EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

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ENV 1996-1-1

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Descriptors: buildings, construction, masonry work, building codes, computation, generalities

English version

Eurocode 6: Design of masonry structures - Part 1-1: General rules for buildings - Rules for reinforced and unreinforced masonry

Eurocode 6: Calcul des ouvrages en maçonnerie - Partie 1-1: Règles générales - Règles pour la maçonnerie armée et non armée

Eurocode 6: Bemessung und Konstruktion von Mauerwerksbauten - Teil 1-1: Allgemeine Regeln - Regeln für bewertes und bewertes Mauerwerk

This European Prestandard (ENV) was approved by CEN on 1994-06-10 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existance of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

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CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart,36 B-1050 Brussels

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Foreword

Objectives of the Eurocodes

(1) The Structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that it is necessary to indicate the quality of the construction products, and the standard of workmanship needed on and off site to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for the methods for testing their performance are available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

Background of the Eurocode programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the Structural Eurocodes.

(5) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

Eurocode programme

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:-

EN 1991 Eurocode 1 : Basis of design and 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 alloy structures.

(8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

(9) This ENV 1996-1-1 is being published as a European Prestandard (ENV) with an initial life of three years.

(10) This prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years, CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC 250/SC6 at the following address:-

DIN
Burggrafenstrasse 6
10772 Berlin
Germany

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV 1996-1-1 have been assigned indicative values which are identified by $\boxed{}$ (" boxed values"). The authorities in each member country are expected to review the "boxed values" and <u>may</u> substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International standards, may not be available by the time this prestandard is issued. It is, therefore, anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this prestandard, will be issued by each member country or its Standards Organisation.

(15) It is intended that this prestandard is used in conjunction with the NAD valid in the country where the building and civil engineering work is located.

Matters specific to this prestandard

(16) The general scope of Eurocode 6 is defined in clause 1.1.1 of this ENV 1996-1-1 and the scope of this part of Eurocode 6 is defined in clause 1.1.2. Additional parts of Eurocode 6 which are planned are indicated in clause 1.1.3 of this ENV 1996-1-1.

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 in unreinforced, reinforced, prestressed and confined masonry.

(2)P Eurocode 6 is only concerned 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[1] 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. Generally, the rules related to execution and workmanship should be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works[1] and methods of construction[1].

(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"[2] 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 "Basis of design and actions on structures"[3].

1.1.2 Scope of Part 1-1 of Eurocode 6

(1)P Part 1-1 of Eurocode 6 gives a general basis for the design of buildings and civil engineering works in unreinforced, reinforced, prestressed and confined masonry made with the following masonry units laid in mortar made with natural sand, or crushed sand, or lightweight aggregate:

- fired clay units, including lightweight clay units;

- calcium silicate units;

- concrete units, made with dense or lightweight aggregates;

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1) For the meaning of these terms, see 1.4.1
2) At present at the draft stage
3) At present at the draft stage.

- autoclaved aerated concrete units;

- manufactured stone units;

- dimensioned natural stone units.

(2)P Part 1-1 deals with reinforced masonry where the reinforcement is added to provide ductility, strength or serviceability. The principles of the design of prestressed masonry and confined masonry are given, but application rules are not provided.

(3) In so far as Part 1-1 gives the basis for the design of reinforced and prestressed masonry, the designer should consider the extent of any concrete infill and the contribution of the masonry to the load resistance and, where the concrete infill makes a much greater contribution to the load resistance than the masonry, Eurocode 2 should be used and the strength of the masonry should be ignored.

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

(5) In addition, 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; their use and any limits of applicability are explained in the text where necessary.

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

- Section 1 : General.

- Section 2 : Basis of design.

- Section 3 : Materials.

- Section 4 : Design of masonry.

- Section 5 : Structural detailing.

- Section 6 : Construction.

(7)P Sections 1 and 2 are common to all Eurocodes, with the exception of some additional clauses which are required for masonry.

Note: The material independent clauses in Section 2 will be replaced by references to ENV 1991-1, when it is published.

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

- resistance to fire (which is dealt with in ENV 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).

1.1.3 Further parts of Eurocode 6

(1)P Part 1-1 of Eurocode 6 will be supplemented by further parts which will complement or adapt it for particular aspects of special types of building or civil engineering works, special methods of construction and certain other aspects of design which are of general practical importance.

(2)P Further parts of Eurocode 6 which, at present, are being prepared or are planned, include the following:

- Part 1-2 : Structural fire design.
- Part 1-3 : Detailed rules on lateral loading.
- Part 1-X : Complex shape sections in masonry structures.
- Part 2 : Design, selection of materials and execution of masonry.
- Part 3 : Simplified and simple rules for masonry structures.
- Part 4 : Constructions with lesser requirements for reliability and durability.

1.2 Distinction between principles and application rules

(1)P Depending on the character of the individual clauses, distinction is made in this Part 1-1 of ENV 1996 between principles and application rules.

(2)P The principles comprise:

- general statements and definitions for which there is no alternative;

- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3)P The principles are defined by the letter P, following the paragraph number, for example, (1)P.

(4)P The application rules are generally recognised rules which follow the principles and satisfy their requirements.

(5)P It is permissible to use alternative design rules differing from the application rules given in this Eurocode, provided that it is shown that the alternative rules accord with the relevant principles and have not less than the same reliability.

(6)P The application rules are all clauses not indicated as being principles.

1.3 Assumptions

(1)P The following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.

- Adequate supervision and quality control is provided in factories, in plants, and on site.

- Construction is carried out by personnel having the appropriate skill and experience.

- The construction materials and products are used as specified in this Eurocode or in the relevant material or product specifications.

- The structure will be adequately maintained.

- The structure will be used in accordance with the design brief.

(2)P The design procedures are valid only when the requirements for execution and workmanship given in Section 6 of this ENV 1996-1-1 are also complied with.

(3)P Numerical values identified by $\boxed{}$ are given as indications. Other values may be specified by Member States.

1.4 Definitions

1.4.1 Terms common to all Eurocodes

(1)P Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

(2)P The following terms are used in common for all Eurocodes with the following meanings:

- **Construction works** : Everything that is constructed or results from construction operations.[4] This term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.

- **Execution** : The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

Note: In English, "construction" may be used in certain combinations of words, when there is no ambiguity (for example, "during construction").

- **Structure** : Organised combination of connected parts designed to provide some measure of rigidity.[5] This term refers to load carrying parts.

- **Type of building or civil engineering works** : Type of "construction works" designating its intended purpose, for example, dwelling house, industrial building, road bridge.

Note: "Type of construction works" is not used in English.

- **Form of structure** : Structural type designating the arrangement of structural elements, for example, beam, triangulated structure, arch, suspension bridge.

- **Construction material** : A material used in construction work, for example, concrete, steel, timber, masonry.

- **Type of construction** : Indication of principal structural material, for example, reinforced concrete construction, steel construction, timber construction, masonry construction.

- **Method of construction** : Manner in which the construction will be carried out, for example, cast in place, prefabricated, cantilevered.

- **Structural system** : The loadbearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

Note: The equivalent terms in six languages are given in table 1.1.

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[4] This definition accords with the International Standard ISO 6707 Part 1.
[5] The International Standard ISO 6707 Part 1 gives the same definition, however, adds "or a construction works having such an arrangement". For Eurocodes this addition is not used, in order to avoid ambiguous translations.

1.4.2 Special terms used in this ENV 1996-1-1

1.4.2.1 Masonry

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

(2)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 forces.

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

(4)P **Confined masonry** : Masonry built rigidly between reinforced concrete or reinforced masonry structural columns and beams on all four sides (not designed to perform as a moment resistant frame).

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

Table 1-1 : List of equivalent terms in Community languages (to be completed for other Community languages)

ENGLISH	FRANCAIS	DEUTSCH	ITALIANO
Construction works	Construction	Bauwerk	Costruzione
Execution	Exécution	(Bau-) Ausführung	Esecuzione
Structure	Structure	Tragwerk	Struttura
Type of building or civil engineering works	Nature de construction	Art des Bauwerks	Tipo di costruzione
Form of structure	Type de structure	Art des Tragwerks	Tipo di struttura
Construction material	Matériau de construction	Baustoff; Werkstoff (Stahlbau)	Materiale da costruzione
Type of construction	Mode de construction	Bauart	Sistema costruttivo
Method of construction	Procédé d'execution	Bauverfahren	Procedimento esecutivo
Structural system	Système structural	Tragsystem	Sistema strutturale

Table 1-1 (Continued) : List of equivalent terms in Community languages (to be completed for other Community languages)

ENGLISH	NEDERLANDS	ESPANOL	PORTUGUES
Construction works	Bouwwerk	Construcción	Obras de construção
Execution	Uitvoering	Ejecucion	Execução
Structure	Draag-constructie	Estructura	Estrutura
Type of building or civil engineering works	Type bouwwerk	Naturaleze de la construcción	Tipos de obras de construção
Form of structure	Type draag-constructie	Tipo de estructura	Tipo de estrutura
Construction material	Constructie materiaal	Material de construcción	Material de construção
Type of construction	Bouwwijze	Modo de construcción	Tipo de construção
Method of construction	Bouwmethode	Procedimiento de ejecucion	Processo de construção
Structural system	Constructief systeem	Sistema estructural	Sistema estrutural

1.4.2.2 Strength of masonry

(1)P **Characteristic strength of masonry** : The value of strength corresponding to a 5% fractile of all strength measurements of the masonry.

Note: the value may be taken from the results of specific tests or from an evaluation of test data or other specified values.

(2)P **Compressive strength of masonry** : The strength of masonry in compression without the effects of platten 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.

1.4.2.3 Masonry units

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

(2)P **Groups 1, 2a, 2b 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.

Note: For the purposes of this Eurocode, reference is made to EN 772-1, "Methods of test for masonry units. Part 1. Determination of compressive strength".

(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.

(12)P **Characteristic compressive strength of masonry units** : The compressive strength corresponding to a 5% fractile of the compressive strength of a specified number of masonry units.

Note: For the purposes of this Eurocode, reference is made to EN 772-1, "Methods of test for masonry units. Part 1. Determination of compressive strength".

1.4.2.4 Mortar

(1)P **Mortar** : A mixture of inorganic binders, aggregates and water, together with additions and admixtures if required.

Note: For the purposes of this Eurocode, reference is made to EN 998-2, "Specification for mortar for masonry. Part 2. Masonry mortar".

(2)P **General purpose mortar** : A mortar for use in joints with a thickness greater than 3 mm and in which only dense aggregates are used.

(3)P **Thin layer mortar** : A designed mortar for use in joints between 1mm and 3 mm in thickness.

(4)P **Lightweight mortar** : A designed mortar with a dry hardened density lower than 1 500 kg/m^3.

(5)P **Designed mortar** : A mortar designed and manufactured to fulfil stated properties and subjected to test requirements.

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

(7)P **Factory made mortar** : A mortar batched and mixed in a factory and supplied to the building site.

(8)P **Pre-batched mortar** : A material consisting of constituents batched in a plant, supplied to the building site and mixed there under factory specified proportions and conditions.

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

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

Note: For the purposes of this Eurocode, reference is made to EN 1015-11, "Methods of test for mortar for masonry. Part 11. Determination of flexural and compressive strength of hardened mortar".

1.4.2.5 Concrete infill

(1)P **Concrete infill** : A concrete mix of suitable consistency and aggregate size to fill cavities or voids in masonry.

1.4.2.6 Reinforcement

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

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

Note: For the purposes of this Eurocode, reference is made to EN 845-3, "Specification for ancillary components for masonry. Part 3. Bed joint reinforcement".

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

1.4.2.7 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.

1.4.2.8 Mortar joints

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

(2)P **Perpend 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 having a maximum thickness of 3 mm.

(5)P **Movement joint** : A joint permitting free movement in the plane of the wall.

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

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

1.4.2.9 Wall types

(1)P **Load-bearing wall** : A wall of plan area greater than 0,04m², or one whole unit if Group 2a, Group 2b or Group 3 units of plan area greater than 0,04m² are used, 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, with either or both leaves supporting vertical loads. 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 (not exceeding 25 mm) 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, spaced at least 50 mm apart, 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 general purpose mortar strips 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.

1.4.2.10 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.

1.5 S.I. units

(1)P S.I. Units shall be used in accordance with ISO 1000.

(2) For calculations, the following units are recommended:

- forces and loads : kN, kN/m, kN/m^2;

- unit mass : kg/m^3;
- unit weight : kN/m^3;
- stresses and strengths : N/mm^2 (= MN/m^2 or MPa);
- moments (bending, etc) : kNm.

1.6 Symbols used in this ENV 1996-1-1

(1)P Particular material-independent symbols used are as follows:

A accidental action;

A_d design value of an accidental action;

A_k characteristic value of an accidental action;

C_d nominal value, or function, of certain design properties of materials;

E action effect;

E_d design value of an action effect;

$E_{d,dst}$ design value of a destabilizing action effect;

$E_{d,stb}$ design value of a stabilizing action effect;

F action;

F_d design value of an action;

F_k characteristic value of an action;

G permanent action;

G_d design value of a permanent action;

$G_{d,inf}$ lower design value of a permanent action;

$G_{d,sup}$ upper design value of a permanent action;

G_k characteristic value of a permanent action;

$G_{k,inf}$ lower characteristic value of a permanent action;

$G_{k,sup}$ upper characteristic value of a permanent action;

P prestressing action;

Q variable action;

Q_d design value of a variable action;

Q_k characteristic value of a variable action;

R resistance capacity;

R_d design value of resistance capacity;

S_d design value of an internal action effect;

W_k characteristic value of wind action;

X_d design value of a material property;

X_k characteristic value of a material property;

a_d design value of geometrical data;

a_{nom} nominal value of geometrical data;

Δ_a additive (or reducing) safety element for geometrical data;

γ_A partial safety factor for accidental actions;

γ_F partial safety factor for actions;

γ_G partial safety factor for permanent actions;

$\gamma_{G,inf}$ partial safety factor associated with $G_{k,inf}$;

$\gamma_{G,sup}$ partial safety factor associated with $G_{k,sup}$;

γ_{GA} partial safety factor for permanent actions used in accidental combinations;

γ_M partial safety factor for material properties;

γ_P partial safety factor for prestressing actions;

γ_Q partial safety factor for variable actions;

ψ_o coefficient defining the combination value of variable actions;

ψ_1 coefficient defining the frequent value of variable actions;

ψ_2 coefficient defining the quasi-permanent value of variable actions;

ζ reduction factor for γ_G.

(2)P Particular material-dependent symbols used for masonry are as follows:

A area of a wall;

A_1 numerical factor;

A_b area of bearing;

A_{ef} effective area of a wall;

a_1 distance from the end of a wall to the nearer edge of a bearing;

b_c distance apart of cross walls or buttresses;

b_s distance between centre lines of mortar strips;

d deflection of arch under design lateral load;

E modulus of elasticity;

E_n modulus of elasticity of a member (where n = 1, 2, 3 or 4);

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_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_d design compressive strength of masonry;

f_k characteristic compressive strength of masonry;

f_m mean compressive strength of mortar;

f_v shear strength of masonry;

f_{vd} design shear strength of masonry;

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});

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_{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_c length of wall in compression;

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;

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;

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_{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;

Z section modulus;

α bending moment coefficient;

γ_M partial safety factor for material properties;

δ factor allowing for height and width of masonry units;

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_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.

(3)P Particular material-dependent symbols used for reinforced masonry are as follows:

A_m area of masonry;

A_{mr} area of reinforced masonry including concrete infill;

A_s area of reinforcment in tension;

A_{sw} area of shear reinforcement;

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_{ef} effective width of a flanged member;

C compressive strength class of concrete;

d effective depth of section;

E_s modulus of elasticity of reinforcing steel;

F_c design compressive bending force in member;

F_s design tensile force in steel;

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_{vk} characteristic shear strength of masonry or concrete infill;

f_p tensile strength of prestressing steel;

f_{tk} characteristic tensile strength of reinforcing steel;

f_y yield strength of the reinforcing steel;

f_{yk} characteristic yield strength of reinforcing steel;

h_m overall depth of a section;

l_b anchorage length for reinforcing steel;

l_{ef} effective span of a member;

M_{Rd} design moment of resistance;

S slump class of concrete;

V_{Rd2} design shear resistance of reinforcement;

s spacing of shear reinforcement;

x depth of the compression zone of a member;

z lever arm in a reinforced masonry member subjected to bending;

α angle of shear reinforcement;

γ_S partial safety factor for steel;

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;

ϕ diameter of reinforcement.

2 Basis of design

2.1 Fundamental requirements

(1)P A structure shall be designed and constructed in such a way that:

- with acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and its cost, and

- with appropriate degrees of reliability, it will sustain all actions and influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

(2)P A structure shall be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human error, to an extent disproportionate to the original cause.

(3) The potential damage should be limited or avoided by appropriate choice of one or more of the following:

- avoiding, eliminating or reducing the hazards which the structure is to sustain;

- selecting a structural form which has low sensitivity to the hazards considered;

- selecting a structural form and design that can survive adequately the accidental removal of an individual element;

- tying the structure together.

(4)P The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project.

2.2 Definitions and classifications

2.2.1 Limit states and design situations

2.2.1.1 Limit states

(1)P Limit states are states beyond which the structure no longer satisfies the design performance requirements.

(2)P Limit states are classified into:

- ultimate limit states;

- serviceability limit states.

(3)P Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

(4)P States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also classified and treated as ultimate limit states.

(5)P Ultimate limit states which may require consideration include:

- loss of equilibrium of the structure or any part of it, considered as a rigid body;

- failure by excessive deformation, rupture, or loss of stability of the structure or any part of it, including supports and foundations.

(6)P Serviceability limit states correspond to states beyond which specified service criteria are no longer met.

(7) Serviceability limit states which may require consideration include:

- deformations or deflections which affect the appearance or effective use of the structure (including the malfunction of machines or services) or cause damage to finishes or non-structural elements;

- vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.

2.2.1.2 Design situations

(1)P Design situations are classified as:

- persistent situations corresponding to normal conditions of use of the structure;

- transient situations, for example, during construction or repair;

- accidental situations.

2.2.2 Actions

2.2.2.1 Definitions and principal classification

(1)P An action (F) is:

- a force (load) applied to the structure (direct action), or

- an imposed deformation (indirect action), for example, temperature effects or settlement.

(2)P Actions are classified:

(i) by their variation in time:

- permanent actions (G), for example, self-weight of structures, fittings, ancillaries and fixed equipment;

- variable actions (Q), for example, imposed loads, wind loads or snow loads;

- accidental actions (A), for example, explosions or impact from vehicles;

(ii) by their spatial variation:

- fixed actions, for example, self-weight (but see 2.3.2.3(2) for structures very sensitive to variations in self-weight);

- free actions, which result in different arrangements of actions, for example, movable imposed loads, wind loads, snow loads.

(3)P Prestressing action (P) is a permanent action but, for practical reasons, it is treated separately.

2.2.2.2 Characteristic values of actions

(1)P Characteristic values F_k are specified:

- in ENV 1991 or other relevant loading codes, or

- by the client, or the designer in consultation with the client, provided that the minimum provisions specified in relevant codes or by the competent authority are observed.

(2)P For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (for example, for some superimposed permanent loads), two characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere a single characteristic value (G_k) is sufficient.

(3) The self-weight of the structure may, in most cases, be calculated on the basis of the nominal dimensions and mean unit masses.

(4)P For variable actions the characteristic value (Q_k) corresponds to either:

- the upper value with an intended probability of not being exceeded, or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or

- the specified value.

(5) For accidental actions the characteristic value A_k (when relevant) generally corresponds to a specified value.

2.2.2.3 Representative values of variable actions

(1)P The main representative value is the characteristic value Q_k .

(2)P Other representative values are expressed in terms of the characteristic value Q_k by means of a coefficient ψ_i . These values are defined as:

- combination value: $\psi_0 Q_k$;

- frequent value: $\psi_1 Q_k$;

- quasi-permanent value: $\psi_2 Q_k$.

(3) Supplementary representative values are used for fatigue verification and dynamic analysis.

(4)P The coefficient ψ_i is specified:

- in ENV 1991 or other relevant loading codes, or

- by the client or the designer in conjunction with the client, provided that the minimum provisions specified in relevant codes or by the competent authority are observed.

2.2.2.4 Design values of actions

(1)P The design value F_d of an action is expressed in general terms as:

$$F_d = \gamma_F F_k \tag{2.1}$$

(2) Specific examples are:

$$G_d = \gamma_G G_k \tag{2.2}$$

$$Q_d = \gamma_Q Q_k \quad \text{or} \quad \gamma_Q \psi_i Q_k \tag{2.3}$$

$$A_d = \gamma_A A_k \quad \text{(if } A_d \text{ is not directly specified)} \tag{2.4}$$

$$P_d = \gamma_P P_k \tag{2.5}$$

where γ_F, γ_G, γ_Q, γ_A and γ_P are the partial safety factors for the action considered taking account of, for example, the possibility of unfavourable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effects of actions and uncertainties in the assessment of the limit state considered.

(3)P The upper and lower design values of permanent actions are expressed as follows:

- where only a single characteristic value G_k is used (see 2.2.2.2(2)) then:

$$G_{d,sup} = \gamma_{G,sup} G_k \qquad (2.6)$$

$$G_{d,inf} = \gamma_{G,inf} G_k \qquad (2.7)$$

- where upper and lower characteristic values of permanent actions are used (see 2.2.2.2(2)) then:

$$G_{d,sup} = \gamma_{G,sup} G_{k,sup} \qquad (2.8)$$

$$G_{d,inf} = \gamma_{G,inf} G_{k,inf} \qquad (2.9)$$

where $G_{k,sup}$ and $G_{k,inf}$ are the upper and lower characteristic values of permanent actions, and $\gamma_{G,sup}$ and $\gamma_{G,inf}$ are the upper and lower values of the partial safety factor for the permanent actions.

2.2.2.5 Design values of the effects of actions

(1)P The effects of actions (E) are responses (for example, internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions (E_d) are determined from the design values of the actions, geometrical data and material properties, when relevant:

$$E_d = E\ (F_d\ ,\ a_d\ ,\ \ldots) \qquad (2.10)$$

where a_d is defined in 2.2.4.

