

prEN 1996-1-1: Redraft 9A

**Eurocode 6: Design of Masonry Structures –
Part 1-1: Common rules for reinforced and unreinforced
masonry structures**

Sent out: September 2001
(Revised: October 2001)

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Foreword

This European Standard EN 1996-1-1: Eurocode 6: Design of Masonry Structures - Part 1-1: General rules for buildings - Rules for reinforced and unreinforced masonry, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1996-1-1 on **YYYY-MM-DD**.

This European Standard supersedes ENV 1996-1-1: 1995.

Background to the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

¹ Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1990	Eurocode:	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes :

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards³. **Therefore, technical aspects arising from the Eurocodes**

² According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

³ According to Art. 12 of the CPD the interpretative documents shall :

- a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
- b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;
- c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex (informative).

The National Annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic etc), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode

and it may also contain

- decisions on the application of informative annexes
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

⁴ see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

Additional information specific to EN 1996-1-1

This European Standard is part of EN 1996 which comprises the following parts:

EN 1996-1-1: Common rules for reinforced and unreinforced masonry structures.

EN 1996-1-2: Structural fire design.

EN 1996-2: Design , selection of materials and execution of masonry.

EN 1996-3: Simplified calculation methods and simple rules for masonry structures.

Note: A Part 1-3 is under preparation, but after the Stage 34, it will be combined into Part 1-1.

EN 1996-1-1 describes the Principles and requirements for safety, serviceability and durability of masonry structures. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1996-1-1 is intended to be used, for direct application, together with ENs 1990, 1991, 1992, 1993, 1994, 1995, 1997, 1998 and 1999.

EN 1996-1-1 is intended for use by :

- committees drafting standards for structural design and related product, testing and execution standards ;
- clients (e.g. for the formulation of their specific requirements on reliability levels and durability) ;
- designers and contractors ;
- relevant authorities.

National annex for EN 1996-1-1

This standard gives some symbols for which a National value needs to be given, with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1996-1-1 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1996-1-1 through clauses:

[PT Note: to be drafted when the final version is available.]

1 General

1.1 Scope

1.1.1 Scope of Eurocode 6

(1)P Eurocode 6 applies to the design of buildings and civil engineering works, or parts thereof, in unreinforced, reinforced, prestressed and confined masonry.

(2)P Eurocode 6 deals only with the requirements for resistance, serviceability and durability of structures. Other requirements, for example, concerning thermal or sound insulation, are not considered.

(3)P Execution is covered to the extent that is necessary to indicate the quality of the construction materials and products that should be used and the standard of workmanship on site needed to comply with the assumptions made in the design rules.

(4)P Eurocode 6 does not cover the special requirements of seismic design. Provisions related to such requirements are given in Eurocode 8 "Design of structures in seismic regions" which complements, and is consistent with, Eurocode 6.

(5)P Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in Eurocode 6. They are provided in Eurocode 1 "Actions on structures".

1.1.2 Scope of Part 1-1 of Eurocode 6

(1)P The basis for the design of buildings and civil engineering works in reinforced masonry is given in this Part 1-1 of Eurocode 6, which deals with unreinforced masonry and reinforced masonry where the reinforcement is added to provide ductility, strength or improve serviceability. The principles of the design of prestressed masonry and confined masonry are given, but application rules are not provided.

[PT Note: Review later]

(2) For those types of structures not covered entirely, for new structural uses for established materials, for new materials, or where actions and other influences outside normal experience have to be resisted, the principles and application rules given in this EN may be applicable, but may need to be supplemented.

(3) Part 1-1 gives detailed rules which are mainly applicable to ordinary buildings. The applicability of these rules may be limited, for practical reasons or due to simplifications; any limits of applicability are given in the text where necessary.

(4)P The following subjects are dealt with in Part 1-1:

- section 1 : General.
- section 2 : Basis of design.
- section 3 : Materials.
- section 4 : Durability.
- section 5 : Structural analysis.
- section 6 : Ultimate Limit States.
- section 7 : Serviceability Limit States.
- section 8 : Detailing.
- section 9 : Execution.

(5)P Part 1-1 does not cover:

- resistance to fire (which is dealt with in EN 1996-1-2);
- particular aspects of special types of building (for example, dynamic effects on tall buildings);
- particular aspects of special types of civil engineering works (such as masonry bridges, dams, chimneys or liquid-retaining structures);
- particular aspects of special types of structures (such as arches or domes);
- masonry reinforced with other materials than steel.

1.1.3 Further parts of Eurocode 6

(1) Part 1-1 of Eurocode 6 will be supplemented by further parts as follows:

- Part 1-2: Structural fire design.
- Part 2: Design, selection of materials and execution of masonry.
- Part 3: Simplified calculation methods and simple rules for masonry structures.

1.2 Normative references

The following normative documents contain provisions which, through references in this text, constitutive provisions of this European standard. For dated references,

subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

[PT Note: list of standards to be added in Stage 34 draft.]

1.3 Assumptions

(1)P The assumptions given in 1.3 of EN 1990 apply to this En 1996-1-1.

1.4 Distinction between principles and application rules

(1)P The rules in 1.4 of EN 1990 apply to this EN 1996-1-1.

1.5 Definitions

1.5.1 Terms common to all Eurocodes

(1)P The definitions in 1.5 of EN 1990 apply to this EN 1996-1-1.

1.5.2 Masonry

(1)P **Masonry** : An assemblage of masonry units laid in a specified pattern and joined together with mortar.

(2)P **Unreinforced masonry** : masonry not containing sufficient reinforcement so as to be considered as reinforced masonry.

(3)P **Reinforced masonry** : Masonry in which bars or mesh, usually of steel, are embedded in mortar or concrete so that all the materials act together in resisting action effects.

(4)P **Prestressed masonry** : Masonry in which internal compressive stresses have been intentionally induced by tensioned reinforcement.

(5)P **Confined masonry** : Masonry provided with reinforced concrete or reinforced masonry confining elements in the vertical and horizontal direction.

(6)P **Masonry bond** : Disposition of units in masonry in a regular pattern to achieve common action

1.5.3 Strength of masonry

(1)P **Characteristic strength of masonry** : Value of the strength of masonry having a prescribed probability of 5% of not being attained in a hypothetically unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances.

(2)P **Compressive strength of masonry** : The strength of masonry in compression without the effects of platen restraint, slenderness or eccentricity of loading.

(3)P **Shear strength of masonry** : The strength of masonry subjected to shear forces.

(4)P **Flexural strength of masonry** : The strength of masonry in pure bending.

(5)P **Anchorage bond strength** : The bond strength, per unit surface area, between reinforcement and concrete or mortar, when the reinforcement is subjected to tensile or compressive forces.

(6)P **Adhesion** : the effect of mortar developing a tensile or shear resistance at the contact surface of masonry units.

1.5.4 Masonry units

(1)P **Masonry unit** : A preformed component, intended for use in masonry construction.

(2)P **Groups 1, 2 and 3 masonry units** : Group designations for masonry units, according to the percentage size and orientation of holes in the units when laid.

(3)P **Bed face** : The top or bottom surface of a masonry unit when laid as intended.

(4)P **Frog** : A depression, formed during manufacture, in one or both bed faces of a masonry unit.

(5)P **Hole** : A formed void which may or may not pass completely through a masonry unit.

(6)P **Griphole** : A formed void in a masonry unit to enable it to be more readily grasped and lifted with one or both hands or by machine.

(7)P **Web** : The solid material between the holes in a masonry unit.

(8)P **Shell** : The peripheral material between a hole and the face of a masonry unit.

(9)P **Gross area** : The area of a cross-section through the unit without reduction for the area of holes, voids and re-entrants.

(10)P **Compressive strength of masonry units** : The mean compressive strength of a specified number of masonry units (see ENs 771 - 1 to 6).

(11)P **Normalized compressive strength of masonry units** : The compressive strength of masonry units converted to the air dried compressive strength of an equivalent 100 mm wide x 100 mm high masonry unit (see ENs 771 - 1 to 6).

1.5.5 Mortar

(1)P **Masonry Mortar** : mixture of one or more inorganic binders, aggregates and water, and sometimes additions and/or admixtures, for bedding, jointing and pointing of masonry.

(2)P **General purpose masonry mortar** : masonry mortar without special characteristics.

(3)P **Thin layer masonry mortar** : designed masonry mortar with a maximum aggregate size less than or equal to a prescribed figure.

(4)P **Lightweight masonry mortar** : designed masonry mortar with a dry hardened density below a prescribed figure.

(5)P **Designed masonry mortar** : A mortar whose composition and manufacturing method is chosen in order to achieve specified properties (performance concept).

(6)P **Prescribed masonry mortar** : mortar made in predetermined proportions, the properties of which are assumed from the stated proportions of the constituents (recipe concept).

(7)P **Factory made masonry mortar** : mortar batched and mixed in a factory.

(8)P **Semi-finished factory made masonry mortar** : prebatched masonry mortar or a premixed lime and sand masonry mortar.

(9)P **Prebatched masonry mortar** : mortar whose constituents are wholly batched in a factory, supplied to the building site and mixed there according to the manufacturers' specification and conditions.

(10)P **Premixed lime and sand masonry mortar** : mortar whose constituents are wholly batched and mixed in a factory, supplied to the building site, where further constituents specified or provided by the factory are added (eg cement) and mixed with the lime and sand.

(11)P **Site-made mortar** : A mortar composed of individual constituents batched and mixed on the building site.

(12)P **Compressive strength of mortar** : The mean compressive strength of a specified number of mortar specimens after curing for 28 days.

1.5.6 Concrete infill

(1)P **Concrete infill** : A concrete used to fill pre-formed cavities or voids in masonry.

1.5.7 Reinforcement

(1)P **Reinforcing steel** : Steel reinforcement for use in masonry.

(2)P **Bed joint reinforcement** : Reinforcing steel that is prefabricated for building into a bed joint.

(3)P **Prestressing steel** : Steel wires, bars or strands for use in masonry.

1.5.8 Ancillary components

(1)P **Damp proof course** : A layer of sheeting, masonry units or other material used in masonry to resist the passage of water.

(2)P **Wall tie** : A device for connecting one leaf of a cavity wall across a cavity to another leaf or to a framed structure or backing wall.

(3)P **Strap** : A device for connecting masonry members to other adjacent components, such as floors and roofs.

1.5.9 Mortar joints

(1)P **Bed joint** : A mortar layer between the bed faces of masonry units.

(2)P **Perpend joint (head joint)** : A mortar joint perpendicular to the bed joint and to the face of wall.

(3)P **Longitudinal joint** : A vertical mortar joint within the thickness of a wall, parallel to the face of the wall.

(4)P **Thin layer joint** : A joint made with thin layer mortar.

(5)P **Jointing** : The process of finishing a mortar joint as the works proceeds.

(6)P **Pointing** : The process of filling and finishing raked out mortar joints.

1.5.10 Wall types

(1)P **Load-bearing wall** : A wall primarily designed to carry an imposed load in addition to its own weight.

(2)P **Single-leaf wall** : A wall without a cavity or continuous vertical joint in its plane.

(3)P **Cavity wall** : A wall consisting of two parallel single-leaf walls, effectively tied together with wall ties or bed joint reinforcement. The space between the leaves is left as a continuous cavity or filled or partially filled with non-loadbearing thermal insulating material.

(4)P **Double-leaf wall** : A wall consisting of two parallel leaves with the longitudinal joint between filled solidly with mortar and securely tied together with wall ties so as to result in common action under load.

(5)P **Grouted cavity wall** : A wall consisting of two parallel leaves with the intervening cavity filled with concrete and securely tied together with wall ties or bed joint reinforcement so as to result in common action under load.

(6)P **Faced Wall** : A wall with facing units bonded to backing units so as to result in common action under load.

(7)P **Shell bedded wall** : A wall in which the masonry units are bedded on two strips of general purpose mortar at the outside edges of the bed face of the units.

(8)P **Veneer wall** : A wall used as a facing but not bonded or contributing to the strength of the backing wall or framed structure.

(9)P **Shear wall** : A wall to resist lateral forces in its plane.

(10)P **Stiffening wall** : A wall set perpendicular to another wall to give it support against lateral forces or to resist buckling and so to provide stability to the building.

(11)P **Non-loadbearing wall** : A wall not considered to resist forces such that it can be removed without prejudicing the remaining integrity of the structure.

[PT Note: France wanted a double wall...]

1.5.11 Miscellaneous

- (1)P **Chase** : Channel formed in masonry.
- (2)P **Recess** : Indentation formed in the face of a wall.
- (3)P **Grout** : A pourable mixture of cement, sand and water for filling small voids or spaces.
- (4)P **Movement joint** : A joint permitting free movement in the plane of the wall.

1.6 Symbols

- (1)P Material-independent symbols are given in 1.6 of EN 1990.
- (2)P Material-dependent symbols used in this EN 1996-1-1 are:

[PT note: Check all equations etc. Are symbols (a) used; (b) correct; and (c) complete?]

A	area of a wall;
A_1	numerical factor;
A_b	area of bearing;
A_{ef}	effective area of a wall;
A_m	area of masonry;
A_{mr}	area of reinforced masonry including concrete infill;
A_s	area of reinforcement in tension;
A_{sw}	area of shear reinforcement;
a_1	distance from the end of a wall to the nearer edge of a bearing;
a_v	distance from the face of a support to the principal load on a beam;
b	width of section;
b_c	width of compression face of member mid-way between restraints;
b_c	distance apart of cross walls or buttresses;
b_{ef}	effective width of a flanged member;
b_s	distance between centre lines of mortar strips;

C	compressive strength class of concrete;
d	deflection of arch under design lateral load;
d	effective depth of section;
E	modulus of elasticity;
E_n	modulus of elasticity of a member (where $n = 1, 2, 3$ or 4);
E_s	modulus of elasticity of reinforcing steel;
e	eccentricity;
e_a	accidental eccentricity;
e_{hm}	eccentricity at mid-height of a wall resulting from horizontal loads;
e_{hi}	eccentricity at top or bottom of a wall resulting from horizontal loads;
e_i	resultant eccentricity at the top or bottom of a wall;
e_k	eccentricity due to creep;
e_m	eccentricity due to loads;
e_{mk}	resultant eccentricity within the middle fifth of the wall height;
F	flexural strength class
F_c	design compressive bending force in member;
F_s	design tensile force in steel;
F_t	characteristic compressive or tensile resistance of a wall tie;
f	compressive strength of masonry;
f_b	normalized compressive strength of a masonry unit;
f_{bo}	anchorage bond strength of reinforcing steel;
f_{bok}	characteristic anchorage bond strength of reinforcing steel;
f_c	compressive strength of concrete infill;
f_{ck}	characteristic compressive strength of concrete infill;
f_{cv}	shear strength of concrete infill;

f_d	design compressive strength of masonry;
f_k	characteristic compressive strength of masonry;
f_m	mean compressive strength of mortar;
f_p	tensile strength of prestressing steel;
f_{tk}	characteristic tensile strength of reinforcing steel;
f_v	shear strength of masonry;
f_{vd}	design shear strength of masonry;
f_{vk}	characteristic shear strength of masonry or concrete infill;
f_{vk}	characteristic shear strength of masonry;
f_{vko}	characteristic shear strength of masonry under zero compressive load;
f_x	flexural strength of masonry;
f_{xd}	design flexural strength of masonry;
f_{xk}	characteristic flexural strength of masonry (also f_{xk1} and f_{xk2});
f_y	yield strength of the reinforcing steel;
f_{yk}	characteristic yield strength of reinforcing steel;
G	shear modulus;
g	total width of the two mortar strips in shell bedded masonry;
H	height of wall to the level of a concentrated load;
h	clear height of a wall (also h_1 and h_2);
h_{ef}	effective height of a wall;
h_e	depth of soil;
h_m	overall depth of a section;
h_{tot}	total height of a structure;
I_n	second moment of area of a member (where $n = 1, 2, 3$ or 4);
K	constant concerned with the characteristic compressive strength of masonry;

k	ratio of slab stiffness to wall stiffness;
L	length of a panel between supports or between a support and a free edge;
L_{ef}	effective length of a wall;
l	clear span of floor (also l_3 and l_4);
l_b	anchorage length for reinforcing steel;
l_c	length of wall in compression;
l_{ef}	effective span of a member;
M	mortar compressive strength grade;
M_d	design moment;
M_i	bending moment at the top (M_1) or bottom (M_2) of a wall due to load eccentricity;
M_m	bending moment within the middle fifth of the wall height;
M_{Rd}	design moment of resistance;
N	design vertical load per unit length;
N_i	design vertical load at the top (N_1) or bottom (N_2) of a wall;
N_m	design vertical load within the middle fifth of the wall height;
N_{Rd}	design vertical load resistance of a wall;
N_{Sd}	design vertical load on a wall;
n	member stiffness factor;
P_s	imposed load at ground level per unit area;
q_{lat}	design lateral strength per unit length of a wall;
S	slump class of concrete;
s	spacing of shear reinforcement;
t	thickness of a wall or leaf (also t_1 and t_2);
t_{ef}	effective thickness of a wall;

t_f	thickness of a flange;
u	numerical factor;
u_m	height of a masonry unit;
V_{Rd}	design shear resistance of masonry (also V_{Rd1});
V_{Rd2}	design shear resistance of reinforcement;
V_{Sd}	design shear load;
W_{k1}	characteristic wind load per unit area;
W_{Sd}	design horizontal load on a wall per unit area;
w	design uniformly distributed load (also w_3 or w_4);
x	numerical factor;
x	depth of the compression zone of a member;
Z	section modulus;
z	lever arm in a reinforced masonry member subjected to bending;
α	bending moment coefficient;
α	angle of shear reinforcement;
γ_M	partial safety factor for material properties;
γ_S	partial safety factor for steel;
δ	factor allowing for height and width of masonry units;
ε_m	strain in masonry;
ε_s	strain in reinforcing steel;
ε_{uk}	characteristic value of unit elongation at maximum tensile stress in reinforcing steel;
ε	strain;
$\varepsilon_{C\infty}$	final creep strain;
ε_{el}	elastic strain;

λ	numerical factor;
μ	ratio of flexural strengths in two orthogonal directions;
ν	angle of inclination;
ρ_e	bulk density of soil;
ρ_n	reduction factor for stiffened walls (where $n = 2, 3$ or 4);
σ	normal stress;
σ_d	design vertical compressive stress;
σ_{dp}	permanent vertical stress;
Φ	slenderness reduction factor;
Φ	diameter of reinforcement.
Φ_i	slenderness reduction factor at the top or bottom of a wall;
Φ_m	slenderness reduction factor at the mid-height of a wall;
ϕ_∞	final creep coefficient.

2 Basis of design

2.1 Basic requirements

(1)P The design of masonry structures shall be in accordance with the general rules given in EN 1990.

(2)P Specific provisions for masonry structures are given in this section and shall be applied.

(3) The basic requirements of EN 1990 Section 2 are deemed to be satisfied for masonry structures when the following are applied:

- limit state design in conjunction with the partial factor method described in EN 1990
- actions given in EN 1991
- combination rules given in EN 1990
- the principles and rules of application given in this EN 1996-1-1.

2.1.1 Reliability

(1)P The reliability required for masonry structures will be obtained by carrying out design according to this EN 1996-1-1.

2.1.3 Design working life and durability

(1) For the consideration of durability reference should be made to Section 4.

2.2 Principles of limit state design

(1)P Limit states may concern only the masonry, or such other materials as are used for parts of the structure, for which reference shall be made to relevant parts of EN 1992, EN 1993, EN 1994, EN 1995 and EN 1999.

(2)P For masonry structures, the ultimate limit state and serviceability limit state shall be considered for all aspects of the structure including ancillary components in the masonry.

(3)P For masonry structures, all relevant design solutions including relevant stages in the sequence of construction shall be considered.

2.3 Basic variables

2.3.1 Actions

(1)P Actions shall be obtained from the relevant parts of EN 1991.

2.3.2 Design values of actions

(1)P Partial safety factors for load should be obtained from EN 1990.

(2) Partial safety factors for creep and shrinkage of concrete elements in masonry structures shall be obtained from EN 1992-1

(3) For serviceability limit states, imposed deformations should be introduced as estimated (mean) values.

2.3.3 Material and product properties

(1) Properties for materials and construction products and geometrical data to be used for design should be those specified in the relevant hENs of ETAs, unless otherwise indicated in this EN 1996-1-1.

2.4 Verification by the partial factor method

2.4.1 Design value of material properties

(1)P The design value for a material property is obtained by dividing the characteristic value by the partial safety factor for materials, γ_M .

2.4.2 Combination of actions

(1)P Combination of actions shall be in accordance with the general rules given in EN 1990.

Note: In normal residential and office structures the imposed loads, as given in EN 1991-1, may be treated as one fixed variable action (that is, equal loading on all spans, or zero, when appropriate) for which reduction factors are given in EN 1991-1, to allow for the unlikely simultaneous application of the loads at the same time eg several floors of loading.

2.4.3 Ultimate limit states

(1)P The relevant values of the partial safety factor for materials γ_M shall be used for the ultimate limit state for ordinary and accidental situations. When analysing the structure for accidental actions, the probability of the accidental action being present shall be taken into account.

Note: The numerical values of γ_M are given in the National Annex. Recommended values, given as classes that can be related to execution control (see also Annex A) according to national choice, are given in the table below.

Material	γ_M				
	Class				
	1	2	3	4	5
Masonry made with:					
Units of Category I, designed mortar ¹	1,5	1,7	2,0	2,2	2,5
Units of Category I, prescribed mortar ²	1,7	2,0	2,2	2,5	2,7
Units of Category II, any mortar ^{1,2}	2,0	2,2	2,5	2,7	3,0
Anchorage of reinforcing steel	1,7	2,0	2,2	2,5	2,7
Reinforcing steel and prestressing steel	1,15				
Ancillary components ³	1,7	2,0	2,2	2,5	2,7
Lintels according to EN 845-2 ³	1,5				
Note 1: Requirements for designed mortars are given in EN 998-2 and EN 1996-2					
Note 2: Requirements for prescribed mortars are given in EN 998-2 and EN 1996-2					
Note 3: Declared values are mean values.					

2.4.4 Serviceability limit states

(1) Where simplified compliance rules are given in the relevant clauses dealing with serviceability limit states, detailed calculations using combinations of actions are not required.

Note: The recommended value for γ_M , for all material properties for serviceability limit states is 1,0.

2.5 Design assisted by testing

(1) Structural properties of masonry may be determined by testing.

Note: Annex D (informative) of EN 1990 gives recommendations for design assisted by testing.

3 Materials

3.1 Masonry Units

3.1.1 Types and grouping of masonry units

(1)P Masonry units shall comply with any of the following types:

- clay units in accordance with EN 771-1.
- calcium silicate units in accordance with EN 771-2.
- aggregate concrete units (dense and lightweight aggregate) in accordance with EN 771-3.
- autoclaved aerated concrete units in accordance with EN 771-4.
- manufactured stone units in accordance with EN 771-5.
- dimensioned natural stone units in accordance with EN 771-6.

(2) Masonry units may be Category I or Category II.

(3) Masonry units should be grouped as Group 1, Group 2, Group 3 or Group 4, for the purposes of using the equations and other numerical values given in 3.6.1 and 3.6.2 and where grouping is referred to in other clauses.

