Free-standing chimneys —

Part 2: Concrete chimneys

The European Standard EN 13084-2:2007 has the status of a British Standard

ICS 91.060.40

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National foreword

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Contents

page

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Foreword

This document (EN 13084-2:2007) has been prepared by Technical Committee CEN/TC 297 "Free-standing industrial chimneys", the secretariat of which is held by DIN.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by November 2007, and conflicting national standards shall be withdrawn at the latest by November 2007.

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This document supersedes EN 13084-2:2001.

This European Standard is part 2 of a series of standards as listed below:

— EN 13084-1, *Free-standing chimneys - Part 1: General requirements*.

— EN 13084-2, *Free-standing chimneys - Part 2: Concrete chimneys*.

— EN 13084-4, *Free-standing chimneys - Part 4: Brick liners – Design and execution*.

— EN 13084-5, *Free-standing chimneys - Part 5: Material for brick liners - Product specifications.*

— EN 13084-6*, Free-standing chimneys - Part 6: Steel liners - Design and execution.*

— EN 13084-7, *Free-standing chimneys – Part 7: Product specifications of cylindrical steel fabrications for use in single wall steel chimneys and steel liners.*

— EN 13084-8, *Free-standing chimneys – Part 8: Design and execution of mast construction with satellite components*

Additionally applies

— EN 1993-3-2, *Eurocode 3: Design of steel structures – Part 3-2: Towers, masts and chimneys – Chimneys.*

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1 Scope

This European Standard specifies particular requirements and performance criteria for the design and construction of cast-in-situ concrete chimneys as well as prefabricated concrete chimneys. It identifies requirements to ensure the mechanical resistance and stability of concrete chimneys in accordance with the general requirements given in EN 13084-1.

As for chimneys attached to buildings the criteria given in Clause 1 of EN 13084-1:2000 apply.

Unless otherwise stated in the following clauses the basic standard for the design of concrete structures, EN 1992-1-1, applies.

2 Normative references

The following referenced documents are indispensable for the application of this European Standard. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 206-1:2000, *Concrete – Part 1: Specification, performance, production and conformity*

EN 1520:2002, *Prefabricated reinforced components of lightweight aggregate concrete with open structure*

EN 1990, *Eurocode - Basis of structural design*

EN 1992-1-1:2004, *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*

EN 1992-1-2, *Eurocode 2: Design of concrete structures – Part 1-2: General rules - Structural fire design*

EN 12446, *Chimneys - Components - Concrete outer wall elements*

EN 13084-1:2000, *Free-standing chimneys - Part 1: General requirements*

EN 13084-4, *Free standing chimneys – Part 4: Brick liners – Design and execution*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 13084-1:2000 and the following apply.

3.1
prefabricated element
precast member of normal-weight or light-weight concrete, reinforced or not, which completely encloses the flues of chimney stacks

3.2
reinforcement ducts
route for the continuous vertical reinforcement

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4 Materials

4.1 Concrete

4.1.1 Normal-weight concrete for cast-in-situ chimneys

The concrete strength classes given in EN 206-1 but not less than C25/30 can be used for cast-in-situ chimneys. For environmental conditions EN 206-1 applies.

NOTE Higher strength classes than C25/30 should only be used if it would be required by environmental conditions in accordance with EN 1992-1-1:2004, Table E.1N and no special provisions for corrosion protection of reinforcement and protection of concrete attack need to be taken.

4.1.2 Concrete for prefabricated chimneys

4.1.2.1 Normal-weight concrete

See 4.1.1

4.1.2.2 Light-weight aggregate concrete

The light-weight aggregate concrete for prefabricated elements shall correspond to the density-class D 1,2 or higher in accordance with Table 9 of EN 206-1:2000.

The light-weight aggregate concrete with closed structure for prefabricated elements shall correspond to the strength classes given in EN 206-1. For environmental conditions EN 206-1 applies.

NOTE Higher strength classes than LC25/28 should only be used if it would be required by environmental conditions in accordance with EN 1992-1-1:2004, Table E.1N and no special provisions for corrosion protection of reinforcement and protection of concrete attack need to be taken.

The light-weight aggregate concrete with open structure for prefabricated elements shall correspond to the strength classes given in EN 1520 but strength classes less than LAC 8 shall not be used. For environmental conditions see EN 1520: 2002, 5.8.2.

4.1.2.3 Reinforcement duct infill concrete

The reinforcement duct infill concrete shall have at least the same strength class as the prefabricated elements, but not more than the next higher strength class. The flow class for consistence shall not be less than F3 in accordance with Table 6 of EN 206-1:2000 and the maximum size of aggregate shall not be greater than 8 mm.

4.1.2.4 Outer wall elements

Prefabricated outer wall elements shall be in accordance with EN 12446.

4.2 Mortar for bedding of prefabricated elements

The mortar of bedding joints between prefabricated elements shall have the same strength class as the concrete of the prefabricated elements.

4.3 Reinforcing steel

For reinforcing steel the specifications given in EN 1992-1-1 apply.