2.2.3 Material properties

2.2.3.1 Characteristic values

(1)P A material property is represented by a characteristic value X_k which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

(2)P In certain cases a designated value is used as the characteristic value.

2.2.3.2 Design values

(1)P The design value X_d of a material property is generally defined as:

$$X_d = \frac{X_k}{\gamma_M} \tag{2.11}$$

where γ_M is the partial safety factor for the material property.

(2)P Design values for the material properties, geometrical data and effects of actions, R, when relevant, should be used to determine the design resistance R_d from:

$$R_d = R\ (X_d\ ,\ a_d\ ,\ ...) \tag{2.12}$$

(3)P The design value R_d may be determined from tests. Guidance is given in EN 846-5, EN 846-6, EN 1052-1, EN 1052-2, EN 1052-3 and EN 1052-4.

2.2.4 Geometrical data

(1)P Geometrical data describing the structure are generally represented by their nominal values:

$$a_d = a_{nom} \tag{2.13}$$

(2)P In some cases the geometrical design values are defined by:

$$a_d = a_{nom} + \Delta_a \tag{2.14}$$

(3)P The values of Δ_a are given in the appropriate clauses.

2.2.5 Load arrangement and load cases

(1)P A load arrangement identifies the position, magnitude and direction of a free action.

(2)P A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

2.3 Design requirements

2.3.1 General

(1)P It shall be verified that no relevant limit state is exceeded.

(2)P All relevant design situtations and load cases shall be considered.

(3)P Possible deviations from the assumed directions or positions of actions shall be considered.

(4)P Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

2.3.2 Ultimate limit states

2.3.2.1 Verification conditions

(1)P When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be verified that:

$$E_{d,dst} \leq E_{d,stb} \tag{2.15}$$

where $E_{d,dst}$ and $E_{d,stb}$ are the design effects of destabilizing and stabilizing actions, respectively.

(2)P When considering a limit state of rupture or excessive deformation of a section, member or connection (fatigue excluded) it shall be verified that:

$$S_d \leq R_d \tag{2.16}$$

where S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments) and R_d is the corresponding design resistance, associating all structural properties with the respective design values.

(3)P When considering a limit state of transformation of the structure into a mechanism, it shall be verified that a mechanism does not occur unless actions exceed their design values, associating all structural properties with the respective design values.

(4)P When considering a limit state of stability induced by second-order effects it shall be verified that instability does not occur unless actions exceed their design values, associating all structural properties with the respective design values. In addition, sections shall be verified according to paragraph (2) above.

2.3.2.2 Combinations of actions

(1)P For each load case, design values E_d for the effects of actions shall be determined from combination rules involving design values of actions as identified by table 2.1.

(2)P The design values of table 2.1 shall be combined using the following rules (given in symbolic form):

- Persistent and transient design situations for verifications other than those relating to prestressing (fundamental combinations):

$$\sum \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{o,i} Q_{k,i} \qquad (2.17)$$

Note: This combination rule is an amalgamation of two separate load combinations:

$$\sum \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, \psi_{o,i}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{o,i}\, Q_{k,i}$$

$$\sum \zeta_j\, \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{o,i} Q_{k,i}$$

- Accidental design situations (if not specified differently elsewhere):

$$\sum \gamma_{GA,j} G_{k,j} + A_d + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (2.18)$$

where:

$G_{k,j}$ is the characteristic values of permanent actions;

$Q_{k,1}$ is the characteristic value of one of the variable actions;

$Q_{k,i}$ is the characteristic values of the other variable actions;

A_d is the design value (specified value) of the accidental action;

$\gamma_{G,j}$ is the partial safety factor for permanent actions;

$\gamma_{GA,j}$ is as $\gamma_{G,j}$ but for accidental design situations;

$\gamma_{Q,i}$ is the partial safety factor for variable actions;

ψ_o , ψ_1 , ψ_2 are factors defined in 2.2.2.3.

(3) Combinations for accidental design situations either involve an explicit accidental action A or refer to a situation after an accidental event (A = 0). Unless specified otherwise, $\gamma_{GA} = \boxed{1{,}0}$ may be used.

(4)P In expressions (2.17) and (2.18) prestressing and indirect actions shall be introduced where relevant.

(5) Simplified combinations for building structures are given in 2.3.3.1.

Table 2.1 : Design values for actions for use in the combination of actions

Design situation	Permanent actions G_d	Variable actions		Accidental actions A_d
		One with its characteristic value	Others with their combination value	
Persistent and transient	$\gamma_G G_k$	$\gamma_Q Q_k$	$\psi_O \gamma_Q Q_k$	-
Accidental	$\gamma_{GA} G_k$	$\psi_1 Q_k$	$\psi_2 Q_k$	$\gamma_A A_k$ (if A_d is not specified directly)

2.3.2.3 Design value of permanent actions

(1)P In the various combinations defined above, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values, those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values (see 2.2.2.4(3)).

(2)P Where the results of a verification may be very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as individual actions. This applies in particular to the verification of static equilibrium. In the aforementioned cases specific γ_G values need to be considered (see 2.3.3.1(4) for building structures).

(3)P In other cases, either the lower or upper design value (whichever gives the more unfavourable effect) shall be applied throughout the structure.

(4) For continuous beams the same design value of the self-weight (evaluated as in 2.2.2.2(3)) may be applied to all spans.

2.3.3 Partial safety factors for ultimate limit states

2.3.3.1 Partial safety factors for actions on building structures

(1)P Partial safety factors for the persistent and transient design situations are given in table 2.2.

(2)P For accidental design situations to which equation (2.18) applies, the partial safety factor for variable actions is equal to $\boxed{1,0}$.

(3) By adopting the γ values given in table 2.2, equation (2.17) may be replaced by:

- considering only the most unfavourable variable action:

$$\sum \gamma_{G,j} G_{k,j} + 1{,}5\ Q_{k,1} \tag{2.19}$$

- considering all unfavourable variable actions:

$$\sum \gamma_{G,j} G_{k,j} + 1{,}35 \sum_{i \geq 1} Q_{k,i} \tag{2.20}$$

whichever gives the larger value.

(4) Where, according to 2.3.2.3(2), favourable and unfavourable parts of a permanent action need to be considered as individual actions, the favourable part should be associated with $\gamma_{G,inf}$ = $\boxed{0{,}9}$ and the unfavourable part with $\gamma_{G,sup}$ = $\boxed{1{,}1}$.

2.3.3.2 Partial safety factors for materials

(1)P Partial safety factors for material properties for the ultimate limit state are given in table 2.3.

(2)P When verifying the stability in the case of accidental actions, γ_M for masonry shall be taken as $\boxed{1{,}2}$, $\boxed{1{,}5}$ and $\boxed{1{,}8}$ for Categories A, B and C of levels of execution respectively, γ_M for anchorage and tensile and compressive resistance of wall ties and straps, and for anchorage bond of reinforcing steel, shall be taken as given in table 2.3 and γ_S for steel shall be taken as $\boxed{1{,}0}$.

2.3.4 Serviceability limit states

(1)P It shall be verified that:

$$E_d \leq C_d \tag{2.21}$$

where:

C_d is a nominal value or a function of certain design properties of materials related to the design effects of actions considered;

E_d is the design effect of actions determined on the basis of one of the combinations defined in paragraph (2)P below.

Table 2.2:Partial safety factors for actions in building structures for persistent and transient design situations

	Permanent actions (γ_G) (see note)	Variable actions (γ_Q) One with its characteristic value	Variable actions (γ_Q) Others with their combination value	Prestressing (γ_p)
Favourable effect	$\boxed{1,0}$	$\boxed{0}$	$\boxed{0}$	$\boxed{0,9}$
Unfavourable effect	$\boxed{1,35}$	$\boxed{1,5}$	$\boxed{1,35}$	$\boxed{1,2}$
Note: See also paragraph 2.3.3.1(3).				

Table 2.3: Partial safety factors for material properties (γ_M)

γ_M			Category of execution (see 6.9) A	B	C
Masonry (see note)	Category of manufacturing control of masonry units (see 3.1)	I	$\boxed{1,7}$	$\boxed{2,2}$	$\boxed{2,7}$
		II	$\boxed{2,0}$	$\boxed{2,5}$	$\boxed{3,0}$
Anchorage and tensile and compressive resistance of wall ties and straps			$\boxed{2,5}$	$\boxed{2,5}$	$\boxed{2,5}$
Anchorage bond of reinforcing steel			$\boxed{1,7}$	$\boxed{2,2}$	-
Steel (referred to as γ_S)			$\boxed{1,15}$	$\boxed{1,15}$	-
Note: The value of γ_M for concrete infill should be taken as that appropriate to the category of manufacturing control of the masonry units in the location where the infill is being used.					

(2)P Three combinations of actions for serviceability limit states are defined by the following expressions:

- Rare combination:

$$\sum G_{k,j} \ (+P) \ + Q_{k,1} \ + \sum_{i>1} \psi_{0,i} \ Q_{k,i} \qquad (2.22)$$

- Frequent combination:

$$\sum G_{k,j} (+P) \ + \psi_{1,1} Q_{k,1} \ + \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (2.23)$$

- Quasi-permanent combination:

$$\sum G_{k,j} (+P) + \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (2.24)$$

where the notation is defined in 2.3.2.2(2).

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

(4) Where the design considers compliance of serviceability limit states by detailed calculations, simplified expressions as given in paragraph (5) below, may be used for building structures.

(5) For building structures the rare combination may be simplified to the following expressions, which may also be used as a substitute for the frequent combination:

- considering only the most unfavourable variable action:

$$\sum G_{k,j} (+P) \ + Q_{k,1} \qquad (2.25)$$

- considering all unfavourable variable actions:

$$\sum G_{k,j} (+P) \ + 0{,}9 \sum_{i\geq 1} Q_{k,i} \qquad (2.26)$$

whichever gives the larger value.

(6)P Values of γ_M shall be taken as $\boxed{1{,}0}$, except where stated otherwise in particular clauses.

2.4 Durability

(1)P To ensure an adequately durable structure, the following inter-related factors shall be considered:

- the use of the structure;
- the required performance criteria;
- the expected environmental conditions;
- the composition, properties and performance of the materials;
- the shape of members, and the structural detailing;
- the quality of workmanship and level of control;
- the particular protective measures;
- the likely maintenance during the intended life.

(2)P The internal and external environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials.

3 Materials

3.1 Masonry Units

3.1.1 Types of masonry units

(1)P Masonry units shall be 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)P Masonry units shall be classified in terms of manufacturing control as either Category I or Category II.

(3) Category I may be assumed where the manufacturer agrees to supply consignments of masonry units to a specified compressive strength and has a quality control scheme, the results of which demonstrate that the mean compressive strength of a consignment, when sampled in accordance with the relevant part of EN 771 and tested in accordance with EN 772-1, has a probability of failing to reach the specified compressive strength not exceeding $\boxed{5}$ %.

Note:Further details relating to the quality control scheme may be given in the National Application Documents.

(4) Category II should be used when the mean value of the compressive strength of the masonry units complies with the declaration in accordance with the relevant part of EN 771, but the additional requirements for Category I are not met.

(5) Natural stone units should be considered as Category II units.

(6) Masonry units should be grouped as Group 1, Group 2a, Group 2b or Group 3, for the purposes of using the equations and other numerical values given in 3.6.2 and 3.6.3 and where grouping is referred to in other clauses. The requirements for Group 1, Group 2a, Group 2b and Group 3 units are given in table 3.1.

Table 3.1 : Requirements for grouping of masonry units.

	Group of masonry units			
	1	2a	2b	3
Volume of holes (% of the gross volume) (see note 1)	≤ 25	> 25-45 for clay units > 25-50 for concrete aggregate units	> 45-55 for clay units > 50-60 for concrete aggregate units (see note 2)	≤ 70
Volume of any hole (% of the gross volume)	≤ 12,5	≤ 12,5 for clay units ≤ 25 for concrete aggregate units	≤ 12,5 for clay units ≤ 25 for concrete aggregate units	Limited by area (see below)
Area of any hole	Limited by volume (see above)	Limited by volume (see above)	Limited by volume (see above)	≤ 2 800mm² except for units with a single hole when the hole should be ≤ 18 000mm²
Combined thickness (% of the overall width) (see note 3)	≥ 37,5	≥ 30	≥ 20	No requirement

Notes:

1. Holes may consist of formed vertical holes through the units or frogs or recesses.

2. If there is national experience, based on tests, that confirms that the safety of the masonry is not reduced unacceptably when a higher proportion of holes is incorporated, the limit of 55% for clay units and 60% for concrete aggregate units may be increased for masonry units that are used in the country having the national experience.

3. 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.

(7) The disposition of holes in Group 2a, Group 2b and Group 3 units should be such as to avoid the serious risk of cracks in thin webs and shells, either in manufacture, handling or use.

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.

(2) When the compressive strength of masonry units is quoted as the mean strength when tested in accordance with EN 772-1 this should be converted to the normalized compressive strength by converting to the air dried strength, if it is not already air dried, and multiplying by the factor δ as given in table 3.2 to allow for the height and width of the units.

(3) When the compressive strength of masonry units is quoted as the characteristic strength when tested in accordance with EN 772-1 this should be converted to the normalized compressive strength by changing the value of the strength to the mean equivalent, using a conversion factor based on the coefficient of variation, and then proceed as in paragraph (2) above.

Table 3.2 : Values of factor δ .

Height of unit (mm)	Least horizontal dimension of unit (mm)				
	50	100	150	200	250 or greater
50	0,85	0,75	0,70	-	-
65	0,95	0,85	0,75	0,70	0,65
100	1,15	1,00	0,90	0,80	0,75
150	1,30	1,20	1,10	1,00	0,95
200	1,45	1,35	1,25	1,15	1,10
250 or greater	1,55	1,45	1,35	1,25	1,15
Note: Linear interpolation is permitted.					

(4) Where the action effects result in compressive forces acting parallel to the bed face, either along the length of the unit or across its width, the normalized compressive strength of the unit should be determined for the relevant direction of loading, by testing in accordance with EN 772-1 (see 3.6.2.1 (4)).

(5) If the compressive strength of a special shaped unit is expected to have a predominant influence upon the masonry strength, the compressive strength of the special shaped unit should be estimated by testing the compressive strength of cut pieces representing the body of the unit, as far as possible in accordance with EN 772-1. Alternatively, it may be appropriate to determine the characteristic compressive strength of the masonry directly using

EN 1052-1.

3.1.2.2 Durability of masonry units

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

Note: Guidance on design and construction to provide adequate durability is given in Sections 5 and 6 of this ENV 1996-1-1 and in ENV 1996-2.

3.2 Mortar

3.2.1 Types of mortar

(1)P Factory made and pre-batched mortars shall be in accordance with EN 998-2. Site mixed mortar shall be in accordance with the particular clauses in 6.3.2. Pre-mixed sand/lime, used for site mixed mortar, shall be in accordance with EN 998-2.

(2) Masonry mortar should be classified as general purpose mortar, thin layer mortar or lightweight mortar.

(3) Thin layer mortar is intended for use in masonry with bed joints with a nominal thickness of 1mm to 3mm.

(4) Lightweight mortars should be made using perlite, pumice, expanded clay, expanded shale or expanded glass as the aggregate. Other materials may be used provided tests are carried out to confirm their suitability.

(5) Mortars should be classified either according to their designed compressive strength, expresssed as the letter M followed by the compressive strength in N/mm^2, for example, M5, or according to their prescription, for example, 1:1:5 cement:lime:sand, by volume.

3.2.2 Properties of mortar

3.2.2.1 Compressive strength of mortar

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

(2) General purpose mortars may be specified by strength (designed mixes) or by prescription (prescribed mixes) as follows:

- Designed mixes, which should be designed and manufactured to achieve the specified compressive strength, f_m , when determined in accordance with EN 1015-11.

- Prescribed mixes, which should be manufactured from specified proportions of constituents, including admixtures and additions, for the relevant value of f_m required for design purposes, and which may be assumed to achieve this strength.

(3) General purpose mortar should not be less than M1 in joints without reinforcement and not less than M5 in both joints containing reinforcement and prestressed masonry. Joints containing prefabricated bed joint reinforcement should be made of general purpose mortar M2,5 or stronger.

Note: For durability requirements where reinforcement or prestressing steels are used, see 5.2.2.

(4) Thin layer mortars should be designed mixes in accordance with EN 998-2 and should be M5 or stronger.

(5) For the purpose of using equation (3.3) in 3.6.2.4, lightweight mortars should be designed mixes in accordance with EN 998-2 and should be M5 or stronger.

3.2.2.2 Durability of mortar

(1)P Mortar in masonry shall be sufficiently durable to resist local exposure conditions for the intended life of the building.

Note: Guidance on design and construction to achieve adequate durability of mortar joints is given in Sections 5 and 6 of this ENV 1996-1-2 and ENV 1996-2.

3.2.2.3 Adhesion between units and mortar

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

(2) Adequate adhesion will normally be obtained with mortars in accordance with EN 998-2 and with site mixed designed or site mixed prescribed general purpose mortars made in accordance with 6.3.2 and not containing admixtures nor additions. Where test data is not available, either for a specific project or on a National basis[6)] and when the values of f_{vko} given in table 3.5 are to be used, shear tests should be carried out using the methods in EN 1052-3, to check that the shear strength f_{vko} is not less than that for general purpose mortar.

Note:See the note to 3.6.3 (3)

3.3 Concrete infill

3.3.1 General

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

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6) It is implicit that the results of this evaluation will be given in National Application Documents.

3.3.2 Specification for concrete infill

(1) Concrete infill should have a characteristic compressive cylinder/cube strength class not less than 12/15N/mm².

(2) For concrete infill to be used in voids whose least dimension is not less than 50mm or when the cover to the reinforcement is between 15mm and 25mm, the maximum aggregate size should not exceed 10mm. For concrete infill to be used in voids whose least dimension is not less than 100mm or when the cover to the reinforcement is not less than 25mm, a maximum aggregate size not exceeding 20mm may be used.

(3) The mixes may be designed or prescribed and should contain just sufficient water to provide the specified strength and to give the workability necessary to satisfy paragraph (4)P below.

(4)P The workability of concrete infill shall be such as to ensure that voids will be completely filled.

Note: This will normally mean using a slump class S3, in accordance with EN 206.

(5) When a pourable concrete infill is used for filling voids, inclusion of expanding agents should be considered to reduce the risk of cracking in the concrete infill due to shrinkage resulting from water loss into the masonry.

3.3.3 Properties of concrete infill

(1) For the purposes of specification, the characteristic compressive strength of the concrete infill, f_{ck} , is classified by the concrete strength class which relates to the cylinder/cube strength at 28 days, in accordance with EN 206. The strength classes normally used for concrete infill in reinforced masonry are given in table 3.3, together with the value of f_{ck} to be used in design.

(2) The characteristic shear strength of concrete infill, f_{cvk} , that may be assumed in design is given in table 3.4 for the relevant concrete strength classes.

Table 3.3 : Characteristic compressive strength, f_{ck} , of concrete infill.

Strength class of concrete	C12/15	C16/20	C20/25	C25/30 or stronger
f_{ck} (N/mm²)	12	16	20	25

Table 3.4 : Characteristic shear strength, f_{cvk} , of concrete infill.

Strength class of concrete	C12/15	C16/20	C20/25	C25/30 or stronger
f_{cvk} (N/mm²)	0,27	0,33	0,39	0,45

3.4 Reinforcing steel

3.4.1 General

(1)P Reinforcing steel shall be in accordance with EN 10080 and stainless steel shall be in accordance with EN 10088.

(2)P Bed joint reinforcement shall be in accordance with EN 845-3.

(3) Reinforcing steel may be assumed to possess adequate elongation ductility for design purposes, if the following requirements are satisfied:

- High ductility : $\varepsilon_{uk} > 5\%$; $(f_t/f_y)_k > 1{,}08$

- Normal ductility : $\varepsilon_{uk} > 2{,}5\%$; $(f_t/f_y)_k > 1{,}05$

where:

ε_{uk} is the characteristic value of the unit elongation at maximum tensile stress;

f_t is the tensile strength of the reinforcing steel;

f_y is the yield strength of the reinforcing steel;

$(f_t/f_y)_k$ is the characteristic value of f_t/f_y.

(4) High bond reinforcing steel with diameters less than 6mm, including those in wire meshwork or trusses used for bed joint reinforcement, should not be treated as having high ductility.

(5) Reinforcing steel may be carbon steel or austenitic stainless steel. Reinforcing steel may be plain or high bond.

3.4.2 Properties of reinforcing steel

(1)P The characteristic strength of reinforcing steel, f_{yk} , shall be in accordance with EN 10080.

(2) The mean value of the modulus of elasticity of reinforcing steel may be assumed to be 200kN/mm².

3.4.3 Durability of reinforcing steel

(1)P Reinforcing steel shall be sufficiently durable, when placed in accordance with the application rules in Section 5 of this ENV 1996-1-1, to resist local exposure conditions for the intended life of the building.

(2) Austenitic stainless steel may be assumed to have adequate durability.

(3) Where carbon steel requires protection to provide adequate durability, it should be galvanised in accordance with EN....[7], such that the zinc coating is not less than that required to provide the necessary durability (see 5.2.2) or the steel should be given an equivalent protection such as by fusion bonded epoxy powder.

3.5 Prestressing steel

3.5.1 General

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

(2)P The properties of prestressing steel shall be in accordance with clause 3.3 of ENV 1992-1-1 and EN 10138.

3.5.2 Durability of prestressing steel

(1)P Prestressing steel shall be sufficiently durable, when placed in accordance with the application rules in Section 5 of this ENV 1996-1-1, to resist local 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.

3.6 Mechanical properties of unreinforced masonry

3.6.1 General

(1) The distinction is made between :

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[7] Ideally a CEN Standard but one is not yet available; National Application Documents should state the appropriate Standard to be used.

- the masonry itself, considered as an assemblage of masonry units and mortar, which has intrinsic mechanical properties;

- the structural masonry element (for example, a wall), the mechanical properties of which depends on the intrinsic mechanical properties of the masonry, the geometry of the element and the interaction of adjacent parts.