(4) The limitations on the geometry of units, ie percentage of holes, volume of holes, minimum thickness of material between the face of a unit and a hole, or the minimum thickness of material between holes in the unit, need to be determined so that the performance of the type of units in masonry can be represented by the relationships given in 3.6.1 and 3.6.2 with the compressive strength of the units. It can be assumed that the limits on the geometry of units, as given in table 3.1, will satisfy this requirement.

Table 3.1 : Geometrical requirements for Grouping of Masonry Units¹

		Materials and limits for Masonry Units						
	Group 1	Materials	Group 2		Group 3		Group 4	
			Vertical holes				Horizontal holes	
Volume of holes (% of the gross volume)	≤ 25	clay	> 25; ≤ 55		> 55; ≤ 70		≤ 70	
		calcium silicate	> 25; ≤ 55		not used		not used	
		concrete	> 25; ≤ 50		> 50; ≤ 70		not used	
Volume of any hole (% of the gross volume)	≤ 12,5	clay	each of multiple holes ≤ 1% gripholes up to a total of 12,5%		each of multiple holes ≤ 1% gripholes up to a total of 12,5%		each of multiple holes ≤ 8%, single holes ≤ 25%	
		calcium silicate	each of multiple holes ≤ 15% gripholes up to a total of 30%		not used		not used	
		concrete	each of multiple holes ≤ 15% gripholes up to a total of 30%		each of multiple holes ≤ 1% gripholes up to a total of 30%		each of multiple holes ≤ 25%	
Minimum thickness in and around holes (mm)	No requirement		web	shell	web	shell	web	shell
		clay	5	8	3	6	6	8
		calcium silicate	5	10	not applicable		not applicable	
		concrete	15	20	15	15	20	20
Combined thickness ² of webs and shells (% of the overall width)	No requirement	clay	≥ 16		≥ 12		not applicable	
		calcium silicate	≥ 20		not applicable			
		concrete	≥ 20		≥ 15			

Notes:

1. The limits given above are for masonry units used in masonry designed using the numerical values of equations in 3.6.1 and 3.6.2 (see 3.1.1(3)).
2. The combined thickness is the thickness of the webs and shells, measured horizontally across the unit at right angles to the face of the wall. In the case of conical holes, or cellular holes, use the mean value of the thickness of the webs and the shells. The check is to be seen as qualification test and need only be repeated in the case of principal changes to the design dimensions of units.

3.1.2 Properties of masonry units

3.1.2.1 Compressive strength of masonry units

(1)P The compressive strength of masonry units, to be used in design, shall be the normalized compressive strength, f_b .

Note: It will normally be given by a manufacturer as the declared normalised strength - see EN 771 series.

(2) When the compressive strength of masonry units is declared as a characteristic strength this should be converted to the mean equivalent, using a factor based on the coefficient of variation of the units.

3.2 Mortar

3.2.1 Types of masonry mortar

(1) Masonry mortars are defined as general purpose, thin layer or lightweight mortar according to their constituents.

(2) Masonry mortars are considered as designed or prescribed mortars according to the method of defining their composition.

(3) Masonry mortars may be factory made (prebatched or premixed), semi-finished factory made, site-made, or pre-mixed, according to the method of manufacture.

(4) Factory made and semi-finished factory made masonry mortars shall be in accordance with EN 998-2. Site-made masonry mortar shall be in accordance with EN 1996-2. Pre-mixed lime and sand masonry mortar shall be in accordance with EN 998-2, and shall be used in accordance with EN 998-2.

3.2.2 Specification of masonry mortar

(1) Mortars should be classified by their compressive strength, expressed as the letter M followed by the compressive strength in N/mm², for example, M5. Prescribed masonry mortars, additionally to the M number, will be described by their prescribed constituents, eg 1: 1: 5 cement: lime: sand by volume.

Note: The National Annex of any country may ascribe acceptable equivalent mixes described by the proportion of the constituents, to stated M values.

(2) General purpose masonry mortars may be designed mortars in accordance with EN 998-2 or prescribed masonry mortars to EN 998-2.

(3) Thin layer masonry mortars should be designed mortars in accordance with EN 998-2.

(4) Lightweight masonry mortars should be designed mortars in accordance with EN 998-2.

3.2.3 Properties of mortar

3.2.3.1 Compressive strength of masonry mortar

(1)P The compressive strength of masonry mortar, f_m , shall be determined in accordance with EN 1015-11.

(2) Masonry mortars should not have a compressive strength f_m less than 1 N/mm^2 .

3.2.3.2 Adhesion between units and mortar

(1)P The adhesion between the mortar and the masonry units shall be adequate for the intended use.

Note 1: Adequate adhesion will depend on the type of mortar used and the units to which that mortar is applied. Mortars in accordance with EN 998-2 and site mixed designed or site mixed prescribed general purpose mortars made in accordance with EN 1996-2, will normally give adequate adhesion with most units when constructed in accordance with EN 1996-2.

Note 2: EN 1052-3, deals with the determination of the initial shear strength of masonry and prEN 1052-5, under preparation, deals with the determination of flexural bond strength

3.3 Concrete infill

3.3.1 General

(1)P Concrete used for infill shall be in accordance with EN 206.

(2) Concrete infill is specified by the characteristic compressive strength, f_{ck} , (concrete strength class), which relates to the cylinder/cube strength at 28 days, in accordance with EN 206.

3.3.2 Specification for concrete infill

(1) The strength class, as defined in EN 206, of concrete infill should be not less than $12/15 \text{ N/mm}^2$.

(2) The concrete may be designed or prescribed and should contain just sufficient water to provide the specified strength and to give adequate workability.

(3)P The workability of concrete infill shall be such as to ensure that voids will be completely filled, when the concrete is placed in accordance with EN 1996-2.

(4) The slump class S3 to S5 or flow class F4 to F6, in accordance with EN 206, will be satisfactory for most cases. In holes, where the smaller dimension is less than 85 mm, slump classes S5 or S6 should be used. Where high slump concretes are to be used, measures need to be taken to reduce the resulting high shrinkage of the concrete.

(5) The maximum aggregate size of concrete infill should not exceed 20mm. When concrete infill is to be used in voids whose least dimension is less than 100mm or when the cover to the reinforcement is less than 25mm, the maximum aggregate size should not exceed 10mm.

3.3.3 Properties of concrete infill

(1)P The characteristic compressive strength and shear strength of concrete infill shall be determined from tests on concrete specimens.

Note: Test results may be obtained from tests carried out for the project, or be available from a database.

(2) Where test data is not available the characteristic compressive strength, f_{ck} , and the characteristic shear strength, f_{cvk} , of concrete infill may be taken from table 3.2.

Table 3.2: Characteristic strengths of concrete infill

Strength class of concrete	C12/15	C16/20	C20/25	C25/30, or stronger
f_{ck} N/mm ²	12	16	20	25
f_{cvk} N/mm ²	0,27	0,33	0,39	0,45

3.4 Reinforcing steel

[PT Note: review all sections in line with EN 1992.]

3.4.1 General

(1)P Reinforcing carbon steel shall be in accordance with EN 10080 and stainless steel shall be in accordance with EN 10088. Specially coated bars shall be specified separately.

(2)P The requirements for the properties of the reinforcement are for the material as placed in the hardened masonry. Operations carried out on site or during manufacture, that might damage the properties of the material shall be avoided.

Note: EN 10080 refers to a yield stress R_e , which includes the characteristic, minimum and maximum values based on the long-term quality of production. In contrast f_{yk} is the characteristic yield stress based on only that reinforcement required for the structure. There is no direct relationship between f_{yk} and the characteristic R_e . However the methods of evaluation and verification of yield strength given in EN 10080 provide a sufficient check for obtaining f_{yk} .

(3) Reinforcing steel may be carbon steel or austenitic stainless steel. Reinforcing steel may be plain or ribbed (high bond) and weldable.

3.4.2 Properties of reinforcing steel in bar form

(1)P The characteristic strength of reinforcing steel, f_{yk} , shall be in accordance with ENV 10081. The relevant required properties of reinforcing steel are met if the testing procedures and results are in accordance with ENV 10081.

(2) The coefficient of thermal expansion may be assumed to be $12 \times 10^{-6} \text{ K}^{-1}$.

Note: The difference between this value and the value for the surrounding masonry or concrete should normally be neglected.

3.4.3 Properties of prefabricated bed joint reinforcement

(1) Prefabricated bed joint reinforcement should be in accordance with EN 845-3.

3.5 Prestressing steel

3.5.1 General

(1)P Prestressing steel shall be in accordance with EN 10138.

(2) The properties of prestressing steel should be obtained from EN 1992-1-1.

3.6 Mechanical properties of masonry

3.6.1 Characteristic compressive strength of masonry

3.6.1.1 General

(1)P The characteristic compressive strength of masonry, f_k , shall be determined from results of tests on masonry specimens.

Note: Test results may be obtained from tests carried out for the project, or be available from a database.

(2) The characteristic compressive strength of masonry should be determined from tests in accordance with EN 1052-1, or it may be established from an evaluation of test data.

3.6.1.2 Characteristic compressive strength of masonry made with filled vertical joints

(1) Where test data are not available, the relationship between the characteristic compressive strength of masonry, f_k , and the unit strength and the mortar strength may be obtained from equation (3.1), for masonry made with general purpose mortar and equation (3.2) for masonry made with thin layer mortar, of thickness 3mm, or less.

Note: EN 998-2 gives no limit for the thickness of joints made of thin layer mortar; the limit of 3 mm is to ensure that the thin layer mortar has the enhanced properties assumed to exist to enable equation (3.2) to be valid.

$$f_k = K f_b^{0,7} f_m^{0,3} \quad (3.1)$$

$$f_k = K f_b^{0,85} \quad (3.2)$$

where:

- | | |
|-------|--|
| K | is a constant, depending on the type of unit and the type of mortar; values of K are given in table 3.3 |
| f_b | is the normalised compressive strength of units, in the direction of the applied action effect, in N/mm^2 |
| f_m | is the compressive strength of mortar, in N/mm^2 |

provided that the following requirements are satisfied:

- f_b is not taken to be greater than 75 N/mm^2 when units are laid in general purpose mortar
- f_b is not taken to be greater than 50 N/mm^2 when units are laid in thin layer mortar
- f_m is not taken to be greater than 20 N/mm^2 nor greater than $2f_b$ for general purpose mortar
- f_m is not taken to be greater than 10 N/mm^2 for thin layer mortar
- f_m is not taken to be greater than 5 N/mm^2 for lightweight mortar
- the masonry is detailed in accordance with section 8 of this EN 1996-1-1;
- the coefficient of variation of the strength of the masonry units is not more than 25%;
- all joints satisfy the requirements of 8.1.5 so as to be considered as filled;
- the thickness of the masonry is equal to the width or length of the unit, so that there is no mortar joint parallel to the face of the wall through all or any part of the length of the wall.

(2) Where action effects are parallel to the direction of the bed joints, the characteristic compressive strength may also be determined from equations (3.1) or (3.2), using the normalized compressive strength of the masonry unit, f_b , obtained from tests where the direction of application of the load to the test specimen is the same as the direction of the action effect in the masonry, but with the factor, δ , as given in EN 771-1 to 6, not taken to be greater than 1,0. For Group 2 units, K should then be multiplied by 0,5.

(3) For masonry made with general purpose mortar where there is a mortar joint parallel to the face of the wall through all or any part of the length of the wall, the values of K can be obtained by multiplying the values of table 3.3 by 0,8.

(4) For masonry made of general purpose mortar where Group 2 aggregate concrete units are used with the vertical cavities filled completely with concrete, the value of f_b , should be obtained by considering the units to be Group 1 with a compressive strength corresponding to the compressive strength of the units or of the concrete infill, whichever is the lesser.

Table 3.3: Values of K for use with:

Masonry Unit		General purpose mortar	Thin layer mortar (≤ 3 mm bed joint)	Lightweight mortar of density	
				$600 \leq \rho \leq 700 \text{ kg/m}^3$	$700 < \rho \leq 1500 \text{ kg/m}^3$
Clay	Group 1	0,50	0,75	0,30	0,40
	Group 2	0,45	0,55	0,30	0,40
	Group 3	0,35	0,25	0,20	0,25
	Group 4	0,30	not used	not used	not used
Calcium Silicate	Group 1	0,50	0,80	not used	not used
	Group 2	0,45	0,55	not used	not used
Aggregate Concrete	Group 1	0,50	0,80	0,45	not used
	Group 2	0,50	0,80	0,45	not used
	Group 3	0,30	not used	not used	not used
	Group 4	0,30	not used	not used	not used
Autoclaved Aerated Concrete	Group 1	0,50	0,85	0,45	not used
Manufactured Stone	Group 1	0,50	0,75	not used	not used
Dimensioned Natural Stone	Group 1	0,50	not used	not used	not used

3.6.1.3 Characteristic compressive strength of masonry with unfilled vertical joints

(1) The characteristic compressive strength of masonry made with masonry units in which the perpendicular joints are unfilled may be obtained from equations (3.1) and (3.2),

provided that the shear resistance is based upon the requirements of 3.6.2(7) and due consideration is given to any horizontal actions that might be applied to, or be transmitted by, the masonry.

3.6.1.4 Characteristic compressive strength of shell bedded masonry

(1) The characteristic compressive strength of shell bedded masonry, made with Group 1 and Group 4 masonry units and bedded on two or more equal strips of general purpose mortar, two of which are at the outside edges of the bed face of the units, may be obtained from equation (3.1) for general purpose and lightweight mortar and equation (3.2) for the thin layer mortar in beds not more than 3mm thick, provided that:

- the width of each strip of mortar is 30 mm or greater;
- the thickness of the masonry is equal to the width or length of the masonry units so that there is no longitudinal mortar joint through all or part of the length of the wall;
- the ratio g/t is not less than 0,4;
- K is taken from table 3.3 when $g/t = 1,0$ or K is taken as 0,22 when $g/t = 0,4$, with intermediate values obtained by linear interpolation,

where:

g is the total width of the mortar strips;

t is the thickness of the wall.

(2) The characteristic compressive strength of shell bedded masonry made with Group 2 masonry units and bedded as noted for Group 1 masonry units, may be obtained from equation (3.1) provided that the normalised compressive strength of the units, f_b , used in the equation is that obtained from tests on units shell bedded with strips of mortar, no wider than those intended to be used in the masonry, but basing the strength of the unit on the gross area of the unit, not the bedded area.

3.6.2 Characteristic shear strength of masonry

(1)P The characteristic shear strength of masonry, f_{vk} , shall be determined from the results of tests on masonry.

Note: Test results may be obtained from tests carried out for the project, or be available from a database.

(2) The characteristic initial shear strength of masonry, f_{vko} , should be determined from tests in accordance with EN 1052-3 or EN 1052-4 or it may be established from an evaluation of test data.

(3) Where test data are not available, values for the initial shear strength of masonry made with general purpose mortar, thin layer mortar in beds not greater than 3mm thick and lightweight mortar, may be taken from table 3.4.

(4) The characteristic shear strength of masonry, f_{vk} , using general purpose mortar in accordance with 3.2.2.1(2) and (3), or thin layer mortar in beds not greater than 3mm thick, in accordance with 3.2.2.1(4), or lightweight mortar in accordance with 3.2.2.1(5) with all joints satisfying the requirements of 8.1.5 so as to be considered as filled, may be taken from equations (3.3a to c), whichever gives the lowest value, for the appropriate Groups.

$$f_{vk} = f_{vko} + 0,4 \sigma_d \quad (3.3a)$$

or $f_{vk} = (0,034 f_b + 0,14 \sigma_d)$ for Group 1 and 4 units (3.3b)

or $f_{vk} = 0,9 (0,034 f_b + 0,14 \sigma_d)$ for Group 2 and 3 (3.3c)

where:

f_{vko} is the characteristic initial shear strength, under zero compressive stress;

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination;

f_b is the normalized compressive strength of the masonry units, as described in 3.1.2.1, for the direction of application of the load on the test specimens being perpendicular to the bed face.

[PT Notes: shear in basements; suggestion to tabulate.]

(5) The characteristic shear strength for unreinforced masonry using general purpose mortar in accordance with 3.2.2.1(2) and (3), or thin layer mortar in accordance with 3.2.2.1, in beds not greater than 3 mm thick, or lightweight mortar in accordance with 3.2.2.1, and having the perpend joints unfilled, but with adjacent faces of the masonry units closely abutted together, may be taken from equations (3.4a to c), whichever gives the lowest value for the relevant units.

$$f_{vk} = 0,5 f_{vko} + 0,4 \sigma_d \quad (3.4a)$$

or $f_{vk} = 0,7 (0,034 f_b + 0,14 \sigma_d)$ for Group 1 and 4 units, (3.4b)

or $f_{vk} = 0,6 (0,034 f_b + 0,14 \sigma_d)$ for Group 2 and 3 units (3.4c)

where

f_{vko} , σ_d and f_b are as defined in (3) above.

(6) In shell bedded masonry, made with Group 1 masonry units and bedded on two or more equal strips of general purpose mortar, each at least 30mm in width, two of which are at the outside edges of the bed face of the unit, may be taken from equations (3.5a and b), whichever gives the lowest value, for the relevant Group 1, 2 or 3 units.

$$f_{vk} = \frac{g}{t} f_{vko} + 0,4 \sigma_d \quad (3.5a)$$

or f_{vk} = the value obtained from equations (3.4b) or (3.4c) (3.5b)

where

f_{vko} , σ_d and f_b are as defined in (4) above and:

g is the total width of the mortar strips;

t is the thickness of the wall.

[PT Note: It is recognised that the $0.4\sigma_d$ part of the equation does not really need to be divided by γ_m , but it is not reasonable to ignore a material safety factor so it has not been changed.]

Table 3.4: Values of f_{vko} for general purpose mortar

Masonry Units	Mortar Strength	f _{vko} (N/mm ²) for mortars		
		General purpose	Thin layer	Lightweight
Clay	M10 - M20	0,30	not used	not used
	M2,5 – M9	0,20	0,30	0,20
	M1 – M2	0,10	not used	not used
Calcium Silicate Aggregate Concrete Autoclaved Aerated Concrete Manufactured stone	M10 - M20	0,20		
	M2,5 – M9	0,15		
	M1 – M2	0,10		
Calcium silicate			0,40	0,20
Aggregate concrete Manufactured stone Autoclaved aerated concrete			0,30	0,20
Dimensioned Natural Stone	M2,5 – M9	0,15		
	M1 – M2	0,10		

[PT Note: There is a need for vertical shear to be covered – see UK and TBE.]

3.6.3 Characteristic flexural strength of masonry

(1) In considering out-of plane bending, the following situations are to be considered: flexural strength having a plane of failure parallel to the bedjoints, f_{xk1} ; flexural strength having a plane of failure perpendicular to the bedjoints, f_{xk2} (see figure 3.1).

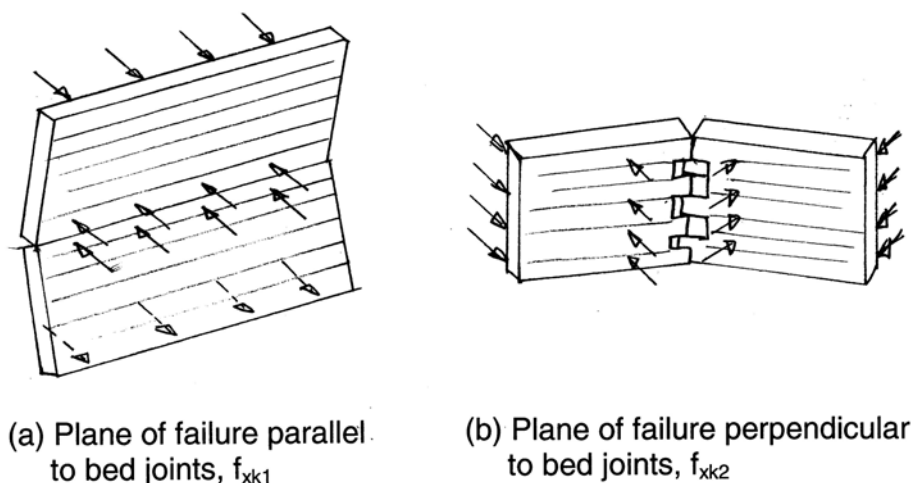


Figure 3.1 : Planes of failure of masonry in bending.

(2)P The characteristic flexural strength of masonry, f_{xk1} and f_{xk2} , shall be determined from the results of tests on masonry.

Note: Tests results may be obtained from tests carried out for the project, or be available from a database.

(3) The characteristic flexural strength of masonry may be determined by tests in accordance with EN 1052-2, or it may be established from an evaluation of test data based on the flexural strengths of masonry obtained from appropriate combinations of units and mortar.

Note:

1. Where test data are not available values of the characteristic flexural strength of masonry made with general purpose mortar, thin layer mortar or lightweight mortar, may be taken from the tables in this note, provided that the following requirements are fulfilled:

- thin layer mortar and lightweight mortars are M5, or stronger;
- values of f_{xk1} are for masonry with filled and unfilled perpend joints and those for f_{xk2} are for masonry with unfilled perpend joints only.

2. For masonry made with autoclaved aerated concrete units laid in thin layer mortar, f_{xk1} values may be taken from the tables in this note or from the following equations, whichever gives the higher value:

$$f_{xk1} = 0,035 f_b, \text{ with filled and unfilled perpend joints}$$

$$f_{xk2} = 0,036 f_b, \text{ with filled perpend joints}$$

[PT note: 0,035 and 0,036 are effectively the same! f_{xk2} is typically greater than f_{xk1} .]

3. The values of f_{xk2} for masonry with unfilled perpend joints may be obtained by multiplying the values for masonry with filled perpend joints by 2/3.

The National Annex should give the values of f_{xk1} and f_{xk2} to be used in the country whose National Annex it is.

Masonry Unit	f_{xk1} (N/mm ²)			
	General purpose mortar		Thin layer mortar	Lightweight mortar
	$f_m < 5$ N/mm ²	$f_m \geq 5$ N/mm ²		
Clay	0,10	0,10	0,15	0,10
Calcium silicate	0,05	0,10	0,20	not used
Aggregate concrete	0,05	0,10	0,20	not used
Autoclaved aerated concrete	0,05	0,10	0,15	0,10
Manufactured stone	0,05	0,10	not used	not used
Dimensioned natural stone	0,05	0,10	not used	not used

Masonry Unit	f_{xk2} (N/mm ²)			
	General purpose mortar		Thin layer mortar	Lightweight mortar
	$f_m < 5$ N/mm ²	$f_m \geq 5$ N/mm ²		
Clay	0,20	0,40	0,15	0,10
Calcium silicate	0,20	0,40	0,30	not used
Aggregate concrete	0,20	0,40	not used	not used
Autoclaved aerated concrete	0,20	0,20	0,30	0,15
Manufactured stone	0,20	0,40	not used	not used
Dimensioned natural stone	0,20	0,40	not used	not used

[PT Note: the old (4) has been deleted. This contained a warning concerning poor bond - is a form of warning needed?]

3.6.4 Characteristic anchorage strength

(1)P The characteristic anchorage strength of reinforcement bedded in concrete shall be obtained from the results of tests.

Note: Test results may be obtained from tests carried out for the project, or be available from a database.

(2) The characteristic anchorage strength of reinforcement may be established from an evaluation of test data.

(3) Where tests data are not available, for reinforcement embedded in concrete sections with dimensions greater than or equal to 150 mm, or where the concrete infill surrounding the reinforcement is confined within masonry units, so that the reinforcement can be considered to be confined, the characteristic anchorage strength, f_{bok} , is given in table 3.5.