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5 Material properties

The material properties of concrete and reinforcing steel for normal temperature design shall be taken from EN 1992-1-1 or EN 1520 with the exception of the mean tensile strength of concrete, f_{ctm}, which shall be calculated in accordance with Equation (1). The influence of elevated temperatures on the mechanical and thermal properties of concrete and reinforcing steel shall be determined from EN 1992-1-2.

$$f_{ctm} = c_c \times c_\beta \times c_v \times c_\eta \, f_{cm}^{0,67} \text{ , in N/mm}^2 \tag{1}$$

where:

c_c is the concrete density factor;

$$c_c = 0,4 + 0,6 \frac{\rho}{2200} \tag{1a}$$

c_β is the concrete strength factor;

$$c_\beta = 0,45 \tag{1b}$$

c_v is the predamage factor;

$$c_v = 0,85 - 0,2 \times t \tag{1c}$$

c_η is the eccentricity factor;

$$c_\eta = \frac{0,6 + 6 \times c_t \times \eta}{1,0 + 6 \times \eta} \tag{1d}$$

c_t is the wall thickness factor;

$$c_t = \frac{2,6 + 24 \times t}{1,0 + 40 \times t} \tag{1e}$$

f_{cm} is the mean compressive strength of concrete;

$$f_{cm} = f_{ck} + 8 \quad \text{in Newtons per square millimetres} \tag{1f}$$

f_{ck} is the characteristic compressive strength (cylinders) of concrete in Newtons per square millimetres;

ρ is the density of concrete;

for normal-weight concrete:

ρ = 2200 kg/m^3

for light-weight concrete:

ρ is the design value of the density of concrete corresponding to the density class in accordance with Table 11.1 of EN 1992-1-1:2004,

η is the eccentricity;

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$$\eta = \frac{M}{N \times t} \tag{1g}$$

M is the design value of the bending moment in the cross-section concerned in Newton metres;

N is the design value of the axial force in the cross-section concerned in Newtons;

t is the wall thickness of the cross-section concerned in metres;

For lightweight-aggregate concrete the symbols f_{ctm}, f_{cm} and f_{ck} shall be replaced by f_{lctm}, f_{lcm} and f_{lck} respectively.

6 Structural design

6.1 Actions

Actions to be considered are given in EN 13084-1. For design values of actions and combination of actions see EN 1990.

6.2 Effect of actions

6.2.1 General

The effect of actions in both horizontal and vertical cross-sections have to be calculated taking into account moments of 2nd order.

The windshield can be treated in accordance with the beam theory being subjected to axial forces, bending moments and thermal effects.

If effects on the stability are expected, the influences from creep, shrinkage and cracking shall be taken into account.

6.2.2 Partial safety factors

Partial safety factors for actions shall be applied.

NOTE The values of the partial safety factors for actions in the ultimate limit state for use in a Country may be found in its National Annex.

The recommended partial safety factors for actions are:

a) permanent actions
   - unfavourable effect γ_G = 1,35
   - favourable effect γ_G = 1,0

b) wind actions γ_W = 1,5

c) thermal effects γ_T = 1,5

d) seismic actions γ_E = 1,0

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6.2.3 Moments of 2nd order

6.2.3.1 General

For the determination of moments of 2nd order the mean values of the material properties may be used. In the concrete compression zone, the following linear material law has to be used:

$$\sigma = E_{\text{cm}} \times \varepsilon \tag{2}$$

where:

σ is the stress;

E_{cm} is the modulus of elasticity of concrete;

ε is the strain.

The stiffening effect of the concrete in the tension zone may be taken into account in chimneys with continuous vertical reinforcement.

6.2.3.2 Approximate method

The approximation is based on the following assumptions:

— full utilization of the cross-sections with respect to the local load carrying capacity;

— consideration of the tension stiffening effects of the concrete;

— chimney height less than 300 m;

— no consideration of deflection effects due to imperfections and rotation of the foundation;

— constant diameter and wall thickness or nearly linear reduction of one or both of them over the chimney height.

The design value of the 2nd order moment may be calculated as follows:

— for windshields with continuous vertical reinforcement:

$$M^{\text{II}}(z) = M^{\text{I}}(z) + M^{\text{I}}(0) \times \frac{(85 - 0{,}14 \times h)\alpha^2}{100} \times \left(1 + 2{,}4 \times \frac{z}{h}\right) \times \left(1 - \frac{z}{h}\right)^{2{,}4} \tag{3}$$

— for windshields without continuous vertical reinforcement with the value of α according to Equation (5) not exceeding 0,6:

$$M^{\text{II}}(z) = (1 + \kappa \times \alpha^2) \times M^{\text{I}}(z) \tag{4}$$

where:

$M^{\text{II}}(z)$ is the design value of the 2nd order bending moment at height z;

$M^{\text{I}}(0)$ is the design value of the 1st order bending moment at the chimney base;

$M^{\text{I}}(z)$ is the design value of the 1st order bending moment at height z;

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z is the height at the considered cross-section above the top level of the foundation;

h is the height of chimney above the top level of the foundation;

$$\alpha = h\sqrt{\frac{N}{E_{cm} \times I}} \tag{5}$$

κ =0,5, if the horizontal joints do not open deeper than the centre of gravity;

κ = 0,75, if the horizontal joints open deeper than the centre of gravity;

N is the design value of the axial force at the chimney base;

E_{cm} is the modulus of elasticity of concrete;

I is the 2nd moment of area at the chimney base of the uncracked cross-section ignoring reinforcement;

$$I = \pi \times d_m^3 \times \frac{t}{8} \tag{5a}$$

d_m is the mean windshield diameter at the chimney base;

t is the wall thickness at the chimney base.

