DS/EN 1993-1-1 DK NA:2013

National Annex to
Eurocode 3: Design of steel structures -
Part 1-1: General rules and rules for buildings

Foreword

This national annex (NA) is a revision of DS/EN 1993-1-1 DK NA:2010 and replaces the latter on 2013-05-22. For a transition period until 2013-09-01, this National Annex as well as the previous National Annex will be applicable. In addition to minor editorial amendments, complementary information has been added regarding Annex B and the factor $\gamma_0$ has been introduced in clause 6.1.(1).

Previous versions, addenda and an overview of all National Annexes can be found at www.eurocodes.dk

This NA lays down the conditions for the implementation in Denmark of EN 1991-3-1 for construction works in conformity with the Danish Building Act or the building legislation. Other parties can put this NA into effect by referring thereto.

This NA includes:

- an overview of possible national choices and clauses containing complementary information;
- national choices;
- complementary (non-contradictory) information which may assist the user of the Eurocode.

The numbering refers to the clauses of the Eurocode where national choices have been made and/or complementary information is given. To the extent possible, headings are identical to the headings of the clauses in the Eurocode followed by a clarification, as appropriate.
Overview of possible national choices and clauses containing complementary information

The list below identifies the clauses where national choices are possible and the applicable/not applicable informative annexes. Furthermore, clauses giving complementary information are identified. Complementary information is given at the end of this document.

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NOTE Unchanged: No national choice is made, and recommendations in the standard are followed.
National choices

3.1(2) Materials, General
The standard applies to steel materials in accordance with Table 3.1 of EN 1993-1-1 or equivalent.

3.2.1(1) Material properties
The values of $f_y$ and $f_u$ specified in (1) a) should be used.

5.2.1(3) Effects of deformed geometry of the structure
A lower value of $a_{cr}$ than that that given in (5.1) may be used if justification of its application is documented.

5.3.2(11) Imperfections for global analysis of frames
Which of the methods referred to in (3), (6) and (11) to use should be determined for each individual case.

6.1(1) Ultimate limit states, General
The below expressions for $\gamma_{Mi}$ are used, including the factor ($\gamma_0$) on the partial factors for strength parameters and resistances, cf. National Annex to EN 1990, Table A1.2(B+C):

$$\gamma_{M0} = 1,1 \cdot \gamma_0 \cdot \gamma_3$$
$$\gamma_{M1} = 1,2 \cdot \gamma_0 \cdot \gamma_3$$
$$\gamma_{M2} = 1,35 \cdot \gamma_0 \cdot \gamma_3$$

The factor $\gamma_0$ takes into account the combination of actions, cf. National Annex to EN 1990, Table A1.2(B+C).

<table>
<thead>
<tr>
<th>Limit state</th>
<th>STR/GEO</th>
<th>STR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combination of actions</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>$\gamma_0$</td>
<td>1,0</td>
<td>1,0</td>
</tr>
</tbody>
</table>

The factor $\gamma_3$ takes into account the level of checking of the product. The reduced level of checking is not used.

Extended level of checking: $\gamma_3 = 0,95$
Normal level of checking: $\gamma_3 = 1,00$

The partial factors are determined in accordance with the National Annex to EN 1990, Annex F, where $\gamma_M = \gamma_1 \gamma_2 \gamma_3 \gamma_4$, where the values of $\gamma_{Mi}$ given above include the factor $\gamma_0$. 

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\( \gamma_1 \) takes into account the type of failure
\( \gamma_2 \) takes into account the uncertainty related to the design model
\( \gamma_3 \) takes into account the extent of checking
\( \gamma_4 \) takes into account the variation of the strength parameter or resistance.

When determining \( \gamma_1 \), the following types of failure have been assumed:

\[
\begin{align*}
\gamma_{M0} & : \text{Warning of failure with residual resistance} \\
\gamma_{M1} & : \text{Warning of failure without residual resistance} \\
\gamma_{M2} & : \text{No warning of failure}
\end{align*}
\]

For accidental and seismic design situations the following values are used:

\[
\begin{align*}
\gamma_{M0} & = 1.0 \\
\gamma_{M1} & = 1.0 \\
\gamma_{M2} & = 1.0
\end{align*}
\]

6.1(1) NOTE 2B  Ultimate limit states, General
See clause 6.1(1)

6.3.2.3(2) Lateral torsional buckling curves for rolled sections or equivalent welded sections
\( f = 1 \). The determination of \( M_{cr} \) takes into account the moment distribution between lateral restraints.

6.3.3(5) Uniform members in bending and axial compression
Both Method 1 and Method 2 may be used to determine the values of the interaction factors \( k_{yy}, k_{yz}, k_{zy}, \) and \( k_{zz} \). See also the complementary information.

6.3.4(1) General method for lateral and lateral torsional buckling of structural components
The relevance of using the method in 6.3.4 is to be evaluated for each case.