(4) For reinforcement embedded in mortar, or in concrete sections with dimensions less than 150 mm, or where the concrete infill surrounding the reinforcement is not confined within masonry units so that the reinforcement is considered not to be confined, the characteristic anchorage strength, f_{bok} , is given in table 3.6.

(5) For prefabricated bed joint reinforcement, the characteristic anchorage strength should be determined by tests in accordance with EN 846-2, or the bond strength of the longitudinal wires alone should be used.

Table 3.5 : Characteristic anchorage strength of reinforcement in concrete infill, confined within masonry units

Strength class of concrete	C12/15	C16/20	C20/25	C25/30 or stronger
f_{bok} for plain carbon steel bars (N/mm ²)	1,3	1,5	1,6	1,8
f_{bok} for high-bond carbon and stainless steel bars (N/mm ²)	2,4	3,0	3,4	4,1

[PT Note: the UK put forward figures for to replace this table, but they are not yet in a form that can be used]

Table 3.6 : Characteristic anchorage strength of reinforcement in mortar or concrete not confined within masonry units

Strength class of:	Mortar	M5-M9	M10-M14	M15-M19	M20
	Concrete	C12/15	C16/20	C20/25	C25/30 or stronger
f_{bok} for plain carbon steel bars (N/mm ²)		0,7	1,2	1,4	1,5
f_{bok} for high-bond carbon steel and stainless steel bars (N/mm ²)		1,0	1,5	2,0	2,5

3.7 Deformation properties of masonry

3.7.1 Stress-strain relationship

(1) The stress-strain relationship of masonry may be taken as linear parabolic, parabolic rectangular (see figure 3.2) or as rectangular, for the purposes of calculating the strength of a masonry section (see 6.4.1(3)P).

Note: Figure 3.2 is an approximation and may not be suitable for all types of masonry units.

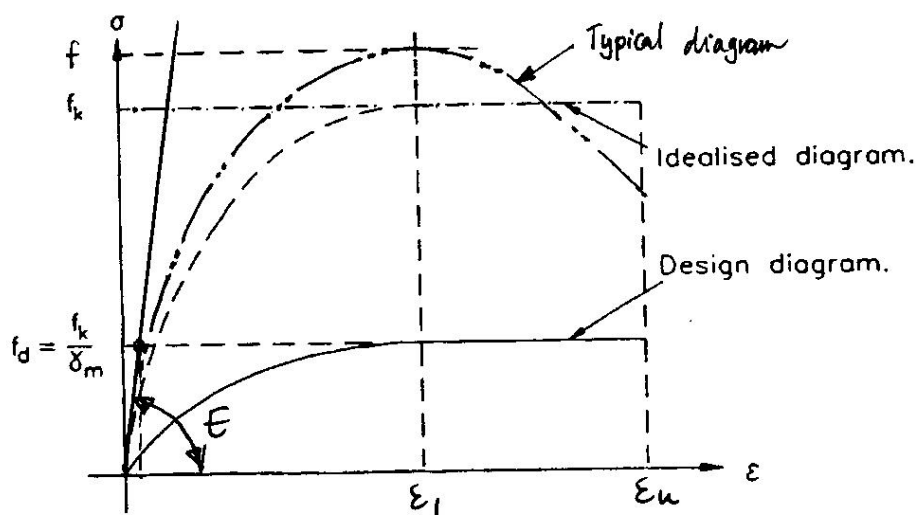


Figure 3.2: Stress-strain relationship for masonry in compression

3.7.2 Modulus of elasticity

(1)P The short term secant modulus of elasticity, E , shall be determined by tests in accordance with EN 1052-1.

Note: Test results may be obtained from tests carried out for the project, or be available from a database.

(2) In the absence of a value determined by tests in accordance with EN 1052-1, the short term secant modulus of elasticity of masonry, E , for use in structural analysis, may be taken to be $1000 f_k$.

(3) The long term modulus should be based on the short term secant value, reduced to allow for creep effects, (see 3.7.4), such that:

$$E_{\text{long term}} = \frac{E_{\text{short term}}}{1 + \phi_{\infty}} \quad (3.6)$$

where

ϕ_{∞} is the final creep coefficient.

3.7.3 Shear modulus

(1) The shear modulus, G , may be taken as 40% of the elastic modulus, E .

3.7.4 Creep, moisture expansion or shrinkage and thermal expansion

(1)P Coefficients of creep, moisture expansion or shrinkage and thermal expansion shall be determined by test.

Note 1: Test results may be obtained from tests carried out for the project, or be available from a database.

Note 2: No European test method to determine creep or moisture expansion currently exists and none is planned.

(2) The final creep coefficient, ϕ_{∞} , final moisture expansion or shrinkage, or the coefficient of thermal expansion, α , should be obtained from an evaluation of test data.

Note: A range of values for the deformation properties of masonry is given in the table below. The values to be used in a particular country should be given in the National Annex.

Range of coefficients of creep, moisture expansion or shrinkage, and thermal properties of

masonry.

Type of masonry unit	Final creep coefficient (see note 1) ϕ_{∞}	Final moisture expansion or shrinkage (see note 2) mm/m	Coefficient of thermal expansion, α , $10^{-6}/K$
	Range	Range	Range
Clay	0,5 to 1,5	-0,2 to +1,0	4 to 8
Calcium Silicate	1,0 to 2,0	-0,4 to -0,1	7 to 11
Dense aggregate concrete and manufactured stone	1,0 to 2,0	-0,6 to -0,1	6 to 12
Lightweight aggregate concrete	1,0 to 3,0	-1,0 to -0,2	8 to 12
Autoclaved aerated concrete	1,0 to 2,5	-0,4 to +0,2	7 to 9
Natural stone Magmatic Sedimentary Metamorphic	(see note 3)	-0,4 to +0,7	5 to 9 2 to 7 1 to 18
<p>Notes:</p> <p>1. The final creep coefficient $f_{\infty} = \varepsilon_{C\infty} / \varepsilon_{el}$, where $\varepsilon_{C\infty}$ is the final creep strain and $\varepsilon_{el} = \sigma / E$.</p> <p>2. Where the final value of moisture expansion or shrinkage is shown minus it indicates shortening and where plus it indicates extension.</p> <p>3. Values are normally very low.</p>			

3.8 Ancillary components

3.8.1 Damp proof courses

(1)P Damp proof courses shall resist the passage of water.

3.8.2 Wall ties

(1)P Wall ties shall be in accordance with EN 845-1.

3.8.3 Straps, hangers and brackets

(1)P Straps, hangers and brackets shall be in accordance with EN 845-1.

3.8.4 Prefabricated lintels

(1)P Prefabricated lintels shall be in accordance with EN 845-2

3.8.5 Prestressing devices

(1)P Anchorages, couplers, ducts and sheaths shall be in accordance with the requirements of EN 1992-1-1.

4 Durability

4.1 General

(1)P Masonry shall be designed to have adequate durability for its intended use and taking into account the relevant environmental condition.

4.2 Classification of environmental conditions

4.2.1 Micro conditions of exposure

(1)P The micro conditions to which the masonry is expected to be exposed shall be taken into account in the design.

(2)P When deciding the micro conditions of exposure of the masonry, the effect of applied finishes and protective claddings shall be taken into account

(3) Micro conditions of exposure of completed masonry should be categorised into classes, as follows:

MX1 - In a dry environment;

MX2 - Exposed to moisture or wetting;

MX3 - Exposed to wetting plus freeze/thaw cycling;

MX4 - Exposed to saturated salt air, seawater or other salt laden water;

MX5 - In an aggressive chemical environment.

Note: When necessary, more closely defined conditions within these classes may be specified using the sub-classes in Annex B (e.g. MX2.1 or MX2.2).

(4) To produce masonry that meets specified performance criteria and withstands the environmental conditions to which it is exposed, the determination of the exposure class should take into account:

- climatic factors;
- severity of exposure to wetting;
- exposure to freeze/thaw cycling;
- presence of chemical materials that may lead to damaging reactions.

4.2.2 Climatic factors (macro conditions of exposure)

(1) The effect of the macro conditions on the micro conditions should be taken into account when determining the relative wetness of masonry and its exposure to freeze/thaw cycling.

(2) Concerning the macro conditions the following should be taken into account:

- rain and snow;
- the combination of wind and rain;
- temperature variation;
- relative humidity variation.

4.2.3 Exposure to wetness

(1) The exposure to wetness should be taken into account in determining the micro conditions of exposure of the masonry. The effect of any applied finishes, cladding, weathered overhanging sills, copings, string courses, drainage or other features intended to throw water clear of the masonry should be considered.

Note: It is acknowledged that climates (macro conditions) vary considerably throughout Europe and that certain aspects of climate can influence the risk of exposure of masonry to wetting and/or freeze/thaw cycling. However, it is the classification of the micro conditions that is relevant for determining the durability of masonry rather than the ranking of the macro conditions. Examples of relative exposure to wetness of masonry elements in a typical building are given in prEN 1996-2.

4.2.4 Exposure to freeze/thaw cycling

(1) The range and nature of temperature variations should be taken into account in determining the classification of micro conditions of exposure of the masonry.

4.3 Durability of masonry

4.3.1 Masonry units

(1)P Masonry units shall be sufficiently durable to resist the relevant exposure conditions for the intended life of the building.

Note: Guidance on design and construction to provide adequate durability is given in sections 8 and 9 of this EN 1996-1-1 and in EN 1996-2.

4.3.2 Mortar

(1)P Mortar in masonry shall be sufficiently durable to resist relevant micro exposure conditions for the intended life of the building, and shall not contain constituents which can have a detrimental effect on the properties or durability of the mortar or abutting materials.

Note: Guidance on design and construction to achieve adequate durability of mortar joints is given in section 8 of this EN 1996-1-1 and EN 1996-2.

4.3.3 Reinforcing steel

(1)P Reinforcing steel shall be sufficiently durable, either by being corrosion resistant or adequately protected, so that, when placed in accordance with the application rules in section 8, it will resist local exposure conditions for the intended life of the building.

(2) Where carbon steel requires protection to provide adequate durability, it should be galvanised in accordance with EN ISO 1461, such that the zinc coating is not less than that required to provide the necessary durability (see (4), below) or the steel should be given an equivalent protection such as by fusion bonded epoxy powder.

(3) The type of reinforcing steel and the minimum level of protection for the reinforcing steel that should be used in masonry in the various exposure classes, as defined in 4.2, is given in table 4.1. This table applies to carbon steel, austenitic stainless steel and galvanised steel when cover to the reinforcing steel is provided in accordance with 8.2.4. Alternatively, where unprotected carbon steel is used, it may be protected by concrete cover in accordance with table 4.2.

(4) Where galvanising is used to provide protection, the reinforcing steel should be galvanised after it has been bent to shape.

(5) For prefabricated bed joint reinforcement, EN 845-3 lists the protection systems that are to be declared by the manufacturer.

4.3.4 Prestressing steel

(1)P Prestressing steel shall be sufficiently durable, when placed in accordance with the application rules in section 8, to resist relevant micro exposure conditions for the intended life of the building.

(2) When prestressing steel is to be galvanised it should be of such a composition that it will not be adversely affected by the galvanising process.

4.3.5 Damp proof courses

(1)P Damp proof courses shall be durable for the type of building; they shall be formed from materials which are not easily punctured in use and they shall be able to resist stresses without exuding.

4.3.6 Wall ties

(1)P Wall ties and their fixings shall be able to withstand the relevant environmental action and differential movements between leaves. They shall be corrosion resistant in the environment in which they are used.

4.3.7 Straps, hangers, brackets and support angles

(1)P Straps, hangers, brackets and support angles shall be corrosion resistant in the environmental condition in which they are used.

4.3.8 Prefabricated lintels

(1)P Prefabricated lintels shall be corrosion resistant in the environmental condition in which they are used.

4.3.9 Prestressing devices

(1)P Anchorages, couplers, ducts and sheaths shall be corrosion resistant in the environmental condition in which they are used.

4.4 Masonry below ground

(1)P Masonry below ground shall be such that it is not adversely affected by the ground conditions or it shall be suitably protected therefrom.

(2) Measures should be taken to protect masonry that may be damaged by the effects of moisture when in contact with the ground.

(3) When the soil is likely to contain chemicals which might be harmful to the masonry, the masonry should be constructed of materials resistant to the chemicals or it should be protected in such a way that the aggressive chemicals cannot be transmitted into it.

Table 4.1 : Selection of reinforcing steel for durability.

Exposure class	Minimum level of protection for reinforcing steel	
	Located in mortar	Located in concrete with cover less than required in table 4.2
MX1	Unprotected carbon steel (see note 1)	Unprotected carbon steel
MX2	Carbon steel, heavily galvanised or with equivalent protection (see note 2)	Unprotected carbon steel or, where mortar is used to fill in the voids, carbon steel, heavily galvanised or with equivalent protection (see note 2)
	Unprotected carbon steel, in masonry with a rendering mortar on the exposed face (see note 3)	
MX3	Austenitic stainless steel (see note 4)	Carbon steel, heavily galvanised or with equivalent protection (see note 2)
	Unprotected carbon steel, in masonry with a rendering mortar on the exposed face (see note 3)	
MX4 & MX5	Austenitic stainless steel AISI 316 or 304 (see note 4)	Austenitic stainless steel AISI 316 or 304 (see note 4)
<p>Notes:</p> <p>1. For the inner leaf of external cavity walls likely to become damp, carbon steel, heavily galvanised or with equivalent protection as note 2, should be used.</p> <p>2. Carbon steel should be galvanised with a minimum mass of zinc coating of 900g/m² or galvanised with a minimum mass of zinc coating of 60g/m² and provided with a bonded epoxy coating of at least 80µm thickness, with an average of 100µm. See also 3.4.</p> <p>3. The masonry mortar should be general purpose mortar, not less than M5, the side cover in figure 8.2 should be increased to 30mm and the masonry should be rendered with a rendering mortar in accordance with EN 998-1.</p>		

[PT Note: Review this table in conjunction with the protection systems listed in 845-1 & 2; frost relevance for reinforcement.]

Table 4.2 : Minimum concrete cover for unprotected carbon steel

Exposure class	Minimum thickness of concrete cover (mm)			
	Water/cement ratio not greater than			
	0,65	0,55	0,50	0,45
	Cement content (kg/m ³) not less than			
	260	280	300	300
MX 1	20	20	20	20
MX 2	-	25	25	25
MX 3	-	-	40	40
MX 4	-	-	40	40
MX 5	-	-	-	40

[PT Note: review against EN 1992 – not in EN 206. Include review of “sucking out” improvement in w/c ratio. Professor Weck will do this as soon as the new draft of EN 1992-1-1 is available (Oct/Nov 2001?)]

5 Structural analysis

5.1 General

(1)P For each relevant limit state verification, a calculation model of the structure shall be set up from:

- an appropriate description of the structure, the materials from which it is made, and the relevant environment of its location;
- the behaviour of the whole or parts of the structure, related to the relevant limit state;
- the actions and how they are imposed.

(2)P The general arrangement of the structure and the interaction and connection of its various parts shall be such as to give appropriate stability and robustness during construction and use.

(3) Calculation models may be based on separate parts of the structure (such as walls) independently, provided that 5.1(2)P is satisfied.

Note: Where the structure is made of separately designed components the overall stability and robustness should be ensured.

(4) The response of the structure should be calculated using either

- non linear theory of elasticity assuming a relationship between stress and strain (see 3.7.1)

or

- linear theory of elasticity assuming a linear relationship between stress and strain with a slope equal to the short term secant modulus of elasticity (see 3.7.2).

(5) The results obtained from analysis of the calculation models should provide, in any member,

- the axial loads due to vertical loading
- the shear loads due to vertical and/or horizontal loading
- the eccentricity of the axial loads
- the bending moments due to vertical or lateral loading

(6)P Structural members shall be verified in the ultimate limit state and the serviceability limit state, using, as actions the results obtained from the analysis.

(7) Design rules for verification in the ultimate limit state and the serviceability limit state are given in Sections 6 and 7.

5.2 Structural behaviour in accidental situations (other than earthquakes and fire)

(1)P In addition to designing the structure to support loads arising from normal use, it shall be ensured that there is a reasonable probability that it will not be damaged under the effect of misuse or accident to an extent disproportionate to the original cause.

Note: no structure can be expected to be resistant to the excessive loads or forces, or loss of bearing members or portions of the structure, that could arise due to an extreme cause. For example in a small building the primary damage may cause total destruction.

(2) The structural behaviour under accidental situations should be considered using one of the following methods:

- members designed to resist the effects of accidental actions given in EN 1991-1-7
- the hypothetical removal of essential loadbearing members in turn, with special attention being paid to the integrity of ties and restraints to members;
- reducing the risk of accidental actions, such as the use of impact barriers against vehicle impact.

5.3 Imperfections

(1)P The possible effects of imperfections shall be allowed for by assuming that the structure is inclined at an angle $\psi = 1/(100 \sqrt{h_{\text{tot}}})$ radians to the vertical,

where:

h_{tot} is the total height of the structure in metres.

The resulting horizontal action shall be added to the other actions.

5.4 Second order effects

(1)P Structures incorporating masonry walls designed according to this EN 1996-1-1 shall have their parts braced together adequately so that sway of the structure is either prevented or allowed for by calculation.

(2) No allowance for sway of the structure is necessary if the vertical stiffening elements satisfy equation (5.1) in the relevant direction of bending:

$$\begin{aligned} h_{\text{tot}} (N/\sum EI)^{1/2} &\leq 0,6 && \text{for } n \geq 4 \\ &\leq 0,2 + 0,1n && \text{for } 1 \leq n < 4 \end{aligned} \quad (5.1)$$

where

- h_{tot} is the total height of the structure from the top of the foundation;
- N is the sum of the characteristic vertical permanent and variable actions on the building;
- $\sum EI$ is the sum of the bending stiffnesses of all vertical stiffening building elements in the relevant direction;

Note: Openings in vertical stiffening elements of less than 2m² with heights not exceeding 0.6h may be neglected.

n is the number of storeys.

(3) When the vertical stiffening elements do not satisfy 5.4 (2), the total eccentricity of a stability core due to sway should be calculated from:

$$e_t = \xi \cdot \left(\frac{M_d}{N_d} e_o + e_c \right) \quad (5.2)$$

where:

- M_d is the design bending moment at the bottom of the core, calculated using the linear theory of elasticity
- N_d is the design vertical load at the bottom of the core, calculated using the linear theory of elasticity
- e_c is an additional eccentricity
- ξ is a magnification factor for the rotational stiffness of the restraint of the structural element being considered

The additional eccentricity e_c and the magnification factor ξ may be calculated from equation (5.5) and (5.6) (see figure 5.1):

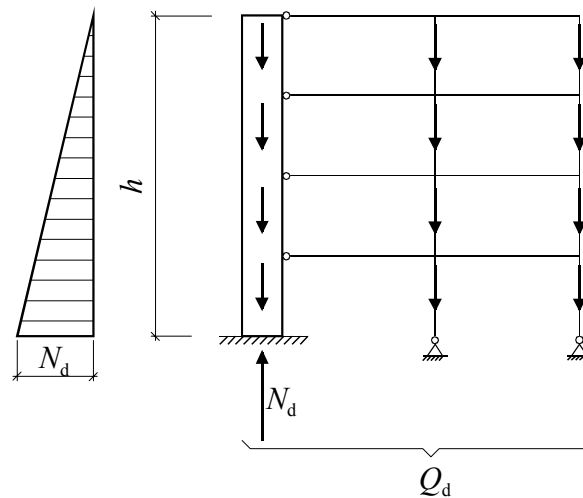


Figure 5.1: Representation of a stability core

$$\xi = \frac{k}{k - 0,5N_d \cdot h \cdot \frac{Q_d}{N_d}} \quad (5.3)$$

$$e_c = \frac{Q_d}{N_d} \cdot 4,5 d \cdot \left(\frac{h}{100 d} \right)^2 \quad (5.4)$$

where:

- k is the rotational stiffness of the restraint in Nmm/rad;
- h is the height of the wall or core in mm;
- d is the largest dimension of the cross section of the structural element in the bending direction, in mm;
- N_d is the design value of the vertical load in the stability core, in N; and
- Q_d is the design value of the total vertical load, of the part of the building that is stabilized by the core being considered.

5.5 Analysis of structural members

5.5.1 Masonry walls subjected to vertical loading

(1) When analysing walls subjected to vertical loading, allowance in the design should be made for the following:

- vertical loads directly applied to the wall;
- second order effects;
- eccentricities calculated from a knowledge of the layout of the walls, the interaction of the floors and the stiffening walls;
- eccentricities resulting from construction deviations and differences in the material properties of individual components.

(2) The bending moments may be calculated from the material properties given in Section 3, the joint behaviour, and from the principles of structural mechanics.

Note: A simplified method for calculating the bending moments in walls due to vertical loading is given in Annex C.

(3)P An accidental eccentricity, e_a , shall be assumed for the full height of a wall to allow for construction imperfections.

(4) The accidental eccentricity, e_a , may be assumed to be $h_{ef} / 450$, where h_{ef} is the effective height of the wall, calculated from 6.1.4.

5.5.2 Reinforced masonry members subjected to vertical loading

5.5.2.1 Effective span of masonry beams

(1) The effective span, l_{ef} , of simply supported or continuous masonry beams, with the exception of deep beams, may be taken as the smaller of the following:

- the distance between centres of supports;
- the clear distance between supports plus the effective depth, d .

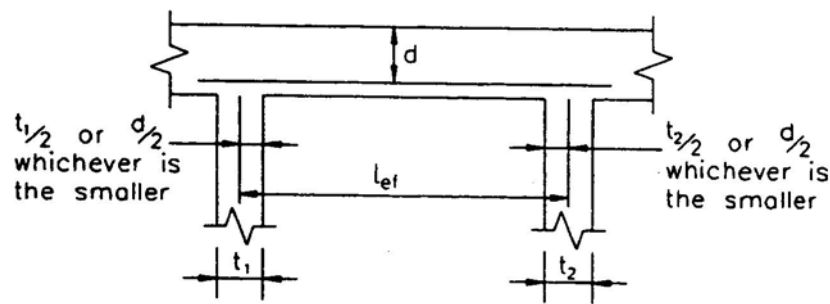


Figure 5.2: Effective span of simply supported or continuous masonry beams

(2) The effective span, l_{ef} , of a masonry cantilever may be taken as the smaller of the following (see figure 5.3):

- the distance between the end of the cantilever and the centre of its support;
- the distance between the end of the cantilever and the face of the support plus half its effective depth, d .

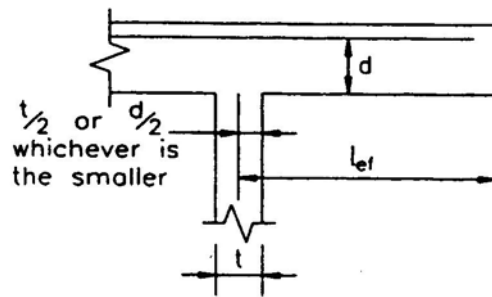


Figure 5.3: Effective span of masonry cantilever

(3) The effective span of deep beams may be determined according to 5.5.2.2.

5.5.2.2 Deep masonry beams subjected to vertical loading

(1) Deep masonry beams are vertically loaded walls, or parts of walls, bridging openings, such that the ratio of the overall height of the wall above the opening to the effective span of the opening is at least 0,5.

The effective span of the beam may be taken as:

$$l_{ef} = 1,15 l \quad (5.5)$$

where

l is the clear span of the opening.