6.2.3.3 Rotation of foundation

Rotation of the foundation causes moments of 2nd order in the windshield.

For the determination of the angle of rotation θ, Equations (6) or (7) apply:

— for a circular raft foundation if there is no uplift

$$\theta = M^{II} \times 0{,}54 \frac{\left(1 - \nu^2\right)}{\left(E_{soil} \times R^3\right)} \tag{6}$$

— for a foundation on end bearing piles

$$\theta = \frac{M^{II}}{\beta_p \sum x_p^2 \times k_p} \tag{7}$$

where:

M^{II} is the design value of the 2nd order bending moment acting on the foundation referred to underside of the foundation;

R is the radius of the circular raft foundation;

E_{soil} is the modulus of elasticity of the soil;

ν is the Poisson's ratio; ν = 0,5;

β_p is the factor for pile interference in a grid;

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$$\beta_{\mathrm{p}} = \frac{1}{1 + 6 \times \left(\frac{d_{\mathrm{p}}}{s_{\mathrm{p}}} \right)} \tag{7a}$$

x_{p} is the distance of a pile to the axis of rotation;

k_{p} is the spring constant of an end bearing pile;

d_{p} is the pile diameter;

s_{p} is the minimum spacing between the piles.

6.2.4 Superposition of effects from thermal and other actions

The superposition of thermal stresses and stresses from other actions is particularly difficult due to the highly non-linear behaviour of the material. A calculation method to determine the bending moments and reinforcement due to temperature differences between inner and outer surface of the concrete wall superposed by other actions is given in Annex A. The influence of sun exposure on the effects of actions need not be taken into account.

The calculation of thermal stresses is not necessary for chimneys made of prefabricated elements if the following conditions apply:

a) Flue gas temperature $T \leq 300$ °C;

b) Thickness of thermal inside insulation ≥ 80 mm and thermal conductivity $\lambda \leq 0{,}058$ W/mK measured at 150 °C;

c) Windshield wall thickness of prefabricated element ≤ 200 mm;

d) Chimney height ≤ 30 m.

6.3 Verification

6.3.1 General

The following standards for structural design shall be used unless other specifications are made in this document.

Reinforced concrete:

Normal weight concrete	EN 1992-1-1
Lightweight aggregate concrete with closed structure	EN 1992-1-1
Lightweight aggregate concrete with open structure	EN 1520

6.3.2 Ultimate limit state

6.3.2.1 Cast-in-situ chimneys

Verification shall show that the design values of the effect of actions according to 6.2 do not exceed the design values of the ultimate load bearing capacity of the cross-sections taking into account partial safety factors for materials.

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NOTE The values of the partial safety factors for materials in the ultimate limit state for use in a Country may be found in its National Annex. The recommended partial safety factors for materials are:

a) concrete $\gamma_c = 1,5$

b) steel reinforcement $\gamma_s = 1,15$

6.3.2.2 Prefabricated chimneys

6.3.2.2.1 Chimneys with continuous vertical reinforcement

The specifications of 6.3.2.1 apply.

The design value of the maximum capacity of compression of the horizontal mortar joints between prefabricated elements shall be determined according to EN 1992-1-1:2004, 10.9.4.3.

The design value of the resistant shear forces in horizontal mortar joints between prefabricated elements made of normal weight concrete shall be determined according to EN 1992-1-1:2004, 6.2.5.

For prefabricated elements made of lightweight aggregate concrete with closed structure EN 1992-1-1 applies. For prefabricated elements made of lightweight aggregate concrete with open structure EN 1520:2002, Annex A applies.

6.3.2.2.2 Chimneys or parts of chimneys without continuous vertical reinforcement

The specifications of 6.3.2.2.1 apply. Parts of a cross-section which do not transmit compressive forces shall not be taken into account to support shear forces.

6.3.2.3 Openings

6.3.2.3.1 General

Stresses around openings in the windshield can be analysed by shell theory. In the case of circular cylindrical shells the approximate method given in 6.3.2.3.2 and 6.3.2.3.3 may be used.

6.3.2.3.2 Virtual openings

The basic assumption of Navier in beam theory does not apply to parts of chimneys with openings.

However, this model may be applied to the dimensioning of horizontal cross-sections, if the openings are enlarged to virtual openings according to Figure 1 and if the following conditions are fulfilled:

— No virtual opening has a width larger than 1,2 times the inside radius;

— for each horizontal section with more than one opening, the circumferential distance *a* between any two adjacent virtual openings with width b_1 and b_2 shall be such that

$$a \geq 0{,}25\,(b_1 + b_2) \tag{8}$$

otherwise the openings have to be considered as one opening ignoring the wall between the openings.