(2) The intrinsic mechanical properties of the masonry obtained from standard test methods and used in design are:

- the compressive strength, f ;
- the shear strength, f_v ;
- the flexural strength, f_x ;
- the stress-strain relationship, (σ - ε).

(3) Although direct tensile strength can be developed in masonry, it is not a property normally used in design.

3.6.2 Characteristic compressive strength of unreinforced masonry

3.6.2.1 General

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

Note: Test results may be available nationally or from tests carried out for the project.

(2) The characteristic compressive strength of unreinforced masonry may be determined by tests in accordance with EN 1052-1, or it may be established from an evaluation of test data based on the relationship between the characteristic compressive strength of unreinforced masonry and the compressive strengths of the masonry units and the mortar in a similar way to 3.6.2.2-3.6.2.6.

(3) Where test data is not available either for a specific project or on a National basis[8)], it may be assumed that the relationship between the characteristic compressive strength of unreinforced masonry, f_k , and the unit strength and the mortar strength is as given in 3.6.2.2-3.6.2.6, and that f_k will not fall below the value obtained from those relationships, when:

- the masonry is built in accordance with Section 5 of this ENV 1996-1-1, and

[8)] It is implicit that the results of this evaluation will be given in National Application Documents.

- the coefficient of variation of the masonry units is not more than 25%, when the masonry units are sampled in accordance with the relevant part of EN 771 and tested for compressive strength in accordance with EN 772-1.

(4) The characteristic compressive strength given in 3.6.2.2-3.6.2.6 is that strength resulting from action effects normal to the bed joints. Where action effects are parallel to the direction of the bed joints, the characteristic compressive strength may also be determined from 3.6.2.2-3.6.2.6, but 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 as the direction of the action effect in the masonry, but with the factor, δ , as given in table 3.2, not taken to be greater than 1,0. In the case of Group 1 units, the formulae may be used without adjustment. However, where Group 2a or 2b units are used, the values of K should be multiplied by 0,5.

3.6.2.2 Characteristic compressive strength of unreinforced masonry made using general purpose mortar

(1) The characteristic compressive strength of unreinforced masonry made with general purpose mortar, complying with 3.2.1 with all joints satisfying the requirements of 5.1.5 so as to be considered as filled (but see 3.6.2.5), may be calculated using equation (3.1):

$$f_k = K f_b^{0,65} f_m^{0,25} \text{ N/mm}^2 \tag{3.1}$$

provided that f_m is not taken to be greater than 20N/mm² nor greater than $2f_b$, whichever is the smaller;

where:

K is a constant in $(\text{N/mm}^2)^{0,10}$ that may be taken as:

$\boxed{0,60}$ for Group 1 masonry units when 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 (see figures 5.1 (a) and 5.2);

$\boxed{0,55}$ for Group 2a masonry units when the thickness of the masonry is equal to the width or length of the units so that there is no longitudinal mortar joint through all or part of the length of the wall;

$\boxed{0,50}$ for Group 2b masonry units when the thickness of the masonry is equal to the width or length of the units so that there is no longitudinal mortar joint through all or part of the length of the wall;

$\boxed{0,50}$ when, for Group 1 masonry units, there is a longitudinal mortar joint through all or part of the length of the masonry (see figures 5.1(b), 5.3 and 5.4);

$\boxed{0,45}$ when, for Group 2a masonry units, there is a longitudinal mortar joint through all or part of the length of the masonry (see figures 5.1(b), 5.3 and 5.4);

$\boxed{0,40}$ when, for Group 2b masonry units, there is a longitudinal mortar joint through all or part of the length of the masonry (see figures 5.1(b), 5.3 and 5.4);

$\boxed{0,40}$ for Group 3 masonry units;

f_b is the normalized compressive strength of the masonry units in N/mm², as described in 3.1.2.1, in the direction of the applied action effect;

f_m is the specified compressive strength of the general purpose mortar in N/mm².

(2) When Group 2 aggregate concrete units are used with the vertical cavities filled completely with insitu concrete, the value of f_b should be obtained by considering the units to be Group 1 having a compressive strength based on their net area (the net compressive strength), provided that the characteristic compressive strength of the concrete infill is not less than the net compressive strength of the units. Where the compressive strength of the infill concrete is less than the net compressive strength of the units, the value of f_b should be obtained as if the units were solid and of compressive strength equal to the characteristic strength of the concrete infill.

3.6.2.3 Characteristic compressive strength of unreinforced masonry made using thin layer mortar

(1) The characteristic compressive strength of unreinforced masonry, f_k, made with thin layer mortar, complying with 3.2.1 with all joints satisfying the requirements of 5.1.5 so as to be considered as filled (but see 3.6.2.5), and using Group 1 calcium silicate units and autoclaved aerated concrete units may be calculated using equation (3.2):

$$f_k = 0,8\ f_b^{\ 0,85} \qquad (3.2)$$

provided that:

- the masonry units have dimensional tolerances such that they are suitable for use with thin layer mortars;

- the normalized compressive strength of the masonry units, f_b, is not taken to be greater than 50N/mm²;

- the thin layer mortar has a compressive strength of 5N/mm² or more;

- 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.

(2) The characteristic compressive strength of unreinforced masonry, f_k, made with thin layer mortar and using masonry units other than Group 1 calcium silicate units and autoclaved aerated concrete units may be calculated using equation (3.1):

where:

K is a constant in $(N/mm^2)^{0.10}$ that may be taken as:

$\boxed{0{,}70}$ for Group 1 masonry units;

$\boxed{0{,}60}$ for Group 2a masonry units;

$\boxed{0{,}50}$ for Group 2b masonry units;

provided that, in addition, the requirements in pragraph (1) above are met.

3.6.2.4 Characteristic compressive strength of unreinforced masonry made using lightweight mortar

(1) The characteristic compressive strength of unreinforced masonry, f_k, made with Group 1, 2a and 2b masonry units and lightweight mortar, complying with 3.2.1 with all joints satisfying the requirements of 5.1.5 so as to be considered as filled (but see 3.6.2.5), may be calculated using equation (3.3):

$$f_k = K\ f_b^{\ 0,65}\ \text{N/mm}^2 \qquad (3.3)$$

provided that f_b is not taken to be greater than 15N/mm² and 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;

where:

K is a constant in $(N/mm^2)^{0,35}$ that may be taken as:

$\boxed{0{,}80}$ when lightweight mortar with a density of 600-1 500 kg/m³ is used in masonry with lightweight aggregate concrete units in accordance with EN 771-3 and autoclaved aerated concrete units in accordance with EN 771-4;

$\boxed{0{,}70}$ when lightweight mortar with a density of more than 700 kg/m³ and less than 1 500 kg/m³ is used in masonry with clay units in accordance with EN 771-1, calcium silicate units in accordance with EN 771-2 and dense aggregate concrete units in accordance with EN 771-3;

$\boxed{0,55}$ when lightweight mortar with a density of 600-700 kg/m³ is used in masonry with clay units in accordance with EN 771-1, calcium silicate units in accordance with EN 771-2 and dense aggregate concrete units in accordance with EN 771-3;

f_b is the normalized compressive strength of the masonry units in N/mm², as described in 3.6.2.2.

Note: The effects of the mortar strength on the characteristic compressive strength of the masonry are taken into account in the value of K.

3.6.2.5 Characteristic compressive strength of unreinforced masonry with unfilled vertical joints

(1) The characteristic compressive strength of unreinforced masonry made with masonry units in which the perpend joints are unfilled may be taken as that obtained from 3.6.2.2 - 3.6.2.4 for use in 4.4.2, equation (4.5), and 4.4.8, equations (4.18), (4.19), (4.20) and (4.21), provided that the shear resistance is based upon the requirements of 3.6.3(7) and due consideration is given to any horizontal actions that might be applied to, or transmitted by, the masonry.

3.6.2.6 Characteristic compressive strength of shell bedded unreinforced masonry

(1) The characteristic compressive strength of shell bedded masonry (see figure 5.5), made with Group 1 masonry units and bedded on two equal strips of general purpose mortar at the outside edges of the bed face of the units, may be taken to be the value obtained from equation (3.1), with the attendant limitations given with the formula, provided that:

- the width of each strip of mortar, is 30mm 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 b/t does not exceed 0,8;

- K is taken as $\boxed{0,60}$ when $b/t \leq 0,5$ or $\boxed{0,30}$ when $b/t = 0,8$, with intermediate values obtained by linear interpolation;

where:

b_s is the distance between the centre lines of the mortar strips;

t is the thickness of the wall.

(2) The characteristic compressive strength of shell bedded masonry, made with Group 2a or Group 2b masonry units and bedded as noted for Group 1 masonry units, may be taken to be the value obtained from equation (3.1) provided that the normalized compressive strength of the unit, f_b, used in the equation is that obtained from tests, in accordance with EN 772-1,

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.

3.6.3 Characteristic shear strength of unreinforced masonry

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

Note: Test results may be available nationally or from tests carried out for the project.

(2) The characteristic shear strength of unreinforced masonry may be determined by tests in accordance with EN ...[9] or it may be established from an evaluation of test data based on the relationship between the characteristic shear strength of unreinforced masonry, the initial shear strength of the masonry, f_{vko} , which may be determined from EN 1052-3 and EN 1052-4 or obtained from table 3.5 for general purpose mortar, and the applied compressive stress.

(3) Where test data is not available either for a specific project or on a National basis[10], it can be assumed that the characteristic shear strength of unreinforced masonry, f_{vk} , using general purpose mortar in accordance with 3.2.1, with all joints satisfying the requirements of 5.1.5 so as to be considered as filled, will not fall below the least of the values described below:

$$f_{vk} = f_{vko} + 0,4\ \sigma_d \tag{3.4}$$

or $= \boxed{0,065}\ f_b$ but not less than f_{vko}

or = the limiting value given in table 3.5;

where:

f_{vko} is the shear strength, under zero compressive stress, determined in accordance with EN 1052-3 or EN 1052-4 or, for general purpose mortars not containing admixtures or additions, obtained from table 3.5;

Note: When National test data [11] is not available, or tests in accordance with EN 1052-3 have not been carried out (see 3.2.2.3(2)), the value of f_{vko} should be taken as 0,1N/mm².

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination from 2.3.2.2;

f_b is the normalized compressive strength of the masonry units, as described in 3.1.2.1 for

[9] CEN standard is not yet available.
[10] It is implicit that the results of this evaluation will be given in National Application Documents.
[11] It is implicit that the results of this evaluation will be given in National Application Documents.

the direction of application of the load on the test specimens being perpendicular to the bed face.

(4) Where test data is not available either for a specific project or on a National basis[12], it can be assumed that the characteristic shear strength for unreinforced masonry using general purpose mortar in accordance with 3.2.1 and having the perpend joints unfilled, but with adjacent faces of the masonry units closely abutted together, will not fall below the least of the values described below:

$$f_{vk} = 0{,}5\, f_{vko} + 0{,}4\ \sigma_d \tag{3.5}$$

or $= \boxed{0{,}045}\ f_b$ but not less than f_{vko}

or $=$ 0,7 times the limiting value given in table 3.5;

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

(5) In shell bedded masonry, made with Group 1 masonry units and bedded on two equal strips of general purpose mortar, each at least 30mm in width, at the outside edges of the bed face of the unit, it can be assumed that the characteristic shear strength will not fall below the least of the values described below:

$$f_{vk} = \frac{g}{t}\ f_{vko} + 0{,}4\ \sigma_d \tag{3.6}$$

or $= \boxed{0{,}05}\ f_b$ but not less than f_{vko}

or $=$ 0,7 times the limiting value given in table 3.5;

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

g is the total width of the two mortar strips;

t is the thickness of the wall.

(6) For thin layer mortars, used with autoclaved aerated concrete units, calcium silicate or concrete units, the value of f_{vk} obtained from equations (3.4), (3.5) and (3.6), and the limits applicable to those equations, may be assumed, using the values given in table 3.5 for clay units of the same Group and M10 to M20 mortar.

[12] It is implicit that the results of this evaluation will be given in National Application Documents.

Table 3.5 : Values of f_{vko} and limiting values of f_{vk} for general purpose mortar.

Masonry Unit	Mortar	f_{vko} (N/mm²)	Limiting f_{vk} (N/mm²)	
Group 1 clay units	M10 to M20	0,3	1,7	
	M2,5 to M9	0,2	1,5	
	M1 to M2	0,1	1,2	
Group 1 units other than clay and natural stone	M10 to M20	0,2	1,7	
	M2,5 to M9	0,15	1,5	
	M1 to M2	0,1	1,2	
Group 1 natural stone units	M2,5 to M9	0,15	1,0	
	M1 to M2	0,1	1,0	
Group 2a clay units	M10 to M20	0,3	The lesser of longitudinal compressive strength (see note below) or	1,4
	M2,5 to M9	0,2		1,2
	M1 to M2	0,1		1,0
Group 2a and Group 2b units other than clay and Group 2b clay units	M10 to M20	0,2		1,4
	M2,5 to M9	0,15		1,2
	M1 to M2	0,1		1,0
Group 3 clay units	M10 to M20	0,3	No limits other than given by equation (3.4)	
	M2,5 to M9	0,2		
	M1 to M2	0,1		

Note: For Group 2a and 2b masonry units, the longitudinal compressive strength of the units is taken to be the measured strength, with δ taken to be not greater than 1,0. When the longitudinal compressive strength can be expected to be greater than 0,15 f_b , by consideration of the pattern of holes, tests are not necessary.

(7) For lightweight mortars, the value of f_{vk} obtained from equations (3.4) and (3.5) and the limits applicable to those equations, may be assumed, using the values given in table 3.5 for M5 mortar. Alternatively, tests should be carried out to check that the adhesion between the lightweight mortar and the masonry units is not less than would be obtained with the equivalent strength general purpose mortar.

(8) Where the masonry may be subjected to seismic actions it may be assumed that the residual shear strength is the value of f_{vk} given by equations (3.4), (3.5) and (3.6), and the limits applicable to those equations, multiplied by 0,7.

(9) The initial shear strength of masonry, f_{vko} , containing sheet damp proof courses should be determined in accordance with EN 1052-4.

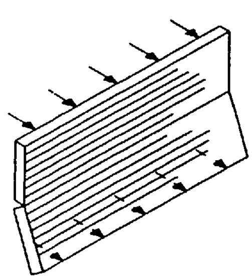
3.6.4 Characteristic flexural strength of unreinforced masonry

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

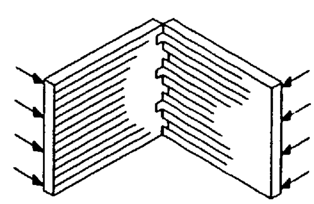
Note: Tests results may be available nationally or from tests carried out for the project.

(2) The characteristic flexural strength of unreinforced masonry may be determined by tests in accordance with EN 1052-2, or it may be established from an evaluation of test data[13)] based on the flexural strengths of masonry obtained from appropriate combinations of units and mortar. The characteristic flexural strength should be determined on specimens that give the plane of failure parallel to the bed joints, f_{xk1} , and others that give the plane of failure perpendicular to the bed joints, f_{xk2} , as illustrated in figure 3.1.

(3) The flexural strength of masonry f_{xk1} should only be used in the design of walls carrying transient loads (for example, wind) normal to their surface. The value of f_{xk1} should be taken as zero where failure of the wall would lead to a major collapse or total loss of stability in the structure as a whole, or in the design for earthquake resistance.



f_{xk1} : Plane of failure parallel to bed joints.



f_{xk2} : Plane of failure perpendicular to bed joints.

Figure 3.1 : Flexural strengths f_{xk1} and f_{xk2}.

13) It is implicit that the results of this evaluation will be given in National Application Documents.

(4) The flexural strength of masonry should be classified according to the masonry units and mortar category being used.

Note: The flexural strength may be expressed as the letter F followed by the flexural strengths in N/mm² i.e. F f_{xk1}/f_{xk2} (for example, F 0,35/1,00).

3.7 Mechanical properties of reinforced, prestressed and confined masonry

3.7.1 General

(1)P The strength of reinforced, prestressed and confined masonry is determined from the mechanical properties of the assemblage of masonry units, including the mortar or concrete infill, acting in conjunction with the reinforcement, where appropriate.

(2) The intrinsic mechanical properties of reinforced, prestressed and confined masonry obtained from standard test methods and used in design are as for unreinforced masonry (see 3.6.1(2)) and additionally, where appropriate:

- the compressive strength of the concrete infill, f_c ;
- the shear strength of the concrete infill, f_{cv} ;
- the tensile and compressive yield strength of the reinforcing steel, f_y ;
- the tensile strength of the prestressing steel, f_p ;
- the anchorage bond strength of the reinforcing steel, f_{bo} .

3.7.2 Characteristic anchorage bond strength

(1) For reinforcement embedded in concrete sections with dimensions greater than or equal to 150mm, or where the concrete infill surrounding the reinforcement is confined within masonry units, the characteristic anchorage bond strength, f_{bok} , that can be assumed in design for reinforcement considered to be confined, is given in table 3.6.

(2) For reinforcement embedded in mortar, or in concrete sections with dimensions less than 150mm, or where the concrete infill surrounding the reinforcement is not confined within masonry units, the characteristic anchorage bond strength, f_{bok} , that can be assumed in design for reinforcement considered to be not confined, is given in table 3.7.

(3) For special reinforcement, such as prefabricated bed joint reinforcement, the characteristic anchorage bond 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 3.6 : Characteristic anchorage bond 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/mm²)	$\boxed{1,3}$	$\boxed{1,5}$	$\boxed{1,6}$	$\boxed{1,8}$
f_{bok} for high-bond carbon steel and stainless steel bars (N/mm²)	$\boxed{2,4}$	$\boxed{3,0}$	$\boxed{3,4}$	$\boxed{4,1}$

Table 3.7 : Characteristic anchorage bond strength of reinforcement in mortar or concrete not confined within masonry units.

Classification					
Classification	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/mm²)		$\boxed{0,7}$	$\boxed{1,2}$	$\boxed{1,4}$	$\boxed{1,5}$
f_{bok} for high-bond carbon seel and stainless steel bars (N/mm²)		$\boxed{1,0}$	$\boxed{1,5}$	$\boxed{2,0}$	$\boxed{2,5}$

3.8 Deformation properties of masonry

3.8.1 Stress-strain relationship

(1) The general shape of a stress-strain relationship of masonry is of the form given in figure 3.2.

(2) The stress-strain relationship of masonry may be taken as parabolic, parabolic rectangular (see figure 3.3) or as rectangular, for the purposes of design.

Note: Figure 3.3 is an approximation and may not be suitable for all types of masonry units. For example, units with large holes (Group 2b and Group 3 units) may suffer brittle failure and be without the horizontal ductile range.

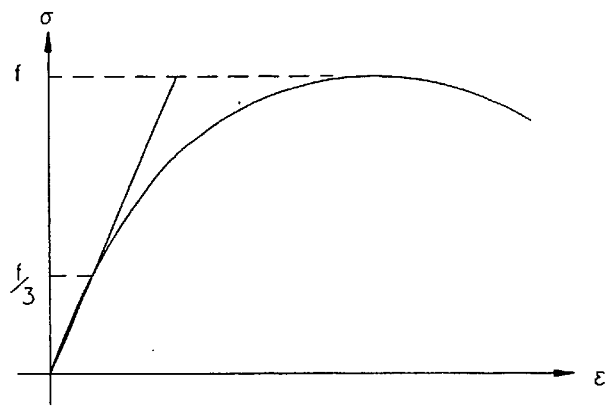


Figure 3.2 : General shape of a stress-strain relationship for masonry.

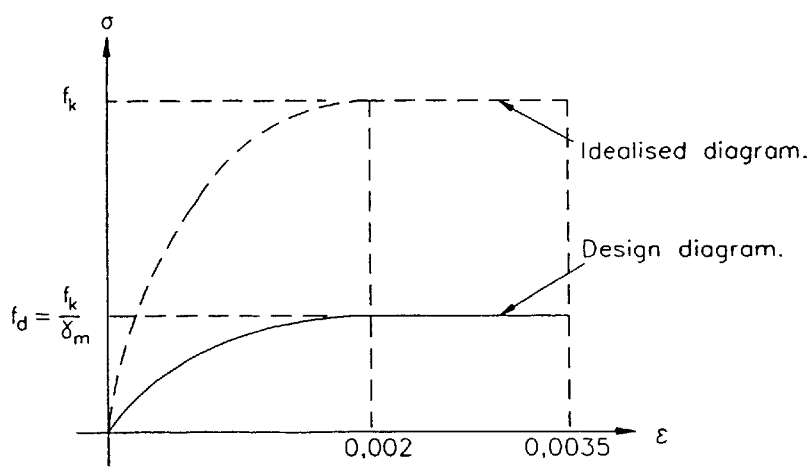


Figure 3.3 : Stress-strain relationship for the design of masonry in bending and compression.

3.8.2 Modulus of elasticity

(1)P The short term secant modulus of elasticity, E , shall be determined by tests in accordance with EN 1052-1 at service load conditions, i.e. at one third of the maximum load determined in accordance with EN 1052-1.

(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 , under service conditions and for use in the structural analysis, may be taken to be $\boxed{1\ 000}$ f_k.

(3) When the modulus of elasticity is used in calculations relating to the serviceability limit state, it is recommended that a factor of 0,6 be applied to the value of E.

(4) The long term modulus may be based on the short term secant value (see paragraph (2) above), reduced to allow for creep effects, (see 3.8.4).

3.8.3 Shear modulus

(1) In the absence of a more precise value, it may be assumed that the shear modulus, G, is 40% of the elastic modulus, E.

3.8.4 Creep, shrinkage and thermal expansion

(1) A range of values for the deformation properties of masonry made from units of various materials laid in general purpose mortar is given in table 3.8. Preferably, the deformation properties should be determined by test but, in the absence of such values,the design values given in table 3.8 may be used.

Note: Some values will exceed and some will fall below the design values given; a likely range is given in table 3.8.

(2) In the absence of test data, the deformation properties of masonry laid in thin layer and lightweight mortar may be taken as the values given in table 3.8 for the appropriate type of unit.

3.9 Ancillary components

3.9.1 Damp proof courses

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

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

3.9.2 Wall ties

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

(2)P Materials used for wall ties shall be able to resist the bending and straightening to which they will be exposed, without unintentional reduction of strength, ductility and corrosion protection.