(2) All the vertical loads acting on that part of the wall situated above the effective span should be taken into account, unless the loads can be taken by other means, for example, by upper floors acting as ties.

(3) In order to determine the bending moments, the deep beam may be considered as simply supported between supports as shown in figure 5.4.

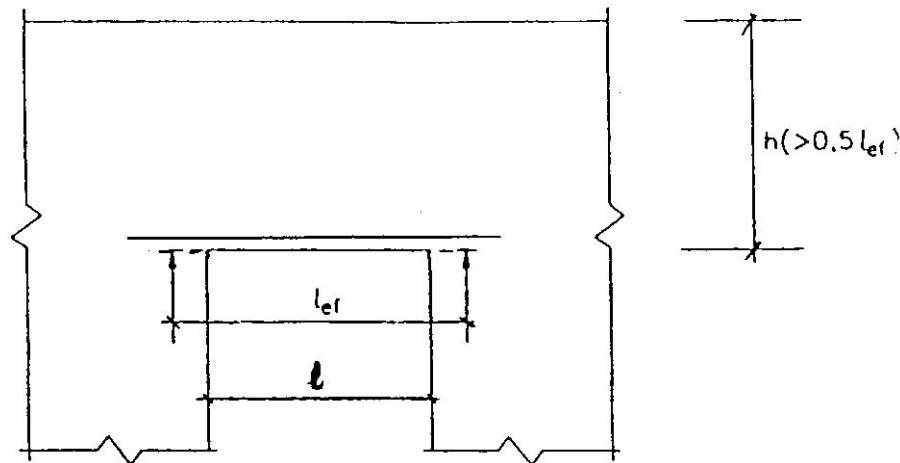


Figure 5.4: Deep masonry beam

5.5.3 Masonry walls subjected to shear loading

(1) Walls subjected to shear loading should be analysed for the appropriate load cases.

(2) When deriving the relevant design load that assists shear resistance, the vertical load applied to slabs spanning in two directions may be distributed equally onto the supporting walls; in the case of floor or roof slabs spanning one way, a 45° spread of the load may be considered in deriving the axial load, at the lower storeys, on the walls not directly loaded.

(3) An intersecting wall, or a portion of such a wall, may be considered to act as a flange to a shear wall provided that the connection of the main shear wall to the flange is able to resist the corresponding shearing actions, and provided the flange will not buckle within the length assumed.

(4) The length of any intersecting wall, which may be considered to act as a flange, is the thickness of the shear wall plus, on each side of it, where appropriate, the least of (see also figure 5.5):

- $h_{tot}/5$, where h_{tot} is the overall height of the shear wall;
- half the distance between shear walls (l), when connected by the intersecting wall;

- the distance to the end of the wall;
- half the clear height (h).

In intersecting walls, openings with a width smaller than $h/4$ may be disregarded. Openings with a width greater than $h/4$ should be regarded as marking the end of the wall.

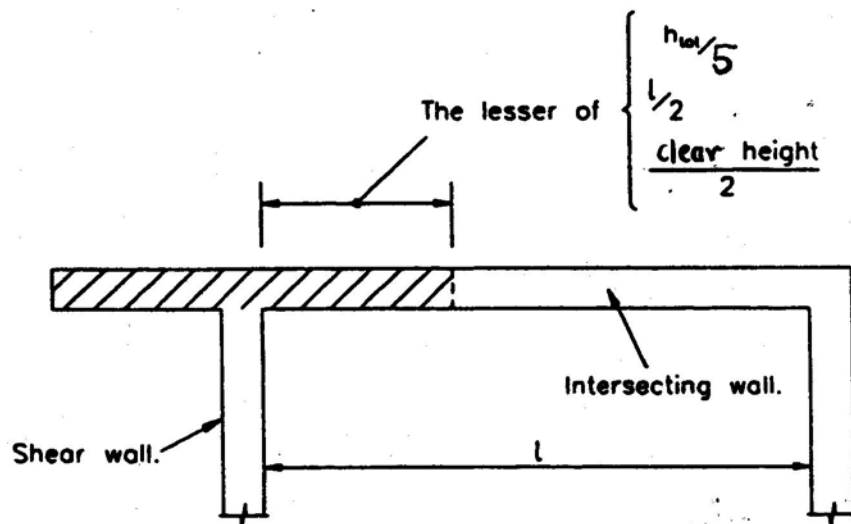


Figure 5.5: Flange widths that can be assumed for shear walls

- (5) The elastic stiffness of the shear walls, including any flanges, should be used as the stiffness of the wall. For walls higher than twice their length, the effect of shear strains on the stiffness can be neglected.
- (6) If the floors can be idealised as rigid diaphragms, the horizontal forces may be distributed to the shear walls in proportion to their stiffness.
- (7)P Where the plan arrangement of the shear walls is asymmetric, or for any other reason the horizontal force is eccentric to the overall stiffness centre of the structure, account shall be taken of the effect of the consequent rotation on the individual walls (torsional effects).
- (8) If the floors are not sufficiently rigid when considered as horizontal diaphragms (for example, precast concrete units which are not inter-connected) horizontal forces to be resisted by the shear walls should be taken to be the forces from the floors to which they are directly connected, unless a semi rigid analysis is carried out.
- (9) The maximum horizontal load on a shear wall may be reduced by up to 15% provided that the load on the parallel shear walls is correspondingly increased.

5.5.4 Masonry walls subjected to lateral loading

[PT Note: Clause temporarily removed to Part -1-3.]

6 Ultimate Limit State

6.1 Unreinforced masonry walls subjected to mainly vertical loading

6.1.1 General

(1)P The resistance of masonry walls to vertical loading shall be based on the geometry of the wall, the effect of the applied eccentricities and the material properties of the masonry.

(2) In calculating the vertical resistance of masonry walls, it may be assumed that:

- plane sections remain plane;
- the tensile strength of masonry perpendicular to bed joints is zero;

6.1.2 Verification of unreinforced masonry walls subjected to mainly vertical loading

(1)P At the ultimate limit state, the design value of the vertical load applied to a masonry wall, N_{Sd} , shall be less than or equal to the design value of the vertical resistance of the wall, N_{Rd} , such that:

$$N_{Sd} \leq N_{Rd} \quad (6.1)$$

(2) The design value of the vertical resistance of a single leaf wall per unit length, N_{Rd} , is given by:

$$N_{Rd} = \Phi t f_d \quad (6.2)$$

where:

Φ is the capacity reduction factor, Φ_i , at the top or bottom of the wall, or Φ_m , in the middle of the wall, as appropriate, allowing for the effects of slenderness and eccentricity of loading, obtained from 6.1.3.

t is the thickness of the wall;

f_d is the design compressive strength of the masonry, obtained from 3.6.1.X.

(3) Where the cross-sectional area of a wall is less than $0,1 \text{ m}^2$, the design compressive strength of the masonry, f_d , should be multiplied by the factor:

$$(0,7 + 3 A) \quad (6.3)$$

where:

A is the loaded horizontal gross cross-sectional area of the wall, expressed in square metres.

(4) For cavity walls, each leaf should be verified separately, using the plan area of the loaded leaf and the slenderness ratio based upon the effective thickness of the cavity wall, calculated according to equation (6.15).

(5) A faced wall, should be designed in the same manner as a single-leaf wall constructed entirely of the weaker units, using the value of K, from Table 3.4, appropriate to a wall with a longitudinal mortar joint.

(6) A double-leaf wall tied together according to clause 6.3.3 may be designed as a single-leaf wall, if both leaves have a load of similar magnitude, or, alternatively, as a cavity wall.

(7) When chases or recesses are outside the limits given in clause 8.6, the effect on loadbearing capacity should be taken into account as follows:

- vertical chases or recesses should be treated either as a wall end or, alternatively, the residual thickness of the wall should be used in the calculations of the design vertical load resistance;
- horizontal or inclined chases should be treated by verifying the strength of the wall at the chase position, taking account of the load eccentricity.

Note: As a general guide the reduction in vertical loadbearing capacity may be taken to be proportional to the reduction in cross-sectional area due to any vertical chase or recess, provided that the reduction in area does not exceed 25%.

6.1.3 Reduction factor for slenderness and eccentricity

(1) The value of the reduction factor for slenderness and eccentricity, Φ , may be obtained as follows:

(i) At the top or bottom of the wall (Φ_i)

$$\Phi_i = 1 - 2 \frac{e_i}{t} \quad (6.4)$$

where:

e_i is the eccentricity at the top or the bottom of the wall, as appropriate, calculated using the equation (6.5):

$$e_i = \frac{M_{id}}{N_{id}} + e_{hi} + e_a \geq 0,05t \quad (6.5)$$

- M_{id} is the design value of the bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support, analysed according to 5.5.1;
- N_{id} is the design value of the vertical load at the top or bottom of the wall;
- e_{hi} is the eccentricity at the top or bottom of the wall, if any, resulting from horizontal loads (for example, wind);
- e_a is the accidental eccentricity (see 5.5.1);
- t is the thickness of the wall.

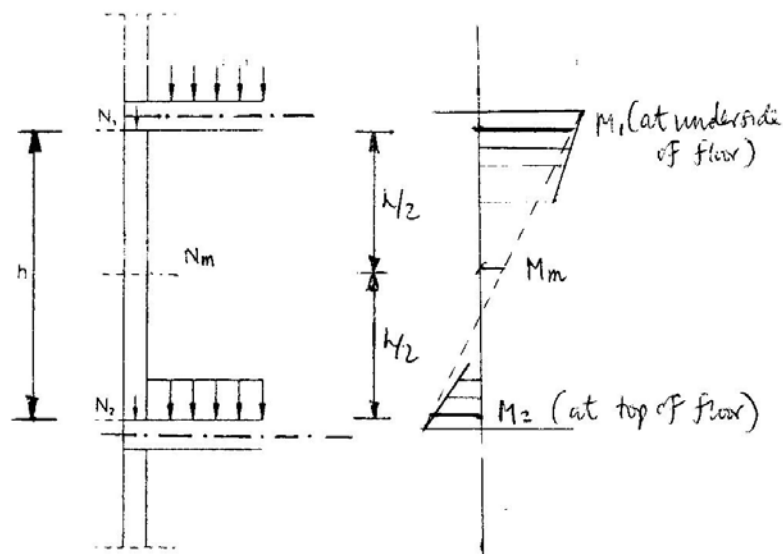


Figure 6.1: Moments from calculation of eccentricities

(ii) In the middle of the wall height (Φ_m)

By using a simplification of the general principles given in 6.1.1, the reduction factor within the middle height of the wall, Φ_m , may be determined from figure 6.1:

Note: Figure 6.1 is based on the formula given in Annex E, with E taken as equal to $1000f_k$.

where:

e_{mk} is the eccentricity within the middle one fifth of the wall height, calculated using equations (6.6) and (6.7):

$$e_{mk} = e_m + e_k \geq 0,05t \quad (6.6)$$

$$e_m = \frac{M_{md}}{N_{md}} + e_{hm} \pm e_a \quad (6.7)$$

- e_m is the eccentricity due to loads;
- M_{md} is the design value of the greatest moment in the middle of the height of the wall resulting from the moments at the top and bottom of the wall (see figure 6.1);
- N_{md} is the design value of the vertical load at the middle of the height of the wall;
- e_{hm} is the eccentricity at mid-height resulting from horizontal loads (for example, wind);

Note: the inclusion of e_{hm} depends on the load combination being used for the verification.

- h_{ef} is the effective height, obtained from 6.1.4 for the appropriate restraint or stiffening condition;
- t_{ef} is the effective thickness of the wall, obtained from 6.1.5;
- e_k is the eccentricity due to creep, calculated from the equation (6.8):

$$e_k = 0,002 \phi_{\infty} \frac{h_{ef}}{t_{ef}} \sqrt{t e_m} \quad (6.8)$$

- ϕ_{∞} is the final creep coefficient (see note under 3.7.4(2))

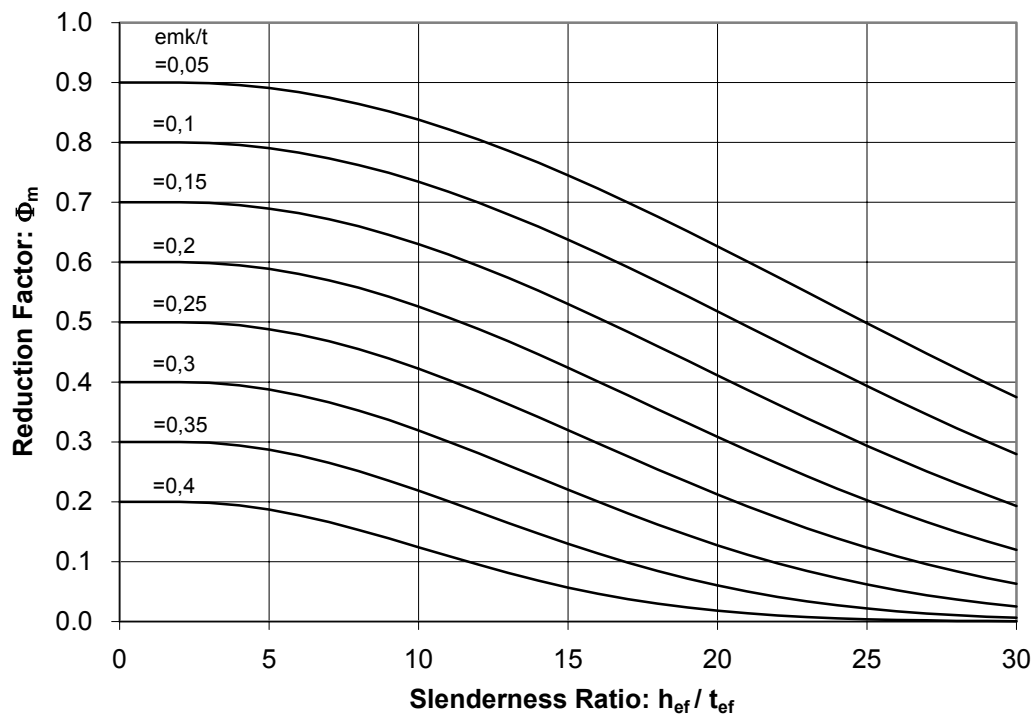


Figure 6.2: Values of Φ_m against slenderness ratio for different eccentricities

- (2) The creep eccentricity, e_k , may be taken as zero for all walls built with clay and natural stone units and for walls having a slenderness ratio up to 15 constructed from other masonry units.
- (3) The value of e_{hi} and e_{hm} should not be applied to reduce e_i and e_m , respectively.

6.1.4 Effective height of walls

- (1)P The effective height of a loadbearing wall shall be assessed taking account of the relative stiffness of the elements of structure connected to the wall and the efficiency of the connections.
- (2) A wall may be stiffened by floors, or roofs, suitably placed cross walls, or any other similarly rigid structural elements to which the wall is connected.
- (3) Walls may be considered as stiffened at a vertical edge if:

- cracking between the wall and its stiffening wall is not expected to occur i.e. both walls are made of materials with approximately similar deformation behaviour, are approximately evenly loaded, are erected simultaneously and bonded together and differential movement between the walls, for example, due to shrinkage, loading etc., is not expected, or

- the connection between a wall and its stiffening wall can resist tension and compression forces by anchors or ties or other suitable means.

(4) Stiffening walls should have a length of at least $1/5$ of the clear height and have a thickness of at least 0,3 times the effective thickness of the wall to be stiffened.

(5) If the stiffening wall is interrupted by openings, the minimum length of the wall between openings, encompassing the stiffened wall, should be as shown in figure 6.3, and the stiffening wall should extend a distance of at least $1/5$ of the storey height beyond each opening.

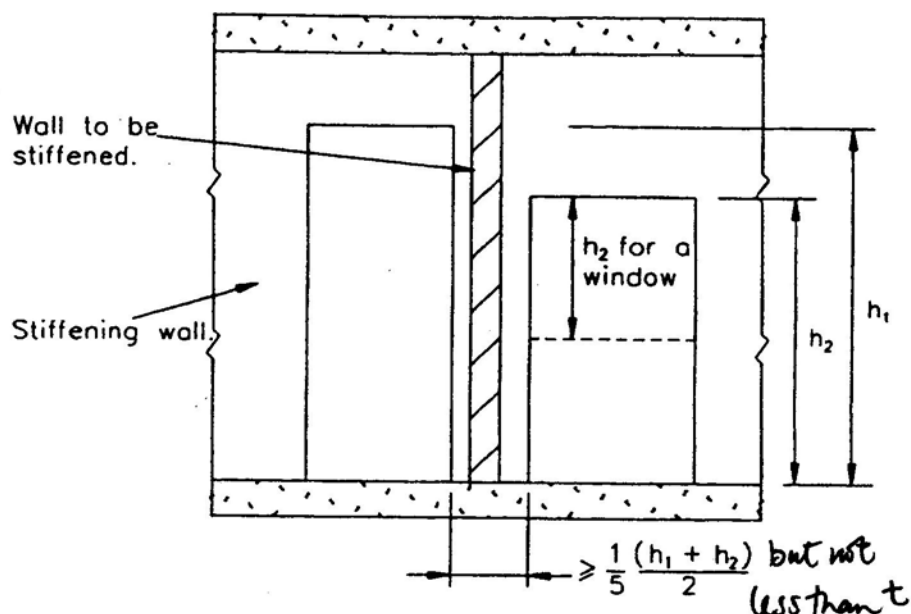


Figure 6.3: Minimum length of stiffening wall with openings

(6) Walls may be stiffened by members other than masonry walls provided that such members have the equivalent stiffness of the masonry stiffening wall, described in paragraph (4) above, and they are connected to the stiffened wall with anchors or ties designed to resist the tension and compression forces that will develop.

(7) Walls stiffened on two vertical edges, with $L \geq 30 t$, or walls stiffened on one vertical edge, with $L \geq 15 t$, where L is the length of the wall t and is the thickness of the stiffened wall, should be treated as walls restrained at top and bottom only.

(8) If the stiffened wall is weakened by vertical chases and/or recesses, other than those allowed by 6.1.2, the reduced thickness of the wall should be used for t , or a free edge should be assumed at the position of the vertical chase or recess. A free edge should always be assumed when the thickness of the wall remaining after the vertical chase or recess has been formed is less than half the wall thickness.

(9) Walls with openings having a clear height of more than 1/4 of the clear height of the wall or a clear width of more than 1/4 of the wall length or an area of more than 1/10 of the total area of the wall, should be considered as having a free edge at the edge of the opening for the purposes of determining the effective height.

(10) The effective height of a wall should be taken as:

$$h_{\text{ef}} = \rho_n h \quad (6.9)$$

where:

h_{ef} is the effective height of the wall;

h is the clear storey height of the wall;

ρ_n is a reduction factor where $n = 2, 3$ or 4 depending on the edge restraint or stiffening of the wall.

(11) The reduction factor, ρ_n , may be assumed to be:

(i) For walls restrained at the top and bottom by reinforced concrete floors or roofs spanning from both sides at the same level or by a reinforced concrete floor spanning from one side only and having a bearing of at least 2/3 the thickness of the wall:

$$\rho_2 = 0,75 \quad \text{unless the eccentricity of the load at the top of the wall is greater than 0,25 times the thickness of wall in which case } \rho_2 \text{ should be taken as 1,0.}$$

(ii) For walls restrained at the top and bottom by timber floors or roofs spanning from both sides at the same level or by a timber floor spanning from one side having a bearing of at least 2/3 the thickness of the wall but not less than 85mm:

$$\rho_2 = 1,0.$$

(iii) For walls restrained at the top and bottom and stiffened on one vertical edge (with one free vertical edge):

$$\rho_3 = \frac{1}{1 + \left[\frac{\rho_2 h}{3L} \right]^2} \rho_2 \quad \rho_2 > 0,3 \quad (6.10)$$

when $h \leq 3,5 L$, with ρ_2 from (i), (ii) or (iii) whichever is appropriate, or

$$\rho_3 = \frac{1,5 L}{h} \quad (6.11)$$

when $h > 3,5 L$,

where

L is the distance of the free edge from the centre of the stiffening wall.

Note: Values for ρ_3 are shown in graphical form in Annex F.

(iv) For walls restrained at the top and bottom and stiffened on two vertical edges:

$$\rho_4 = \frac{1}{1 + \left[\frac{\rho_2 h}{L} \right]^2} \rho_2 \quad (6.12)$$

when $h \leq L$, with ρ_2 from (i), (ii) or (iii) whichever is appropriate, or

$$\rho_4 = \frac{0,5 L}{h} \quad (6.13)$$

when $h > L$.

where L is the distance between the centres of the stiffening walls.

Note: Values for ρ_4 are shown in graphical form in Annex F.

6.1.5 Effective thickness of walls

(1) The effective thickness, t_{ef} , of a single-leaf wall, a double-leaf wall, a faced wall, a shell bedded wall and a grouted cavity wall, as defined in 1.4.2.9, should be taken as the actual thickness of the wall, t .

(2) The effective thickness of a wall stiffened by piers should be obtained from equation (6.14):

$$t_{ef} = \rho_t t \quad (6.14)$$

where

t_{ef} is the effective thickness

ρ_t is a coefficient obtained from table 6.1

t is the thickness of the wall

Table 6.1: Stiffness coefficient, ρ_t , for walls stiffened by piers

Ratio of pier spacing (centre to centre) to pier width	Ratio of pier thickness to actual thickness of wall to which it is bonded		
	1	2	3
6	1,0	1,4	2,0
10	1,0	1,2	1,4
20	1,0	1,0	1,0

Note: Linear interpolation between the values given in Table 6.1 is permissible, but not extrapolation outside the limits given.

(3) The effective thickness, t_{ef} , of a cavity wall in which both leaves are connected with wall ties in accordance with 6.3.3 should be determined using equation (6.15):

$$t_{ef} = \sqrt[3]{t_1^3 + t_2^3} \quad (6.15)$$

where

t_1 and t_2 are the actual thicknesses of the leaves or their effective thicknesses, calculated from equation (6.14), when relevant.

(4) When the effective thickness would be overestimated if the loaded leaf of a cavity wall has a higher E value than the outer leaf, the relative stiffness should be taken into account when calculating t_{ef} .

(5) When only one leaf of a cavity wall is loaded, equation (6.15) may be used to calculate the effective thickness, provided that the wall ties have sufficient flexibility such that the loaded leaf is not affected adversely by the unloaded leaf. In calculating the effective thickness, the thickness of the unloaded leaf should not be taken to be greater than the thickness of the loaded leaf.

6.1.6 Slenderness ratio of walls

(1)P The slenderness ratio of a wall shall be obtained by dividing the value of the effective height, h_{ef} , by the value of the effective thickness, t_{ef} .

(2) The slenderness ratio of the wall should not be greater than 27 when subjected to mainly vertical loading.