— For the determination of the equilibrium, when the opening is in the compression zone, the vertical reinforcement ratio ρ_v existing within a distance of 0,5 x *b* from the edges of the opening shall be assumed to be 0,005 less than the actual amount.

$$\rho_v = \frac{A_c}{A_s} \tag{9}$$

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where:

A_c is the cross-sectional area of concrete;

A_s is the area of reinforcement;

— the vertical height of the opening is not larger than the diameter.

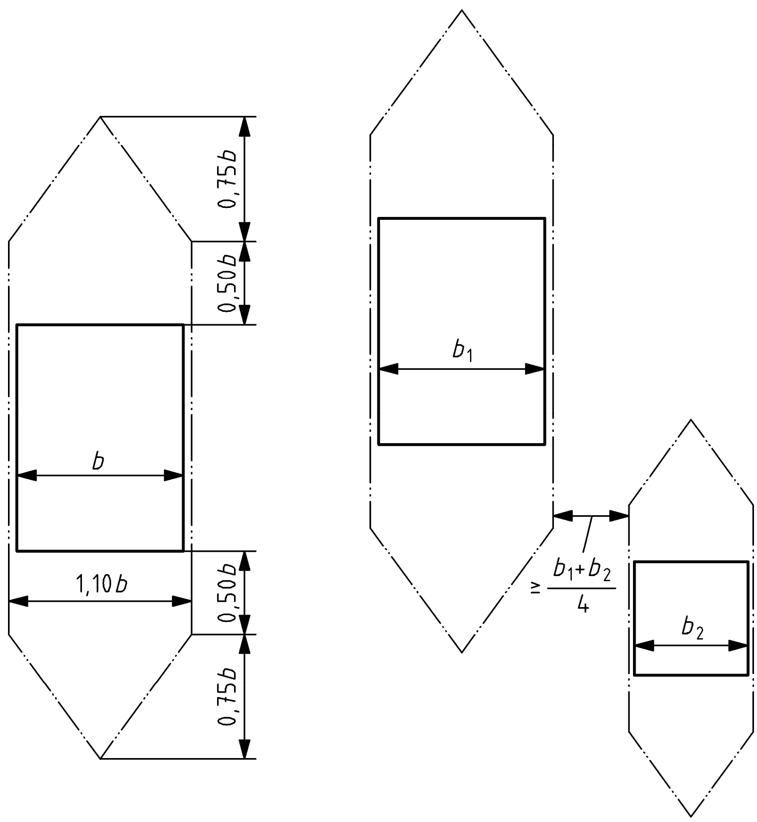

Figure 1 — Relation between real opening (solid line) and virtual opening (dashed double dotted line)

6.3.2.3.3 Dimensioning

The model of virtual openings will not give sufficient safety to take care of the flow of forces around the opening. Additional reinforcement can be needed to take care of such local disturbances.

Buckling of the vertical sides of the opening shall be checked if the vertical height of the opening is larger than the diameter.

The total primary tensile force in the horizontal direction above and below an opening is

$$F_1 = 0{,}15 \times b \times t\,(\sigma_c + \rho_v \times \sigma_s) \qquad (10)$$

where:

b is the clear width of the opening;

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t is the wall thickness;

σ_c is the vertical concrete compressive stress in the undisturbed shell;

σ_s is the vertical steel compressive stress in the undisturbed shell;

ρ_v is the ratio of the vertical reinforcement.

The stresses σ_c and σ_s are the design values of the stresses in the ultimate limit state. All parameters shall be taken at the actual height immediately above or below the opening.

6.3.3 Serviceability limit state

6.3.3.1 Partial safety factors

The partial safety factors for actions, γ_F, as well as for materials, γ_M , shall be applied in the serviceability state.

NOTE The values of γ_F and γ_M in the serviceability state for use in a Country may be found in its National Annex. The recommended value for γ_F as well as for γ_M is 1,0.

6.3.3.2 Cracking

The limitation of crack widths shall only be assessed in the circumferential direction concerning vertical cracks.

The reinforcement shall prevent unacceptable cracks.

The design crack width shall be limited according to environmental conditions, see Table 1.

For the outer and the inner face of the windshield, different environmental conditions shall be applied if appropriate.

The maximum design crack width determines the required ratio of reinforcement and the diameter of the bars.

The dimensioning has to be carried out in accordance with Annex B.

Table 1 – Maximum design crack width max w_k

Exposure class in accordance with EN 206-1	max w_k mm
XA2, XA3, XD3, XS1	0,2
all other classes	0,3

6.3.3.3 Deflection of the chimney

The limitation of deflection of chimneys has only to be considered concerning relative displacements between windshield and lining. The dynamic response due to wind actions shall be taken into account (see EN 13084-4).

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7 Detailing provisions

7.1 Cast-in-situ chimneys

7.1.1 Minimum vertical reinforcement

The minimum cross-section of the vertical reinforcement is 0,3 % of the horizontal concrete cross-sectional area.

The reinforcement shall be distributed in layers towards the inner and the outer faces with not less than half and not more than 2/3 of the total reinforcement in the layer towards the outer face.

7.1.2 Minimum horizontal reinforcement

The windshield shall be provided with a layer of horizontal reinforcement at both outer and inner face. The cross-section of the horizontal reinforcement in each layer shall not be less than 0,15 % of the vertical concrete cross-sectional area. The inner layer shall contain not less than one third of the total horizontal reinforcement.

