforcement is only required to carry the excess shear force above V_{Rd1}. This gives, for vertical stirrups:

$$V_{Sd} = V_{Rd3} = V_{Rd1} + 0.9\ d\ f_{ywd}\ A_{sw}/s \quad (1)$$

and $$V_{Rd2} = 0.45 \nu b_w d\ f_{cd} \quad (2)$$

where
- d is the effective depth
- f_{ywd} is the design strength of the shear reinforcement
- A_{sw} is the cross-sectioned area of the shear reinforcement
- s is the spacing of the shear reinforcement
- b_w is the minimum web breadth
- f_{cd} is the design strength of the concrete
- ν is an empirical effectiveness factor varying from 0.5 to 0.6 over the practical range of concrete strengths

In the 'variable strut inclination method', the angle θ in Figure 2 may be selected by the designer within a range which can be as great as $0.4 < \cot\theta < 2.5$. Once the design shear exceeds V_{Rd1}, all the shear force has to be carried by shear reinforcement. For vertical stirrups, this gives the following relationships for V_{Rd2} and V_{Rd3}:

$$V_{Rd3} = 0.9\, d\, f_{ywd} A_{swc} \cot\theta / s \qquad (3)$$

$$V_{Rd2} = 0.9 b_w d \upsilon f_{cd} / (\cot\theta + \tan\theta) \qquad (4)$$

A possible design procedure is to take either the maximum permitted value of $\cot\theta$ or, if less, the value of $\cot\theta$ which gives $V_{Sd} = V_{Rd2}$ and calculate the amount of shear reinforcement on the basis of this value. It should be noted that the choice of $\cot\theta$ will influence the curtailment of reinforcement.

4. TORSION

The approach adopted for design for torsion is an extension of the 'variable strut inclination method' described above. Two torsional resistances are defined:

T_{Rd1} - the maximum torsion that can be resisted by the compressive struts in the concrete (torsional equivalent of V_{Rd2})

T_{Rd2} - the maximum torsion that can be resisted by the reinforcement (torsional equivalent of V_{Rd3})

Both these quantities are a function of the strut angle, θ and, where combined shear and torsion are considered, the same angle must be chosen for both calculations.

Rules are given for the design of combined shear and torsion or torsion combined with bending and/or axial force. Conditions are also set out for cases where only a minimum area of stirrups is required.

5. PUNCHING

Punching may also be considered as an extension of the shear provisions. A critical perimeter around a column is defined as shown in Figure 3 and the design shear force is assessed for this perimeter.

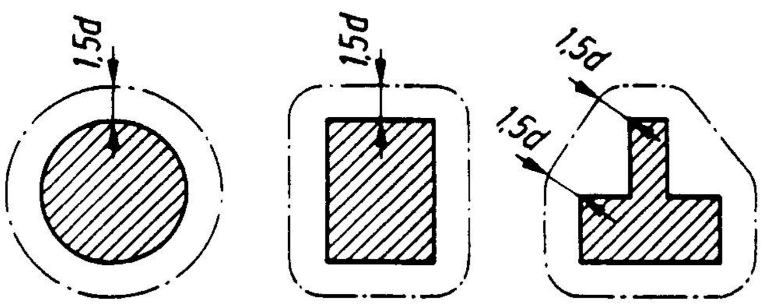


Fig. 3 Perimeters for punching

From this a shear per unit length of the perimeter, v_{Sd}, is calculated from the relation:

$$v_{Sd} = V_{Sd}\,\beta/u \qquad (5)$$

where u is the length of the perimeter
β is a coefficient which takes account of the effects of eccentricity of loading (moment transfer between column and slab)

If v_{Sd} is less than the design shear resistance per unit length of the perimeter for the slab without shear reinforcement, v_{Rd1}, then no shear reinforcement is required. For greater shears, shear reinforcement is required. Shears in excess of 1.6 v_{Rd1} cannot be supported. The expressions for v_{Rd1} and v_{Rd3}, the shear capacity of the slab with shear reinforcement, are effectively the same as for ordinary shear.

There is also a requirement for a minimum design moment in the region of the slab-column connection.