6.1.7 Concentrated loads

(1)P The design value of a concentrated vertical load, $N_{Sd,c}$, applied to an unreinforced wall, shall be less than or equal to the design value of the vertical resistance of the wall, $N_{Rd,c}$, such that

$$N_{Sd,c} \leq N_{Rd,c} \quad (6.16)$$

(2) When a wall, built with Group 1 masonry units and detailed in accordance with Section 8, other than a shell bedded wall, is subjected to a concentrated load, the design value of the vertical load resistance of the wall is given by:

$$N_{Rd,c} = \beta A_b f_d \quad (6.17)$$

$$\text{where } \beta = (1 + 0,15x)(1,5 - 1,1 \frac{A_b}{A_{ef}}) \quad (6.18)$$

which should not be less than 1,0 nor taken to be greater than:

$$1,25, \text{ where } \frac{2a_1}{H} = 0$$

and

$$1,5, \text{ where } \frac{2a_1}{H} \geq 1,0$$

with the upper limit linearly interpolated between 1,25 and 1,5.

where:

β is an enhancement factor for concentrated loads;

$x = \frac{2a_1}{H}$ but not greater than 1,0;

a_1 is the distance from the end of the wall to the nearer edge of the bearing area (see figure 6.4);

H is the height of the wall to the level of the load;

A_b is the bearing area, not taken to be greater than $0,45 A_{ef}$;

A_{ef} is the loaded area;

L_{ef} is the effective length of the bearing as determined at the mid height of the wall or pier (see figure 6.4), to be taken as greater than $2,2 A_b/t$;

t is the thickness of the wall, taking into account the depth of recesses in joints greater than 5mm.

Note: Values for the enhancement factor for β are shown in graphical form in Annex G.

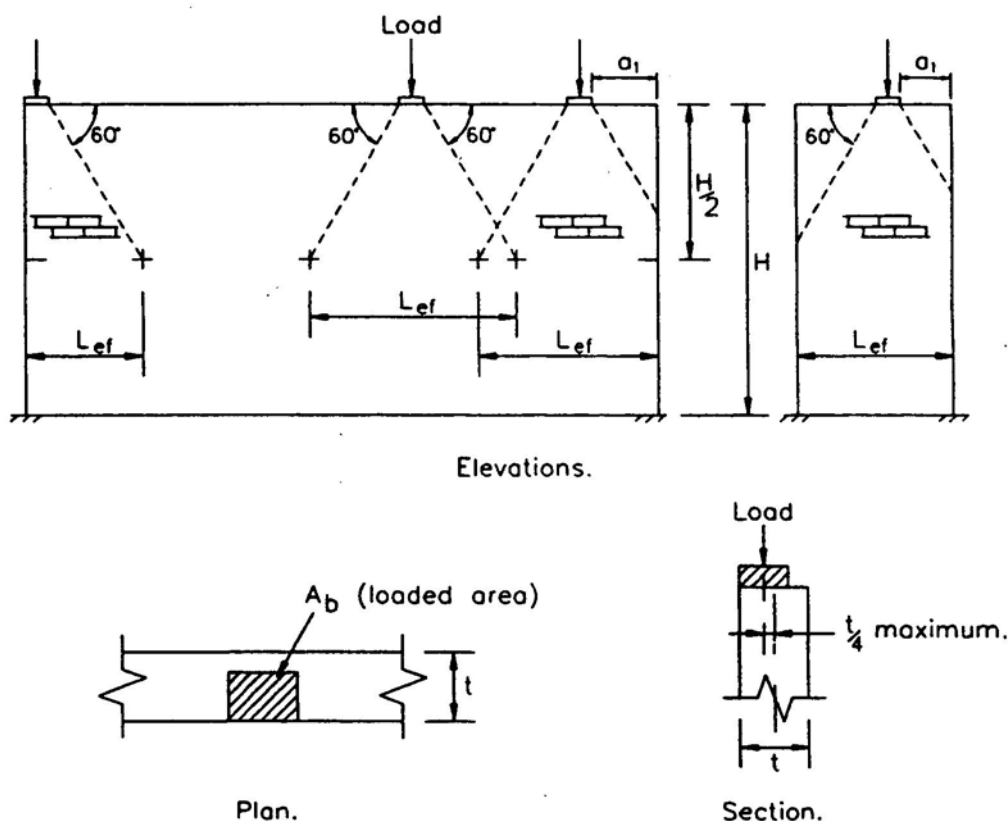


Figure 6.4: Walls subjected to concentrated load

(3) For walls built with Group 2 and Group 3 masonry units and when shell bedding is used, it should be verified that, locally under the bearing of a concentrated load, the design compressive stress does not exceed the design compressive strength of masonry, f_d .

(4) The eccentricity of the load from the centre line of the wall should not be greater than $t/4$ (see figure 6.4).

(5) In all cases, the requirements of 6.1.2 should be met at the middle height of the wall below the bearings, including the effects of any other superimposed vertical loading, particularly for the case where concentrated loads are sufficiently close together for their effective lengths to overlap.

(6) The concentrated load should bear on a Group 1 unit or other solid material of length equal to the required bearing length plus a length on each side of the bearing based on a 60° spread of load to the base of the solid material; for an end bearing the additional length is required on one side only.

(7) Where the concentrated load is applied through a spreader beam of adequate stiffness and of width t , height greater than 200mm and length greater than three times the bearing length of the load, the design value of the compressive stress beneath the concentrated load should not exceed $1,5 f_d$.

6.2 Unreinforced masonry subjected to shear loading

6.2.1 General

(1)P At the ultimate limit state the design value of the shear load applied to the masonry wall, V_{Sd} , shall be less than or equal to the design value of the shear resistance of the wall, V_{Rd} , such that :

$$V_{Sd} \leq V_{Rd} \quad (6.19)$$

(2) The design value of the shear resistance is given by:

$$V_{Rd} = f_{vd} t l_c \quad (6.20)$$

where:

f_{vd} is the design value of the shear strength of masonry, obtained from 3.6.2, based on the vertical load being resisted by the compressed part of the wall, ignoring any part of the wall that is in tension;

t is the thickness of the wall;

l_c is the length of the compressed part of the wall, ignoring any part of the wall that is in tension;

(3) The distribution of shear stress along a wall may be assumed to be constant.

(4) The length of the compressed part of the wall, l_c , should be calculated assuming a triangular or trapezoidal stress distribution, and taking into account any openings, chases or recesses; any portion of the wall subjected to vertical tensile stresses should not be used in calculating the area of the wall to resist shear.

(5)P The connections between shear walls and flanges of intersecting walls shall be verified for vertical shear.

(6) The vertical shear resistance of a connection may be obtained from suitable tests for a specific project or it may be taken from an evaluation of test data. In the absence of such data, the characteristic vertical shear resistance may be based on f_{vko} , where f_{vko} is the shear strength under zero compressive stress, as given in 3.6.2, and provided that the connection between the walls is in accordance with 8.5.2.1.

(7) The length of the compressed part of the wall should be verified for the vertical loading applied to it and the vertical load effect of the shear loads.

6.3 Unreinforced masonry: walls subjected to lateral loading

[PT Note: Temporarily removed to Part 1-3. This also affects the equation numbering system for the remainder of this section.]

[Start of PT Note:

In answer to the Danish request for a further method of dealing with vertical and horizontal loads, the following clause is being considered:

6.4 Unreinforced masonry walls subjected to combined vertical and lateral loading

6.4.1 General

(1) *Unreinforced masonry walls that are subjected to both vertical and lateral loading may be verified by using any one of the methods given in 6.4.2, 6.4.3 or 6.4.4, as appropriate.*

6.4.2 Method using Φ factor

(1) *By using the relevant value of the eccentricity due to horizontal action, e_{hi} or e_{hm} , a slenderness reduction factor that takes into account the combined vertical and horizontal loading can be obtained, using equations 6.5 and 6.7, for use in equation 6.2.*

6.4.3 Method using apparent f_{xk1}

(1) 6.3(4) allows a permanent vertical load to increase the characteristic flexural strength of masonry, f_{xk1} , to an apparent flexural strength, $f_{xk1.app}$, for use with the verification given in 6.3.

6.4.4 Method using equivalent bending coefficients

(1) Annex I gives a method of modifying the bending moment coefficient, α , as described in 6.3, to allow for both vertical and horizontal loads.

[End of PT note]

6.4 Reinforced masonry members subjected to bending, bending and axial loading, or axial loading

6.4.1 General

(1)P The design of reinforced masonry members, subjected to bending, bending and axial loading, or axial loading, shall be based on the following assumptions:

- plane sections remain plane;
- the reinforcement is subjected to the same variations in strain as the adjacent masonry;
- the tensile strength of the masonry is then taken to be zero;
- the maximum compressive strain of the masonry is chosen according to the material;
- the maximum tensile strain in the reinforcement is chosen according to the material;
- the stress-strain relationship of masonry is taken to be parabolic, parabolic rectangular or rectangular (see 3.7.1);
- the stress-strain relationship of the reinforcement is obtained from EN 1992-1-1;
- for cross-sections subject to pure longitudinal compression, the compressive strain in the masonry is limited to $\epsilon_m = -0,002$ (see figure 3.2);
- for cross-sections not fully in compression, the limiting compressive strain is taken to be not greater than $\epsilon_m = -0,0035$ for Group 1 units and $\epsilon_m = -0,002$ for Group 2 and 3 units (see figure 3.2). In intermediate situations, the strain diagram is defined by

(3) In reinforced masonry members, the linear elastic distribution of internal forces may be modified assuming equilibrium, if the members have sufficient ductility. The ratio of the depth of the neutral axis, x , to the effective depth, d , should not exceed 0,4 where no redistribution of moments has been carried out. Redistribution of moments in a

continuous member should be limited to 15% when steel of ductility Class C is to be used. In this case, the ratio of the redistributed moment to the moment before redistribution should not be less than:

$$0,44 + 1,25 \frac{x}{d} \quad (6.30)$$

provided that the characteristic compressive strength of the masonry or concrete infill is not taken to be greater than 35N/mm².

[PT Note: This clause comes from reinforced concrete – can DIN amendment be accepted? - it seems okay.]

(4) No redistribution should be allowed with steel of ductility Class A.

(5) In determining the design value of the moment of resistance of a section, a rectangular stress distribution as indicated in figure 6.7 may be assumed as a simplification, with the maximum compressive strain, ϵ_m , taken as -0,0035.

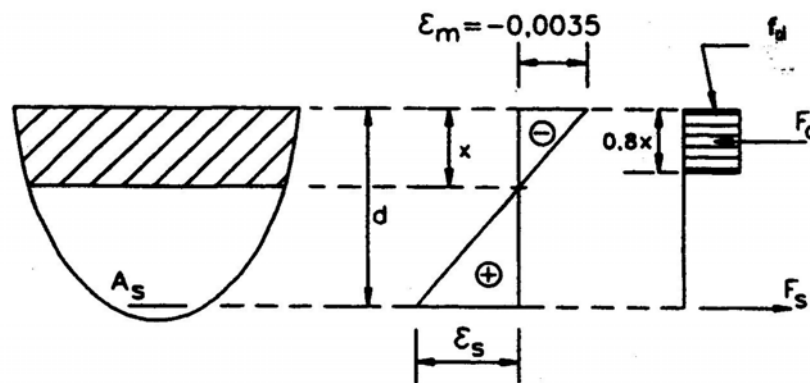


Figure 6.7: Simplified rectangular stress block

(6) For the case of a singly reinforced rectangular cross-section, subject to bending only, the design value of the moment of resistance, M_{Rd} , may be taken as :

$$M_{Rd} = A_s f_{yd} z \quad (6.31)$$

where, based on the simplification illustrated in figure 6.7, the lever arm, z , may be taken, for a section when the maximum compression and tension are reached together, as:

$$z = d \left[1 - 0,5 \frac{A_s f_{yd}}{b d f_d} \right] \leq 0,95d \quad (6.32)$$

where:

- b is the width of the section;
- d is the effective depth of the section;
- A_s is the cross-sectional area of the reinforcement in tension;
- f_d is the design compressive strength of masonry in the direction of loading, obtained from 3.6.1, or concrete infill, obtained from 3.3, whichever is the lesser and using the appropriate value of γ_M ;
- f_{yd} is the design strength of reinforcing steel;

Note: For the special case of reinforced masonry cantilever walls subjected to bending, refer to (7), below.

(7) In determining the design value of the moment of resistance, M_{Rd} , of reinforced masonry cantilever walls subject to bending, the design compressive strength f_d , in figure 6.7, may be taken over the full depth to the neutral axis, x , with x not taken to be greater than $d/2$, when the design value of the moment of resistance, M_{Rd} , in compression, should not be taken to be greater than:

$$0,4 f_d b d^2 \quad (6.33)$$

where:

- f_d is the design compressive strength of masonry;
- b is the width of the section;
- d is the effective depth of the section.

(8) When the reinforcement in a section is concentrated locally such that the member cannot be treated as a flanged member (see 6.4.3), the reinforced section should be considered as having a width of not more than 3 times the thickness of the masonry (see figure 6.8).

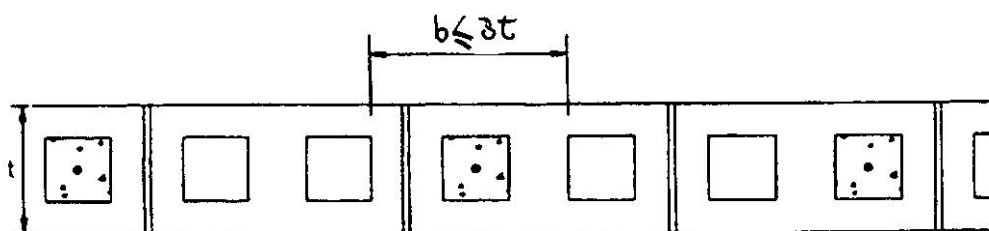


Figure 6.8: Width of section for members with locally concentrated reinforcement

(9) Reinforced masonry members with a slenderness ratio, calculated in accordance with 6.1.6, greater than 12, may be designed using the principles and application rules for unreinforced members in 6.1, taking into account second order effects.

(10) Reinforced masonry members subjected to a small axial force may be designed for bending, only, if the design axial stress does not exceed:

$$0,1 f_k \quad (6.34)$$

where

f_k is the characteristic compressive strength of masonry.

(11) When walls are reinforced with prefabricated bed joint reinforcement to assist their resistance to lateral loads, the cross-section of the masonry may be designed using this clause 6.4.2 or using simplified methods. When the strength of such a reinforced section is needed to arrive at a bending moment coefficient, α , (see 6.3.X) an equivalent f_{xk2} may be calculated by equating the moment of resistance of the bedjoint reinforced section to an unreinforced section of the same thickness.

6.4.3 Flanged Members

(1) In reinforced members, where the reinforcement is concentrated locally such that the member can act as a flanged member, for example with a T or L shape, the thickness of the flange, t_f , should be taken as the thickness of the masonry but in no case greater than $0,5d$, where d is the effective depth of the member. The masonry between the concentrations of reinforcement should be checked to ensure that it is capable of spanning between the support so provided.

(2) The effective width of the flanged members, b_{ef} , should be taken as the least of:

(i) For T-members:

- the actual width of the flange;
- the width of the pocket or rib plus 12 times the thickness of the flange;
- the spacing of the pockets or ribs;
- one-third the height of the wall.

(ii) For L-members:

- the actual width of the flange;
- the width of the pocket or rib plus 6 times the thickness of the flange;
- half the spacing of the pockets or ribs;

- one-sixth the height of the wall.

(3) In the case of flanged members, the design value of the moment of resistance, M_{RD} , can be obtained using equation (6.31) but should not be taken to be greater than:

$$f_d b_{ef} t_f (d - 0,5 t_f) \quad (6.35)$$

where:

- f_d is the design compressive strength of the masonry, obtained from 3.6.1;
- f_d is the design compressive strength of the masonry, obtained from 3.6.1;
- d is the effective depth of the member;
- t_f is the thickness of the flange in accordance with the requirements of (1) and (2);
- b_{ef} is the effective width of the flanged member, in accordance with the requirements of (1) and (2).

6.4.4 Deep beams

(1) In the case of deep beams, the design value of the moment resistance, M_{Rd} , can be obtained from equation 6.31,

where:

- A_s is the area of reinforcement in the bottom of the deep beam
- f_{yd} is the design strength of the reinforcement
- z is the lever arm, which may be taken as the lesser of the following values:

$$z = 0,7 l_{ef} , \quad (6.36)$$

or

$$z = 0,4 h + 0,2 l_{ef} ; \quad (6.37)$$

- l_{ef} is the effective span of the beam;
- h is the clear height of the wall.

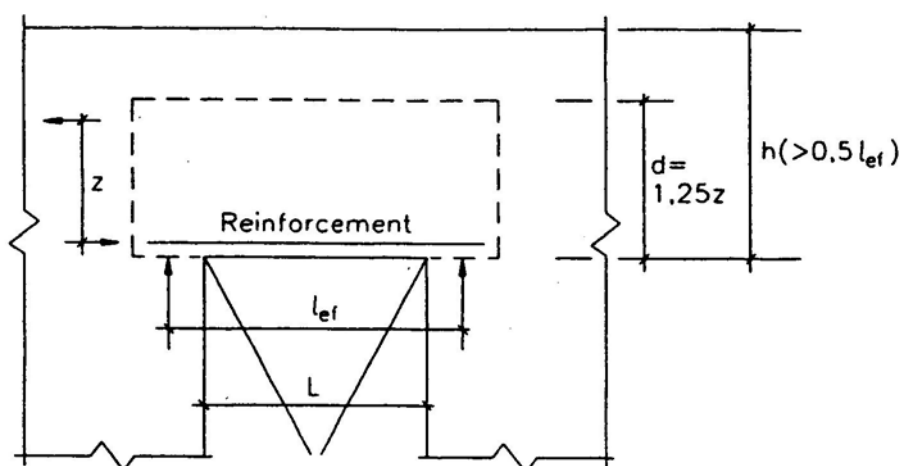


Figure 6.9: Reinforcement of a deep beam.

(2) To resist cracking, reinforcement should be provided in the bed joints above the main reinforcement, to a height of $0,5 l_{ef}$ or $0,5d$, whichever is the lesser, from the bottom face of the beam (see 8.2.3(4)).

(3) The reinforcing bars should be continuous or properly lapped over the full effective span, l_{ef} , and be provided with the appropriate anchorage length in accordance with 8.2.5.

(4) The design value of the moment of resistance, M_{Rd} , should not be taken to be greater than :

$$0,4 f_d b d^2 \quad (6.38)$$

where:

b is the width of the beam;

d is the effective depth of the beam which may be taken as $1.25z$;

f_d is the design compressive strength of the masonry in the direction of loading, obtained from 3.6.1, or concrete infill, obtained from 3.3, whichever is the lesser, and using the appropriate value of γ_M .

(5) The resistance of the compression zone of the deep beam should be verified against buckling, if unrestrained, using the method for vertical loading on walls (6.1.2).

(6) The deep beam should be verified for vertical loadings in the vicinity of its supports.

6.4.5 Composite lintels

(1) Where reinforced or prestressed prefabricated lintels are used to act compositely with the masonry above the lintel in order to provide the tension element, and where the stiffness of the prefabricated lintel is small compared with that of the wall above, the design may be based on the rules of application given in 6.4.3.2 and 6.4.4, provided that the bearing length at each end of the prefabricated lintel is justified by calculation for anchorage and bearing, but is not less than 100mm (see figure 6.10).

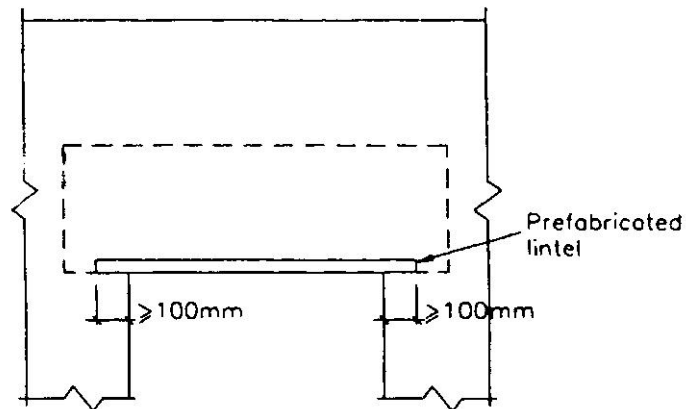


Figure 6.10: Composite lintel forming a deep beam

6.4.6 Slenderness ratio of vertically loaded members

- (1) The slenderness ratio of vertically loaded reinforced masonry members in the plane of the member should be determined in accordance with 6.1.6.
- (2) When calculating the slenderness ratio of grouted cavity walls, the thickness of the wall should not be based on a cavity width greater than 100mm.
- (3) The slenderness ratio should not be greater than 27.

Table 6.2 : Limiting ratios of effective span to effective depth for walls and beams subjected to out-of-plane bending.

Support condition	Ratio of effective span to effective depth (l_{ef}/d)	
	Wall subjected to out-of-plane bending	Beam
Simply supported	35	20
Continuous	45	26
Spanning in two directions	45	-
Cantilever	18	7
Note: For free-standing walls not forming part of a building and subjected predominantly to wind loads, the ratios may be increased by 30%, provided such walls have no applied finish which may be damaged by deflections.		

6.4.7 Limiting span of members subjected to bending

(1) The span of reinforced masonry members should be limited to the appropriate value obtained from table 6.2.

(2) To ensure the lateral stability of simply supported or continuous members, the proportions should be such that the clear distance between lateral restraints does not exceed:

$$60 b_c \text{ or } \frac{250}{d} b_c^2, \text{ whichever is the lesser;} \quad (6.39)$$

where:

d is the effective depth;

b_c is the width of the compression face midway between restraints.

(3) For a cantilever with lateral restraint provided only at the support, the clear distance from the end of the cantilever to the face of the support should not exceed:

$$25 b_c \text{ or } \frac{100}{d} b_c^2, \text{ whichever is the lesser} \quad (6.40)$$

where

b_c is taken at the face of the support.

6.5 Reinforced masonry: members subjected to shear loading

6.5.1 General

(1)P At the ultimate limit state the design shear load applied to a reinforced masonry member, V_{Sd} , shall be less than or equal to the design shear resistance of the member, V_{Rd} , such that:

$$V_{Sd} \leq V_{Rd} \quad (6.41)$$

(2) In calculating the design shear load on members with uniformly distributed loading, it may be assumed that the maximum shear load occurs at a distance $d/2$ from the face of a support, where d is the effective depth of the member.

(3) When taking the maximum shear load at $d/2$ from the face of a support, the following conditions should be satisfied:

- the loading and support reactions are such that they cause diagonal compression in the member (direct support);
- at an end support, the tension reinforcement required at a distance $2,5d$ from the face of the support is anchored into the support;
- at an intermediate support, the tension reinforcement required at the face of the support extends for a distance at least $2,5d$, plus the anchorage length, into the span.

(4) The design shear resistance of reinforced masonry members, V_{Rd} , may be calculated either by:

- ignoring the contribution of any shear reinforcement incorporated into the member, where the minimum area of shear reinforcement, as required by 8.2.3, is not provided,

or

- taking into account the contribution of the shear reinforcement, where at least the minimum area of shear reinforcement is provided.

(5) The extent of any contribution of concrete infill to the shear resistance of the reinforced masonry member should be considered, and, where the concrete infill makes a much greater contribution to the shear resistance than the masonry, EN 1992-1-1 should be used and the strength of the masonry should be ignored.

[PT Note: Add in here the possibility of getting shear resistance from tension steel, 17.5p (UK 4.7.2.2(1).)]

6.5.2 Verification of reinforced masonry walls subjected to horizontal loads in the plane of the wall

(1) For reinforced masonry walls, when the contribution of any shear reinforcement is being ignored, it should be verified that:

$$V_{Sd} \leq V_{Rd1} \quad (6.42)$$

and $V_{Rd1} = f_{vd} t L \quad (6.43)$

where:

V_{Sd} is the design value of the applied horizontal load

t is the thickness of the wall;

L is the length of the wall;

f_{vd} is the design shear strength of masonry, obtained from 3.6.2, or concrete infill, obtained from 3.3, whichever is the lesser.