7.1.3 Minimum reinforcement around openings

Unless calculation calls for higher values an additional vertical reinforcement of at least 50 % of the reinforcement displaced by the opening shall be arranged respectively at both sides of the opening and as near as possible to the edges.

The horizontal reinforcement above and below the opening to take up the primary tensile forces in accordance with Equation (10) shall be arranged in equal thirds at the edge, at the outer and inner surface of the wall respectively as near as possible to the opening (see Figure 2). The anchorage length of the reinforcement at each side of the opening shall be at least 60 % of the clear width of the opening but not less than the anchorage length specified in EN 1992-1-1.

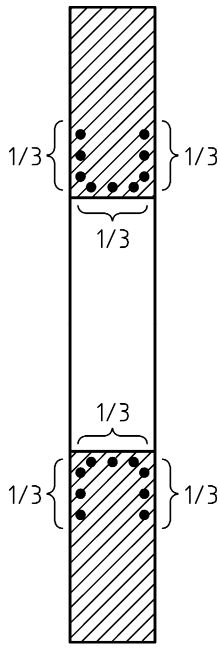

Figure 2 — Arrangement of the horizontal reinforcement above and below the opening

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In addition vertical stirrups shall be arranged over a height of 3/4 of the clear width of the opening to take up the oblique primary tensile stresses and an additional horizontal reinforcement of the same size as the primary tensile reinforcement shall be provided with its centre of gravity at a distance of 1/3 of the clear width of the opening from the edge of the opening.

7.1.4 Distance between bars

The distance between vertical bars shall not exceed 300 mm.

The distance between horizontal bars shall not exceed 200 mm.

7.1.5 Minimum concrete cover

The minimum concrete cover c_{min} shall be 30 mm, for exposure class XA3 in accordance with EN 206-1 40 mm. To calculate the nominal cover c_{nom}, an addition of Δc_{dev} = 10 mm to the minimum cover shall be made in design.

7.1.6 Minimum wall thickness

The wall thickness of the windshield shall not be less than 200 mm.

7.1.7 Splices

The values of the lap length of lapped joints given in EN 1992-1-1 shall be increased by 200 mm. For lapped joints the percentage of lapped bars in any cross-section shall not exceed 50 % and the lapped joints shall be arranged regularly over the circumference of the windshield.

7.2 Prefabricated concrete chimneys

7.2.1 Minimum reinforcement for transportation of prefabricated elements

Prefabricated elements with a weight exceeding 200 kg shall have a minimum horizontal reinforcement for transportation. This shall consist of horizontal stirrups with a minimum diameter of 8 mm and with a maximum spacing of 200 mm as well as vertical transverse reinforcement with bars of at least the same diameter and with a maximum spacing of 500 mm. Lifting gears, e. g. crane-eyes or anchor sockets, shall be connected with the transportation reinforcement to ensure full force transmission.

Sections of chimneys consisting of several prefabricated outer wall elements in accordance with EN 12446, which are transported horizontally shall have sufficient reinforcement for transportation purpose. Transverse reinforcement is not required.

7.2.2 Minimum horizontal reinforcement

The inner layer of reinforcement may be omitted, but in that case the cross-section of outer layer reinforcement shall not be less than 0,3 % of the vertical concrete cross-sectional area.

Prefabricated chimneys built of outer wall elements in accordance with EN 12446 do not need any horizontal reinforcement, if it can be shown by tests that the calculated design value of the shear stress does not exceed the actual design shear resistance taking into account a safety factor of γ_{M} = 2,0. To determine the actual ultimate shear resistance 3 tests shall be carried out on nominally identical specimens, the effective span of which is 3,0 m with a limit deviation of ± 5 cm and with free ends on both sides with a length of 25 cm - 33 cm. A single load shall be applied in mid-span. The mean value of the evaluated shear stress shall be taken as the characteristic value.

7.2.3 Minimum concrete cover

The minimum concrete cover c_{min} shall be 30 mm, for exposure class XA3 in accordance with EN 206-1 40 mm. To calculate the nominal cover c_{nom}, an addition of Δc_{dev} = 5 mm to the minimum cover shall be made in design.

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7.2.4 Minimum wall thickness

The wall thickness shall be at least 100 mm, for outer wall elements in accordance with EN 12446 at least 50 mm.

7.2.5 Vertical continuous bundles of bars

7.2.5.1 General

The vertical continuous reinforcement required by static analysis may be concentrated in bundles of bars. The distance between the bundles shall not exceed 2,5 m or the chimney diameter.

Steel of continuous reinforcement without bond shall be stainless steel or it shall be protected against corrosion.

7.2.5.2 Minimum reinforcement

The minimum cross-section of the vertical continuous reinforcement shall be 0,075 % of the horizontal concrete cross-sectional area and may be taken into account as part of the total vertical minimum reinforcement including the vertical reinforcement of the precast units.

The diameter of the continuous vertical bars shall be at least 10 mm.