Table 3.8 : Deformation properties of unreinforced masonry made with general purpose mortar.

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 10^{-6}/K	
	Range	Design value	Range	Design value	Range	Design value
Clay	0,5 to 1,5	$\boxed{1,0}$	-0,2 to +1,0	(see note 3)	4 to 8	$\boxed{6}$
Calcium Silicate	1,0 to 2,0	$\boxed{1,5}$	-0,4 to -0,1	$\boxed{-0,2}$	7 to 11	$\boxed{9}$
Dense aggregate concrete and manufactured stone	1,0 to 2,0	$\boxed{1,5}$	-0,6 to -0,1	$\boxed{-0,2}$	6 to 12	$\boxed{10}$
Lightweight aggregate concrete	1,0 to 3,0	$\boxed{2,0}$	-1,0 to -0,2	$\boxed{-0,4}$ (see note 4) $\boxed{-0,2}$ (see note 5)	8 to 12	$\boxed{10}$
Autoclaved aerated concrete	1,0 to 2,5	$\boxed{1,5}$	-0,4 to +0,2	$\boxed{-0,2}$	7 to 9	$\boxed{8}$
Natural stone	(see note 6)	$\boxed{0}$	-0,4 to +0,7	$\boxed{+0,1}$	3 to12	$\boxed{7}$

Notes:

1. The final creep coefficient $\phi_\infty = \varepsilon_{c\infty} / \varepsilon_{el}$, where $\varepsilon_{c\infty}$ is the final creep strain and $\varepsilon_{el} = \sigma / E$.
2. Where the final value of moisture expansion or shrinkage is shown minus it indicates shortening and where plus it indicates extension.
3. Values depend upon the type of material concerned and a single design value cannot be given.
4. Value given is for pumice and expanded clay aggregates.
5. Value given is for lightweight aggregates other than pumice or expanded clay.
6. Values are normally very low.

(3) Wall ties should be in accordance with the requirements of EN 845-1 and, where manufactured from steel, be in accordance with the requirements of 5.2.2 for durability for the appropriate exposure class.

3.9.3 Straps, hangers, brackets and support angles

(1)P Straps, hangers, brackets and support angles shall be in accordance with the requirements of EN 845-1. They shall be corrosion resistant in the environmental condition in which they are used.

3.9.4 Prefabricated lintels

(1)P Prefabricated lintels shall be in accordance with the requirements of EN 845-2. They shall be corrosion resistant in the environmental condition in which they are used.

3.9.5 Prestressing devices

(1)P Anchorages, couplers, ducts and sheaths shall be in accordance with the requirements of clause 3.4 of ENV 1992-1-1. They shall be corrosion resistant in the environmental condition in which they are used.

4 Design of masonry

4.1 Structural behaviour and overall stability

4.1.1 Design models for structural behaviour

(1)P For each relevant limit state verification, a design model shall be set up from:

- an appropriate description of the structure, the constitutive materials from which it is made, and the relevant environment;

- the behaviour of the whole or parts of the structure, related to the relevant limit state;

- the actions and how they are imposed.

(2) Design may be carried out for sections and parts of the structure (such as walls) independently, if the three dimensional layout and structure interaction are considered.

(3)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.

(4) To ensure stability and robustness, it is necessary for the layout of the structure on plan and section, the interaction of the masonry parts and their interaction with other parts of the structure to be such as to produce a properly braced arrangement when detailed and built in accordance with Sections 5 and 6 of this ENV 1996-1-1. The possible effects of imperfections should be allowed for by assuming that the structure is inclined at an angle $\nu = 1/(100\sqrt{h_{tot}})$ radians to the vertical where h_{tot} is the total height of the structure in metres.

(5) Structures incorporating masonry walls designed according to Section 4 of ENV 1996-1-1 should have their parts suitably braced together so that sway of the structure will not occur.

(6)P The designer responsible for the overall stability of the structure shall ensure the compatibility of the design with details of the parts and components.

(7) There should be no doubt of this responsibility for overall stability when some or all of the design and detailing is carried out by more than one designer.

4.1.2 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 collapse catastrophically under the effect of misuse or accident, and that it will not be damaged 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 case.

(2) Either the structure as a whole should be considered with the hypothetical removal of essential loadbearing members in turn or, alternatively, members should be designed to resist the effects of accidental actions. In the former case, assessment of the stability of the residual structure should include consideration of the integrity of ties and restraints to members. In the latter case, it is necessary to consider the effect of the accidental actions on the ties and restraint of members designed to resist the effects of accidental actions. The possibility of reducing the risk of accidental actions, such as from vehicle impact, should be considered.

4.1.3 Design of structural members

(1)P The design of members shall be verified in the ultimate limit state.

(2)P The structure shall be designed so that cracks or deflections which might damage facing materials, partitions, finishings or technical equipment, or which might impair water-tightness, are avoided or minimised.

(3) The design of members need not be verified at the serviceability limit state, in cases where the serviceability limit state is deemed to be satisfied when the ultimate limit state is successfully verified.

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

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

4.2 Actions, combinations and partial coefficients

4.2.1 Characteristic permanent action

(1)P The characteristic permanent action G_k shall be obtained as described in 2.2.2.2.

4.2.2 Characteristic variable action

(1)P The characteristic variable action Q_k shall be obtained as described in 2.2.2.2. The representative value of a variable action, for use in design combinations, shall be obtained as described in 2.2.2.3.

4.2.3 Characteristic wind action

(1)P The characteristic wind action W_k shall be taken as the wind load calculated in accordance with ENV 1991.

4.2.4 Characteristic lateral earth pressure

(1)P The characteristic lateral earth pressure shall be calculated in accordance with ENV 1991 and ENV 1997.

4.2.5 Design combinations

(1)P When using the design relationship for the ultimate limit state, the combination of design values to be taken into account shall be as given in 2.3.2.2 with the appropriate partial coefficients given in 2.3.3.1. Where alternative values of a partial safety factor are given, that producing the most severe conditions shall be used.

4.3 Design strength of masonry

(1)P The design strength of masonry shall be taken as the characteristic strength divided by the appropriate partial safety factor γ_M.

The design strength of masonry is given by :

- in compression $$f_d = \frac{f_k}{\gamma_M} \tag{4.1}$$

- in shear $$f_{vd} = \frac{f_{vk}}{\gamma_M} \tag{4.2}$$

- in flexure $$f_{xd} = \frac{f_{xk}}{\gamma_M} \tag{4.3}$$

where γ_M is the appropriate value given in 2.3.3.2.

4.4 Unreinforced masonry walls subjected to vertical loading

4.4.1 General

(1)P The resistance of unreinforced 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) It may be assumed that:

- plane sections remain plane;

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

- the stress/strain relationship is of the form indicated in figure 3.2.

(3) Allowance in the design should be made for the following:

- long-term effects of loading;

- 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.

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

$$N_{Sd} \leq N_{Rd} \tag{4.4}$$

(5) An acceptable design method for the verification of the ultimate limit state is given in 4.4.2 - 4.4.8.

Note: In 4.4.2 - 4.4.8 some simplifying assumptions are made with regard to the calculation of the slenderness ratio of walls and the reduction factor Φ, particularly that second order effects can be taken into account in a simplified form.

4.4.2 Verification of unreinforced masonry walls

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

$$N_{Rd} = \frac{\Phi_{i,m}\ t\ f_k}{\gamma_M} \tag{4.5}$$

where:

$\Phi_{i,m}$ is the capacity reduction factor Φ_i or Φ_m, as appropriate, allowing for the effects of slenderness and eccentricity of loading, obtained from 4.4.3;

f_k is the characteristic compressive strength of the masonry, obtained from 3.6.2;

γ_M is the partial safety factor for the material, obtained from 2.3.3.2;

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

(2) The design strength of a wall may be at its lowest in the middle one fifth of the height, when Φ_m should be used, or at the top of the wall or bottom of the wall, when Φ_i should be used.

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

$$(0{,}7 + 3\ A) \tag{4.6}$$

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

(4) For cavity walls, the load carried by each leaf should be assessed and the design vertical load resistance of each leaf, N_{Rd}, should be verified using equation (4.5). When only one leaf of a cavity wall is loaded, the loadbearing capacity of the wall should be based on the horizontal cross-sectional area of that leaf alone but using the effective thickness calculated using equation (4.17) for the purposes of determining the slenderness ratio (see 4.4.5(3)).

(5) A faced wall, so bonded together as to result in common action under load, should be designed in the same manner as a single-leaf wall constructed entirely of the weaker units, using the value of K appropriate to a wall with a longitudinal mortar joint (see 3.6.2.2). A faced wall, which is not so bonded together as to result in common action under load, should be designed as a cavity wall, provided that it is tied together as required for such walls (see 5.4.2.2).

(6) A double-leaf wall may be designed as a cavity wall or, alternatively, as a single-leaf wall if the two leaves are tied together so as to result in common action under load (see 5.4.2.3).

(7) Chases and recesses reduce the loadbearing capacity of a wall. The reduction may be assumed to be insignificant if the chases or recesses are within the limits given in 5.5. If the size, number or location of the chases or recesses are outside these limits the vertical loadbearing capacity of the wall should be checked as follows :

- vertical chases or recesses should be treated either as openings passing through the wall or, alternatively, the residual thickness of the wall at the chase or recess should be used in the calculations for the whole wall;

- horizontal or inclined chases should be treated either as openings passing through the wall or, alternatively, the strength of the wall should be checked at the chase position, taking account of the load eccentricity relative to the residual wall thickness.

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%.

(8) Walls that satisfy the ultimate limit state when verified in accordance with equation (4.5) may be deemed to satisfy the serviceability limit state.

4.4.3 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.

$$\Phi_i = 1 - 2\frac{e_i}{t} \tag{4.7}$$

where:

e_i is the eccentricity at the top or the bottom of the wall, as appropriate, calculated using the equation (4.8):

$$e_i = \frac{M_i}{N_i} + e_{hi} + e_a \geq 0{,}05t \tag{4.8}$$

M_i is the design bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support, according to 4.4.7 (see figure 4.1);

N_i is the design 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 4.4.7.2);

t is the thickness of the wall.

(ii) In the middle one fifth of the wall height.

By using a simplification of the general principles given in 4.4.1, the reduction factor within the middle height of the wall, Φ_m , may be determined from figure 4.2:

Note: The formulae on which figure 4.2 is based are given in Annex A. E has been taken as equal to 1 000f_k.

where:

e_{mk} is the eccentricity within the middle one fifth of the wall height, calculated using equations (4.9) and (4.10):

$$e_{mk} = e_m + e_k \geq 0{,}05t \tag{4.9}$$

$$e_m = \frac{M_m}{N_m} + e_{hm} \pm e_a \qquad (4.10)$$

e_m is the eccentricity due to loads;

M_m is the greatest moment within the middle one fifth of the height of the wall resulting from the moments at the top and bottom of the wall (see figure 4.1);

N_m is the design vertical load within the middle one fifth of the height of the wall;

e_{hm} is the eccentricity at mid-height resulting from horizontal loads (for example, wind);

h_{ef} is the effective height, obtained from 4.4.4 for the appropriate restraint or stiffening condition;

t_{ef} is the effective thickness of the wall, obtained from 4.4.5;

e_k is the eccentricity due to creep, calculated from the equation (4.11):

$$e_k = 0{,}002\ \phi_\infty \frac{h_{ef}}{t_{ef}} \sqrt{t\ e_m} \qquad (4.11)$$

ϕ_∞ is the final creep coefficient taken from table 3.8.

(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.

4.4.4 Effective height of walls

4.4.4.1 General

(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) In the assessment of effective height a distinction may be made between walls restrained or stiffened on two, three or four edges and free-standing walls. A floor supported by a wall, suitably placed cross walls, or any other similarly rigid structural elements to which the wall is connected, may be regarded as providing lateral restraint to a wall, irrespective of any contribution such elements may be considered to make to the overall stability of the structure.

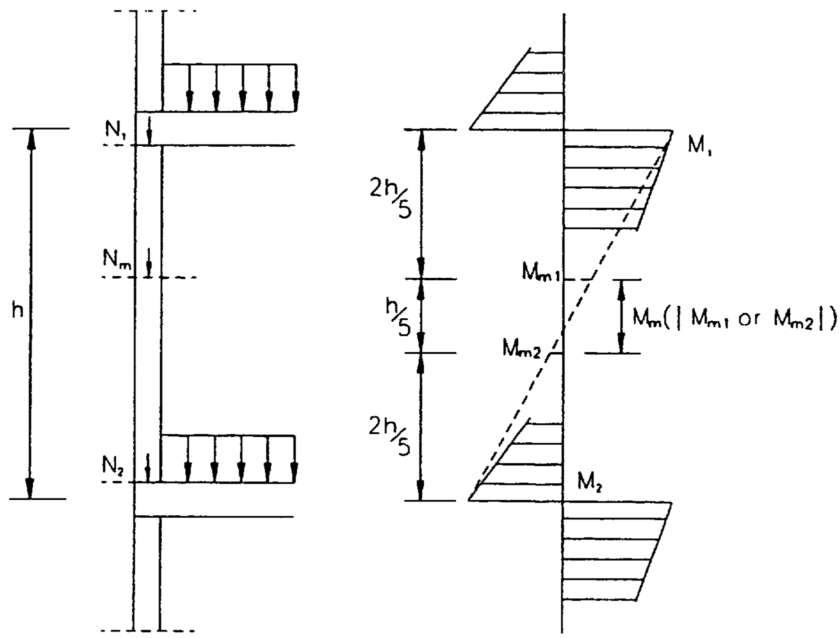


Figure 4.1 : Moments from calculation of eccentricities.

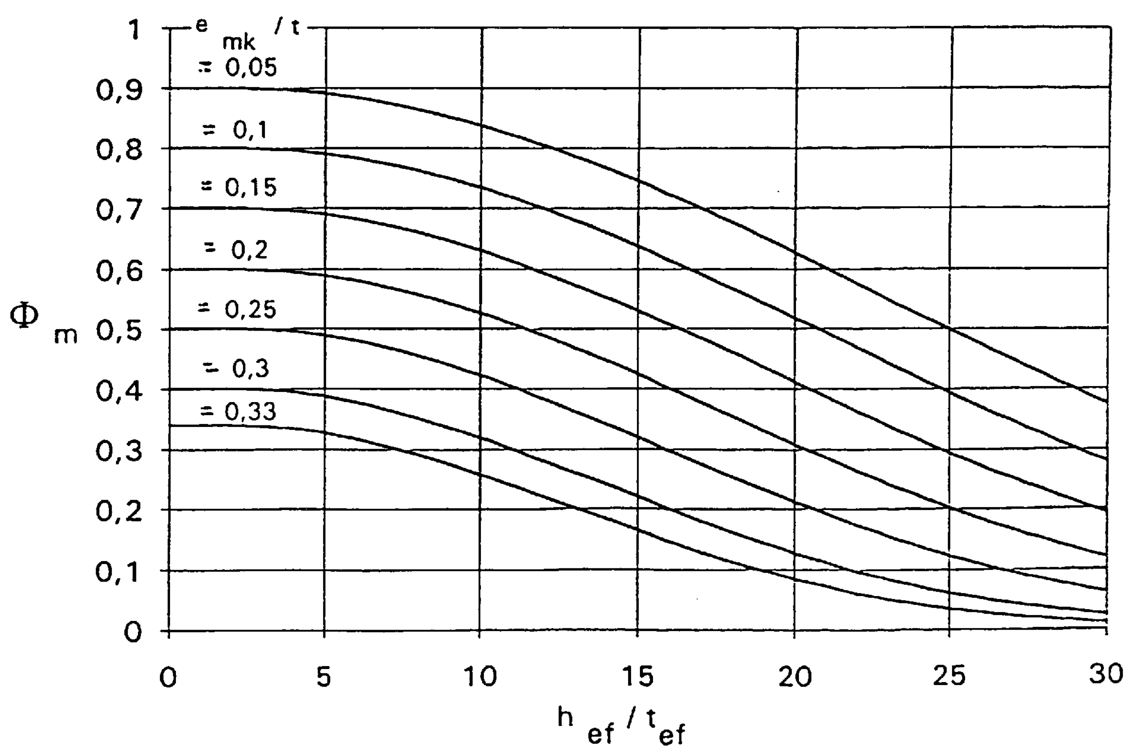


Figure 4.2 : Graph showing values of Φ_m against slenderness ratio for different eccentricities.

4.4.4.2 Stiffened walls

(1) Walls may be considered as stiffened at a vertical edge if:

- cracking between the wall and its stiffening wall is not expected 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 beween the walls, for example, due to shrinkage, loading etc., is not expected, or

- the connection between a wall and its stiffening wall is designed to resist developed tension and compression forces, by anchors or ties or other similar means.

(2) Stiffening walls should have at least a length of 1/5 of the storey height and have at least a thickness of 0,3 times the effective thickness of the wall to be stiffened, but not less than $\boxed{85}$ mm.

(3) 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 4.3, and the stiffening wall should extend a distance of at least 1/5 of the storey height beyond each opening.

(4) Alternatively, walls may be stiffened by members other than masonry walls provided that they have the equivalent stiffness of the masonry stiffening wall, referred to in paragraph (2) above, and that they are connected to the stiffened wall with anchors or ties designed to resist the tension and compression forces that will develop.

4.4.4.3 Determination of effective height

(1) The effective height can be taken as:

$$h_{ef} = \rho_n \; h \qquad (4.12)$$

where:

h_{ef} is the effective height;

h is the clear storey height;

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

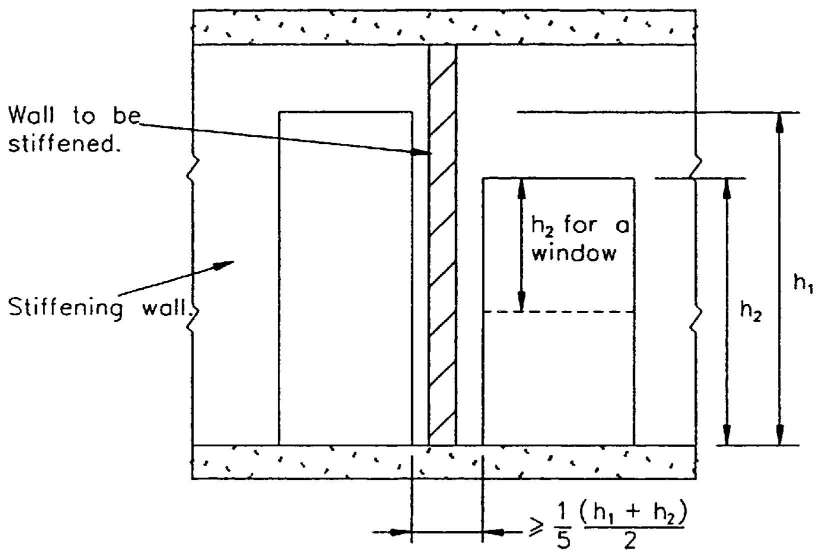


Figure 4.3 : Minimum length of stiffening wall with openings.

(2) 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 but not less than 85mm:

ρ_2 = $\boxed{0,75}$ 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 ρ_2 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 $\boxed{85}$ mm:

ρ_2 = $\boxed{1,00}$ unless the eccentricity of the load at the top of the wall is greater than 0,25 times the thickness of the wall in which case ρ_2 should be taken as 1,0.

Note: When the boxed value of ρ_2 is confirmed to be 1,00 , the remainder of the sentence should be deleted.

(iii) When neither condition (i) nor condition (ii) applies, ρ_2 should be taken as 1,0.

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

$$\rho_3 = \frac{1}{1 + \left[\dfrac{\rho_2\, h}{3\, L}\right]^2} \rho_2 \qquad > 0{,}3 \tag{4.13}$$

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

$$\rho_3 = \frac{1{,}5\ L}{h} \tag{4.14}$$

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 B.

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

$$\rho_4 = \frac{1}{1 + \left[\dfrac{\rho_2\, h}{L}\right]^2} \rho_2 \tag{4.15}$$

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

$$\rho_4 = \frac{0{,}5\ L}{h} \tag{4.16}$$

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 B.

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

4.4.4.4 Effects of openings, chases and recesses in walls

(1) If the stiffened wall is weakened by vertical chases and/or recesses, other than those allowed by table 5.3, 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.

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

(3) For the consideration of the effects of chases and recesses on the vertical load capacity of walls, see 4.4.2.(7).

4.4.5 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, a veneer wall and a grouted cavity wall, as defined in 1.4.2.9 (see also 5.1.2), should be taken as the actual thickness of the wall, t.

(2) The effective thickness, t_{ef}, of a cavity wall in which both leaves are connected with wall ties in accordance with 5.4.2.2 should be determined using equation (4.17):

$$t_{ef} = \sqrt[3]{t_1^3 + t_2^3} \qquad (4.17)$$

where t_1 and t_2 are the thicknesses of the leaves.

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

(4) When only one leaf of a cavity wall is loaded, equation (4.17) 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.

4.4.6 Slenderness ratio of walls

(1)P The slenderness ratio of a wall, h_{ef}/t_{ef}, shall not be greater than $\boxed{27}$.

4.4.7 Out-of-plane eccentricity

4.4.7.1 General

(1)P The out-of-plane eccentricity of loading on walls shall be assessed.

(2) The eccentricity may be calculated from the material properties given in Section 3 of this

ENV 1996-1-1, the joint behaviour and from the principles of structural mechanics. A simplified method for calculating the out-of-plane eccentricity of loading on walls is given in Annex C.

4.4.7.2 Allowance for imperfections

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

(2) The accidental eccentricity to allow for construction imperfections may be assumed to be h_{ef} / $\boxed{450}$, where h_{ef} is the effective height of the wall, calculated from 4.4.4.

Note: The figure $\boxed{450}$ reflects on average level of Category of execution. A higher or lower figure can be used to reflect better or worse levels of Category of execution.

4.4.8 Concentrated loads

(1)P Under the ultimate limit state, the design load resistance of an unreinforced wall subjected to concentrated loads shall be greater than the design concentrated load on the wall.

