Second Generation of Eurocode 8

Eurocode 8 — Design of structures for earthquake resistance
Part 3: Assessment and retrofitting of buildings and bridges
Chapter 6: Modelling, structural analysis and verification

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Standards addressed in the presentation

prEN_1998-3_2022_ENQ: Eurocode 8 — Design of structures for earthquake resistance — Part 3: Assessment and retrofitting of buildings and bridges, Date: 2022-11, N1236

FprEN 1998-1-1:2024_TC 250: Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action, Date: 2023-08, N1283

prEN_1998-1-2_2022_ENQ: Eurocode 8 — Design of structures for earthquake resistance — Part 1-2: Buildings, Date: Date: 2022-11, N1235

prEN1998-2_draft_post-ENQ_46th_meeting: Eurocode 8 — Design of structures for earthquake resistance — Part 2: Bridges, Date: 2023-09, N1286

Scope of Chapter 6

6.2 Modelling

6.3 in 6.4 Analysis

6.5 Safety verification

Two assessment approaches

Displacement based approach – DBA (Types of analyses 1- 3)

Force based approach – FBA (4. type of analysis)

Four limit states

NC – Near collapse

SD – Significant damage

DL – Damage Limitation

OP – Operational Limit States

Four general types of analysis

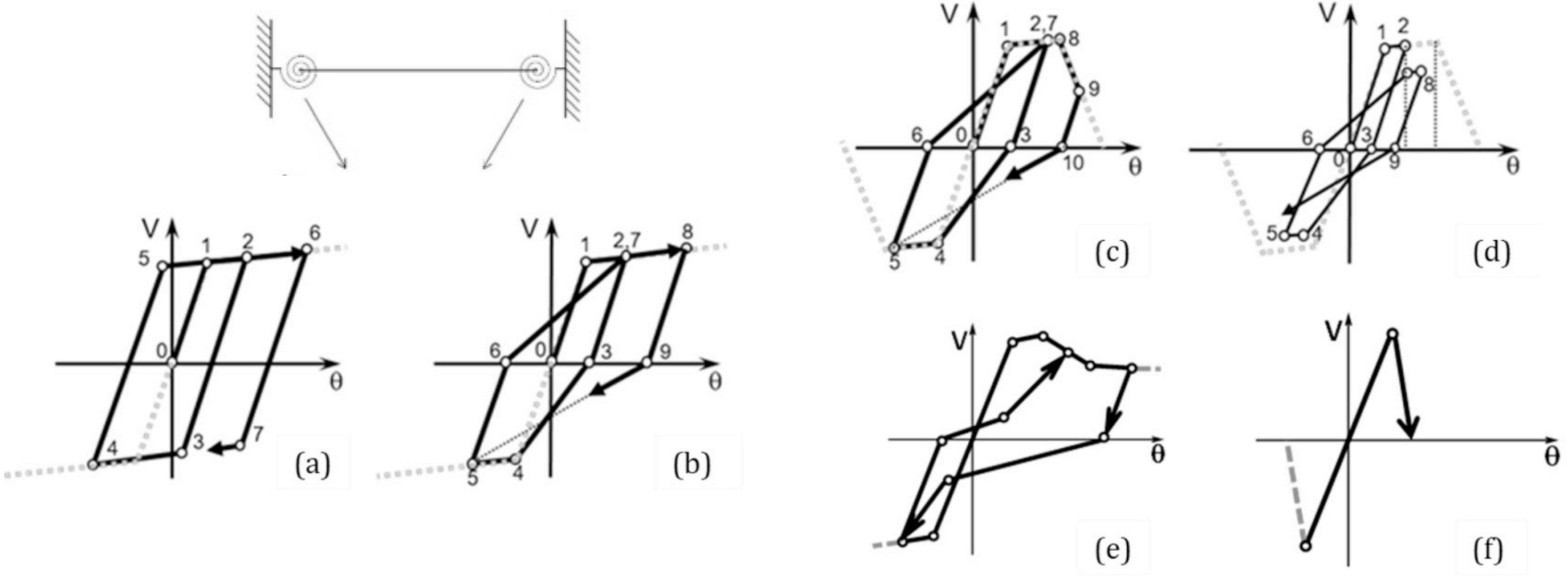
1. **Simplified Nonlinear Static**
2. Nonlinear Response History
3. Linear Elastic - Unreduced Acc. Spectrum
4. Linear Elastic – Reduced Acc. Spectrum