7.2.5.3 Splices

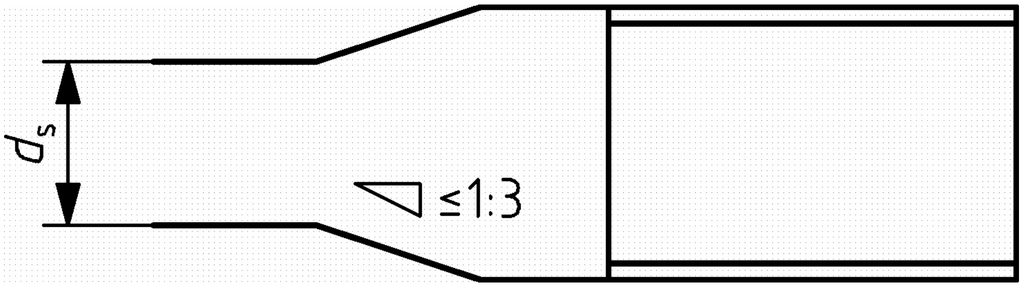
For splices of continuous reinforcement see EN 1992-1-1. The values of the lap length of lapped vertical reinforcement joints given in EN 1992-1-1 shall be increased by 300 mm. For lapped joints only straight bars are allowed and the percentage of lapped bars in any cross-section shall not exceed 50 %. The laps shall be arranged regularly over the circumference of the windshield.

7.2.5.4 Connections of the continuous vertical reinforcement

Mechanical devices for joining (for example sockets) shall be able to carry 120 % of the ultimate design load of the smaller connected bar. For the concrete cover and the spacing of bars in the ducts see EN 1992-1-1; the reference value is the diameter of the larger connected bar.

Upsetting of joint bars in order to increase their cross-sectional area is permitted, a gradient of 1:3 or less being used for the transition zone (see Figure 3). The deformation occuring in addition to elastic strain (due to slippage at the sleeve ends) shall be not more than 0,1 mm under service load. For rolled threads the core-section may be calculated as the full area but for milled threads only with 80 % of the full area.

Connections shall have been tested for fatigue under dynamic loads.



Key

d_s = diameter of the original reinforcing bar

Figure 3 — Upsetting of joint bars

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7.2.6 Openings

The size and position of the openings shall be arranged so that at least 1/4 of the prefabricated element height remains to form a closing ring-section or polygonal-section (see Figure 4).

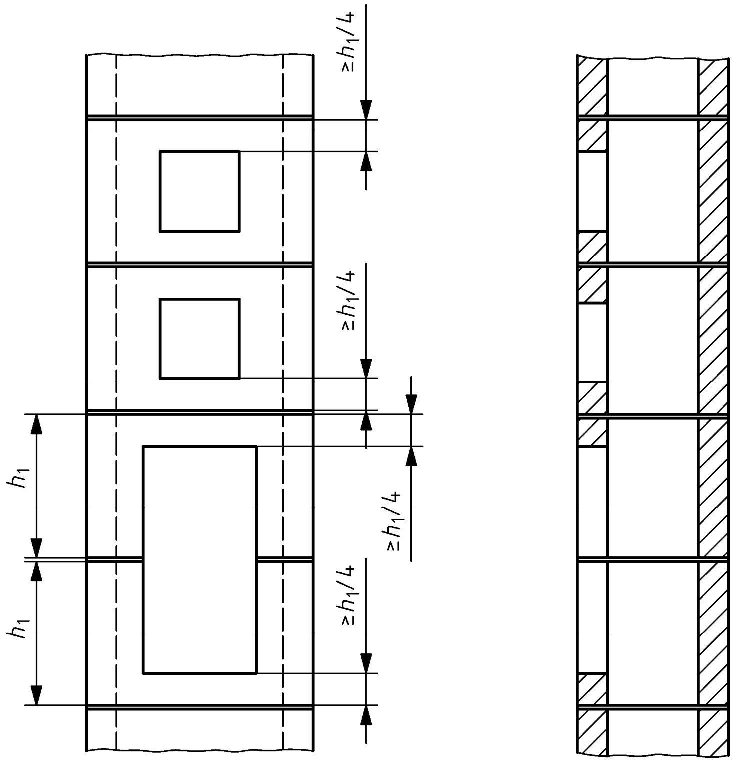

Figure 4 — Openings in prefabricated concrete elements

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8 Workmanship

8.1 General

The progress of erection shall be adjusted to the development of concrete strength (see 7.2 of EN 206-1:2000).

8.2 Bedding joints

The bedding joints between prefabricated elements shall have a thickness of at least 20 mm to allow an accurate filling in of mortar or concrete. Bedding joints may be thinner, if the upper element is placed on a fresh bed joint and the accurate positioning of the element is guaranteed by appropriate devices (for example neoprene plates).

8.3 Reinforcement ducts

The position of the continuous vertical reinforcement shall be guaranteed by fixing devices before filling in the concrete into the duct. The concrete shall be filled in section by section and fully compacted. The height of each stage of filling in shall not exceed 2,50 m. The infill concrete shall be filled in up to the middle of the height of the last element installed to ensure force transmission.

The bond action between reinforcement duct concrete and the concrete of the prefabricated element shall be guaranteed by roughening the inside face of the reinforced ducts or better by shaping them. Lost tubes of formwork are not allowed except of steel strip sheaths.