(2) When a wall, built with Group 1 masonry units and detailed in accordance with Section 5 of this ENV 1996-1-1 and other than a shell bedded wall, is subjected to a concentrated load, it should be verified that, locally under the bearing of the concentrated load, the design compressive stress does not exceed the value derived from the following:

$$\frac{f_k}{\gamma_M} [(1 + 0{,}15\ x)\ (1{,}5 - 1{,}1\ \frac{A_b}{A_{ef}})] \tag{4.18}$$

but not less than f_k/γ_m nor greater than:

$$1{,}25\ \frac{f_k}{\gamma_M} \text{ where } x = 0 \tag{4.19}$$

and

$$1{,}5\ \frac{f_k}{\gamma_M} \text{ where } x = 1{,}0 \tag{4.20}$$

with the upper limit linearly interpolated between 1,25 f_k/γ_M and 1,5 f_k/γ_M where $0 < x < 1$;

where:

f_k is the characteristic compressive strength of the masonry, obtained from 3.6.2;

γ_M is the partial safety factor for the material, obtained from 2.3.3.2;

$x = \frac{2a_1}{H}$ 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 4.4);

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 effective area of the wall L_{ef} t (see figure 4.4);

L_{ef} is the effective length as determined at the mid height of the wall or pier (see figure 4.4);

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 f_k/γ_M are shown in graphical form in Appendix D.

(3) For walls built with Groups 2a, Group 2b 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:

$$\frac{f_k}{\gamma_M} \tag{4.21}$$

(4) The eccentricity of the load from the centre line of the wall should not be greater than t/4 (see figure 4.4).

(5) In all cases, the requirements of 4.4.2 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 suitable spreader beam of width t , height greater than 200mm and length greater than three times the bearing length of the load, the design compressive stress beneath the loaded (not spreader) area should not exceed 1,5

f_k/γ_M.

(8) Bearings that satisfy the ultimate limit state when verified in accordance with equations (4.18), (4.19), (4.20) or (4.21) may be deemed to satisfy the serviceability limit state.

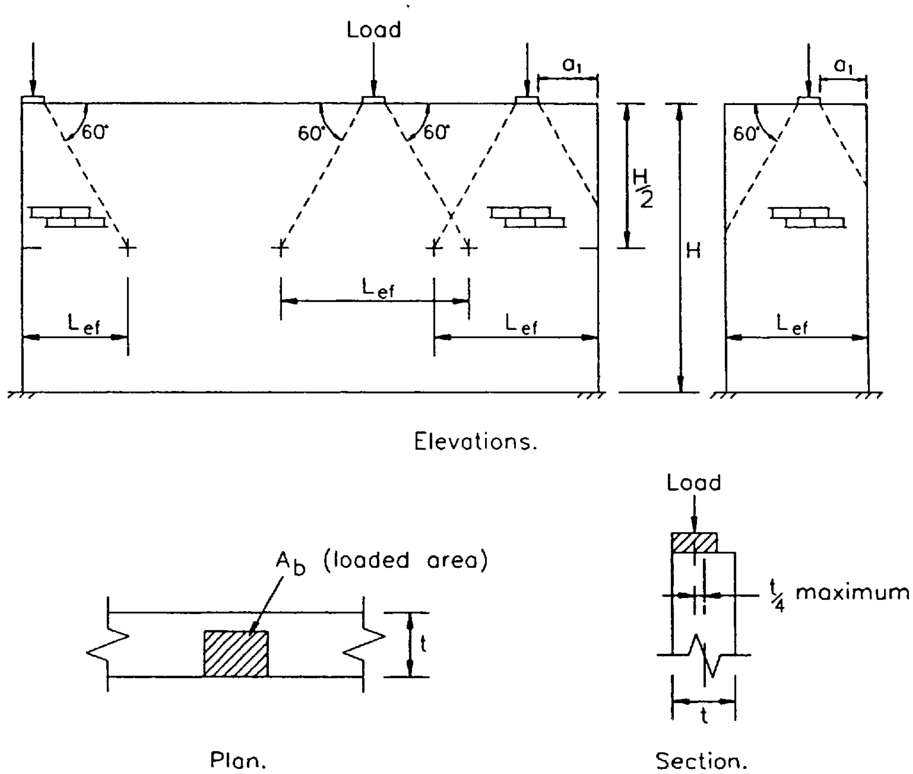


Figure 4.4 : Walls subjected to concentrated load.

4.4.9 Stresses due to restraints

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

Note: The rigid connection of materials with different deformation behaviour may cause considerable stresses due to shrinkage, creep and changes in temperature (see 3.8.4), and these may lead to redistribution of design stresses and damage to the masonry. Similar effects may be induced by differential settlement of foundations or deflections of supporting structural members.

(2) Damage, due to stresses arising from restraints, should be avoided by appropriate specification and detailing, as referred to in Section 5 of this ENV 1996-1-1.

4.5 Unreinforced masonry shear walls

4.5.1 General

(1)P Resistance to horizontal actions is generally provided by a system formed by the floors and shear walls. The structural system shall be such that the action effects do not exceed the resistance to horizontal effects.

(2) Openings in shear walls can considerably affect their behaviour and their presence should be taken into account when they occur; simplified approaches may be used when justifiable.

(3) Chases and recesses reduce the shear capacity of a wall. The reduction may be assumed to be insignificant if the chases or recesses are within the limits given in 5.5. If the size, number or location of the chases or recesses are outside these limits, the shear resistance of the wall should be checked using the reduced thickness of the wall at the chase or recess positions.

(4) A limited portion of an intersecting wall can act as a flange to a shear wall, increasing its stiffness and strength; it can be taken into account in the design, 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 (see also 4.4.4.2).

(5) 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 4.5):

- $2h_{tot}/10$, where h_{tot} is the overall height of the shear wall;
- half the distance between shear walls, when connected by the intersecting wall;
- the distance to the end of the wall;
- half the storey height.

(6) Due to the lack of knowledge of the non-linear flexural characteristics of masonry walls bent in their plane, for the distribution of horizontal actions only, the elastic stiffness of the shear walls, including any flanges, should be used. For walls higher than twice their length, the effect of shear strains on the stiffness can be neglected.

(7) If the floors can be idealised as rigid diaphragms (for example, in the case of in-situ concrete slabs) a conservative procedure is to distribute the horizontal forces to the shear walls in proportion to their stiffness on the assumption that all deflect by the same amount; more refined analytical procedures may be used, if appropriate.

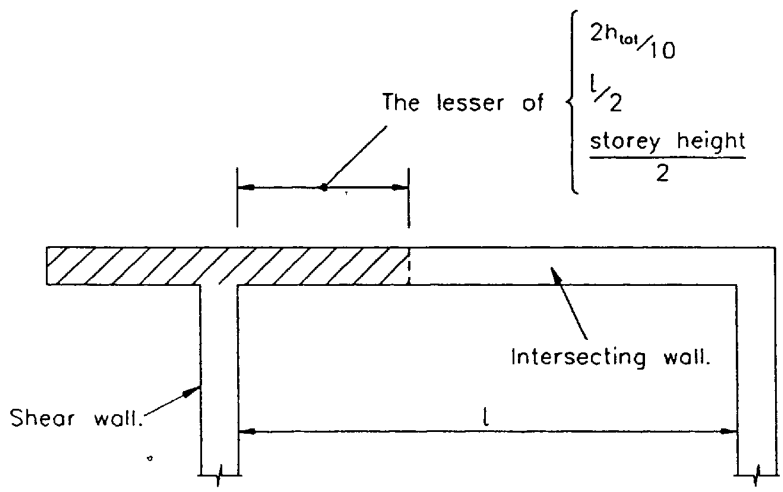


Figure 4.5 : Flange widths that can be assumed for shear walls.

(8)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 of the system on the individual walls (torsional effects). See 4.1.1(4) for the general requirements for a properly braced structure.

(9) 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.

4.5.2 Analysis of shear walls

(1)P For the analysis of shear walls, the design horizontal actions and the design vertical loads shall be applied to the overall structure.

(2)P The design horizontal actions shall not be less than the notional horizontal force obtained from 4.1.1 (4).

Note: It is usually sufficient to consider the effect of horizontal actions on the two principal axes.

(3) The most unfavourable combination of vertical load and shear should be considered, as follows:

either:

- maximum axial load per unit length of the shear wall, due to vertical loads and considering the longitudinal eccentricity due to cantilever bending, or

- maximum axial load per unit length in the flanges or stiffening walls, or

- maximum horizontal shear in the shear wall when the minimum axial load assisting the design shear resistance is combined with the maximum horizontal load, or

- maximum vertical shear per unit length at the connection between the shear wall and any intersecting wall or flange taken into account in the verification.

(4) When deriving the minimum axial load assisting the shear resistance, the vertical load applied to slabs spanning in two directions may be equally distributed on 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 stories, on the walls not directly loaded.

(5) The maximum horizontal shear on a wall, calculated using a linear elastic analysis, may be modified taking advantage of possible redistribution of forces due to limited cracking of the wall at the ultimate limit state; the shear on a single wall may be reduced by 15% provided that shear on the parallel walls is correspondingly increased so that equilibrium is assured under the design loads.

4.5.3 Verification of shear walls

(1)P Under the ultimate limit state, the shear wall and any flange of an intersecting wall shall be verified for vertical loading and for shear loading.

(2) The net resisting length and thickness of the walls or flanges should be determined taking into account any openings, chases or recesses and disregarding any portion subject to vertical tensile stress.

(3)P The connections between the shear walls and the flanges of intersecting walls, taken into account in the analysis, shall be verified for vertical shear.

(4)P The design value of the applied shear load, V_{Sd}, shall be less than or equal to the design shear resistance, V_{Rd} , calculated using the values given in 3.6.3, such that :

$$V_{Sd} \leq V_{Rd} \tag{4.22}$$

(5) The design shear resistance is given by:

$$V_{Rd} = \frac{f_{vk}\ t\ l_c}{\gamma_M} \tag{4.23}$$

where:

f_{vk} is the characteristic shear strength of masonry, obtained from 3.6.3, 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;

γ_M is the partial safety factor for the material, obtained from 2.3.3.2.

(6) The length of the compressed part of the wall, l_c, should be calculated assuming a triangular stress distribution.

(7) The vertical shear resistance of a connection may be taken from test data for a specific project or from that available on a National basis. In the absence of such data, the design value f_{vko}/γ_M, where f_{vko} is the shear strength under zero compressive stress, as given in 3.7.3, may be used, provided that the connection between the walls is in accordance with 5.4.2.

(8) Walls that satisfy the ultimate limit state when verified in accordance with equation (4.23) may be deemed to satisfy the serviceability limit state.

4.6 Unreinforced walls subjected to lateral loads

4.6.1 General

(1)P A wall subjected to lateral load under the ultimate limit state shall be verified to have a design strength greater than or equal to the design lateral load effect.

Note: Precise methods for the design of masonry walls subjected predominantly to lateral wind loads are not available but there are approximate methods which may be used.

(2) The design of masonry walls subjected predominantly to lateral wind loads may be based on the approximate methods given in 4.6.2.2, with the wall being supported along whichever edges provide support, or 4.6.2.3, with the wall arching between supports.

Note: In some countries design of walls subjected predominantly to lateral wind loads is not necessary as the thickness of walls used will provide adequate lateral strength.

(3) Chases and recesses reduce the flexural strength of a wall used in lateral load design. The reduction may be assumed to be insignificant if the chases or recesses are within the limits given in 5.5. If the size, number or location of the chases or recesses are outside these limits, the flexural strength of the wall should be checked using the reduced thickness of the wall at the chase or recess position.

(4) Where damp proof courses are used in walls, allowance should be made for any effect on the flexural strength.

4.6.2 Walls subjected to lateral wind loads

4.6.2.1 Support conditions and continuity

(1)P In assessing the lateral resistance of masonry walls subjected to lateral wind loads, the support conditions and continuity over supports shall be taken into account.

(2) The reaction along an edge of a wall due to the design load may normally be assumed to be uniformly distributed when designing the means of support. Restraint at a support may be provided by ties, by bonded masonry returns or by floors or roofs.

(3) In the case of cavity construction, full continuity may be assumed even if only one leaf is continuously bonded across a support, provided that the cavity wall is provided with ties in accordance with 5.4.2.2. Where the leaves are of different thickness, the thicker leaf normally should be the continuous leaf unless it is clear that full continuity can be assumed from the stiffness and strength of a continuous thinner leaf. The load to be transmitted from a panel to its support may be taken by ties to one leaf only, provided that there is adequate connection between the two leaves (see 5.4.2.2) particularly at the edges of the panels. In all other cases, partial continuity may be assumed.

(4) The design lateral strength of a cavity wall should be taken as the sum of the design lateral strengths of the two leaves provided that the wall ties, or other connectors between the leaves, are capable of transmitting the actions to which the cavity wall is subjected (see 4.6.2.4).

4.6.2.2 Method of design for a wall supported along edges

(1) Masonry walls are not isotropic and there is an orthogonal strength ratio depending on the unit and the mortar used.

(2) The calculation of the design moment, M_d, should take into account the masonry properties referred to above and may be taken as either:

$$M_d = \alpha \ W_k \ \gamma_F \ L^2 \text{ per unit height of the wall} \tag{4.24}$$

when the plane of failure is perpendicular to the bed joints, ie. in the f_{xk2} direction , or

$$M_d = \mu \ \alpha \ W_k \ \gamma_F \ L^2 \text{ per unit length of the wall} \tag{4.25}$$

when the plane failure is parallel to the bed joints, ie. in the f_{xk1} direction;

where:

α is a bending moment coefficient which depends on the orthogonal ratio, μ, the degree of fixity at the edges of the panels and the height to length ratio of the panels and which is obtained from a suitable theory [14];

γ_F is the partial safety factor for loads, obtained from 2.3.3.1;

μ is the orthogonal ratio of the characteristic flexural strengths of the masonry, f_{xk1}/f_{xk2} see 3.6.4;

L is the length of the panel between supports;

W_k is the characteristic wind load per unit area.

(3) When a vertical load acts so as to increase the flexural strength f_{xk1}, the orthogonal strength ratio may be modified by using a flexural strength in that direction of:

$$f_{xk1} + \gamma_M \sigma_{dp} \tag{4.26}$$

where:

f_{xk1} is the characteristic flexural strength with the plane of failure parallel to the bed joints, obtained from 3.6.4;

γ_M is the partial safety factor for the material, obtained from 2.3.3.2;

σ_{dp} is the permanent vertical stress on the wall at the level under consideration.

(4) The bending moment coefficient at a damp proof course may be taken as for an edge over which full continuity exists when there is sufficient vertical load on the damp proof course to ensure that its flexural strength is not exceeded under the applied moment.

(5) The design moment of lateral resistance of a masonry wall, M_{Rd}, is given by:

$$M_{Rd} = \frac{f_{xk} Z}{\gamma_M} \tag{4.27}$$

where:

f_{xk} is the characteristic flexural strength, obtained from 3.6.4, appropriate to the plane of bending;

Z the section modulus of the wall.

[14] It is implicit that a suitable theory is given in the National Application Documents.

(6) In assessing the section modulus of a pier in a wall, the outstanding length of flange from the face of the pier should be taken as:

- h/10 for walls spanning vertically between restraints;
- 2h/10 for cantilever walls;
- in no case more than half the clear distance between piers;

where h is the clear height of the wall

(7) Walls that satisfy the ultimate limit state when verified using equations (4.24), (4.25) and (4.27) may be deemed to satisfy the serviceability limit state.

(8) In a laterally loaded panel or free standing wall built of masonry set in mortar designations M2 to M20, and designed in accordance with paragraphs (1) - (7) above, the dimensions should be limited to avoid undue movements resulting from deflections, creep, shrinkage, temperature effects and cracking.

4.6.2.3 Method for design of arching between supports

(1) When a masonry wall is built solidly between supports capable of resisting an arch thrust, as described below, or when a number of walls are built continuously past supports, the wall may be designed assuming that an horizontal or vertical arch develops within the thickness of the wall.

Note: In the present state of knowledge, walls subjected to mainly lateral loads should be designed only for arching horizontally, except when accidental actions are considered.

(2) Calculation should be based on a three-pin arch and the bearing at the supports and at the central hinge should be assumed as 0,1 times the thickness of the wall.

(3) The arch thrust should be assessed from knowledge of the applied lateral load, the strength of the masonry in compression and the effectiveness of the junction between the wall and the support resisting the thrust. A small change in length of a wall in arching can considerably reduce the arching resistance. Therefore, care should be taken if the masonry is built of masonry units that may shrink in service.

(4) The arch rise is given by:

$$0{,}9\ t - d \qquad (4.28)$$

where:

t is the thickness of the wall (but see 6.5.3(2));

d is the deflection of the arch under the design lateral load; it may be taken to be zero for walls having a length to thickness ratio of 25 or less.

(5) The maximum design arch thrust per unit length of wall may be assumed to be:

$$1{,}5 \frac{f_k}{\gamma_M} \frac{t}{10} \tag{4.29}$$

and where the lateral deflection is small, the design lateral strength is given by:

$$q_{lat} = \frac{f_k}{\gamma_M} \left[\frac{t}{L} \right]^2 \tag{4.30}$$

where:

q_{lat} is the design lateral strength per unit area of wall;

t is the thickness of the wall (but see 6.5.3(2));

f_k is the characteristic compressive strength of the masonry, obtained from 3.6.2;

L is the length of the wall;

γ_M is the partial safety factor for the material, obtained from 2.3.3.2.

(6) Walls that satisfy the ultimate limit state when verified using equation (4.30) may be deemed to satisfy the serviceability limit state.

4.6.2.4 Wall ties

(1)P Where walls, especially cavity walls, are subjected to lateral wind loads, the wall ties connecting the two leaves shall be capable of distributing the wind loads from the loaded leaf to the other leaf, backing wall or support.

(2) The minimum number of wall ties per unit area should be obtained from (but see also 5.4.2.2):

$$\gamma_M \frac{W_{Sd}}{F_t} \tag{4.31}$$

where:

W_{Sd} is the design horizontal action from wind, per unit area, to be transferred;

F_t is the characteristic compressive or tensile resistance of a wall tie, as appropriate to the design condition, determined by tests in accordance with EN 846-5 or EN 846-6;

γ_M is the partial safety factor for wall ties, from 2.3.3.2.

(3) The horizontal action from the wind acting on the wall should be calculated in accordance with ENV 1991. For cavity walls, the design horizontal action per unit area to be transferred between the leaves, W_{Sd}, should be determined by dividing the wind action acting on the wall between the two leaves in proportion to their lateral load capacity.

Note: The selection of wall ties should allow for differential movement between the leaves, without damage.

(4) In the case of a veneer wall, W_{Sd}, should be calculated on the basis that the wall ties are required to transmit all the design horizontal action from the wind acting on the wall to the backing structure.

4.6.3 Walls subjected to lateral earth pressure

(1)P Walls subject to lateral earth pressure shall be designed using acceptable engineering principles.

Note: The flexural strength of masonry f_{xk1} should not be used in the design of walls subjected to lateral earth pressure.

(2) An empirical method for designing basement walls subjected to lateral earth pressure is given in Annex E.

4.6.4 Horizontal accidental loads (excluding seismic actions)

(1) Walls subjected to horizontal accidental loads, other than those resulting from seismic actions (for example, gas explosions), may be designed similarly to walls subjected to wind loads in accordance with 4.6.2. Axially loaded walls, with a slenderness ratio no greater than 20, may be designed on the basis of arching vertically between concrete floors. Allowance may be made for enhancement due to bonded piers or return walls. The resistance of damp proof courses or similar low friction planes, should be verified to ensure that the shear strength is not exceeded.

4.7 Reinforced masonry

4.7.1 Reinforced masonry members subjected to bending, bending and axial load or axial load

4.7.1.1 General

(1)P The strength of reinforced masonry members shall be calculated using a theory in which the non-linear behaviour of the materials and second order effects are taken into account.

(2)P The deformation properties of concrete infill shall be assumed to be as for masonry. The requirements for masonry in paragraph (3) below shall be equally applicable to concrete infill.

(3)P The design of reinforced masonry members, subjected to bending, bending and axial load, or axial load, 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 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 3.8.1(2));

- the stress-strain relationship of the reinforcement is derived from figure 4.6;

- for cross-sections subject to pure longitudinal compression, the compressive strain in the masonry is limited to -0,002 (see figure 3.3);

- for cross-sections not fully in compression, the limiting compressive strain is taken as -0,0035 (see figure 3.3). In intermediate situations, the strain diagram is defined by assuming that the strain is -0,002 at a level 3/7 of the height of the section from the most compressed face (see figure 4.9).

(4) The rules of application are concerned with bending both in plane and out of plane and include walls and beams.

(5) The design compressive stress block for masonry or concrete infill may be based on figure 3.3, where f_d is f_k / γ_M for masonry, taking particular care to use the value appropriate to the direction of loading, and f_{ck} / γ_M for concrete infill.

(6) When a compression zone contains masonry as well as concrete infill, the compressive strength should be calculated using a stress block based on the compressive strength of the weakest material.

4.7.1.2 Effective span of members subjected to bending

(1) The effective span, l_{ef}, of simply supported or continuous members, with the exception of deep beams, may be taken as the smaller of the following (see figure 4.7):

- the distance between centres of supports;

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

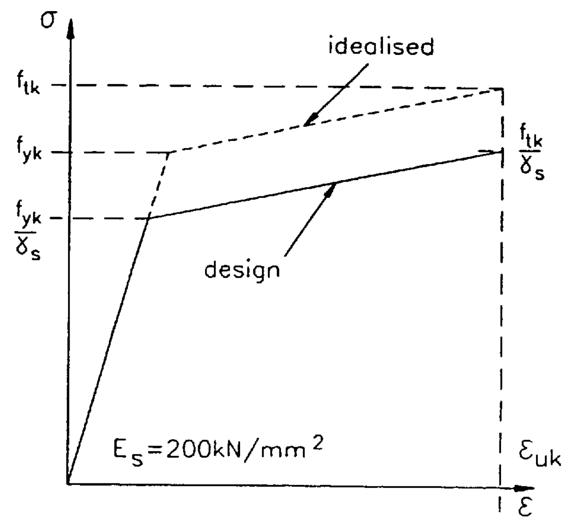


Figure 4.6 : Design stress-strain diagram for reinforcing steel (tension and compression).