6.2 Modelling

- All already observed damage should be adequately included in the numerical model
- The distinction should be made between upgrading undamaged structures and the retrofit and repair of earthquake-damaged structures

- FBA: Mass, stiffness and damping should be properly modelled
- DBA: Mass, stiffness, damping, strength and deformation capacity
- **Mean material properties**
- Exception: **new materials** in elements withstanding all seismic action effects, which are **verified using FBA**

6.2 DBA approach – Nonlinear analysis (Response history or Simplified static - pushover based)



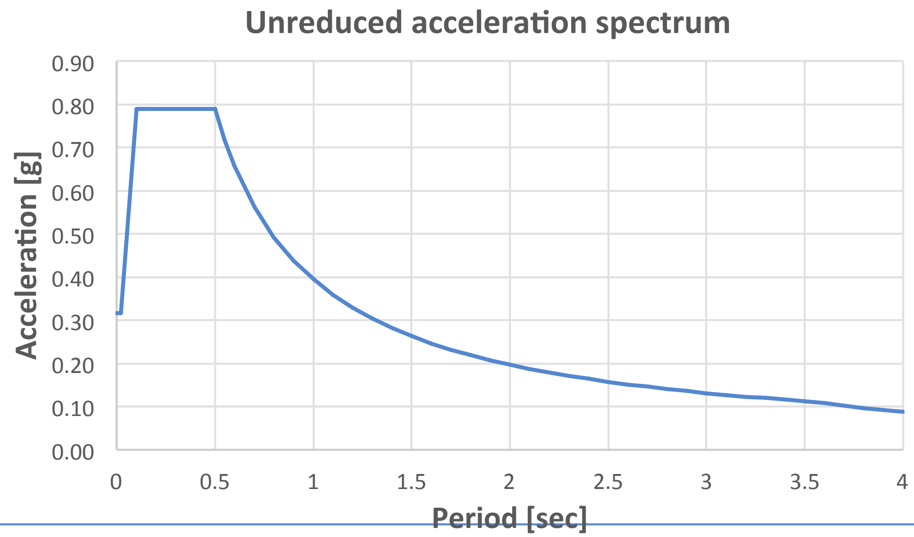
prEN1998-1-1 Chapter 7

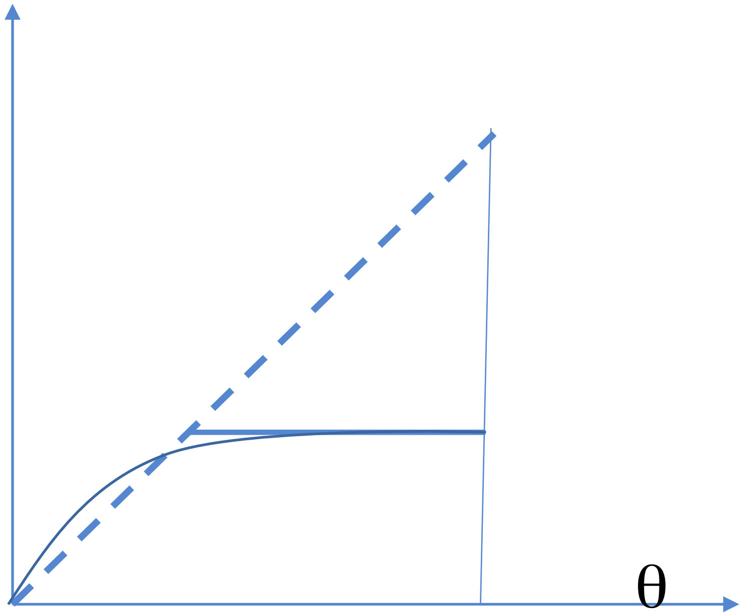
Existing structures: Cyclic degradation to be modelled

6.2 Simplified DBA approach – Elastic Analysis with Unreduced Spectrum

- **Elastic secant effective stiffness corresponding to elastic limit state**

- **Simplification:**
- **Preliminary analysis – (informative Annex A):** the elastic flexural and shear stiffness properties of concrete members - 25% of the stiffness of the uncracked members – demand chord rotations