9 Quality control

9.1 Cast-in-situ chimneys

For each 300 m^3 of concrete volume and at least each 3rd day tests shall be taken and for each test 3 specimens are necessary. The compressive strength is the mean value of the 3 specimens.

9.2 Prefabricated chimneys

For each 15 m of chimney height and at least each 3rd day tests shall be taken and for each test 3 specimens of infill concrete and joint mortar are necessary. The compressive strength is the mean value of the 3 specimens.

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Annex A
(normative)

Analysis of stresses due to thermal and other actions

A.1 Moment-curvature-relation

Stresses resulting from imposed deformations of horizontal as well as of vertical cross-sections due to temperature differences between inner and outer surface of the concrete wall and from other actions may be determined by using the moment-curvature-relation given in Figure A.1. This relationship takes account of the concrete in tension between cracks (tension stiffening effect). To evaluate the effective stiffness of the cross-section mean material properties may be assumed and the static system may be considered as a bar with constant axial force and constant bending moment.

The following 3 ranges are shown in Figure A.1:

— Range a: $M_{\Delta T} + M_{L} < M_{cr}$

NOTE 1 No cracks occur.

— Range b: $M_{\Delta T} + M_{L} = M_{cr}$

NOTE 2 In this range of the formation of single cracks, the total bending moment from other actions, M_{L}, and temperature difference, $M_{\Delta T}$, is equal to the bending moment at cracking, M_{cr}. The reason for this behaviour is, that with each increase of ΔT, for which the ceiling value M_{cr} is obtained, a new single crack arises which decreases the stiffness. The steel stress is reduced by the bond of concrete and steel between cracks. Thus, tensile stresses are induced into the concrete, which eventually will produce another crack.

— Range c: $M_{\Delta T} + M_{L} > M_{cr}$

NOTE 3 At the beginning of the range the regions of disturbance from adjacent cracks touch each other. If now the moment is further increased, the steel stress will also increase, but the reduction of steel stress due to concrete in tension between cracks is constant. Therefore the line for range c is parallel to the "pure state II" line.

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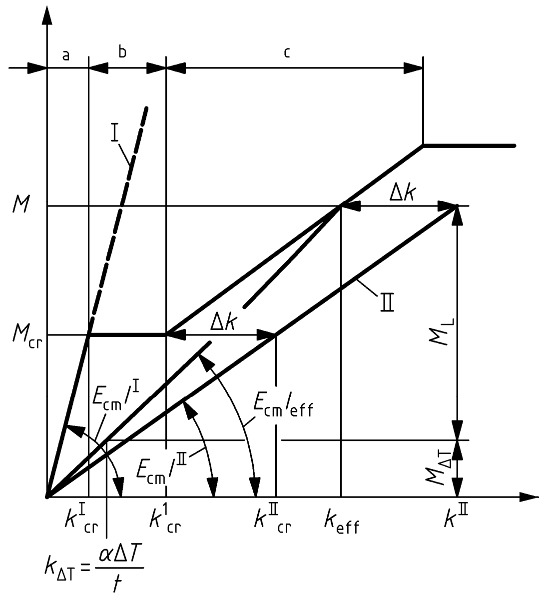

Key

a) Range a: $M < M_{cr}$

b) Range b: $M = M_{cr}$

c) Range c: $M > M_{cr}$

where:

M_{cr} is the bending moment at cracking, see Equation (A.2)

$M_{\Delta T}$ is the bending moment from temperature difference ΔT

M_L is the bending moment from other actions (loading)

$k_{\Delta T}$ is the curvature due to temperature difference ΔT, see Equation (A.7)

k^I_{cr} is the curvature state I at cracking, see Equation (A.3)

k^{II}_{cr} is the curvature state II at cracking, see Equation (A.4)

k^1_{cr} is the curvature, when the formation of cracks is completed, see Equation (A.6)

k_{eff} is the effective curvature

k^{II} is the curvature state II

Δk is the stiffening effect, see Equation (A.5)

E_{cm} is the modulus of elasticity of concrete

I^I is the 2nd moment of area at state I

I^{II} is the 2nd moment of area at state II, see Equation (A.1)

I_{eff} is the effective 2nd moment of area

state I: uncracked cross-section

state II: cracked cross-section

Figure A.1 — Moment-curvature-relation

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The 2nd moment of area at state **II**, I^{II}, of a cross-section under the action of the characteristic values of a bending moment, M, and an axial force, N, may be calculated as follows:

$$I^{\text{II}} = I_0^{\text{II}} \left(1 + \frac{\Delta x}{e}\right) \tag{A.1}$$

where:

I_0^{II} is the 2nd moment of area at state **II** under the action of pure bending;

Δx is the difference between the position of the neutral axis of the cross-section at state **I** and state **II** respectively under the action of pure bending;

$e = \frac{M}{N}$ is the eccentricity of the characteristic value of the axial force.