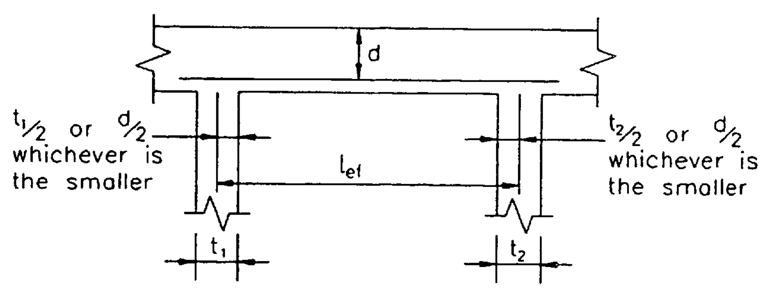


Figure 4.7 : Effective span of simply supported or continuous members.

(2) The effective span, l_{ef} , of a cantilever may be taken as the smaller of the following (see figure 4.8):

- 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.

(3) The effective span of deep beams may be determined according to 4.7.3.1.

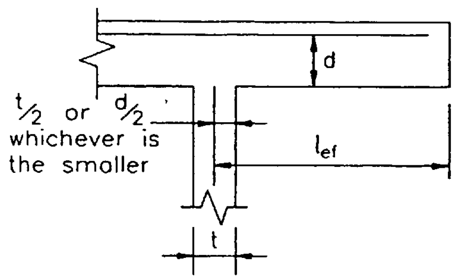


Figure 4.8 : Effective span of cantilevers.

4.7.1.3 Limiting span of members subjected to bending

(1) The span of a reinforced masonry member should be limited to the appropriate value obtained from table 4.1.

(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:

$$60\ b_c \text{ or } \frac{250}{d}\ b_c^2 \text{ , whichever is the lesser;} \tag{4.32}$$

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:

$$25\ b_c \text{ or } \frac{100}{d}\ b_c^2 \text{ , whichever is the lesser.} \tag{4.33}$$

where b_c is taken at the face of the support.

4.7.1.4 Slenderness ratio of vertically loaded members

(1) The slenderness ratio of vertically loaded members in the plane of the member should be determined in accordance with 4.4.

(2) The slenderness ratio should not be greater than $\boxed{27}$.

Table 4.1 : Limiting ratios of span to effective depth for walls and beams.

End condition	Ratio of span to effective depth	
	Wall	Beam
Simply supported	35	20
Continuous	45	26
Spanning in two directions	45	-
Cantilever	18	7

Notes:

1. A wall is a member subjected to out of plane bending and a beam may be part of a wall subjected to bending in the plane of the wall.

2. For free-standing walls not forming part of a building and subjected predominantly to wind loads, the ratios given for walls may be increased by 30%, provided such walls have no applied finish which may be damaged by deflections.

4.7.1.5 Flanged Members

(1) Where the reinforcement in a section 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 flange, 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.

4.7.1.6 Verification of reinforced masonry members subjected to bending and/or axial load

(1)P At the ultimate limit state, the design load applied to a reinforced masonry member, S_d, shall be less than or equal to the design load resistance of the member, R_d , such that:

$$S_d \leq R_d \tag{4.34}$$

(2) The design should be based on the assumptions described in 4.7.1.1. This leads to the range of strain diagrams shown in figure 4.9 with the strain diagram passing through one of the three points A, B or C. The tensile strain of the reinforcement ε_s should be limited to 0,01.

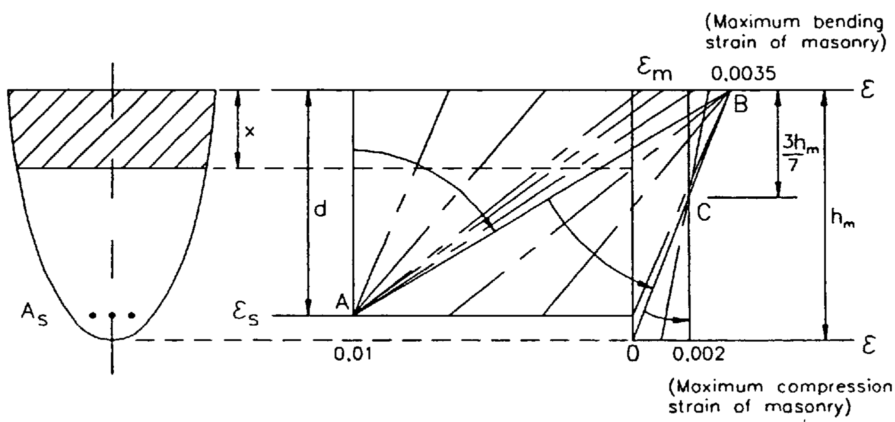


Figure 4.9 : Strain diagrams in the ultimate limit state.

(3) The linear elastic distribution of internal forces may be modified assuming equilibrium, if the members have sufficent 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 beam should be limited to 15% when high ductility steel is to be used. In this case, the ratio of the redistributed moment to the moment before redistribution should not be less than:

$$0{,}44 + 1{,}25 \frac{x}{d} \tag{4.35}$$

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

(4) No redistribution should be allowed with normal ductility steel.

(5) In determining the moment of resistance of a section, a rectangular stress distribution as indicated in figure 4.10 may be assumed as a simplification.

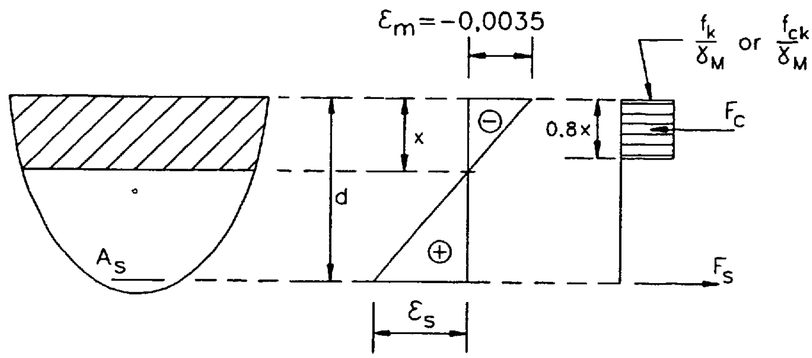


Figure 4.10 : Simplified rectangular stress block.

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

$$M_{Rd} = \frac{A_s\ f_{yk}\ z}{\gamma_s} \tag{4.36}$$

where, based on the simplification illustrated in figure 4.10, the lever arm , z , may be taken as:

$$z = d\left[1 - 0{,}5\ \frac{A_s\ f_{yk}\ \gamma_M}{b\ d\ f_k\ \gamma_s}\right] \leq 0{,}95d \tag{4.37}$$

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_k is the characteristic compressive strength of masonry in the direction of loading, obtained from 3.6.2, or concrete infill obtained from 3.3.3, whichever is the lesser;

f_{yk} is the characteristic strength of steel, obtained from 3.4.2;

γ_M is the partial safety factor for masonry or concrete infill, obtained from 2.3.3.2;

γ_S is the partial safety factor for steel, obtained from 2.3.3.2.

Note: For the special case of reinforced masonry cantilever walls subjected to bending, refer to Annex F.

(7) In the case of flanged members (see 4.7.1.5), the design moment of resistance can be obtained using equation (4.36) but should not be taken to be greater than:

$$\frac{f_k}{\gamma_M} b_{ef}\, t_f\, (d - 0{,}5\, t_f) \qquad (4.38)$$

where:

t_f is the thickness of the flange in accordance with the requirements of 4.7.1.5;

b_{ef} is the effective width of the flange, in accordance with the requirements of 4.7.1.5.

(8) When the reinforcement in a section is concentrated locally such that the member cannot be treated as a flanged member, the reinforced section should be considered as having a width of not more than 3 times the thickness of the masonry (see figure 4.11).

(9) Reinforced masonry members with a slenderness ratio, calculated in accordance with 4.4, greater than 12, may be designed conservatively using the principles and application rules for unreinforced members in 4.4, taking into account second order effects.

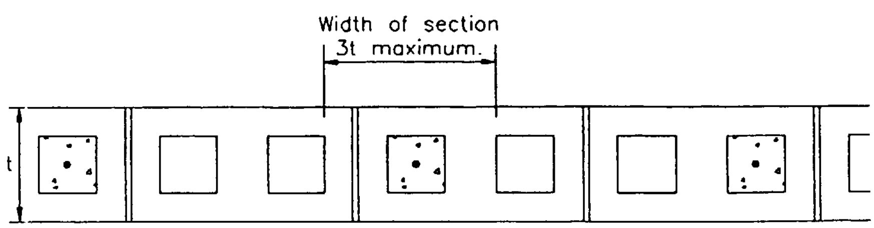


Figure 4.11 : Width of section for members with locally concentrated reinforcement.

(10) In the analysis of a cross-section which is subjected to a small axial force, the member may be designed for bending only if the design axial force does not exceed:

$$0{,}1\, f_k\, A_m \qquad (4.39)$$

where A_m is the cross-sectional area of the masonry.

4.7.2 Reinforced masonry members subjected to shear

4.7.2.1 General

(1)P The analysis of reinforced masonry shear walls shall be in accordance with 4.5.2.

(2)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_{Sd} , such that:

$$V_{Sd} \leq V_{Rd} \tag{4.40}$$

(3) 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.

(4) 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.

(5) 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 5.2.6, is not provided, or

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

4.7.2.2 Verification of members ignoring shear reinforcement

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

$$V_{Sd} \leq V_{Rd1} \tag{4.41}$$

where:

$$V_{Rd1} = \frac{f_{vk}\, b\, d}{\gamma_M} \qquad (4.42)$$

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

d is the effective depth of the member;

f_{vk} is the characteristic shear strength of masonry, obtained from 3.6.3, or concrete infill, obtained from 3.3.3, whichever is the lesser;

γ_M is the partial safety factor for masonry or concrete infill, obtained from 2.3.3.2.

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

(2) For simply supported beams or cantilevers where the ratio of the shear span to the effective depth is less than 2, f_{vk} for use in determining V_{Rd1} may be increased by a factor :

$$\frac{2d}{a_V} \leq 4 \qquad (4.43)$$

where:

d is the effective depth of the member;

a_V is the distance from the face of the support to the load;

provided that the design shear load is calculated at the face of the support and f_{vk} is not taken to be greater than 0,7 N/mm², and where the shear span is the ratio of the maximum design bending moment to the maximum design shear force.

4.7.2.3 Verification of members taking into account shear reinforcement

(1) For members when shear reinforcement is taken into account, it should be verified that:

$$V_{Sd} \leq V_{Rd1} + V_{Rd2} \qquad (4.44)$$

where: V_{Rd1} is given by equation (4.42) and V_{Rd2} is given by:

$$V_{Rd2} = 0{,}9\ d\ \frac{A_{sw}}{s}\ \frac{f_{yk}}{\gamma_S}\ (1 + \cot\alpha)\sin\alpha \qquad (4.45)$$

where:

d is the effective depth of the member;

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 member between 45° and 90°;

f_{yk} is the characteristic strength of steel obtained from 3.4.2;

γ_S is the partial safety factor for steel, obtained from 2.3.3.2.

(2) It should also be verified that:

$$V_{Rd1} + V_{Rd2} \leq \frac{0{,}30\ f_k\ b\ d}{\gamma_M} \tag{4.46}$$

where:

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

d is the effective depth of the member;

f_k is the characteristic compressive strength of the masonry in the direction of loading, obtained from 3.6.2, or the concrete infill, obtained from 3.3.3, whichever is the lesser;

γ_M is the partial safety factor for masonry or concrete infill, obtained from 2.3.3.2.

4.7.3 Reinforced masonry deep beams subjected to vertical loading

4.7.3.1 General

(1) This clause refers to 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 over the opening is at least 0,5.

(2) An appropriate structural theory may be used for the design of such beams or, alternatively, the design may be based on 4.7.3.2 and 4.7.3.3 in which case the effective span of the beam may be taken as:

$$l_{ef} = 1{,}15\ L \tag{4.47}$$

where:

L is the clear span of the opening;

z is the lever arm, which may be taken as the lesser of the following values:-

$z = \boxed{0,7}\ l_{ef}$, or

$z = \boxed{0,4}\ h + \boxed{0,2}\ l_{ef}$;

l_{ef} is the effective span of the beam;

h is the clear height of the wall.

(3) The resistance of the compression zone of the deep beam should be verified against buckling, if unrestrained, and the resistance to the compressive load at the bearings should be verified.

4.7.3.2 Verification of deep beams subjected to vertical loading

(1)P At the ultimate limit state, the design moment applied to a reinforced masonry deep beam, M_{Sd}, shall be less than or equal to the design moment of resistance of the beam, M_{Rd}, such that:

$$M_{Sd} \leq M_{Rd} \tag{4.48}$$

(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. However, the design method cannot take account of loads which are applied within the effective depth of the beam.

(3) In order to determine the amount of reinforcement, the deep beam may be considered as simply supported between supports as shown on figure 4.12.

(4) The reinforcement, A_s, required in the bottom of the deep beam may be determined as follows:

$$A_s = \frac{M_{Rd}\ \gamma_S}{f_{yk}\ z} \tag{4.49}$$

where:

M_{Rd} is the design bending moment;

f_{yk} is the characteristic strength of the reinforcement, obtained from 3.4.2;

γ_S is the partial safety factor for steel, obtained from 2.3.3.2;

z is the lever arm which may be taken from 4.7.3.1.

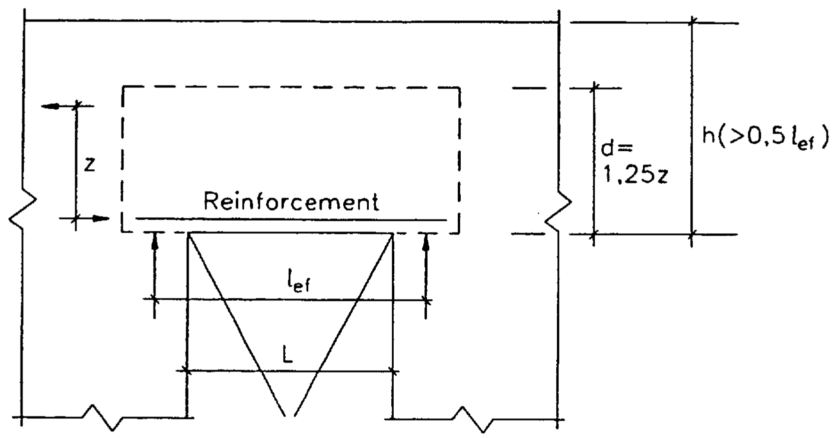


Figure 4.12: Representation of a deep beam.

(5) To resist cracking, additional nominal 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.

(6) 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 5.2.6.

(7) In no case should the moment of resistance, M_{Rd}, be taken to be greater than :

$$\frac{0,4\ f_k\ b\ d^2}{\gamma_M} \tag{4.50}$$

where:

- b is the width of the beam;
- d is the effective depth of the beam which may be taken as 1.25z;
- z is the lever arm of the beam;
- f_k is the characteristic compressive strength of the masonry in the direction of loading, obtained from 3.6.2, or concrete infill, obtained from 3.3.3, whichever is the lesser;
- γ_M is the partial coefficient for masonry or concrete infill, obtained from 2.3.3.2.

4.7.3.3 Verification of deep beams subjected to shear loads

(1)P For the ultimate limit state, no shear reinforcement is required if the design shear resistance of the reinforced masonry deep beam, V_{Rd1}, exceeds or is equal to the design shear load, V_{Sd}, such that:

$$V_{Sd} \leq V_{Rd1} \tag{4.51}$$

where:

V_{Sd} is the design shear force at the edge of the support;

V_{Rd1} is as defined in 4.7.2.2 using the effective depth of the member as d = 1,25 z.

4.7.3.4 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 to that of the wall above, the design may be based on the rules of application given in 4.7.3.2 and 4.7.3.3, 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 4.13).

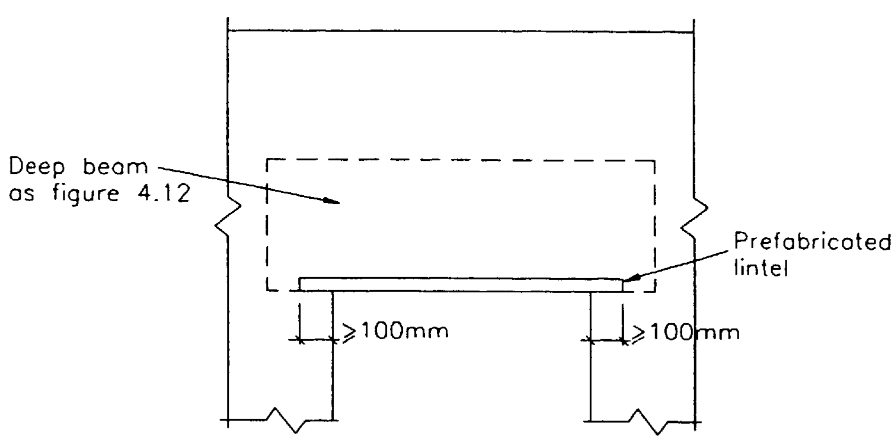


Figure 4.13 : Composite lintel forming a deep beam.

4.7.4 Reinforced masonry members under the serviceability limit state

4.7.4.1 General

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

4.7.4.2 Deflection

(1) Where reinforced masonry members are sized so as to be within the limiting dimensions given in 4.7.1.3, it may normally be assumed that the lateral deflection of a wall and the vertical deflection of a beam are acceptable.

4.7.4.3 Cracking

(1) For reinforced masonry members subjected to bending e.g. laterally loaded wall panels and reinforced masonry beams, cracking will be limited such as to satisfy the serviceability limit state if the limiting dimensions in 4.7.1.3 and the detailing requirements in Section 5 of this ENV 1996-1-1 are followed. However, where for reasons of detailing, cover to the tension reinforcement exceeds the minimum requirements given in 5.2.2.4, some noticeable cracking may occur and the designer should decide if this is acceptable.

4.8 Prestressed masonry

4.8.1 General

(1)P The design of prestressed masonry members shall be based on a consideration of the serviceability and ultimate limit states using principles which 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 and 4 of this ENV 1996-1-1.

(2) 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.

4.8.2 Prestressed masonry members under the serviceability limit state

4.8.2.1 General

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

4.8.2.2 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 2.3.4) at transfer of prestress and under design loads after prestressing losses. Compressive and tensile stresses in the masonry and the initial prestressing force in the tendons shall be limited to avoid damage.

(3)P The analysis of a prestressed masonry member at serviceability 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 above are followed, serviceability limit states will be satisfied although additional deflection verification may need to be carried out.

4.8.3 Prestressed masonry members under the ultimate limit state

4.8.3.1 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.

4.8.3.2 Design for the ultimate limit state

(1)P At the ultimate limit state, the partial safety factor for materials shall be obtained from 2.3.3.2. Where prestressing forces are considered as actions, the partial safety factors shall be obtained from 2.3.3.1. The partial safety factors for actions shall be obtained from 2.3.3.1, 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_k / γ_M;
- 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 prestress 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 4.4 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.

4.8.4 Other design matters

4.8.4.1 General

(1)P The maximum initial prestressing force and bearing stresses shall be within acceptable design criteria. Prestressing losses shall be calculated and allowed for in serviceability and ultimate limit states assessments as relevant.

4.8.4.2 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 considered 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.

4.8.4.3 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.

4.9 Confined masonry

(1)P The design of confined masonry members shall be based on the serviceability and ultimate limit state using principles which are similar to those set out for unreinforced and for reinforced masonry members.

(2) In the verification of confined members subjected to bending and/or axial load the assumptions for the reinforced masonry will be adopted. In the compression zones, the compressive stress block should be based on the strength of the masonry, only. Compression reinforcement should also be ignored.

(3) In the verification of confined members subjected to shear all the reinforcements should be ignored.

(4) Confined masonry should be designed, in seismic situations, on the basis that only the masonry is taken into account, using the appropriate principles and application rules given in this ENV 1996-1-1. No account should be taken of the strength of the reinforced concrete or reinforced masonry members.

5 Structural detailing

5.1 General

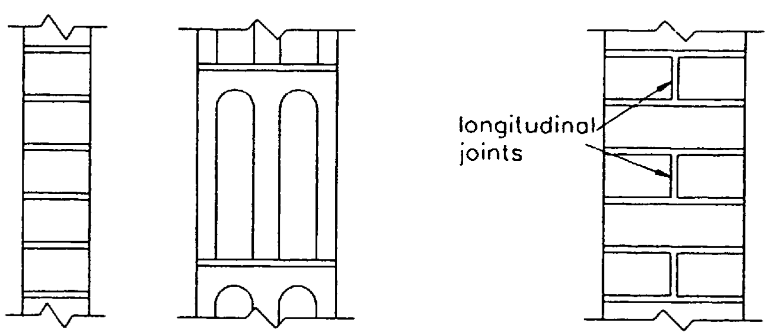
5.1.1 Masonry materials

(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) Materials should be in accordance with Section 3 of this ENV 1996-1-1. Where necessary, the relevant section of ENV 1996-2[15)] should be considered for the design and selection of materials to provide the durability required of the wall.

5.1.2 Types of walls

(1) This Section gives appropriate rules for the detailing of single-leaf walls, cavity walls, double-leaf walls, faced walls, shell bedded walls and veneer walls as defined in 1.4.2.9 and shown on figures 5.1 - 5.6.



a) Walls without longitudinal joints b) Wall with longitudinal joint

Figure 5.1 : Example cross-sections through a single-leaf wall.

[15)] At present at the draft stage.

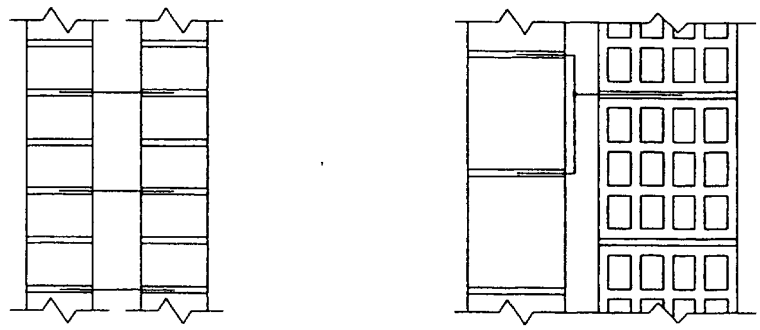


Figure 5.2 : Example cross-sections through a cavity wall.