6.2 FBA approach (Elastic linear analysis – reduced spectrum) – Modelling stiffness

- Elastic secant effective stiffness corresponding to elastic limit state
- Concrete members: Cracked sections at the initiation of reinforcement yielding

- Simplifications in Buildings (prEN 1998-1-1 6.2, prEN1998-1-2 5.1)
- Simplifications in Bridges (prEN 1998-1-1 6.2, prEN1998-2 5.1):

6.4.2 DBA approach: Nonlinear static analysis

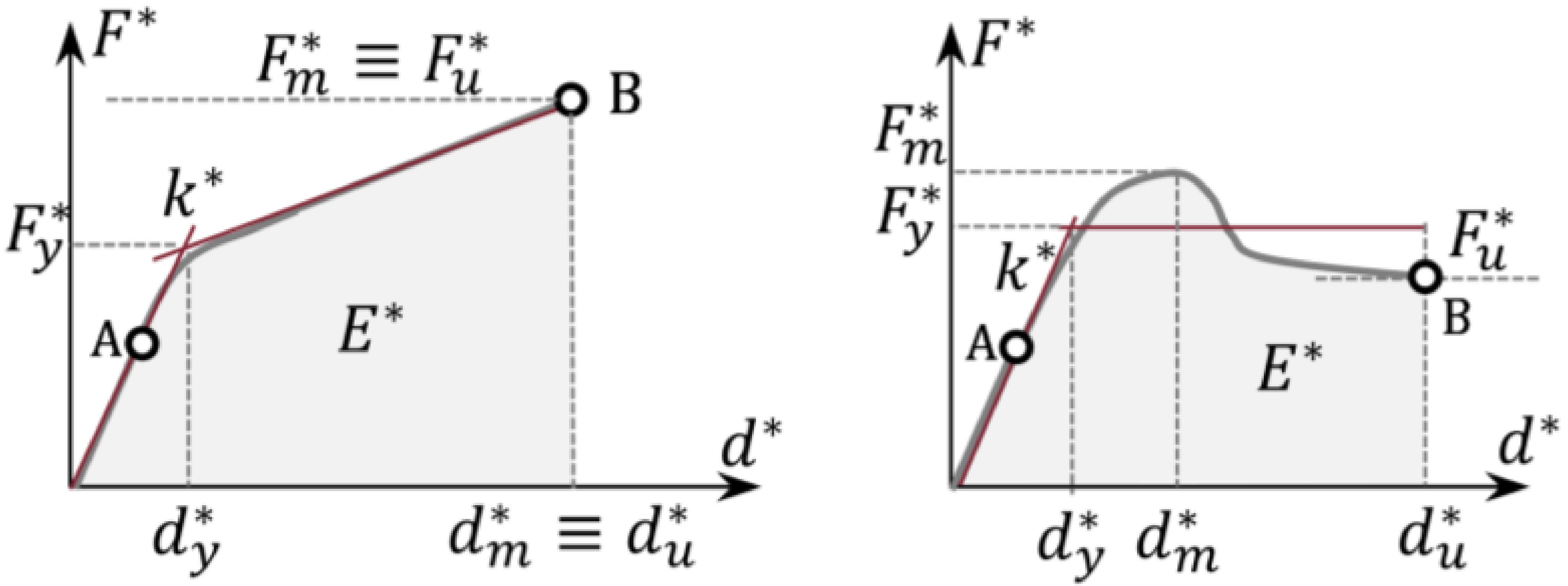
Based on the pushover analysis of the equivalent SDOF system

- General Rules: prEN 1998-1-1 6.5 and Annex D
- Rules for buildings: prEN 1998-1-2 5.3.5
- Rules for bridges: prEN 1998-2 5.2.3

Lateral load proportional to the fundamental mode of vibration
Existing structures:
Buildings with soft storey – additional analysis with uniform load pattern
Special rules for masonry buildings
Corrections due to the torsion and higher modes (C_{pk}, C_{Ei})