The bending moment at cracking may be calculated as follows:

$$M_{\text{cr}} = W^{\text{I}} \left(f_{\text{ctm}} - \frac{N}{A^{\text{I}}} \right) \tag{A.2}$$

where:

W^{I} is the section modulus at state **I**;

A^{I} is the area of cross-section at state **I**;

f_{ctm} is the mean value of the tensile strength;

N is the characteristic value of axial force.

$$k^{\text{I}}_{\text{cr}} = \frac{M_{\text{cr}}}{E_{\text{cm}} \times I^{\text{I}}} \tag{A.3}$$

$$k^{\text{II}}_{\text{cr}} = \frac{M_{\text{cr}}}{E_{\text{cm}} \times I^{\text{II}}} \tag{A.4}$$

$$\Delta k = 0{,}4 \left(k^{\text{II}}_{\text{cr}} - k^{\text{I}}_{\text{cr}} \right) \tag{A.5}$$

$$k^{1}_{\text{cr}} = k^{\text{II}}_{\text{cr}} - \Delta k \tag{A.6}$$

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A.2 Calculation procedure

The characteristic value of the restraint bending moment, $M_{\Delta T}$, due to the temperature difference between the inner and outer surfaces of the concrete wall can be calculated according to the following steps:

1) Evaluation of effects of actions

The following effects of actions have to be determined

N_L is the characteristic value of axial force due to actions (loading);

M_L is the characteristic value of bending moment due to actions (loading);

$k_{\Delta T}$ is the curvature due to temperature difference ΔT, see Equation (A.7).

$$k_{\Delta T} = \frac{\alpha_T \times \Delta T}{t} \tag{A.7}$$

where:

ΔT is the characteristic value of the temperature difference, ΔT;

α_T is the coefficient of thermal expansion, for concrete $\alpha_T = 10^{-5}$ K^{-1};

t is the wall thickness;

2) Check of range a

Range a applies if the following requirement is satisfied:

$$M_L + k_{\Delta T} \times E_{cm} \times I^I \leq M_{cr}$$

In this case the characteristic value of the bending moment, $M_{\Delta T}$, can be calculated from the Equation (A.8):

$$M_{\Delta T} = \alpha_T \times \Delta T \times E_{cm} \times \frac{I^I}{t} \tag{A.8}$$

3) Check of range b

Range b applies if the following requirements are satisfied:

$$M_L + k_{\Delta T} \times E_{cm} \times I^I > M_{cr} \quad \text{and}$$

$$\frac{M_L}{M_{cr}} \times k^1_{cr} + k_{\Delta T} \leq k^1_{cr}$$

In this case the characteristic value of the bending moment, $M_{\Delta T}$, is as given in Equation (A.9):

$$M_{\Delta T} = M_{cr} - M_L \tag{A.9}$$

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4) Check of range c

Range c applies if the requirements of range a and range b are not satisfied.

The characteristic value of the bending moment, $M_{\Delta T}$, and the effective 2nd moment of area, I_{eff}, may be determined by Equations (A.10) and (A.11):

$$M_{\Delta T} = \frac{\alpha_T \times \Delta T}{t} \times E_{cm} \times I_{eff} \tag{A.10}$$

$$\frac{M_{\Delta T} + M_L}{E_{cm} \times I_{eff}} = \frac{M_{\Delta T} + M_L}{E_{cm} \times I^{II}} - \Delta k \tag{A.11}$$

where I^{II} may be calculated in accordance with Equation (A.1).

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Annex B
(normative)

Limitations of crack widths

It has to be verified that, under the action of the bending moment, M, and the axial force, N, the maximum design crack width in accordance with Table 1 will not be exceeded at the serviceability limit state. For assessing the crack width the steel stresses σ_s and σ_{sr} at state II (cracked cross-section) have to be determined. In this calculation the mean compressive strength of concrete may be used and a linear material law for concrete, described by the mean modulus of elasticity, E_{cm}, may be applied.

The design crack width, w_k, shall be calculated by Equation (B.1):

$$w_k = 3{,}5 \left(\frac{\sigma_{sr}^{0{,}88} \times d_s}{f_{cm}^{\frac{2}{3}}} \right)^{0{,}89} \times \frac{\sigma_s - 0{,}4 \times \sigma_{sr}}{E_s} \quad \text{(B.1)}$$

where:

σ_s is the stress in Newtons per square millimetres in the tension reinforcement calculated on the basis of a cracked cross-section (state II) under actual loading conditions with the eccentricity $e = \frac{M}{N}$;

σ_{sr} is the stress in Newtons per square millimetres in the tension reinforcement calculated on the basis of a cracked cross-section (state II) under actions maintaining the actual eccentricity e, at which the tensile strength of the concrete, f_{ctm} in accordance with Equation (1), in the uncracked cross section (state I) is reached;

f_{cm} = f_{ck} + 8 in Newtons per square millimetres, mean compressive strength of concrete;

f_{ck} is the characteristic compressive strength of concrete in Newtons per square millimetres;

d_s is the diameter of the reinforcing bar in millimetres;

E_s is the modulus of elasticity of reinforcing steel in Newtons per square millimetres.

When using light-weight aggregate concrete the symbols f_{cm}, f_{ctm} and f_{ck} shall be replaced by f_{lcm}, f_{lctm} and f_{lck} respectively in Equation (B.1) and in the legend.

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