7.2.1(1)B Vertical deflections
For beams, the following values of the maximum deflection (\( w_3 \) in EN 1990, Figure A1.1) due to one variable action without allowance for impact, if any, may serve as guidance as to what may be regarded as acceptable deflections:

- floors \( \frac{l}{400} \)
- roofs and external walls \( \frac{l}{200} \)
Where

\[ l \] is the span of simply supported and continuous beams, or twice the projection of cantilevered structures.

The values apply both to main and secondary elements, but only the deflection of the element considered is to be used in the assessment.

For secondary sheeting in the form of uninsulated roof sheeting and for facade sheeting, the deflection due to permanent and variable actions should not exceed \( l/90 \).

For roof sheeting with external insulation and roofing felt, the deflection due to permanent and variable actions should not exceed:

\[
\begin{align*}
& l/150 & \text{for} & & l < 4500 \text{ mm} \\
& 30 \text{ mm} & \text{for} & & 4500 \text{ mm} \leq l < 6000 \text{ mm} \\
& l/200 & \text{for} & & 6000 \text{ mm} \leq l 
\end{align*}
\]

7.2.1(1)B Horizontal deflections

For columns, the following numerical values of the maximum deflection of the column head due to one variable action may serve as guidance to what may be regarded as acceptable deflections:

- frames in buildings without cranes \( h/150 \)
- columns in single-storey skeleton structures \( h/300 \)
- columns in multi-storey skeleton structures, for each storey \( h/300 \) for the total height \( h_e/500 \)

Where

\[ h \] is the height of the individual column
\[ h_e \] is the total height of the building.

BB.1.3(3)B Hollow sections as members

Further information on buckling lengths of compression members should be found in textbooks.
Complementary (non-contradictory) information

2.1.2 Reliability management (choice of execution classes)
The execution standard EN 1090-2 specifies 4 execution classes, EXC1 – EXC4, for which requirement strictness increases from EXC1 to EXC4. It is the responsibility of the designer to choose the requisite execution class for each individual structural component and connection. Generally, adequate quality of a structure is achieved when it is executed in accordance with execution class EXC2, if the normal consequences class (CC2) has been selected for the design of the structure in conformity with EN 1990.

For structures assigned to a lower consequences class (CC1) in conformity with EN 1990, it will be possible to relax the execution class requirement to EXC1. For the execution of welds, EN 1993-1-8 requires a weld of at least quality level C. Therefore, at least execution class EXC2 should be applied for the welded connections in the structure. As guidance, the execution class of the structure may be selected according to the table below.

<table>
<thead>
<tr>
<th>Consequences class</th>
<th>Recommended execution class, excluding welds</th>
<th>Recommended execution class, welds for steel types up to and including S355</th>
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<td>EXC1</td>
<td>EXC2</td>
</tr>
<tr>
<td>CC2</td>
<td>EXC2</td>
<td>EXC2</td>
</tr>
<tr>
<td>CC3</td>
<td>EXC3</td>
<td>EXC3</td>
</tr>
<tr>
<td>CC1, subject to fatigue</td>
<td>EXC2</td>
<td>EXC3</td>
</tr>
<tr>
<td>CC2, subject to fatigue</td>
<td>EXC3</td>
<td>EXC3</td>
</tr>
<tr>
<td>CC3, subject to fatigue</td>
<td>EXC4</td>
<td>EXC4</td>
</tr>
</tbody>
</table>

For particularly critical structural components and/or connections consideration should be given to adopting more rigorous the execution requirements.

5.2.2(8) Structural stability of frames
Detailed guidelines are not given for structural analyses of the stability of frames using a method based on equivalent buckling lengths. Guidance should be found in specialist literature or the method of analysis should be documented by other means.
Justification for changing 6.3.2.3(2) Lateral torsional buckling curves for rolled sections or equivalent welded sections
The specified method assumes (cf. ECCS Publication 119) that when calculating $M_{cr}$ and consequently $\lambda_{LT}$, a uniform moment distribution between the lateral restraints is taken into account corresponding to $\Psi = 1$ in Table 6.6, and not as in 6.3.2.2 the real moment distribution. The real moment distribution has been taken into account by the factor $f$. The text of the change specifies that also when using this method, $M_{cr}$ is to be determined on the basis of the real moment distribution between the lateral restraints, and $f$ shall be taken as 1.

6.3.3(5) Uniform members in bending and axial compression
Method 1 is recommended for significant structures and where cost is decisive, and as a basis for the preparation for design programs.
Method 2 is recommended as a simpler method for less significant structures.
See also the national choice.

Annex B Method 2 – Interaction factors $k_{ij}$ for the interaction expressions in 6.3.3(4)
Table B.3: Equivalent uniform moment factors $C_m$ in Tables B.1 and B.2

$M_s$ is to be obtained according to the value of bending moment diagram which yields a local extreme ($dM_s/dx = 0$) between end points of beam elements ($x = 0$ and $x = L$). Where a local extreme does not exist, $M_s$ is to be taken as the value obtained at the centre of the beam element.