6.4.2 DBA approach: Nonlinear static analysis

Idealisation of the pushover curve prEN1998-1-1 6.5.3(2) Fig. 6.1



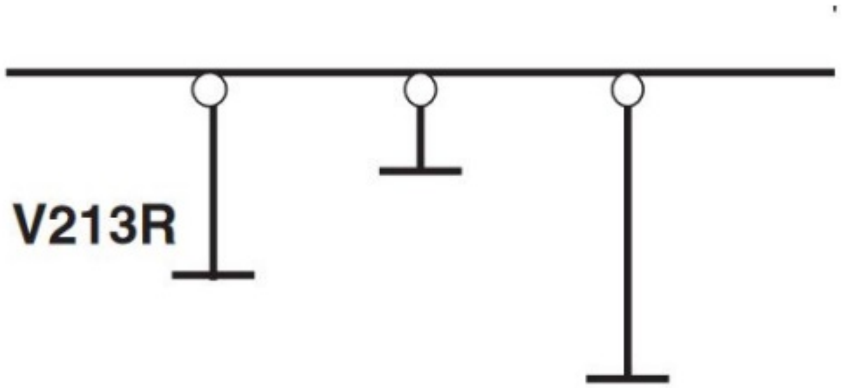
6.4.2 DBA approach: Nonlinear response history analysis

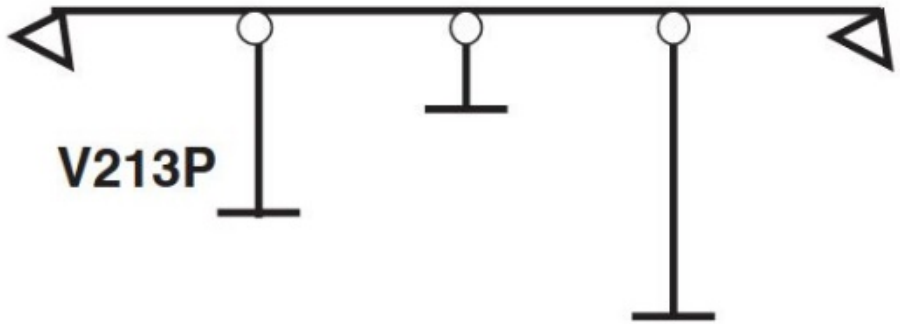
Very irregular structures:

Examples:

Bridges – the effective mass of fundamental mode of vibration <60%,

Curved bridges





6.4.3 DBA approach: Nonlinear response history analysis

Selection of the set of accelerograms:

prEN1998-1-1 Annex D
Criteria for selection and scaling of input motions (informative)

6.4.1 DBA approach: Elastic linear analysis with unreduced acceleration spectrum

Simplified DBA:

Can be used only for regular buildings and bridges, where notable redistribution of demand between structural elements is not expected

$$\rho_i = \frac{E_{d,i}}{R_{d,i}}$$

$$\rho_{max} / \rho_{min} \le \begin{array}{l} 2.5 \text{ in buildings} \\ 2.0 \text{ in bridges} \end{array}$$

6.3.1 FBA approach: Elastic linear analysis with reduced acceleration spectrum

- prEC8-3 4.1(4) Allowed for low and moderate seismic action class

- q = 1.5 Reinforced concrete, Timber, Masonry
- q = 2.0 Steel

- Account for only an overstrength

6.3.1 FBA approach: Linear elastic analysis with reduced acceleration spectrum

- prEC8-3 4.1(4) Allowed for low and moderate seismic action class

- Seismic action class is defined in prEC8-1-1 4.1(4)

$$S_{\delta} = \delta F_{\alpha} F_{T} S_{\alpha,475}$$

δ	is a coefficient that depends on the consequence class of the considered structure;
F_{α}	is the site amplification factor given in 5.2.2.2(5);
F_{T}	is the topography amplification factor given in 5.2.2.2(10);
$S_{\alpha,475}$	is the reference maximum spectral acceleration for the return period of 475 years (see 5.2.1(2)).