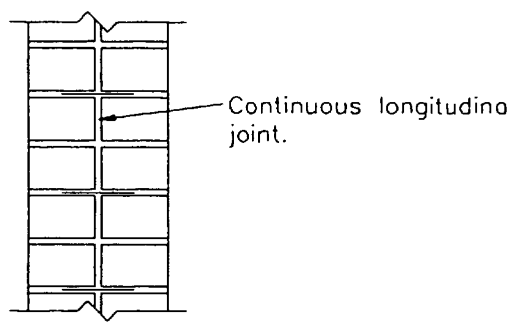


Figure 5.3 : Example cross-section through a double-leaf wall.

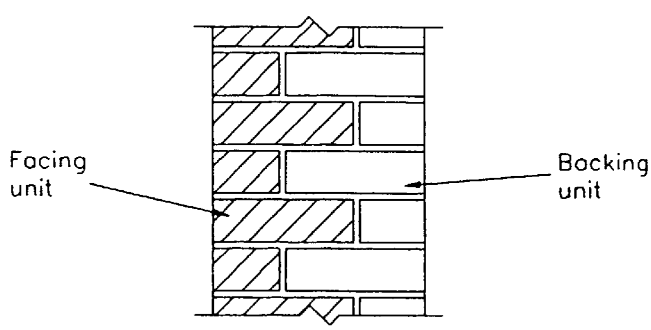


Figure 5.4 : Example cross-section through a faced wall.

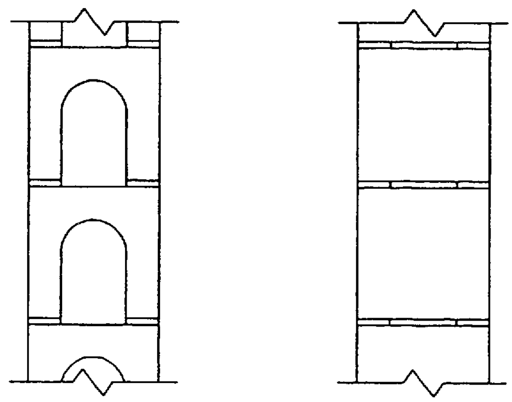


Figure 5.5 : Example cross-sections through a shell bedded wall.

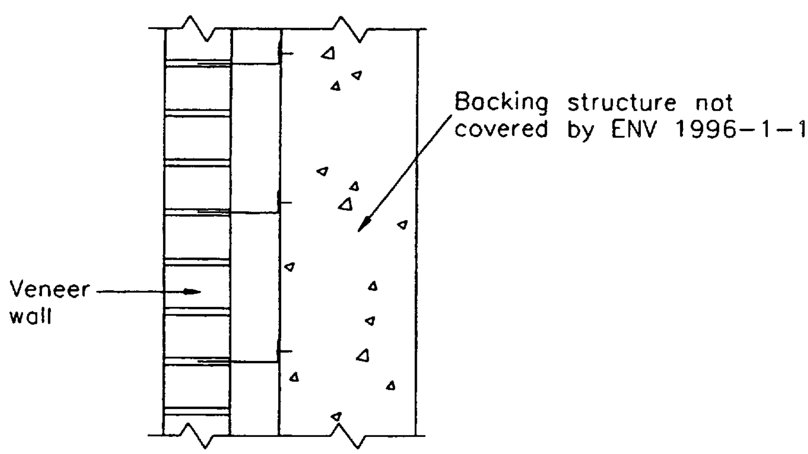


Figure 5.6 : Example cross-section through a veneer wall

5.1.3 Minimum thickness of walls

(1) The thickness of loadbearing walls should be not less than $\boxed{100}$ mm. For veneer walls the minimum thickness should be $\boxed{70}$ mm.

5.1.4 Bonding of masonry

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

(2) Masonry units in a wall should be overlapped on alternate courses so that the wall acts as a single structural element. To ensure adequate bonding, 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, as shown in figure 5.7. 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.

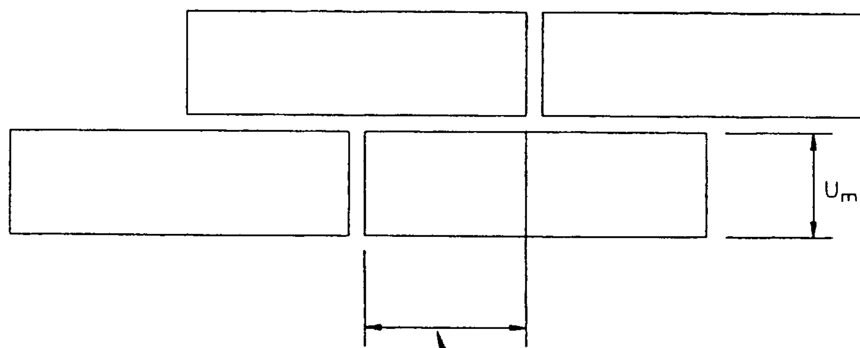


Figure 5.7 : Overlap of masonry units.

(3) Examples of bonding arrangements for walls are shown in figure 5.8, figure 5.9 and figure 5.10. Other bonding arrangements, not meeting the minimum overlap requirements, may be used in reinforced masonry or where experience or experimental data indicate that they are satisfactory.

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

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.

5.1.5 Mortar joints

(1) For the purpose of using the values and equations given in 3.6.2 and 3.6.3, bed and perpend joints made with general purpose and lightweight mortars should be not less than 8mm nor more than 15mm thick and bed and perpend joints made with thin layer mortars should be not less than 1mm nor more than 3mm thick.

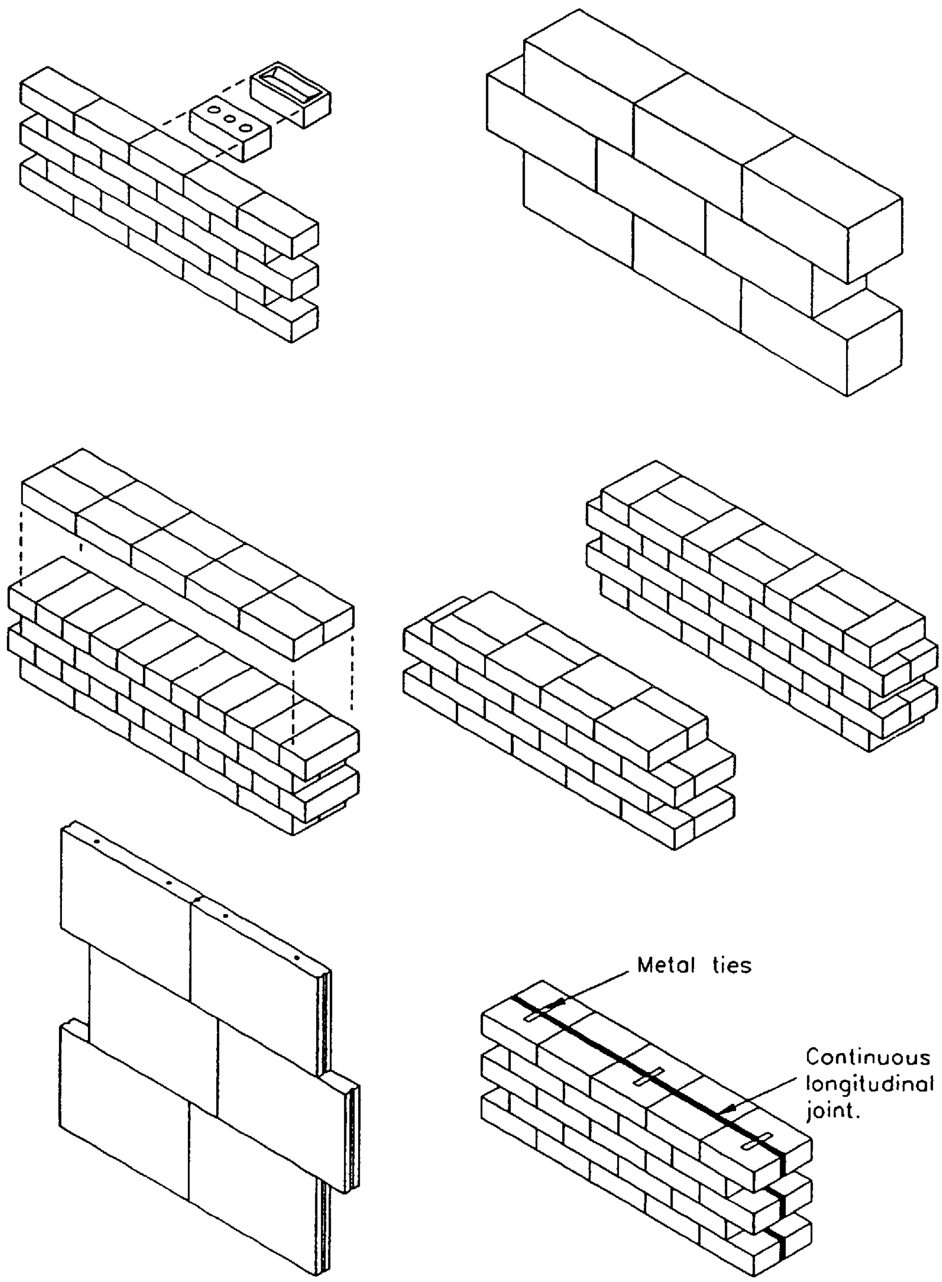


Figure 5.8 : Examples of bonding arrangements using Group 1 masonry units.

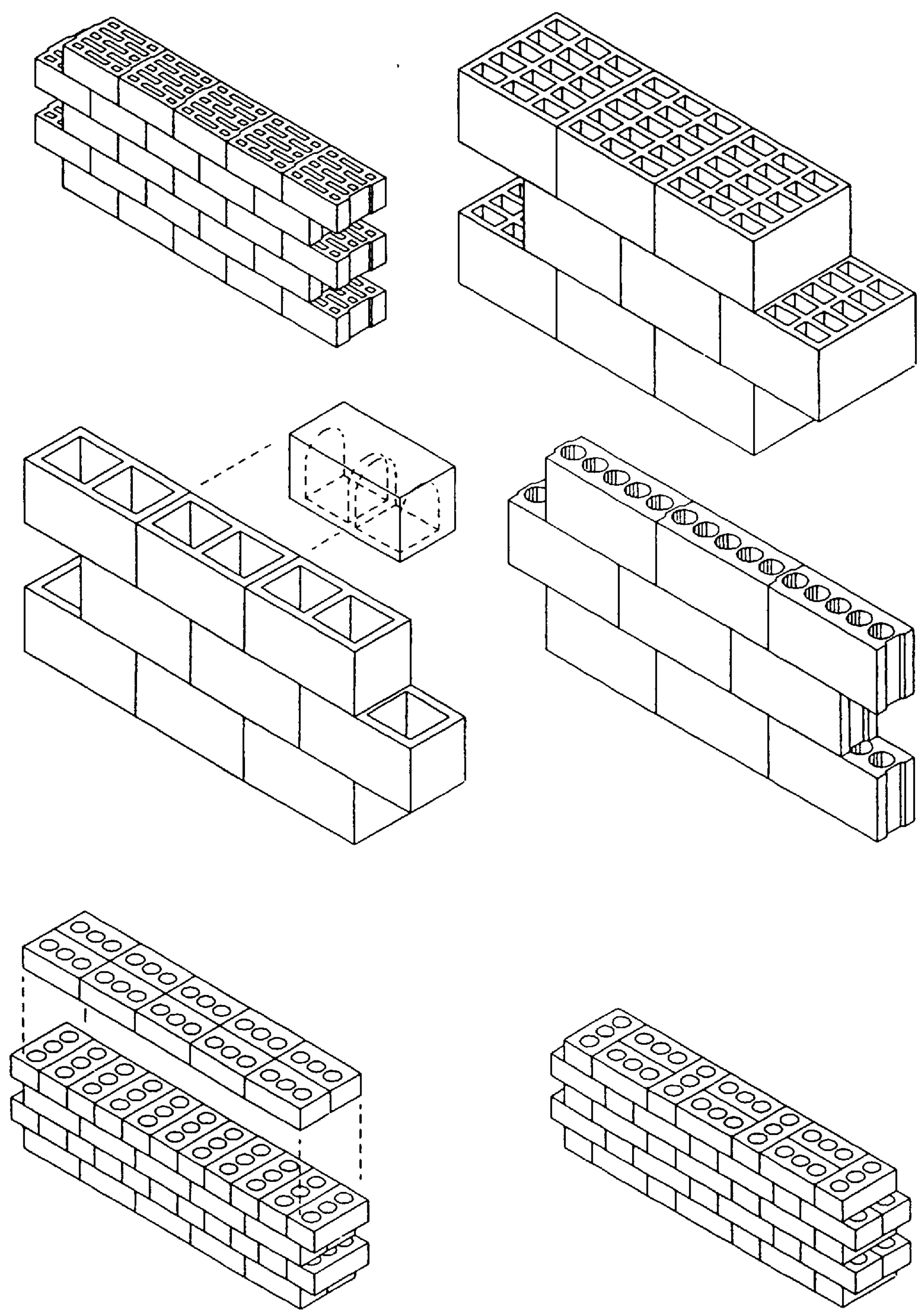


Figure 5.9 : Example of bonding arrangements using Group 2a and Group 2b masonry units.

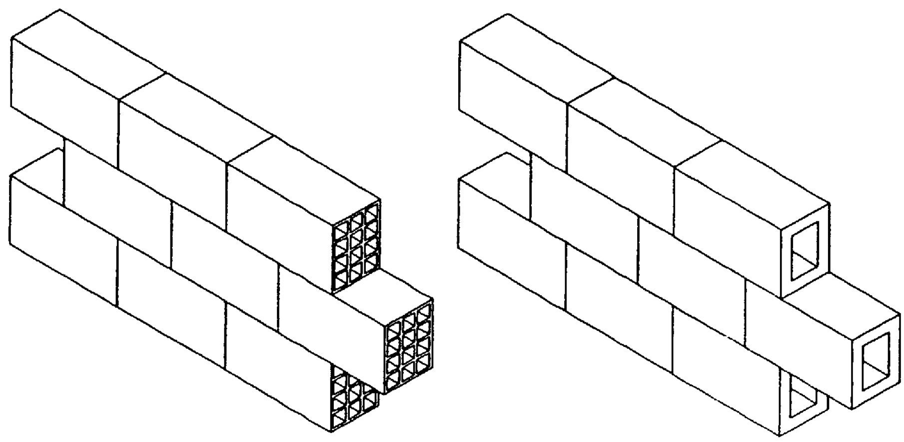


Figure 5.10 : Examples of bonding arrangements using Group 3 masonry units

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

(3) Perpend joints may be considered to be filled if mortar is provided to the full height over a minimum of 40% of the width of the unit, otherwise they should be considered to be unfilled (see 3.6.2.5). Perpend joints in reinforced masonry subject to bending and shear across the joint should be fully filled with mortar.

5.1.6 Bearings under concentrated loads.

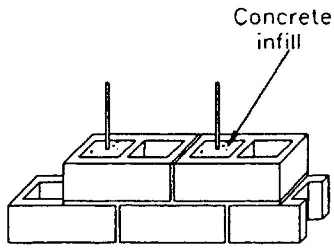
(1) Concentrated loads should bear on a wall a minimum length of 100mm or that required from calculations according to 4.4.8, whichever is the greater.

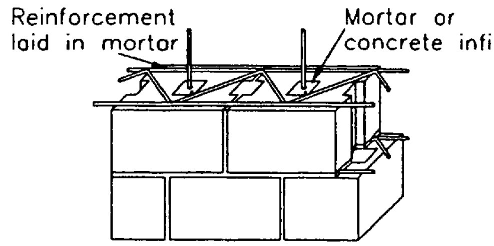
5.2 Reinforcement detailing

5.2.1 General

(1)P Reinforcement shall be located such that it acts compositely with the masonry and such that it will not yield at the formation of cracks in the masonry.

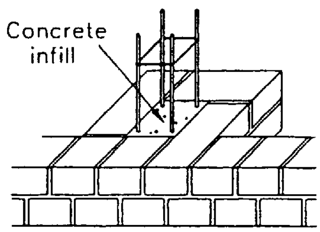
(2) Various ways in which reinforcement can be used in reinforced masonry are shown in figure 5.11.

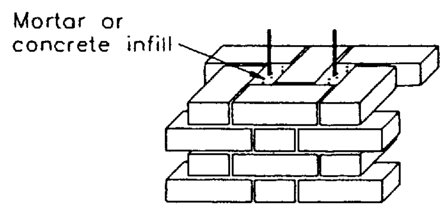




a) Reinforced hollow blockwork wall

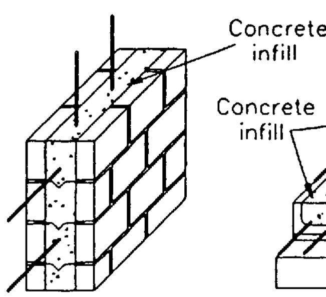
b) Wall with vertical and bed joint reinforcement

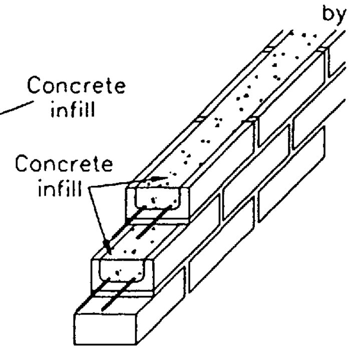


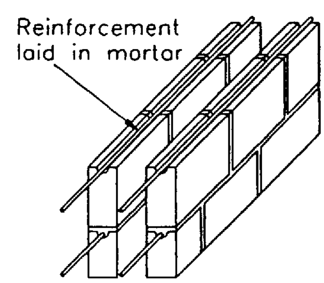


c) Pocket type wall

d) Reinforced pockets formed by bonding arrangements



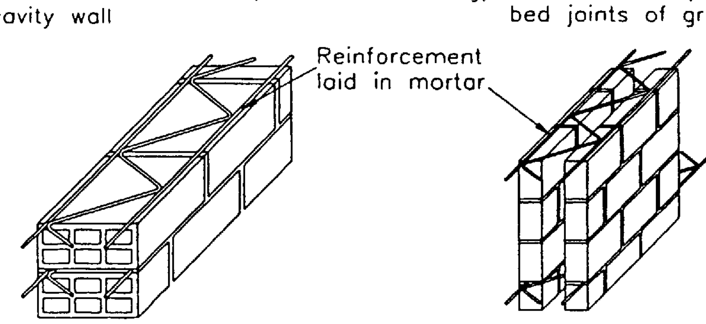




e) Concrete filled cavity wall

f) Beam unit

g) Reinforcement placed in the bed joints of grooved units



h) Units with bed joint reinforcement

Figure 5.11 : Examples of the incorporation of reinforcement in masonry.

(3) Where simple supports are assumed in the design, consideration should be given to the effects of any fixity that might be provided by the masonry. Reinforcement 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 5.2.5.1. In all cases at least one quarter of the reinforcement required at midspan should be carried through to the support and similarly anchored.

5.2.2 Protection of reinforcing steel

5.2.2.1 General

(1)P Reinforcing steel shall be corrosion resistant or protected adequately against corrosion due to environmental conditions.

(2) 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 5.2.2.2, is given in table 5.1. This table applies to carbon steel, austenitic stainless steel and galvanised steel when cover to the reinforcing steel is provided in accordance with 5.2.2.4. Alternatively, where unprotected carbon steel is used, it may be protected by concrete cover in accordance with table 5.2.

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

5.2.2.2 Classification of environmental conditions - exposure classes

(1) Environmental conditions are classified into the following five exposure classes:

- **Exposure Class 1** : A dry environment such as the interior of buildings for normal habitation and for offices, including the inner leaf of external cavity walls not likely to become damp.

  Note: This exposure class is valid only as long as the masonry, or any of its components, is not exposed during construction to more severe conditions over a prolonged period of time.

- **Exposure Class 2** : A humid environment internally, such as in a laundry, or externally not exposed to frost and including members in non-aggressive soil and/or water.

- **Exposure Class 3** : A humid environment with members exposed to frost.

- **Exposure Class 4** : A seawater environment with members completely or partially submerged in seawater, or in the splash zone, or exposed to saturated salt air in a coastal area, exposed to frost or not.

- **Exposure Class 5** : An aggressive chemical environment in gas, liquid or solid form. It includes members in aggressive soils.

5.2.2.3 Selection of reinforcing steel

(1) Where the reinforcing steel is bedded in mortar or in concrete with cover less than required in table 5.2, the reinforcing steel, and its level of protection, should be selected using table 5.1, according to the exposure class.

(2) Where the reinforcing steel is protected by concrete cover in accordance with table 5.2, carbon steel, without any other form of protection, may be used.

5.2.2.4 Cover to reinforcing steel

(1) Where the reinforcing steel is located in mortar in bed joints and is selected using table 5.1:

-the minimum depth of mortar cover from the reinforcing steel to the face of the masonry should be 15mm, as shown in figure 5.12;

- except from thin layer mortar, the mortar cover above and below reinforcement placed in bed joints should not be less than 2mm, as shown in figure 5.12;

- the reinforcing steel should be located so that the mortar cover is maintained.

(2) For filled-cavity or special bond construction, the minimum cover for reinforcing steel selected using table 5.1 should be 20mm mortar or concrete cover, as appropriate, or the diameter of the bar, whichever is the greater.

(3) Where unprotected carbon steel is used in concrete infill which provides the full protection, the concrete cover should be in accordance with table 5.2.

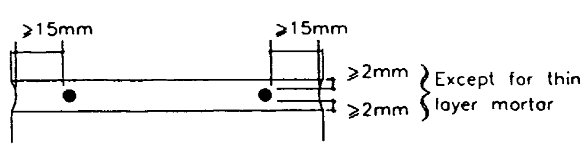


Figure 5.12 : Cover to reinforcing steel in bed joints.

(4) The cut ends of all bars, except those of 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.