Note: Where appropriate, an enhancement in the design shear strength, f_{vd} , may be taken into account in the calculation of V_{Rd1} to allow for the presence of vertical reinforcement.

(2) For reinforced masonry walls, when horizontal shear reinforcement is taken into account, it should be verified that:

$$V_{Sd} \leq V_{Rd1} + V_{Rd2} \quad (6.44)$$

where:

V_{Rd1} is given by equation (6.43), and

V_{Rd2} is given by:

$$V_{Rd2} = \frac{A_{sw}}{s} f_{yd} L \quad (6.45)$$

where:

L is the length of the wall;

A_{sw} is the area of horizontal shear reinforcement;

s is the vertical spacing of horizontal shear reinforcement;

f_{yd} is the design strength of the reinforcing steel;

(3) Where shear reinforcement enhancement is used it should also be verified that:

$$\frac{V_{Rd1} + V_{Rd2}}{t L} \leq 2,0 \quad (6.46)$$

where:

t is the thickness of the wall;

L is the length of the wall;

6.5.3 Verification of reinforced masonry beams subjected to shear loading

(1) For reinforced masonry beams when the contribution of any shear reinforcement is being ignored, it should be verified that:

$$V_{Sd} \leq V_{Rd1} \quad (6.47)$$

$$\text{and } V_{Rd1} = f_{vd} b d \quad (6.48)$$

where:

V_{Sd} is the design value of the applied shear load

b is the minimum width of the beam over the effective depth;

d is the effective depth of the beam; for deep beams d should be taken as 1.25 z, where z is defined in 6.4.4

f_{vd} is the design shear strength of masonry, obtained from 3.6.2, or concrete infill, obtained from 3.3, whichever is the lesser, using the appropriate value of γ_M ;

Note: Where required, an enhancement in the design shear strength, f_{vd} , may be taken into account in the calculation of V_{Rd1} to allow for the presence of longitudinal reinforcement.

(2) The value of f_{vd} for use in determining V_{Rd1} , at a section a_v from the face of a support, may be increased by a factor :

$$\frac{2d}{a_v} \leq 4 \quad (6.49)$$

where:

d is the effective depth of the beam;

a_v is the distance from the face of the support to the cross-section being considered;

provided f_{vk} is not taken to be greater than 0,7 N/mm².

(3) For masonry beams when shear reinforcement is taken into account, it should be verified that:

$$V_{Sd} \leq V_{Rd1} + V_{Rd2} \quad (6.50)$$

where:

V_{Rd1} is given by equation (6.46) and

V_{Rd2} is given by:

$$V_{Rd2} = 0,9 d \frac{A_{sw}}{s} f_{yd} (1 + \cot \alpha) \sin \alpha \quad (6.51)$$

where:

d is the effective depth of the beam;

A_{sw} is the area of shear reinforcement;

s is the spacing of shear reinforcement;

α is the angle of shear reinforcement to the axis of the beam between 45° and 90°;

f_{yd} is the design strength of the reinforcing steel;

(4) It should also be verified that:

$$V_{Rd1} + V_{Rd2} \leq 0,30 f_d b d \quad (6.52)$$

where:

b is the minimum width of the beam within the effective depth;

d is the effective depth of the beam;

f_d is the design compressive strength of the masonry in the direction of loading, obtained from 3.6.2, or the concrete infill, obtained from 3.3, whichever is the lesser;

6.6 Prestressed masonry

6.6.1 General

(1)P The design of prestressed masonry members shall be based on a consideration of the serviceability and ultimate limit states.

(2) The principles are similar to those set out in ENV 1992-1-1 with the design requirements and properties of materials as set out in sections 3, 5 and 6 of this EN 1996-1-1.

[PT Note: Better not to have to refer to 1992-1-1. UK may help.]

(3)P The maximum initial pre-stressing force and bearing stresses shall be within acceptable design criteria. Pre-stressing losses shall be calculated and allowed for in the assessment of serviceability and ultimate limit states, as relevant.

(4) The design principles are applicable to members prestressed in one direction only.

Note: In the design the serviceability limit state should be assessed first in bending and then the bending, axial and shear strengths should be verified at the ultimate limit state.

6.6.1.1 Maximum initial prestress and bearing stress

(1)P The initial prestressing force applied shall be limited to an acceptable proportion of the characteristic ultimate load of the tendons to ensure safety against tendon failure.

(2) Loadbearing stresses and lateral bursting tensile forces at anchorages should be limited so as to avoid an ultimate load failure condition. Local bearing stresses may be limited by consideration of prestressing load acting in either the parallel or perpendicular direction to the bed joints. The anchorage design should consider the containment of the bursting tensile forces.

6.6.1.2 Loss of prestress

(1)P Due allowance shall be made in the design for losses in prestressing forces that can occur.

(2) Losses in prestressing forces will result from a combination of:

- relaxation of tendons;
- elastic deformation of the masonry;
- moisture movement of masonry;
- creep of masonry;
- tendon losses during anchoring;

- friction effects;
- thermal effects.

(3) The factors in paragraph (2) above should be evaluated from a consideration of materials characteristics, the structural form used and conditions of use.

6.6.2 Members

6.6.2.1 General

(1)P The strength of prestressed masonry members at the ultimate limit state shall be calculated using acceptable theory in which all material behaviour characteristics and second order effects are taken into account.

6.6.2.2 Design

(1)P At the ultimate limit state, the partial safety factor for materials shall be obtained from 2.4.3. Where prestressing forces are considered as actions, the partial safety factors shall be obtained from EN 1992-1-1. The partial safety factors for actions shall be obtained from EN 1990, using the favourable or unfavourable values depending on the effect in the direction of loading.

(2)P The design of prestressed masonry members in bending shall be based upon the following assumptions:

- in the masonry, plane sections remain plane;
- the stress distribution over the compressive zone is uniform and does not exceed f_d ;
- the limiting compressive strain in the masonry is taken as -0,0035;
- the tensile strength of the masonry is ignored;
- bonded tendons or any other bonded reinforcement are subject to the same variations in strain as the adjacent masonry;
- stresses in bonded tendons or any other bonded reinforcement are derived from the appropriate stress-strain relationship;
- stresses in unbonded tendons in post-tensioned members are limited to an acceptable proportion of their characteristic strength;
- the effective depth to unbonded tendons is determined taking into account any freedom of the tendons to move.

(3)P Members subjected to loading in the plane of the member shall be designed in accordance with acceptable theory, taking into account the combined effects of the applied load and the prestressing force.

(4) When members subjected to vertical loading in the plane of the member are of solid rectangular cross section, the design method may be as given in 6.1.2 for unreinforced masonry. For non-solid rectangular members, geometric properties will need to be calculated. The prestressing of a member may need to be limited depending upon its effective slenderness and axial load carrying capacity.

(5)P The shear strength of prestressed masonry shall be evaluated using an acceptable theory and the design shear resistance shall be greater than the design value of the applied shear load.

6.7 Confined masonry

6.7.1 General

(1)P The resistance of confined masonry members shall be based on similar principles to those set out for reinforced masonry members.

6.7.2 Verification of members

(1) In the verification of confined masonry members subjected to bending and/or axial load the assumptions given in this EN for the reinforced masonry members should be adopted. In the compression zone, the compressive stress block should be based on the strength of the masonry, only. Reinforcement in compression should also be ignored.

(2) In the verification of confined masonry members subjected to shear load, all of the reinforcement should be ignored.

7 Serviceability Limit State

7.1 General

(1)P A masonry structure shall be designed and constructed so as not to exceed the Serviceability Limit State.

(2) In masonry structures the serviceability limit state for cracking and deflection need not separately be checked when the Ultimate Limit State has been satisfied.

Note: It should be borne in mind that some cracking could result when the ultimate limit state is satisfied, eg roofs.

(3) Deflections that might damage partitions, finishings (including added materials) or technical equipment, or might impair water-tightness should be checked.

(4) The serviceability of masonry members should not be unacceptably impaired by the behaviour of other structural elements, such as deformations of floors or walls.

7.2 Unreinforced masonry

(1)P Allowance shall be made for differences in the properties of masonry materials so as to avoid overstressing or damage where they are inter-connected.

(2) Damage, due to stresses arising from restraints, should be avoided by appropriate specification and detailing (see Section 8).

[PT Note: this clause has been redrafted to remove a reference to laterally loaded walls. That part has been temporarily removed to Part 1-3.]

7.3 Reinforced masonry members

(1)P Reinforced masonry members shall not crack unacceptably or deflect excessively under serviceability loading conditions.

(2) Where reinforced masonry members are sized so as to be within the limiting dimensions given in 6.4.7, it may be assumed that the lateral deflection of a wall and the vertical deflection of a beam will be acceptable.

(3) When the modulus of elasticity is used in calculations of deflections the long term E should be applied as obtained from 3.7.3.

(4) For reinforced masonry members subjected to bending - e.g. reinforced masonry beams - cracking will be limited such as to satisfy the serviceability limit state if the limiting dimensions in 6.4.2.2 and the detailing requirements in section 8 are followed.

Note: Where, for reasons of detailing, cover to the tension reinforcement exceeds the minimum requirements given in 8.2.2, the possibility of surface cracking may need to be considered.

7.4 Prestressed masonry members

7.4.1 General

(1)P Prestressed masonry members shall not exhibit flexural cracking nor deflect excessively under serviceability loading conditions.

7.4.2 Design for the serviceability limit state

(1) Serviceability load conditions at transfer of prestress and under design loads after prestressing losses should be considered. Other design cases may exist for specific structural forms and loading conditions.

(2)P Partial safety factors for loads shall be taken as 1,0 (see EN 1990) at transfer of prestress and under design loads after prestressing losses. The tensile stresses in the masonry shall be limited to zero. The initial prestressing force shall be limited to 70% of the characteristic breaking load of the tendon.

(3)P The analysis of a prestressed masonry member under the serviceability limit state shall be based on the following assumptions:

- in the masonry, plane sections remain plane;
- stress is proportional to strain;
- tensile stress in the masonry is limited so as to avoid excessive crack widths and to ensure durability of the prestressing steel;
- the prestressing force is constant after all losses have occurred.

(4) If the assumptions in paragraph (3)P are followed, serviceability limit states will be satisfied although additional deflection verification may need to be carried out.

7.5 Confined masonry members

(1)P Confined masonry members shall not exhibit flexural cracking nor deflect excessively under serviceability loading conditions.

(2)P The verification of confined masonry members at the serviceability limit states shall be based on the assumptions given for unreinforced masonry members.

7.6 Bearings under concentrated loads

(1) Bearings that satisfy the ultimate limit state when verified in accordance with equations (6.16), (6.17) or (6.18) may be deemed to satisfy the serviceability limit state.

8 Detailing

8.1 Masonry details

8.1.1 Masonry materials

(1)P Masonry units shall be suitable for the type of masonry, its location and its durability requirements. Mortar, concrete infill and reinforcement shall be appropriate to the type of unit and the durability requirements.

(2) Masonry reinforced with bars should be laid in mortar M5 or stronger, and masonry reinforced with prefabricated bed joint reinforcement should be laid in mortar M2.5 or stronger.

8.1.2 Minimum thickness of wall

(1)P The minimum thickness of a wall shall be that required to give a robust wall and one that satisfies the outcome of the calculations.

8.1.3 Minimum area of wall

(1)P A loadbearing wall shall have a minimum area on plan of 0.04m^2 .

8.1.4 Bonding of masonry

(1)P Masonry units shall be bonded together with mortar in accordance with proven practice.

(2)P Masonry units in an unreinforced masonry wall shall be overlapped on alternate courses so that the wall acts as a single structural element.

(3) In unreinforced masonry, masonry units should overlap by a length equal to at least 0,4 times the height of the unit or 40mm, whichever is the greater (see figure 8.1). When the flexural strength of the masonry is utilized, the overlap should be half the length of the unit. At corners or junctions, the overlap of the units should not be less than the thickness of the units; cut units should be used to achieve the specified overlap in the remainder of the wall.

Note: the length of walls and columns and the size of openings and piers preferably should suit the dimensions of the units so as to avoid excessive cutting.

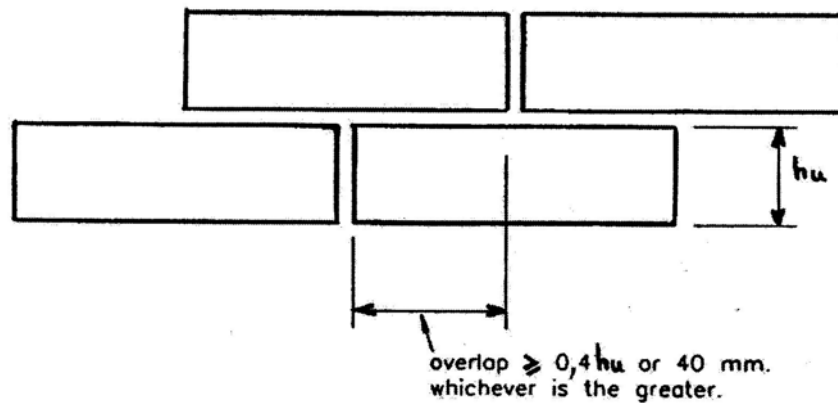


Figure 8.1: Overlap of masonry units

(4) Bonding arrangements not meeting the minimum overlap requirements may be used in reinforced masonry where experience or experimental data indicate that they are satisfactory.

Note: When a wall is reinforced, the degree of overlap can be determined as part of the design of the reinforcement.

(5) Where non-loadbearing walls abut loadbearing walls, allowance for differential deformation due to creep and shrinkage should be taken into account. It is recommended that such walls are not bonded together, but they may be tied together with suitable connectors allowing for differential deformations..

(6) The differential deformation behaviour of materials should be taken into account if different materials are to be rigidly connected together.

(7) The dimensioning of walls, columns and openings should suit the dimensions of the units so as to minimize cutting of units.

8.1.5 Mortar joints

(1) For the purpose of using the values and equations given in 3.6.1 and 3.6.2, bed and perpend joints made with general purpose and lightweight mortars should have a thickness not less than 6 mm nor more than 15mm, and bed and perpend joints made with thin layer mortars should have a thickness not less than 1mm nor more than 3mm.

Note: Joints of thickness between 3 and 6mm may be constructed if the mortars have been specially developed for the particular use, when the design may be based on the use of general purpose mortar.

(2) Bed joints should be horizontal unless the designer specifies otherwise.

(3) Perpend joints can be considered to be filled if mortar is provided to the full height of

the joint over a minimum of 40% of the width of the unit. Perpend joints in reinforced masonry subject to bending and shear across the joints should be fully filled with mortar so as to be considered filled.

8.1.6 Bearings under concentrated loads

(1) Concentrated loads should bear on a wall a minimum length of 100mm or such distance as is required from calculations according to 6.1.7, whichever is the greater.

8.2 Reinforcement details

8.2.1 General

(1)P Reinforcing steel shall be located such that it acts compositely with the masonry.

(2)P Where simple supports are assumed in the design, consideration shall be given to the effects of any fixity that might be provided by the masonry.

(3) Reinforcing steel in masonry designed as a beam should be provided over a support where the masonry is continuous, whether the beam has been designed as continuous or not. Where this occurs, an area of steel not less than 50% of the area of the tension reinforcement required at midspan should be provided in the top of the masonry over the support and anchored in accordance with 8.2.5.1. In all cases at least 25% of the reinforcing steel required at midspan should be carried through to the support and similarly anchored.

8.2.2 Cover to reinforcing steel

(1) Where the reinforcing steel is located in mortar in bed joints and is selected using table 4.1:

- the minimum depth of mortar cover from the reinforcing steel to the face of the masonry should be 15mm (see figure 8.2);

- mortar cover above and below reinforcing steel placed in bed joints should be provided, so that the thickness of the joint is at least 5mm greater than the diameter of the reinforcing steel, for general purpose and lightweight mortars, and 1,5mm, for thin layer mortar (see figure 8.2).

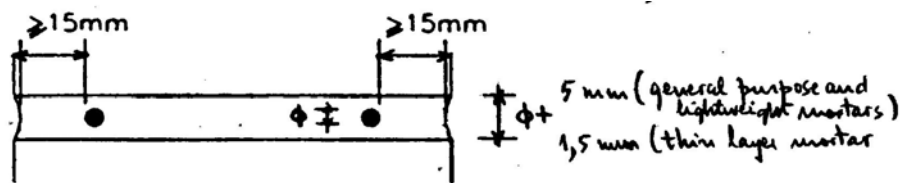


Figure 8.2: Cover to reinforcing steel in bed joints

(2) For filled cavity or special bond construction, the minimum cover for reinforcing steel selected using table 4.1 should be 20mm for mortar or the concrete cover from table 4.1, as appropriate, or the diameter of the bar, whichever is the greater.

(3) The cut ends of all reinforcing steel, except stainless steel, should have the same minimum cover as that appropriate to unprotected carbon steel in the exposure situation being considered, unless alternative means of protection are used.

8.2.3 Minimum area of reinforcement

(1)P The area of reinforcing steel provided shall be at least the minimum necessary to ensure that the relevant design criteria are satisfied.

(2) In reinforced masonry members where reinforcing steel is provided to enhance the strength in the plane of the member, the area of main steel should not be less than 0,05% of the effective cross-sectional area of the member, taken as the product of its effective width and its effective depth.

(3) In walls where reinforcing steel is provided in the bed joints to enhance resistance to lateral loads, the total area of such reinforcement should not be less than 0,03% of the gross cross-sectional area of the wall (ie 0.015% in each face).

(4) Where reinforcement is provided in bed joints to help control cracking or to provide ductility, the total area of the steel should not be less than 0,03% of the gross cross-sectional area of the wall.

(5) In reinforced grouted cavity masonry members designed to span in one direction only, secondary reinforcing steel should be provided in the direction perpendicular to the main steel principally to distribute stresses. The area of this secondary reinforcing steel should not be less than 0,05% of the cross-sectional area of the member, taken as the product of its effective width and its effective depth.

(6) Where shear reinforcing steel is required in the member (see 6.6.3), the area of shear reinforcement should not be less than 0,10% of the cross-sectional area of the member, taken as the product of its effective width and its effective depth.

8.2.4 Size of reinforcing steel

(1)P The maximum size of reinforcing steel used shall be such as to enable proper embedment in the mortar or concrete infill.

(2) Reinforcing steel should have a minimum diameter of 5 mm.

(3)P The maximum size of reinforcing steel used shall be such that the anchorage stresses, as given in 8.2.5, are not exceeded and the cover to the reinforcement, as given in 8.2.2, is maintained.

8.2.5 Anchorage and laps

8.2.5.1 Anchorage of tension reinforcing steel

(1)P Reinforcing steel shall be provided with sufficient anchorage length so that the internal forces to which it is subjected are transmitted to the mortar or concrete infill and that longitudinal cracking or spalling of the masonry is avoided.

(2) Anchorage should be achieved by straight anchorage, hooks, bends or loops as shown in figure 8.3. Alternatively stress transfer may be by means of an appropriate mechanical device proven by tests.

(3) Straight anchorage or bends (see figure 8.3 (a) and (c)) should not be used to anchor plain reinforcing steel of more than 8mm diameter. Hooks, bends or loops should not be used to anchor reinforcing steel in compression.

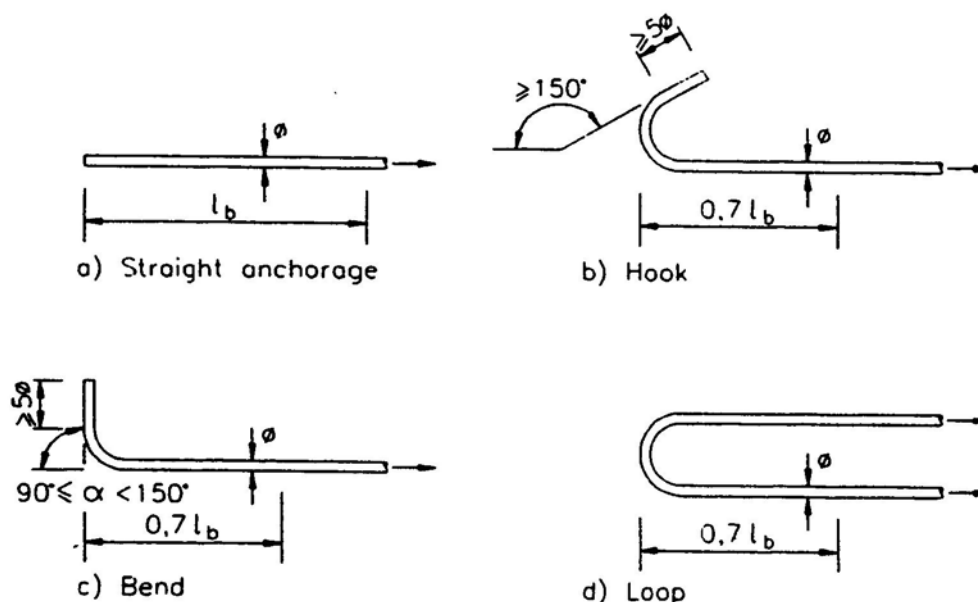


Figure 8.3: Anchorage details

(4) The straight anchorage length l_b required for a bar, assuming constant bond stress, should be obtained from:

$$l_b = \gamma_M \frac{\phi}{4} f_{ym} \frac{1}{f_{bom}} \quad (8.1)$$

where:

- ϕ is the effective diameter of the reinforcing steel;
- f_{ym} is the design strength of reinforcing steel, obtained from 3.4.2;
- f_{bom} is the design anchorage bond strength of reinforcing steel, obtained from table 3.5 or 3.6 and 3.6.4, as appropriate;

(5) For bars ended by hooks, bends and loops (see Figure 8.3 (b), (c) and (d)), the anchorage length in tension may be reduced to 0,7 l_b .

(6) Where a greater area of reinforcing steel is provided than is required by design, the anchorage length may be reduced proportionally provided that:

(i) For reinforcing steel in tension the anchorage length is not less than the greater of:

- 0,3 l_b , or
- 10 bar diameters, or
- 100mm.

(ii) For reinforcing steel in compression the anchorage length is not less than the greater of:

- 0,6 l_b , or
- 10 bar diameters, or
- 100mm.

(7) Where it is practicable, transverse reinforcing steel should be provided evenly distributed along the anchorage length, with at least one reinforcing steel bar placed in the region of a curved anchorage (see figure 8.3 (b), (c) and (d)). The total area of transverse reinforcing steel should be not less than 25% of the area of one anchored reinforcing steel bar.

(8) Where prefabricated bed joint reinforcement is used, the anchorage length should be based on the characteristic anchorage bond strength determined by tests in accordance with EN 846-2.

8.2.5.2 Lapping of tension reinforcing

(1)P The length of laps shall be sufficient to transmit the design forces.

(2) The lap length between two reinforcing steel bars should be calculated in accordance with 8.2.5.1, based on the smaller of the two bars lapped.

(3) The lap length provided between two reinforcing steel bars should be:

- l_b for bars in compression and for reinforcing steel bars in tension where less than 30% of the bars in the section are lapped and where the clear distance between the lapped bars in a transverse direction is not less than 10 bar diameters and the concrete or mortar cover is not less than 5 bar diameters.

- $1,4 l_b$ for reinforcing steel bars in tension where either 30% or more of the bars at the section are lapped or if the clear distance between the lapped bars in a transverse direction is less than 10 bar diameters or the concrete or mortar cover is less than 5 bar diameters.