6. ULTIMATE LIMIT STATE INDUCED BY STRUCTURAL DEFORMATION (BUCKLING)

The design procedure envisaged for dealing with slenderness effects is, briefly, as follows:

(i) The structure is classified as:

(a) braced or unbraced

and (b) sway or non-sway

A braced structure is one where all horizontal loads may be assumed to be carried by stiff, bracing elements such as walls.

A sway structure is one where the deflection of the connections has a significant effect on the bending moments.

(ii) Depending on the classification, the vertical members are checked to establish whether they are slender. The effects of deflection may be ignored in non-slender members but must be taken into account in slender members.

(iii) Where necessary, the members are designed to take account of the effects of the deflections.

In non-sway structures, the individual columns are treated as isolated columns which may be assumed to deflect as shown in Figure 4(a). In sway structures, the whole structure will deflect as shown in Figure 4(b). In addition to considering sway of the whole structure, however, it is also necessary to consider the possibility of each column individually deflecting as in Figure 4(a).

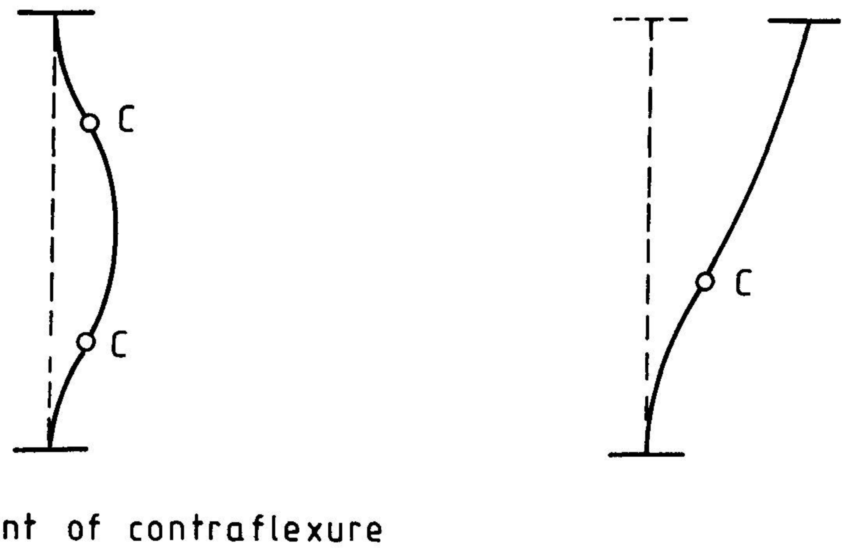
The code only develops a simplified design method for isolated columns. For other situations a more rigorous method is needed and the necessary assumptions for this are set out in Appendix 3. The procedure for isolated braced columns is as follows:

(i) The slenderness ratio $\lambda = l_o/i$ is calculated. l_o is the effective length of the column and i the radius of gyration of the section.

(ii) If $\lambda < 25$, the structure is not slender.

(iii) If $25 < \lambda < 25(2 - e_{o1}/e_{o2})$ then it is only necessary to ensure that the ends of the column can withstand a moment greater than $N_{Sd}\,h/20$. e_{o1} and e_{o2} are, respectively, the numerically smaller and larger end eccentricities.

(iv) If $\lambda > 25(2 - e_{o1}/e_{o2})$ then specific measures have to be taken. A simplified method is given for doing this. This is the 'Model Column Method'. The method makes an estimate of the maximum curvature in the column under ultimate conditions and hence an approximate value for the ultimate deflection.



(a) Assumed deflected shape of an isolated braced column

(b) Assumed deflected shape of a column in a sway structure

Fig. 4 Assumed modes of deflection of columns

The column is then designed to withstand the design vertical load, N_{Sd}, acting at an eccentricity e_{tot}, given by:

$$e_{tot} = e_o + e_a + e_2 \tag{6}$$

where e_o is the initial eccentricity estimated from first order analysis. The value chosen is one appropriate to roughly mid-height of the column

e_a is an accidental eccentricity. It is a nominal figure to allow for possible 'out of plumb' construction of the column

e_2 is the ultimate deflection.

Clearly it will frequently be necessary to consider the possibility of the column deflecting about either axis.

Rules are given for deciding whether or not it is necessary to consider bi-axial bending.

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