Table 4.1 — Range of S_δ values to define seismic action classes

Seismic action class	Range of seismic action index
Very low	$S_\delta < 1{,}30\ m/s^2$
Low	$1{,}30\ m/s^2 \leq S_\delta < 3{,}25\ m/s^2$
Moderate	$3{,}25\ m/s^2 \leq S_\delta < 6{,}50\ m/s^2$
High	$S_\delta \geq 6{,}50\ m/s^2$

CC2, $\delta = 1.0$, $F_T = 1.0$

Soil B $F_\alpha = 1.26$ $S_{\alpha,475} = 0.54g$ or $5.3\ m/s^2$ PGA = $0.21g$ or $2.1\ m/s^2$

Soil C $F_\alpha = 1.45$ $S_{\alpha,475} = 0.46g$ or $4.5\ m/s^2$ PGA = $0.18g$ or $1.8\ m/s^2$

Soil D $F_\alpha = 1.57$ $S_{\alpha,475} = 0.42g$ or $4.2\ m/s^2$ PGA = $0.17g$ or $1.7\ m/s^2$

6.5 Safety verification

6.5.1 General

Action effects should be multiplied by

γ_{Sd} = 1.00 – undamaged structure - 4.2.2(5)

γ_{Sd} = 1.15 – damaged structure - 4.2.2(5)

6.5 Safety verification

6.5.2 NC limit state

General prEN 1998-1-1 4.3(1)
Buildings prEN1998-1-2 4.2(1) and Bridges prEN1998-2 4.2.1

Return period $T_{LS,CC}$ (NDP)
Consequence class CC2 - 1600 years

Performance factor $\gamma_{LS,CC}$ (NDP)
Consequence class CC2 – 1.5

6.5 Safety verification

6.5.2 NC limit state

Verification in local and global terms

Verification in local terms shall be carried out when linear elastic or nonlinear response history analysis is used

Verification in global terms may be carried out when nonlinear static analysis is used: masonry buildings and buildings in which infills dominate their global capacity – post-peak strength degradation is modelled on the member level

6.5 Safety verification

6.5.2 NC limit state

Verification in local terms

Verification of ductile mechanisms – in terms of generalised deformations prEN 1998-1-1 6.7.3(2), 7

Resistance

$$\delta_{\mathrm{NC}} = \frac{1}{\gamma_{\mathrm{Rd}}}\left(\delta_{\mathrm{y}} + \delta_{\mathrm{u}}^{pl}\right)$$

γ_{Rd} – prEN 1998-1-2 and prEN 1998-2 for different types of response mechanisms

Local verification of failure modes, which are not directly captured in the model, should be performed

6.5 Safety verification

6.5.2 NC limit state

Verification in local terms

Verification of brittle mechanisms – in terms of generalised stresses
prEN 1998-1-1 6.7.3(3), 7

Shear resistance

$$V_{\mathrm{R,NC}} = \frac{V_{\mathrm{R}}}{\gamma_{\mathrm{Rd}}}$$

γ_{Rd} – prEN 1998-1-2 and prEN 1998-2 for different types of response mechanisms
Seismic action effects are defined considering the resistance of non-brittle mechanisms multiplied by γ_{Sd} and overstrength factor

6.5 Safety verification

6.5.2 NC limit state

6.5.2.4 Verification in global terms using nonlinear analysis

Verification in global terms may be carried out when nonlinear static analysis is used: masonry buildings and buildings in which infills dominate their global capacity – post-peak strength degradation is modelled on the member level

Allowed when correction factors $c_{P,k}$ and $c_{E,i}$ (corrections accounting for the influence of torsion and higher modes) do not exceed 1.2.

Local verification of failure modes, which are not directly captured in the model, should be performed

6.5 Safety verification

6.5.3 Other limit states

SD – prEN 1998-1-1 6.7.2
Resistance of ductile mechanisms

$$\delta_{\mathrm{SD}} = \frac{1}{\gamma_{\mathrm{Rd}}}\left(\delta_{\mathrm{y}} + \alpha_{\mathrm{SD},\theta}\delta_{\mathrm{u}}^{pl}\right)$$

$\alpha_{\mathrm{SD},\theta} = 0.5$

Shear resistance

$$V_{\mathrm{R,SD}} = \frac{V_{\mathrm{R}}}{\gamma_{\mathrm{Rd}}}$$

DL – prEN 1998-1-1 6.7.3
OP - prEN 1998-1-1 6.7.3