- $2 l_b$ for reinforcing steel bars in tension where both 30% or more of the bars at the section are lapped and the clear distance between the lapped bars is less than 10 bar diameter or the concrete or mortar cover is less than 5 bar diameters.

(4) Where it is possible, laps between reinforcing steel bars should not be located at areas of high stress or where the dimensions of a section change, for example, a step in a wall thickness. The clear distance between two lapped bars should not be less than two bar diameters or 20mm whichever is the greater.

(5) Where prefabricated bed joint reinforcement is used the lap length should be based on the characteristic anchorage bond strength determined by tests in accordance with EN 846-2.

8.2.5.3 Anchorage of shear reinforcing steel

(1) The anchorage of shear reinforcing steel, including stirrups, should be effected by means of hooks or bends (see figure 8.3 (b) and (c)), where appropriate, with a longitudinal reinforcing bar provided inside the hook or bend.

(2) The anchorage is considered to be satisfactory where the curve of the hook is extended by a straight length of 5 bar diameters or 50mm, whichever is the greater, and the curve of the bend is extended by a straight length of 10 bar diameters or 70mm, whichever is the greater (see figure 8.4).

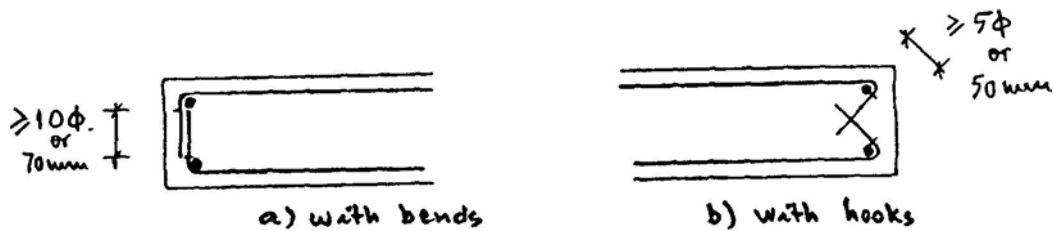


Figure 8.4: Anchorage of shear reinforcement

8.2.5.4 Curtailment of tension reinforcing

(1) In any member subjected to bending, every reinforcing steel bar should extend, except at end supports, beyond the point at which it is no longer needed, for a distance equal to the effective depth of the member or 12 times the diameter of the bar, whichever is the greater. The point at which reinforcing steel is theoretically no longer needed is where the design resistance moment of the section, considering only the continuing bars, is equal to the applied design moment. However, reinforcing steel should not be curtailed in a tension zone unless at least one of the following conditions is satisfied for all arrangements of design load considered:

- the reinforcing steel bars extend at least the anchorage length appropriate to their design strength from the point at which they are no longer required to resist bending;
- the design shear capacity at the section where the reinforcing steel stops is greater than twice the shear force due to design loads, at that section;
- the continuing reinforcing steel bars at the section where the reinforcing steel stops provide double the area required to resist the bending moment at that section.

(2) Where there is little or no end fixation for a member in bending, at least 25% of the area of the tension reinforcing steel required at mid-span should be carried through to the support. This reinforcement may be anchored in accordance with 8.2.5.2, or by providing:

- an effective anchorage length equivalent to 12 times the bar diameter beyond the centre line of the support, where no bend or hook begins before the centre of the support,

or

- an effective anchorage equivalent to 12 times the bar diameter plus $d/2$ from the face of the support, where d is the effective depth of the member, and no bend begins before $d/2$ inside the face of the support.

(3) Where the distance from the face of a support to the nearer edges of a principal load is less than twice the effective depth, all the main reinforcing steel in a member subjected to bending should continue to the support and be provided with an anchorage equivalent to 20 times the bar diameter.

8.2.6 Restraint of compression reinforcing steel

(1)P Reinforcing steel bars in compression shall be restrained to prevent local buckling.

(2) In members where the area of longitudinal reinforcing steel is greater than 0,25% of the area of the masonry and any concrete infill, and more than 25% of the design axial load resistance is to be used, links surrounding the longitudinal bars should be provided.

(3) Where links are required, they should be not less than 4mm in diameter or 1/4 of the maximum diameter of the longitudinal bars, whichever is the greater, and the spacing should not exceed the least of:

- the least lateral dimension of the wall;
- 300mm;
- 12 times the main bar diameters.

(4) Vertical reinforcing steel corner bars should be supported by an internal angle at every link spacing and this angle should not exceed 135°. Internal vertical reinforcing bars need only be restrained by internal angles at alternate link spacings.

8.2.7 Spacing of reinforcing steel

(1)P The spacing of reinforcing steel shall be sufficiently large so as to allow the concrete infill or mortar to be placed and compacted.

(2) The clear distance between adjacent parallel reinforcing steel should not be less than the maximum size of the aggregate plus 5mm, or the bar diameter, or 10mm whichever is the greater.

(3) Except when reinforcing steel is concentrated in cores or pockets, the spacing of tension reinforcement should not exceed 400mm.

(4) When the main reinforcing steel is concentrated in cores or pockets, for example, in pocket type walls, the spacing centre to centre of the concentrations of main reinforcing steel should be at a maximum of 600mm. The total area of main reinforcing steel should not exceed 4% of the gross cross-sectional area of the infill in the core or pocket, except at laps where it should not exceed 8%.

(5) Where shear reinforcing steel is required, the spacing of stirrups should not be greater than $0,75 \times$ effective depth of the member or 300 mm, whichever is lesser.

(6) Prefabricated bed joint reinforcement placed in bed joints should be spaced at 600mm, or less, centres.

8.3 Prestressing details

(1) Detailing of prestressing devices should be in accordance with pr EN 1992-1-1.

8.4 Confined masonry details

(1) P Confined masonry walls shall be provided with vertical and horizontal reinforced concrete or reinforced masonry confining elements so that they act together as a single structural member when subjected to actions.

(2) P Top and sides confining elements shall be cast after the masonry has been built so that they will be duly anchored together.

(3) Confining elements should be provided at every floor level, at every interception between walls and at both sides of every opening having an area of more than $1,5 \text{ m}^2$. Additional confining elements may be required in the walls so that the maximum spacing, both horizontal and vertical is 4 m (figure 8.5).

(4) Confining elements should have a cross-sectional area not less than $0,02 \text{ m}^2$, with a minimum dimension of 150 mm in the plan of the wall, and be provided with longitudinal reinforcements with a minimum area equivalent to 0,8 % of the cross-sectional area of the confining element, but not less than 200 mm^2 . Stirrups not less than 6 mm diameter, spaced not more than 300 mm should also be provided. The detailing of the reinforcements should be in accordance with 8.2.

(5) In confined masonry walls where Group 1 and Group 2 Units are used, the units adjacent to the confining elements should be overlapped according to the rules prescribed in the clause 8.1.3 for bonding of masonry. Alternatively, reinforcement not less than 6 mm diameter bars or equivalent and spaced not more than 400 mm, duly anchored in the concrete infill and in the mortar joints, should be adopted.

8.5 Connection of walls

8.5.1 Connection of walls to floors and roofs

8.5.1.1 General

(1)P Where walls are assumed to be restrained by floors or roofs, the walls shall be connected to the floors or roofs so as to provide for the transfer of the design lateral loads to the bracing elements.

(2) Transfer of lateral loads to the bracing elements should be made by the floor or roof structure, for example, reinforced or precast concrete or timber joists incorporating boarding, provided the floor or roof structure is capable of developing diaphragm action, or by a ring beam capable of transferring the resulting shear and bending action effects. Either the frictional resistance of the bearing of structural members on masonry walls, or metal straps of suitable and fixing, should be capable of resisting the transfer loads.

(3)P Where a floor or roof bears on a wall, the bearing length shall be sufficient to provide the required bearing capacity and shear resistance, allowing for manufacturing and erection tolerances.

(4) The minimum bearing length of floors or roofs on walls should be as for bearings under concentrated loads (see 8.1.6).

8.5.1.2 Connection by straps

(1)P Where straps are used they shall be capable of transferring the lateral loads between the wall and the restraining structural element.

(2) When the surcharge on the wall is negligible, for example, at a gable wall/roof junction, special consideration is necessary to ensure that the connection between the straps and the wall will be effective.

(3) The spacing of straps between walls and floors or roofs should be not greater than 2 m for buildings up to 4 storeys high, and 1,25 m for higher buildings.

8.5.1.3 Connection by frictional resistance

(1)P Where concrete floors, roofs or ring beams bear directly on a wall, the frictional resistance shall be capable of transferring the lateral loads.

(2) Straps are not necessary if the bearing of a concrete floor or roof extends to the centre of the wall or 100mm, whichever is the greater, providing no sliding bearing is formed.

8.5.1.4 Ring ties and ring beams

(1) When the transfer of lateral loads to the bracing elements is to be achieved by the use of ring beams:

- the ring ties are placed in every floor level or directly below. They may consist of reinforced concrete, reinforced masonry, steel or wood and should be able to support a design tensile force of 30kN.

- when ring ties are not continuous, additional measures should be undertaken to ensure continuity.

- ring ties made of reinforced concrete should be reinforced by at least two reinforcing steel bars of at least 150mm². The laps should be designed in accordance with ENV 1992-1-1 and staggered, if possible. Ring ties made of reinforced concrete or reinforced masonry should be reinforced. Parallel continuous reinforcement may be considered with their full cross section provided that they are situated in floors or window lintels at a distance of not more than 0,5m from the middle of the wall and floor, respectively.

(2) If floors without diaphragm action are used or sliding layers are put under the floor bearings due to deformations of the roof floor, the horizontal stiffening of the walls should be ensured by ring beams or statically equivalent measures.

8.5.2 Connection between walls

8.5.2.1 General

(1)P Intersecting loadbearing walls shall be joined together so that the required vertical and lateral loads can be transferred between them.

(2) The joint at the intersection of walls should be made either by:

- masonry bond (see 8.1.4), or
- connectors or reinforcement extending into each wall.

(3) Intersecting loadbearing walls should be erected simultaneously.

8.5.2.2 Cavity walls

(1)P The two leaves of a cavity wall shall be effectively tied together.

(2) The number of wall ties connecting together the two leaves of a cavity wall should be not less than 2 ties/m² of the cavity wall, nor less than the number calculated according to 6.3.3, where relevant.

Note: When connecting elements, for example, prefabricated bed joint reinforcement, are used to connect two leaves of a wall together, each tying element should be treated as a wall tie.

(3)P The wall ties shall be corrosion resistant for the relevant exposure class for the wall (see section 4).

(4)P Additional wall ties shall be provided at a free edge to connect both leaves together.

(5) Where an opening penetrates a wall and the frame for the opening is not capable of transferring the horizontal design action directly to the structure, those wall ties which would have been placed in the opening should be redistributed uniformly along the vertical edges of the opening.

(6) Due allowance should be made for any differential movement between the leaves, or between a leaf and a frame, in the selection of the wall ties.

8.5.2.3 Double-leaf walls.

(1)P The two leaves of a double-leaf wall shall be effectively tied together.

(2) Ties connecting the two leaves, calculated according to 6.3.3, should have a minimum cross-sectional area of $300 \text{ mm}^2/\text{m}^2$ of the double-leaf wall, for steel connectors, and with the connectors evenly distributed, being not less than 2 connectors/ m^2 of the double-leaf wall.

Note: Some forms of prefabricated bed joint reinforcement can also function as ties between the two leaves of a double-leaf wall (see EN 845-3).

(3)P The connectors shall be corrosion resistant for the relevant exposure class for the wall (see 4.3.6)

(4) Due allowance should be made for any differential movement between the leaves in the selection of the connector.

8.6 Chases and recesses on walls

8.6.1 General

(1)P Chases and recesses shall not impair the stability of the wall.

(2) Chases and recesses should not be allowed if the depth of the chase or recess would be greater than half the thickness of the shell of the unit, unless the strength of the wall is verified by calculation.

(3) Chases and recesses should not pass through lintels or other structural items built into a wall nor should they be allowed in reinforced masonry members unless specifically allowed for by the designer.

(4) In cavity walls, the provision of chases and recesses should be considered separately for each leaf.

8.6.2 Vertical chases and recesses

(1) The reduction in vertical load, shear and flexural resistance resulting from vertical chases and recesses may be neglected if such vertical chases and recesses are kept within the limits given in table 8.1, with the depth of the recess or chase taken to include the depth of any hole reached when forming the recess or chase. If these limits are exceeded, the vertical load, shear and flexural resistance should be checked by calculation.

Table 8.1 : Sizes of vertical chases and recesses in masonry, allowed without calculation.

Thickness of wall (mm)	Chases and recesses formed after construction of masonry		Chases and recesses formed during construction of masonry	
	max depth (mm)	max width (mm)	max width (mm)	minimum wall thickness remaining (mm)
≤ 115	30	100	300	70
116 - 175	30	125	300	90
176 - 225	30	150	300	140
226 - 300	30	175	300	175
over 300	30	200	300	215

Notes:

1. The maximum depth of the recess or chase should include the depth of any hole reached when forming the recess or chase.
2. Vertical chases which do not extend more than one third of the storey height above floor level may have a depth up to 80mm and a width up to 120mm, if the thickness of the wall is 225mm or more.
3. The horizontal distance between adjacent chases or between a chase and a recess or an opening should not be less than 225mm.
4. The horizontal distance between any two adjacent recesses, whether they occur on the same side or on opposite sides of the wall, or between a recess and an opening, should not be less than twice the width of the wider of the two recesses.
5. The cumulative width of vertical chases and recesses should not exceed 0,13 times the length of the wall.

8.6.3 Horizontal and inclined chases

(1) Horizontal and inclined chases should preferably be avoided. Where it is not possible to avoid horizontal and inclined chases, the chases should be positioned within one eighth of the clear height of the wall, above or below floor, and the total depth, including the depth of any hole reached when forming the chase, should be less than the maximum size as given in table 8.2. If these limits are exceeded, the vertical load, shear and flexural resistance should be checked by calculation.

Table 8.2 : Sizes of horizontal and inclined chases in masonry, allowed without calculation.

Thickness of wall (mm)	Maximum depth (mm)	
	Unlimited length	Length \leq 1 250mm
≤ 115 mm	0	0
116 - 175	0	15
176 - 225	10	20
226 - 300	15	25
over 300	20	30

Notes:

1. The maximum depth of the chase should include the depth of any hole reached when forming the chase.
2. The horizontal distance between the end of a chase and an opening should not be less than 500mm.
3. The horizontal distance between adjacent chases of limited length, whether they occur on the same side or on opposite sides of the wall, should be not less than twice the length of the longest chase.
4. In walls of thickness greater than 115mm, the permitted depth of the chase may be increased by 10mm if the chase is machine cut accurately to the required depth. If machine cuts are used, chases up to 10mm deep may be cut in both sides of walls of thickness not less than 225mm.
5. The width of chase should not exceed half the residual thickness of the wall.

8.7 Damp proof courses

(1)P Damp proof courses shall be capable of transferring the horizontal and vertical design loads without suffering or causing damage; they shall have sufficient surface frictional resistance to prevent movement of the masonry resting on them.

8.8 Thermal and long term movement

(1)P Allowance shall be made for the effects of movements such that the performance of the masonry is not affected adversely.

(2) Vertical and horizontal movement joints should be provided to allow for the effects of thermal and moisture movement, creep and deflection and the possible effects of internal stresses caused by vertical or lateral loading, so that the masonry does not suffer damage.

(3) In determining the maximum spacing of vertical movement joints, special consideration should be given to the effects of the following:

- the drying shrinkage of calcium silicate units, aggregate concrete units, autoclaved aerated concrete units and manufactured stone units;
- the irreversible moisture expansion of the units;
- variations in temperature and humidity;
- insulation provided to the masonry;
- the provision of prefabricated bed joint reinforcement.

(4) In the case of ties with fixed supports are used, precautions should be taken to allow for vertical movement of external walls. The uninterrupted height between horizontal movement joints in the outer leaf of external cavity walls should be limited to avoid the loosening of the wall ties. Pinned supported ties or ties which are free to move in the vertical direction should in general be used.

(5) The width of vertical and horizontal movement joints should allow for the maximum movement expected. If expansion joints are to be filled then they should be filled with an easily compressed material.

9. Execution

9.1 General

(1)P All work shall be constructed in accordance with the specified details within permissible deviations.

(2)P All work shall be executed by appropriately skilled and experienced personnel.

(3) If the requirements of EN 1996-2 are followed, it can be assumed that (1)P and (2)P are satisfied.

9.2 Design of structural members

(1) It should be determined whether special precautions are necessary to ensure the overall stability of the structure or of individual walls during construction.

9.3 Loading of masonry

(1)P Masonry shall not be subjected to load until it has achieved adequate strength to resist the load without damage.

(2) Backfilling against retaining walls should not be carried out until the wall is capable of resisting loads from the filling operation, taking account of any compacting forces or vibrations.

(3) Attention should be paid to walls which are temporarily unrestrained during construction, but which may be subjected to wind loads or construction loads, and temporary shoring should be provided, if necessary, to maintain stability.

ANNEXES

Annex A (informative): Consideration of partial safety factors relating to Execution

(1) If a country wishes to link a class, or classes, of γ_M from 2.3 to execution control, the following should be considered in differentiating the class, or classes, of γ_M :

- the availability of appropriately qualified and experienced personnel, employed by the contractor, for supervision of the work;
- the availability of appropriately qualified and experienced personnel, independent of the contractor's staff, for the inspection of the work;

Note: In the case of Design -and-Build contracts, the Designer may be considered as a person independent of the construction organisation for the purposes of inspection of the work, provided that the Designer is an appropriately qualified person who reports to senior management independently of the site construction team.

- assessment of the site properties of the mortar and concrete infill;
- the way in which mortars are mixed and the constituents are batched, for example, either by weight or in appropriate measuring boxes.

Annex B (informative): Classification of micro conditions of exposure of completed masonry

Table B.1 Classification of micro conditions of exposure of completed masonry

Class	Micro condition of the masonry	Examples of masonry in this condition
MX1	In a dry environment	Interior of buildings for normal habitation and for offices, including the inner leaf of external cavity walls not likely to become damp. Rendered masonry in exterior walls, not exposed to moderate or severe driving rain, and isolated from damp in adjacent masonry or materials.
MX2	Exposed to moisture or wetting	
MX2.1	Exposed to moisture but not exposed to freeze/thaw cycling or external sources of significant levels of sulfates or aggressive chemicals	Internal masonry exposed to high levels of water vapour, such as in a laundry. Masonry exterior walls sheltered by overhanging eaves or coping, not exposed to severe driving rain or frost. Masonry below frost zone in well drained non-aggressive soil.
MX2.2	Exposed to severe wetting but not exposed to freeze/thaw cycling or external sources of significant levels of sulfates or aggressive chemicals	Masonry not exposed to frost or aggressive chemicals, located: in exterior walls with cappings or flush eaves; in parapets; in freestanding walls; in the ground; under water.
MX3	Exposed to wetting plus freeze/thaw cycling	
MX3.1	Exposed to moisture and freeze/thaw cycling but not exposed to external sources of significant levels of sulfates or aggressive chemicals	Masonry as class MX2.1 exposed to freeze/thaw cycling.
MX3.2	Exposed to severe wetting and freeze/thaw cycling but not exposed to external sources of significant levels of sulfates or aggressive chemicals	Masonry as class MX2.2 exposed to freeze/thaw cycling.
MX4	Exposed to saturated salt air, seawater or other salt laden water	Masonry in a coastal area or in a position where winter salting can affect the masonry
MX5	In an aggressive chemical environment	Masonry in contact with natural soils or filled ground or groundwater, where moisture and sulfates are present. Masonry in contact with highly acidic soils, contaminated ground or groundwater. Masonry near industrial areas where aggressive chemicals are airborne.
NOTE 1 : In deciding the exposure of masonry the effect of applied finishes and protective claddings should be taken into account.		

Annex C (informative)

C.1 A simplified method for calculating the out-of-plane eccentricity of loading on walls

(1) In calculating the eccentricity of loading on walls, the joint between the wall and the floor may be simplified by using uncracked cross sections and assuming elastic behaviour of the materials; a frame analysis or a single joint analysis may be used.

(2) Joint analysis may be simplified as shown in figure C.1; for less than four members, those not existing should be ignored. The ends of the members remote from the junction should be taken as fixed unless they are known to take no moment at all, when they may be taken to be hinged. The moment in member 1, M_1 , may be calculated from equation (C.1) and the moment in member 2, M_2 , similarly but using $E_2 I_2 / h_2$ instead of $E_1 I_1 / h_1$ in the numerator.

$$M_1 = \frac{\frac{n_1 E_1 I_1}{h_1}}{\frac{n_1 E_1 I_1}{h_1} + \frac{n_2 E_2 I_2}{h_2} + \frac{n_3 E_3 I_3}{l_3} + \frac{n_4 E_4 I_4}{l_4}} \left[\frac{w_3 l_3^2}{4(n_3 - 1)} - \frac{w_4 l_4^2}{4(n_4 - 1)} \right] \quad (C.1)$$

where:

n_i is the stiffness factor of members is taken as 4 for members fixed at both ends and otherwise 3;

E_n is the modulus of elasticity of member n , where $n = 1, 2, 3$ or 4 ;

Note: It will normally be sufficient to take the values of E as $1\,000f_k$ for all masonry units.

I_j is the second moment of area of member j , where $j = 1, 2, 3$ or 4 (in the case of a cavity wall in which only one leaf is loadbearing, I_j should be taken as that of the loadbearing leaf only);

h_1 is the clear height of member 1;

h_2 is the clear height of member 2;

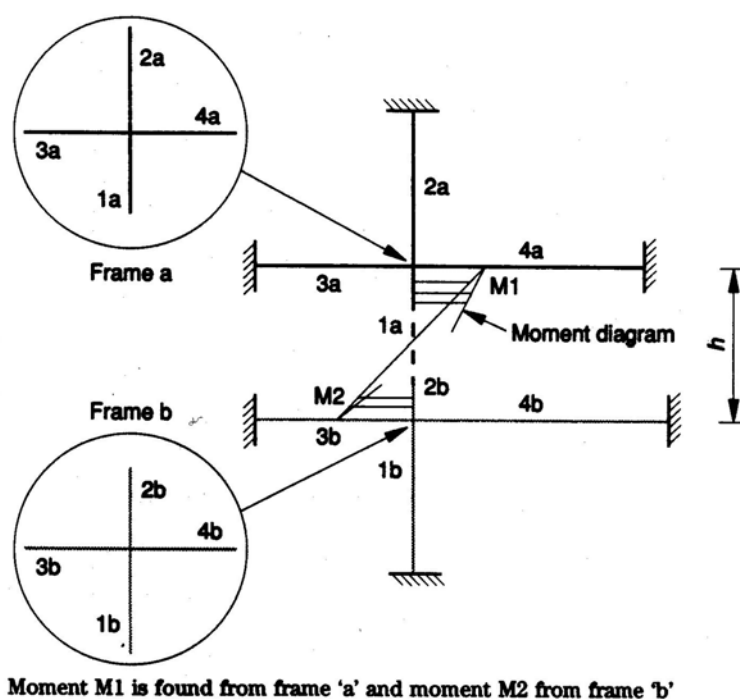
l_3 is the clear span of member 3;

l_4 is the clear span of member 4;

w_3 is the design uniformly distributed load on member 3, using the partial safety factors from EN 1990, unfavourable effect;

w_4 is the design uniformly distributed load on member 4, using the partial safety factors from EN 1990, unfavourable effect.

Note: The simplified frame model used in figure C1 is not considered to be appropriate where timber floor joists are used. For such cases refer to paragraph (4) below.



Moment M1 is found from frame 'a' and moment M2 from frame 'b'

Figure C.1 : Simplified frame diagram

(3) The results of such calculations will usually be conservative because the true fixity, ie. the ratio of the actual moment transmitted by a joint to that which would exist if the joint was fully rigid, of the floor/wall junction cannot be achieved. It will be permissible for use in design to reduce the eccentricity, obtained from the calculations in

accordance with paragraph (1) above, by multiplying it by a factor, η , provided that the design vertical stress acting at the junction in question is greater than 0,25N/mm² when averaged across the thickness of the wall.

η may be obtained experimentally, or it may be taken as $(1 - k/4)$,

where

$$k = \frac{\frac{E_3 l_3}{h_1} + \frac{E_4 l_4}{h_2}}{\frac{E_1 l_1}{h_1} + \frac{E_2 l_2}{h_2}} \leq 2 \quad (C.2)$$

(4) If the eccentricity calculated in accordance with paragraph (2) above is greater than 0,4 times the thickness of the wall, the design may be based on paragraph (5) below.

(5) The eccentricity of loading to be used in design may be based on the load being resisted by the minimum required bearing depth, not taken to be more than 0,2 times the wall thickness, at the face of the wall, stressed to the appropriate design strength of the material (see figure C.2); this will be appropriate, particularly, at a roof.

Note: It should be borne in mind that basing the eccentricity on this Annex may lead to sufficient rotation of the floor or beam to cause a crack on the opposite side of the wall to that of the load application.

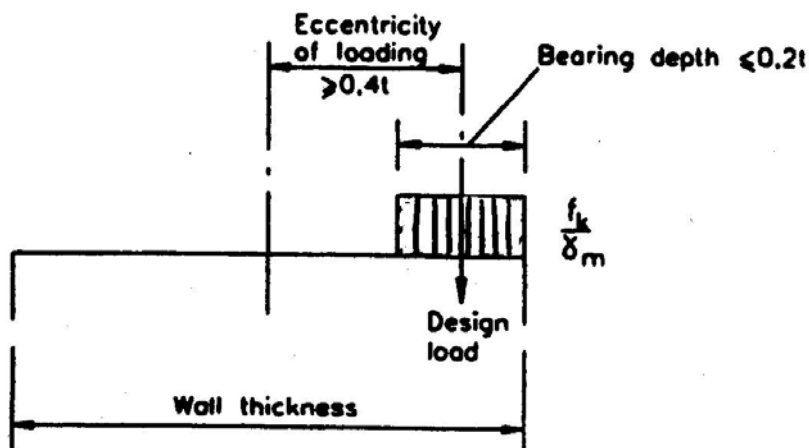


Figure C.2 : Eccentricity obtained from design load resisted by stress block.

Annex D

Temporarily removed to Part -1-3

Annex E (informative)

E.1 Reduction factor for slenderness and eccentricity within the mid-height of a wall

(1) In the middle of the wall height, by using a simplification of the general principles given in 6.1.1, the reduction factor, Φ_m , taking into account the slenderness of the wall and the eccentricity of loading, may be estimated for $E = 1\,000\,f_k$, as assumed in 6.1.3, from:

$$\Phi_m = A_1 e^{-\frac{u^2}{2}} \quad (E.1)$$

where: $A_1 = 1 - 2 \frac{e_{mk}}{t}$ (E.2)

$$u = \frac{\frac{h_{ef}}{t_{ef}} - 2}{23 - 37 \frac{e_{mk}}{t}} \quad (E.3)$$

and e_{mk} , h_{ef} , t and t_{ef} are as defined in 6.1.3, and e is the base of natural logarithms.

(2) The values of Φ_m derived from equation (E.1) are given in table E.1 for different eccentricities and are represented in graphical form in figure 6.2.

(3) For any modulus of elasticity E and characteristic compressive strength of unreinforced masonry f_k equations (E.1) and (E.2) may also be applied, however, with:

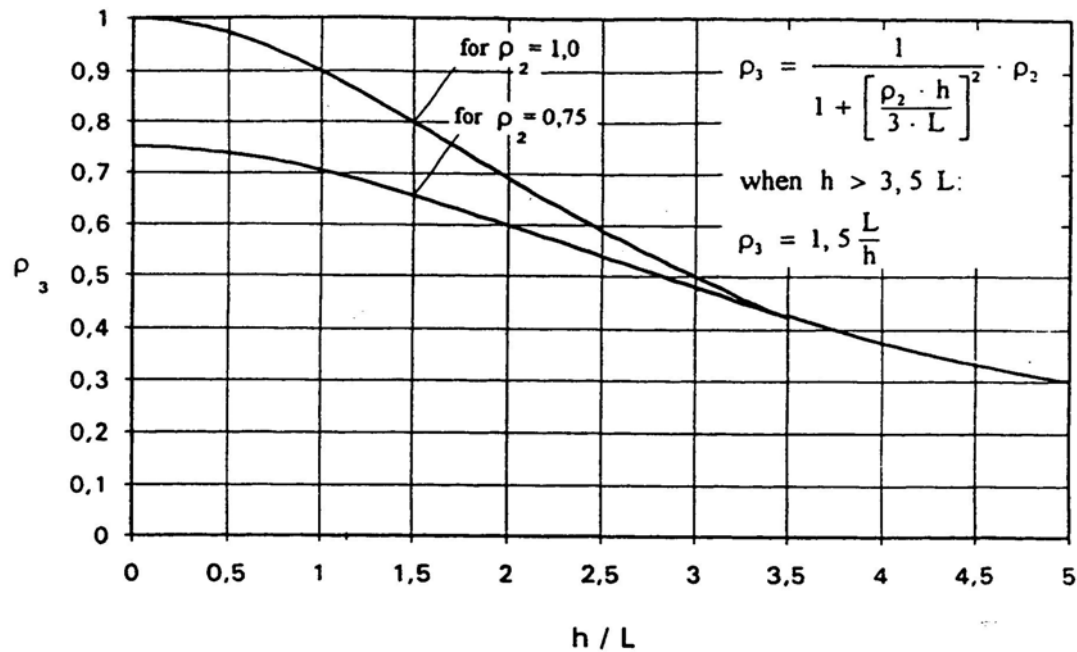
$$u = \frac{\lambda - 0,063}{0,73 - 1,17 \frac{e_{mk}}{t}} \quad (E.4)$$

where: $\lambda = \frac{h_{ef}}{t_{ef}} \sqrt{\frac{f_k}{E}}$ (E.5)

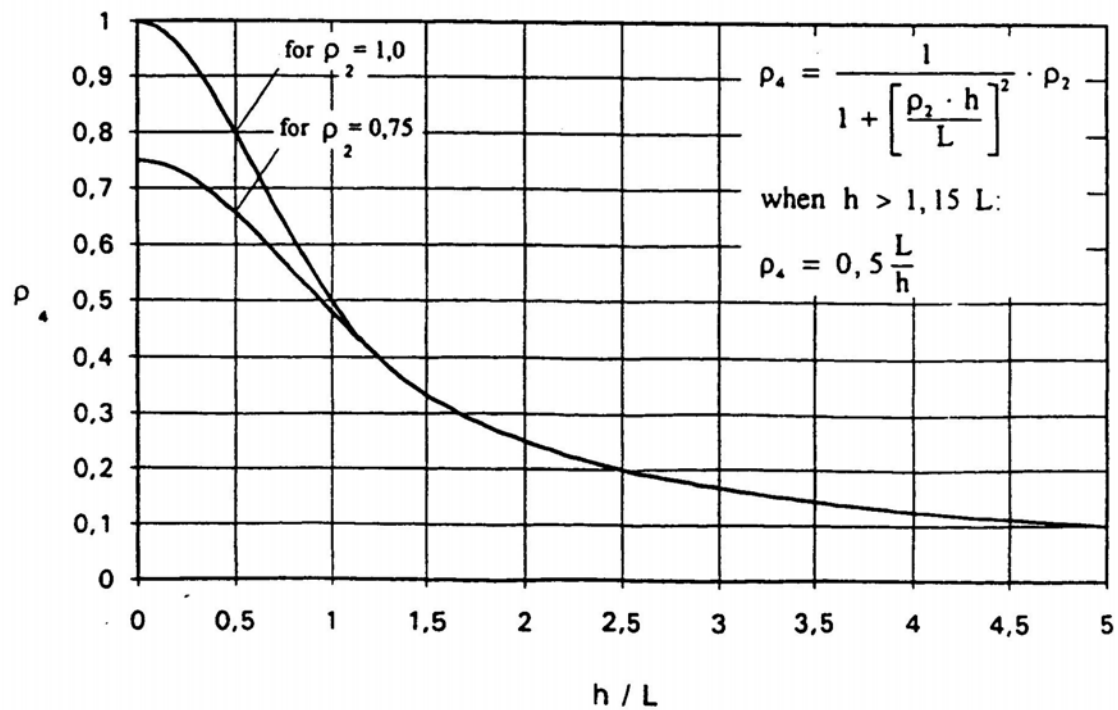
Table E.1 : Capacity reduction factor, Φ_m , for $E = 1\,000\,f_k$

Slenderness ratio h_{ef}/t_{ef}	Eccentricity e_{mk}						
	0,05 t	0,10 t	0,15 t	0,20t	0,25t	0,30t	0,33t
0	0,90	0,80	0,70	0,60	0,50	0,40	0,34
1	0,90	0,80	0,70	0,60	0,50	0,40	0,34
2	0,90	0,80	0,70	0,60	0,50	0,40	0,34
3	0,90	0,80	0,70	0,60	0,50	0,40	0,34
4	0,90	0,80	0,70	0,60	0,49	0,39	0,33
5	0,89	0,79	0,69	0,59	0,49	0,39	0,33
6	0,88	0,78	0,68	0,58	0,48	0,38	0,32
7	0,88	0,77	0,67	0,57	0,47	0,37	0,31
8	0,86	0,76	0,66	0,56	0,45	0,35	0,29
9	0,85	0,75	0,65	0,54	0,44	0,34	0,28
10	0,84	0,73	0,63	0,53	0,42	0,32	0,26
11	0,82	0,72	0,61	0,51	0,40	0,30	0,24
12	0,80	0,70	0,59	0,49	0,38	0,28	0,22
13	0,79	0,68	0,57	0,47	0,36	0,26	0,20
14	0,77	0,66	0,55	0,45	0,34	0,24	0,18
15	0,75	0,64	0,53	0,42	0,32	0,22	0,16
16	0,72	0,61	0,51	0,40	0,30	0,20	0,15
17	0,70	0,59	0,48	0,38	0,28	0,18	0,13
18	0,68	0,57	0,46	0,35	0,25	0,16	0,11
19	0,65	0,54	0,44	0,33	0,23	0,14	0,10
20	0,63	0,52	0,41	0,31	0,21	0,13	0,08
21	0,60	0,49	0,39	0,29	0,19	0,11	0,07
22	0,58	0,47	0,36	0,26	0,17	0,10	0,06
23	0,55	0,44	0,34	0,24	0,16	0,08	0,05
24	0,52	0,42	0,32	0,22	0,14	0,07	0,04
25	0,50	0,39	0,29	0,20	0,12	0,06	0,04
26	0,47	0,37	0,27	0,18	0,11	0,05	0,03
27	0,45	0,35	0,25	0,17	0,10	0,04	0,02
28	0,42	0,32	0,23	0,15	0,08	0,04	0,02
29	0,40	0,30	0,21	0,13	0,07	0,03	0,01
30	0,37	0,28	0,19	0,12	0,06	0,03	0,01

Annex F (informative)



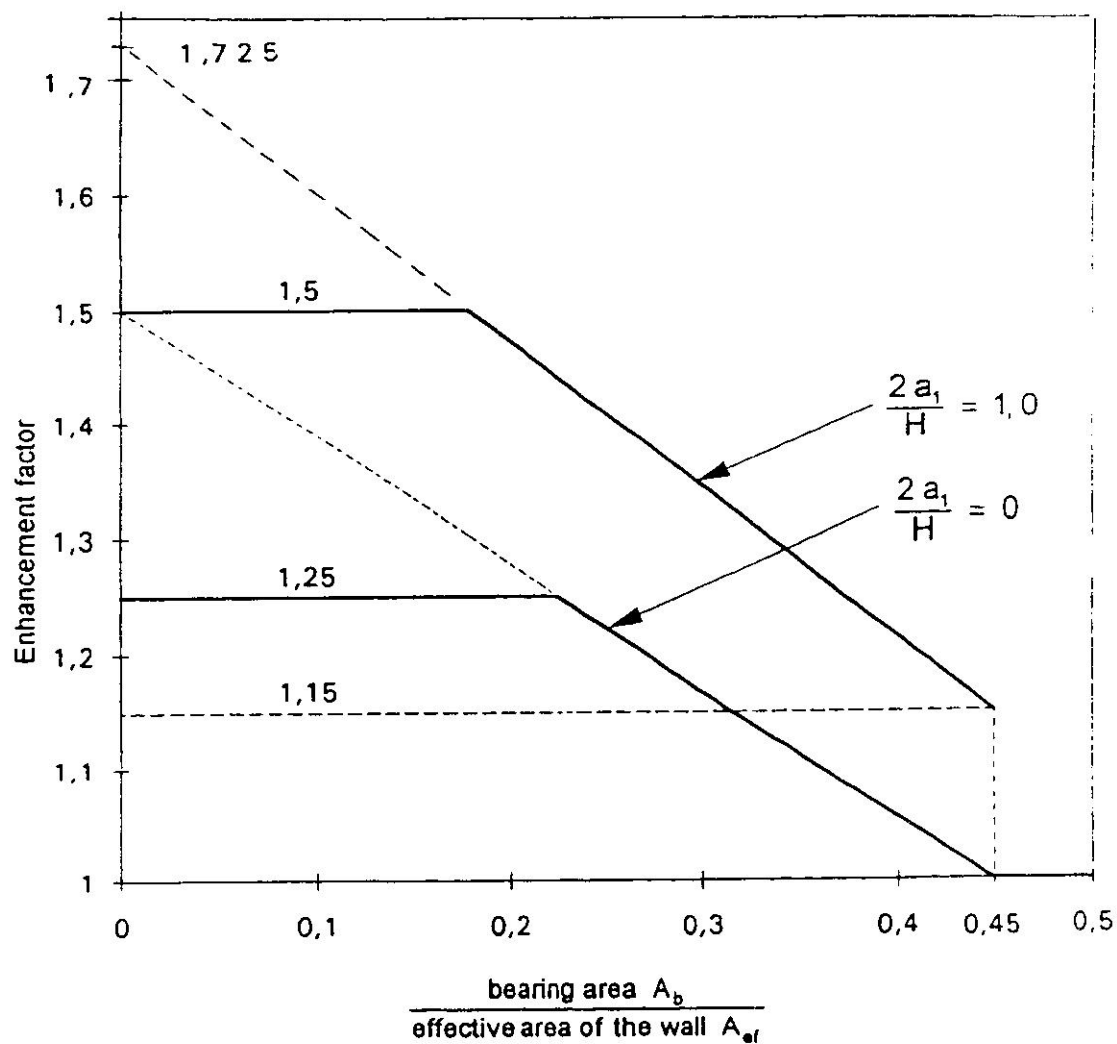
F.1 Graph showing values of ρ_3 using equations 6.10 and 6.11



F.2 Graph showing values of ρ_4 using equations 6.12 and 6.13

Annex G (informative)

G.1 Graph showing the enhancement factor as given in 4.4.8 : Concentrated loads under bearings.



Annex H (Normative)

H.1 Limiting height and length to thickness ratios for walls under the serviceability limit state

(1) Notwithstanding the ability of a wall to satisfy the ultimate limit state, which must be verified, its size should be limited to that which results from use of figures G.1, G.2 or G.3, depending on the restraint conditions as shown on the figures, where h is the clear height of the wall, L is the length of the wall and t is the thickness of the wall; for cavity walls use t_{ef} in place of t .

(2) Where walls are restrained at the top but not at the ends, h should be limited to $30t$.

(3) The minimum thickness of the wall, or one leaf of a cavity wall, should be 100 mm.

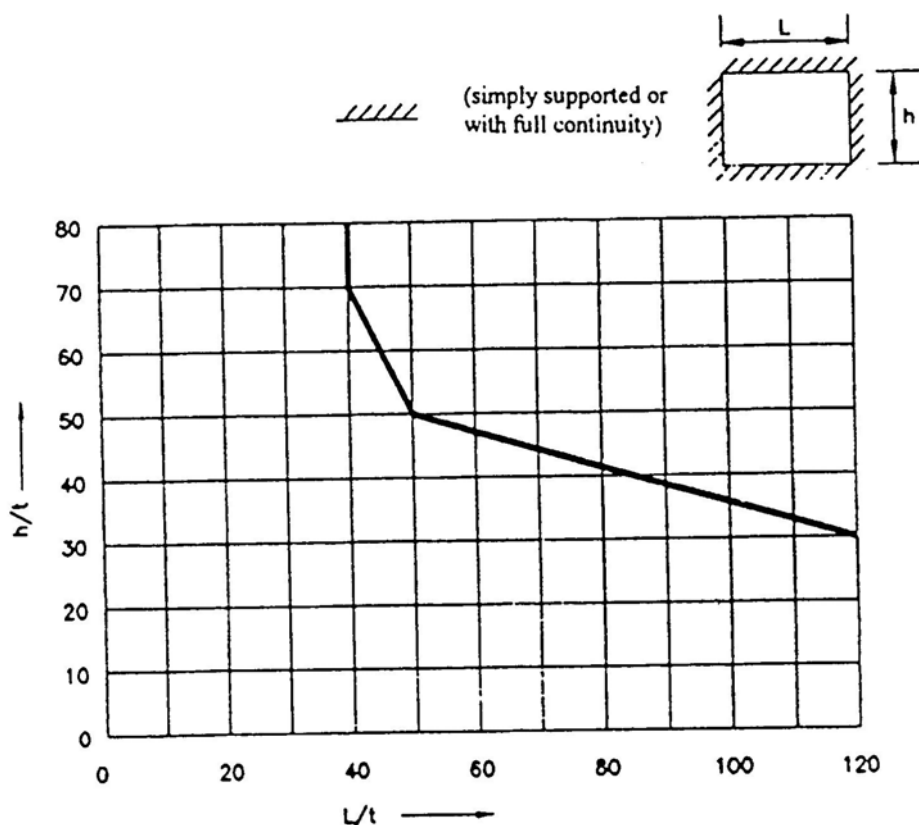


Figure H.1: Limiting height and length to thickness ratios of walls restrained on all four edges

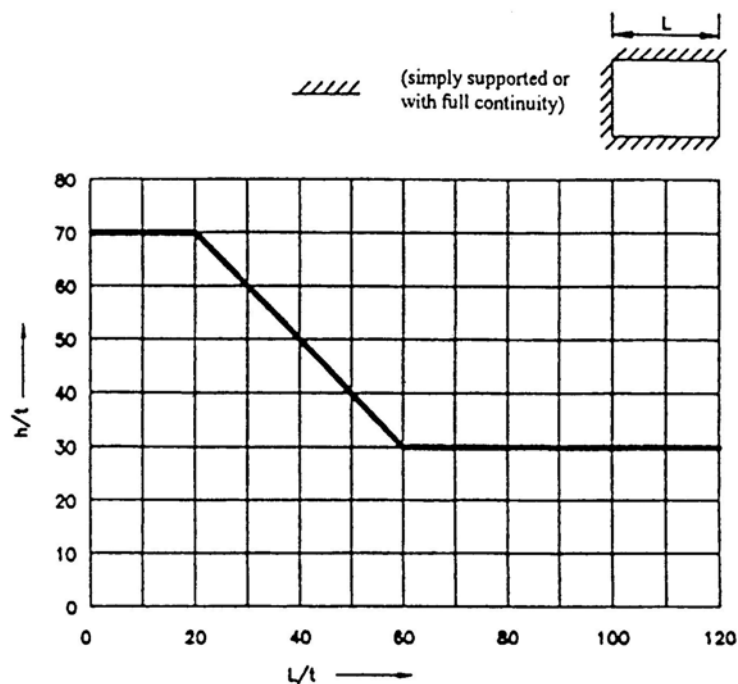


Figure H.2: Limiting height and length to thickness ratios of walls restrained at the bottom, the top and one vertical edge

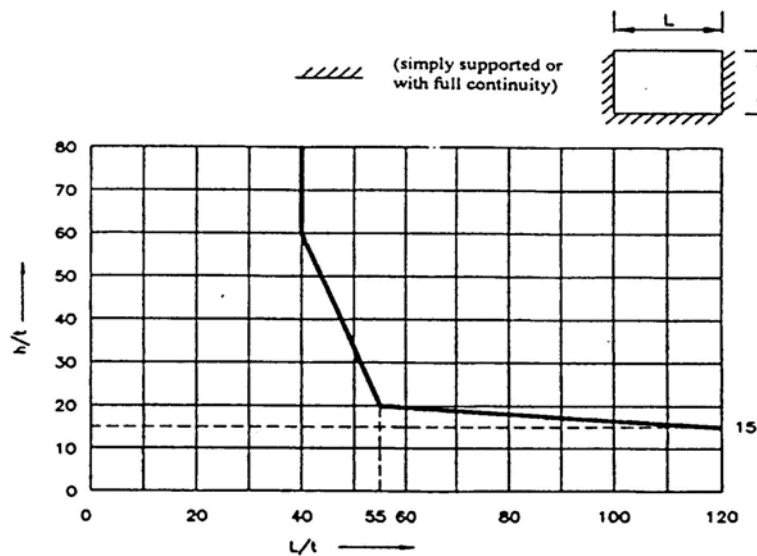


Figure H.3: Limiting height and length to thickness ratios of walls restrained at the edges, the bottom, but not the top

Annex I (Normative)

I.1 Approximate calculation of wall areas with both horizontal and vertical load

(1) The plate area is assumed to be subject to a horizontal out-of-plane load w , a vertical load N and a moment parallel to the longitudinal axis at the top M_{top} .

(2) M_{top} can be included in the degree of restraint of the support. If the degree of restraint is inclusive of M_{top} the value of the degree of restraint can be assumed to be between -1 and +1.

(3) The wall may be calculated according to EN 1996-1-3 using the α factors.

[PT Note: will be 6.3 in EN 1996-1-1.]

(4) Deflective loads are reduced to an equivalent load on a wall area supported at the top and at the bottom, eg by the reduction factor k_a

$$k_a = \frac{\text{moment capacity of a bilateral supported wall area}}{\text{moment capacity of the actual wall area supported at 3 or 4 sides}} = 8\mu\alpha \frac{l^2}{h^2} \cdot 3$$

where

α is the factor taken from table 4.1 in prEN 1996-1-3

μ is the ratio of characteristic flexural strength f_{xk1} divided by f_{xk2}

h height of the wall area

l length of the wall area

(5) The column loadbearing capacity is determined according to 4.4 (*note: ENV number*) as a bilateral wall area supported at the top and at the bottom by the reduced horizontal loads and the reduced moments but with full normal force.

(6) Wall areas with holes are designed by distributing the load onto the hole by half load at each side of the hole.

[PT Note: Some help is needed from Denmark to make this more understandable. This will be done at the next SC 6 meeting.]