Table 5.1 : Selection of reinforcing steel for durability.

<table>
<tr><th rowspan="2">Exposure class</th><th colspan="2">Minimum level of protection for reinforcing steel</th></tr>
<tr><th>Located in mortar</th><th>Located in concrete with cover less than required in table 5.2</th></tr>
<tr><td>1</td><td>Unprotected carbon steel
(see note 1)</td><td>Unprotected carbon steel</td></tr>
<tr><td rowspan="2">2</td><td>Carbon steel, heavily galvanised or with equivalent protection
(see note 2)</td><td rowspan="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)</td></tr>
<tr><td>Unprotected carbon steel, in masonry with a rendering mortar on the exposed face
(see note 3)</td></tr>
<tr><td rowspan="2">3</td><td>Austenitic stainless steel
(see note 4)</td><td rowspan="2">Carbon steel, heavily galvanised or with equivalent protection
(see note 2)</td></tr>
<tr><td>Unprotected carbon steel, in masonry with a rendering mortar on the exposed face
(see note 3)</td></tr>
<tr><td>4 & 5</td><td>Austenitic stainless steel
(see note 4)</td><td>Austenitic stainless steel
(see note 4)</td></tr>
</table>

Notes:

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.

2. Carbon steel should be galvanised with a minimum mass of zinc coating of $900 g/m^2$ or galvanised with a minimum mass of zinc coating of $60 g/m^2$ and provided with a bonded epoxy coating of at least $80 \mu m$ thickness, with an average of $100 \mu m$. See also 3.4.3.

3. The masonry mortar should be general purpose mortar, not less than M5, the side cover in figure 5.12 should be increased to 30mm and the masonry should be rendered with a rendering mortar in accordance with EN 998-1 with a minimum thickness of 15mm.

4. As an alternative to solid stainless steel an equivalent protection can be obtained by coating carbon steel with at least 1mm thickness of austenitic stainless steel.

Table 5.2 : Minimum concrete cover for unprotected carbon steel .

Exposure class	Minimum thickness of concrete cover			
	Water/cement ratio not greater than			
	0,65	0,55	0,50	0,45
	Cement content (kg/m³) not less than			
	260	280	300	300
	mm	mm	mm	mm
1	20	20	20	20
2	-	25	25	25
3	-	-	40	40
4	-	-	40	40
5	-	-	-	40

5.2.3 Minimum area of reinforcement

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

(2) Where reinforcement is provided in reinforced masonry members to enhance the strength, the area of main reinforcement should not be less than $\boxed{0,10}$ % of the cross-sectional area of the masonry, taken as the product of the effective width and the effective depth of the member under consideration. In walls incorporating prefabricated bed joint reinforcement to provide enhanced resistance to lateral loads, the area of such reinforcement should not be less than $\boxed{0,03}$ % of the gross cross-sectional area.

(3)Where reinforcement is provided in bed joints to help control cracking or to provide ductility, the area of the reinforcement should not be less than $\boxed{0,03}$ % and the vertical spacing should not normally exceed 600mm.

(4) In reinforced grouted cavity masonry members designed to span in one direction only, secondary reinforcement should normally be provided in the direction perpendicular to the main reinforcement principally to distribute stresses. The area of this secondary reinforcement should not be less than $\boxed{0,05}$ % of the product of the overall width and the effective depth.

(5) Secondary reinforcement may be used to assist in controlling cracking resulting from thermal and/or moisture movements; it may then be necessary for the area of secondary reinforcement to be greater than the minimum provided for stress distribution to allow for these effects or to satisfy 5.2.1(1)P.

(6) Secondary reinforcement may be omitted from pocket type walls and other similar constructions, unless required to tie the masonry to the infill concrete.

5.2.4 Size of reinforcement

(1)P The maximum size of reinforcement used shall be such as to enable proper embedment in the mortar or concrete infill. Prefabricated bed joint reinforcement shall have a minimum overall thickness as given in EN 845-3. Bars shall have a minimum nominal size of 6mm.

(2)P The maximum size of reinforcement used shall be such that the anchorage stresses, as given in 5.2.5, are not exceeded and the minimum cover to the reinforcement, as given in 5.2.2.4, is maintained.

5.2.5 Anchorage and laps

5.2.5.1 Anchorage of reinforcement

(1)P Reinforcement 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 may be achieved by straight anchorage, hooks, bends or loops as shown in figure 5.13. Alternatively stress transfer may be by means of an appropriate mechanical device.

(3) Straight anchorage or bends (see figure 5.13 (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.

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

$$l_b = \gamma_M \frac{\phi}{4} \frac{f_{yk}}{\gamma_s} \frac{1}{f_{bok}} \qquad (5.1)$$

where:

ϕ is the effective diameter of the reinforcing steel;

f_{yk} is the characteristic strength of reinforcing steel, obtained from 3.4.2;

f_{bok} is the characteristic anchorage bond strength of reinforcing steel, obtained from table 3.6 or 3.7 as appropriate;

γ_M is the partial safety factor for masonry or concrete infill, obtained from 2.3.3.2;

γ_S is the partial safety factor for steel, obtained from 2.3.3.2.

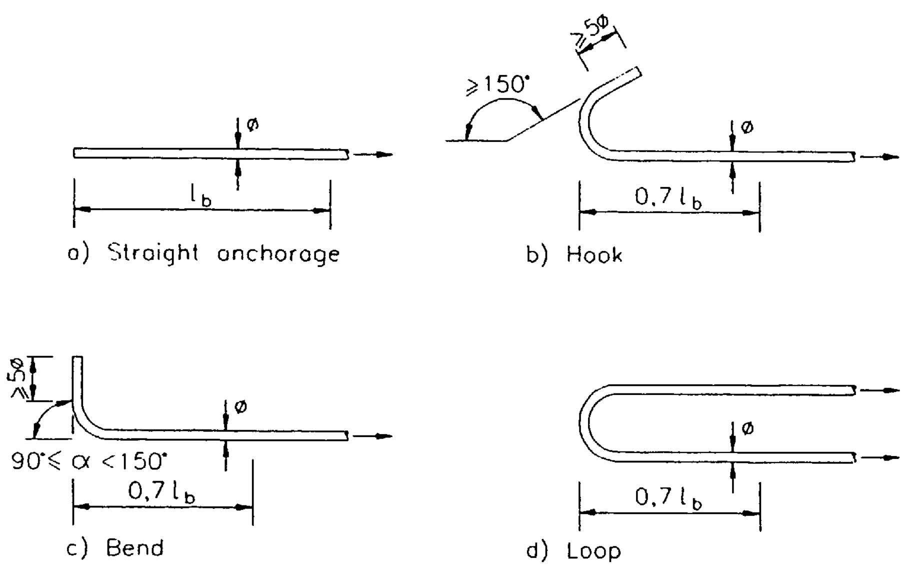


Figure 5.13 : Anchorage details

(5) For hooks, bends and loops (see Figure 5.13(b), (c) and (d)), the anchorage length for bars in tension may be reduced to $0,7\ l_b$.

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

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

- $0,3\ l_b$, or

- 10 bar diameters, or

- 100mm.

(ii) For a reinforcing bar 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 reinforcement should be provided evenly distributed along the anchorage length, with at least one reinforcing bar placed in the region of a curved anchorage (see figure 5.13(b), (c) and (d)). The minimum total area of transverse reinforcement should be 25% of the area of one anchored reinforcing 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.

5.2.5.2 Lapping of reinforcement

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

(2) The lap length between two reinforcing bars should be calculated in accordance with 5.2.5.1, based on the smaller of the two bars lapped.

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

- l_b for bars in compression and for reinforcing 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 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 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 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.

5.2.5.3 Anchorage of shear reinforcement

(1) The anchorage of shear reinforcement, including stirrups, should be effected by means of hooks or bends (see figure 5.13(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 a hook is extended by a straight length of 5 bar diameters or 50mm, whichever is the greater, and the curve of a bend is extended by a straight length of 10 bar diameters or 70mm, whichever is the greater.

5.2.5.4 Curtailment of tension reinforcement

(1) In any member subjected to bending, every reinforcing 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 reinforcement 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, reinforcement 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 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 reinforcement stops is greater than twice the shear force due to design loads, at that section;

- the continuing reinforcing bars at the section where the reinforcement stops provide double the area required to resist the bending moment at that section.

(2) Where there is little or no end fixity for a member in bending, at least 25% of the area of the tension reinforcement required at mid-span should be carried through to the support. This reinforcement may be anchored in accordance with 5.2.5.2, 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 reinforcement in a member subjected to bending should continue to the support and be provided with an anchorage equivalent to 20 times the bar diameter.

5.2.6 Shear reinforcement

(1) Where shear reinforcement is required by design (see 4.7.2.3), it should be provided throughout the span such that the minimum area of reinforcement is not less than $\boxed{0{,}1}$ % of the cross-sectional area of the masonry, taken as the product of the effective width and the effective depth of the section under consideration.

(2) The maximum spacing of stirrups, s , should be not greater than 0,75d or 300mm, whichever is the lesser,

(3) Where the arrangement of units in the masonry make it impossible to incorporate shear reinforcement, the design should be in accordance with 4.7.2.2.

5.2.7 Restraint of compression reinforcement

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

(2) In walls where the area of longitudinal reinforcing steel, A_s, is greater than 0,25% of the area of the reinforced masonry, A_{mr}, which is to include the area of any concrete infill, links should be provided if more than 25% of the design axial load resistance is to be used. In walls where A_s is not greater than 0,25% of the area of the reinforced masonry, or if less than 25% of the design axial load resistance is to be used, links need not 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 reinforced masonry wall;
- 300mm;
- 12 times the main bar diameters.

(4) Where links are provided, they should surround the main vertical reinforcing steel. Every vertical reinforcing corner bar 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.

5.2.8 Spacing of reinforcement

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

(2) In general, the clear distance between adjacent parallel reinforcement 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 reinforcement is concentrated in cores or pockets or when prefabricated bed joint reinforcement is used, the spacing of main and secondary tension reinforcement should not exceed 600mm.

(4) When the main reinforcement is concentrated in cores or pockets, for example, in pocket type walls, the spacing centre to centre of the concentrations of main reinforcement may be at the centres of the pockets, rather than at a maximum of 600mm. For the design of the masonry between the cores or pockets refer to 4.7.1.5. The total area of main reinforcement 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%.

5.2.9 Confined masonry

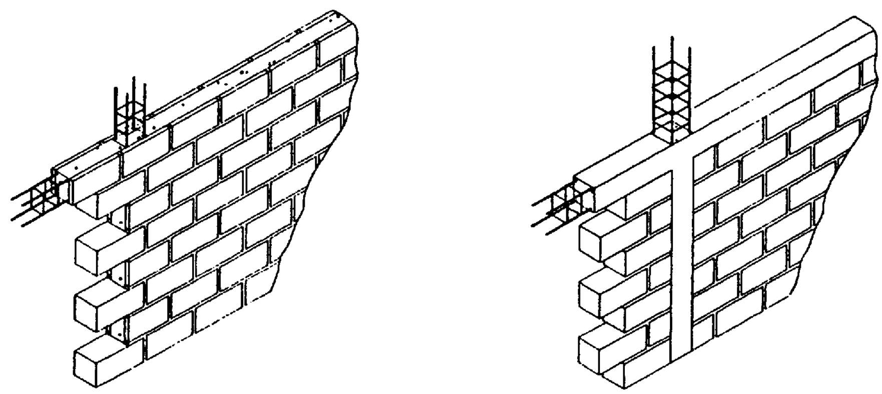
(1)P Confined masonry shall be constructed within reinforced vertical and horizontal members so that deformation of the masonry, when subjected to in plane actions, is restricted.

(2) Confined masonry should be constructed within horizontal reinforced concrete or reinforced masonry members provided at every floor level and within vertical reinforced concrete or reinforced masonry members provided at every intersection between walls and at both sides of every opening having an area of more than $\boxed{1,5}$ m^2. Additional reinforced concrete or reinforced masonry members may be required within the wall so that the maximum spacing, both horizontally and vertically, is $\boxed{4}$ m. Examples of confined masonry are shown in figure 5.14.

(3) Reinforced concrete or reinforced masonry confining members should have a cross-sectional area not less than $\boxed{0,02}$ m^2, with a minimum dimension of $\boxed{100}$ mm, and be provided with a minimum area of reinforcement of $\boxed{0,02}$ t mm^2, where t is the thickness of the wall, but not less than $\boxed{200}$ mm^2. The detailing of the reinforcement should be in accordance with 5.2.

(4)P The reinforced concrete or reinforced masonry members shall be cast after the masonry has been built and shall be anchored together.

(5) Where confined masonry in accordance with 4.9(1)P, (2) and (3) and where Group 1, Group 2a and Group 2b units are used, reinforcement bars not less than $\boxed{6}$ mm diameter and spaced no more than $\boxed{600}$ mm, duly anchored in the concrete infill and in the mortar joints, should be adopted.



a) Masonry confined within reinforced masonry beams and columns

b) Masonry confined within reinforced concrete beams and columns

Figure 5.14 : Examples of confined masonry.

(6) For non-seismic design, there is no other specific requirement. For seismic design, see ENV 1998-1-3.

5.3 Prestressing details

5.3.1 General

(1) In prestressed masonry members the prestressing tendons may be either bonded or unbonded with the masonry.

(2) An example in which prestressing tendons can be used in prestressed masonry is shown in figure 5.15.

5.3.2 Prestressing tendons

(1) Where prestressing tendons are placed in pockets, cores or cavities, that are to be filled with concrete infill, mortar or grout such that they are to form bonded construction with the masonry, the recommendations given in 5.2.2.1, 5.2.2.2 and 5.2.2.3 should be followed.

(2) Where tendons are used in prestressed masonry in open pockets, cores or cavities so as to be unbonded with the masonry, the designer should ensure that the form of construction, reinforcement type and any additional protective measures taken are adequate to provide the required level of durability protection to the prestressing tendons. Ducts for unbonded tendons should be suitably drained.

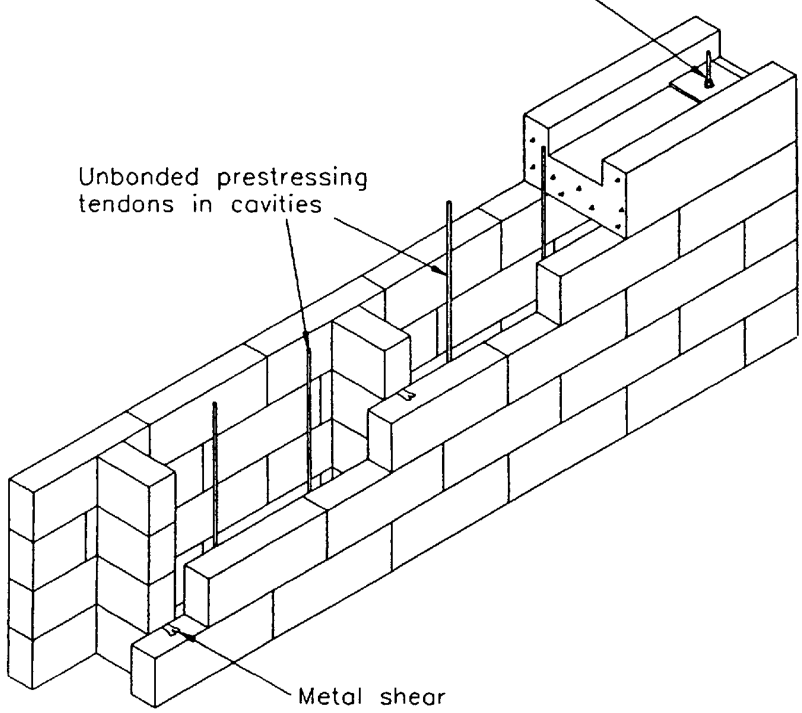


Figure 5.15 : Example of prestressed masonry.

5.4 Connection of walls

5.4.1 Interconnection of walls, floors and roofs

5.4.1.1 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 may be via 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. Alternatively a ring beam capable of transferring the resulting shear and bending action effects may be provided.

(3) The design lateral loads should be transferred between the walls and the interconnecting structural elements either by means of straps or by the frictional resistance between the walls and the floors or roofs.

(4)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, but being not less than 65mm.

5.4.1.2 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 there is little or no surcharge on the wall, 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 2m but, for buildings over 4 storeys high, the spacing should not be greater than 1,25m.

5.4.1.3 Connection by frictional resistance

(1)P Where concrete floors, roofs or ring beams bear directly on to 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 65mm, whichever is the greater, providing no sliding bearing is formed.

5.4.2 Connection between intersecting walls

5.4.2.1 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 5.1.4), or

- connectors or reinforcement extending into each wall so as to provide a strength equivalent to that of a bonded wall.

(3) It is recommended that intersecting walls should be erected simultaneously.

5.4.2.2 Cavity walls

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

(2)The number of wall ties connecting the two leaves of a cavity wall should not be less than that calculated according to 4.6.2.4, using the strength of the ties to be provided, nor less than $\boxed{2}$ ties/m², whichever is the greater.

Note: When connected 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 5.2.2.2).

(4)P 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.

(7) In seismic regions special consideration is necessary (see ENV 1998).

5.4.2.3 Double-leaf walls.

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

(2) The two leaves of a double-leaf wall should be tied together with connectors capable of transferring lateral loads between the two leaves, with a minimum cross-sectional area of 300mm²/m² of the double-leaf wall, for steel connectors, and with the connectors evenly distributed, being not less than $\boxed{2}$ connectors/m² 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 5.2.2.2)

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

5.4.2.4 Veneer walls

(1)P The ties for veneer walls shall be selected and used in such a way that no damage occurs to the wall.

5.5 Chases and recesses

5.5.1 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.

5.5.2 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 5.3, 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.

5.5.3 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 5.4. If these limits are exceeded, the vertical load, shear and flexural resistance should be checked by calculation.

5.6 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.

Table 5.3 : 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.

5.7 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 (see table 3.6) and the possible effects of internal stresses caused by vertical or lateral loading, so that the masonry does not suffer damage.

Table 5.4 : Sizes of horizontal and inclined chases in masonry, allowed without calculation.

Thickness of wall (mm)	Maximum depth (mm)	
	Unlimited length	Length ≤ 1 250mm
≤ 115mm	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.

(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 clay units;

- variations in temperature and humidity;

- insulation provided to the masonry;

- the provision of prefabricated bed joint reinforcement.

(4) 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.

(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.

5.8 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.

5.9 Particular details for seismic design

(1)P For particular details relative to seismic design, reference shall be made to ENV 1998-1-3.

5.10 Particular details for structural fire design

(1)P For particular details relative to structural fire design, reference shall be made to ENV 1996-1-2.

6 Construction

6.1 Masonry units

(1)P The properties of the masonry units shall be in accordance with the specification required by the designer.

(2)P The level of control of the manufacture of masonry units shall be as specified (see 3.1.1(2), (3) and (4)).

(3) If the masonry units are not supplied with a production specification stating the strength of the units and the required level of control, samples should be taken from site in accordance with EN 771 and tested in accordance with EN 772-1.

6.2 Handling and storage of masonry units and other materials

6.2.1 General

(1)P The handling and storage of materials for use in masonry shall be such that the materials are not damaged so as to become unsuitable for their purpose.

6.2.2 Storage of masonry units

(1) Masonry units should be carefully stacked on a suitable level surface and should be protected from rain, snow, and splashes of dirt and de-icing salts from passing vehicles.

(2) Masonry units should not be stacked on surfaces which contain harmful chemicals, clinker or ashes.

(3) Masonry units which are not frost resistant should be suitably protected.

6.2.3 Storage of materials for mortar and concrete infill

6.2.3.1 Binders

(1) Binders should be protected against interaction with moisture and air during transport and storage. Different types of binders should be stored separately so that mixing cannot occur.

6.2.3.2 Sand

(1) Loose sand should be stored on a hard base which permits free drainage of the stockpile and prevents the sand becoming contaminated. Different types of sand should be stored separately.

6.2.3.3 Factory made mortars, pre-batched mortars and pre-mixed lime sand mortars

(1) Dry factory made mortars, and pre-batched mortars containing a hydraulic binder, should be supplied and stored dry.

(2) Pre-batched mortars, where the materials are supplied to site separately, should be stored dry and in accordance with the manufacturer's instructions.

(3) Ready-to-use factory made mortars should be kept in covered containers, when not in use.

(4) Pre-mixed lime sand mortars should be stored on a hard base and must be covered to protect them against rain.

6.2.4 Storage and use of reinforcement

(1)P Steel reinforcing bars and prefabricated bed joint reinforcement shall be stored, bent and placed in position so that they do not suffer damage such that they will be unsuitable for their purpose.

(2)P The surface condition of reinforcement shall be examined prior to use and it shall be free from deleterious substances which may affect the steel, concrete or mortar or the bond between them.

(3)P Reinforcement shall be cut and bent in accordance with appropriate international or national standards so as to avoid the following:

- mechanical damage;
- rupture of welds in prefabricated bed joint reinforcement;
- surface deposits damaging to bond properties;
- lack of identification.

6.3 Mortar and concrete infill

6.3.1 General

(1)P The properties of mortar and concrete infill shall be as specified.

6.3.2 Site made mortar and concrete infill

(1)P Materials for mortar and concrete infill shall be measured into the specified proportions in clean suitable measuring devices.

(2) The materials may be measured into the specified proportions by weight or by volume.

(3) Materials should be mixed until a uniform mixture of the constituents has been obtained, using a suitable mechanical mixer unless hand mixing is permitted on projects where Category C level of execution applies (see 6.9). Mortar should not be contaminated during subsequent handling.

(4) Mortar and concrete infill should be used before the initial set takes place. Any mortar or concrete infill left after the intial set has commenced should be discarded and should not be reconstituted.

(5) In the proportioning of the materials for the concrete infill, account should be taken of the moisture absorption of the masonry units and mortar joints as these may reduce the water content. The concrete infill should also have a sufficient workability to fill perfectly the spaces into which it is put without segregation.

(6) Where admixtures are to be used, these should be added according to the specified requirements.

(7) Binders, aggregates, admixtures or water should not be added to mortars containing cement after they are discharged from the mixer.

6.3.3 Factory made mortars, pre-batched mortars, pre-mixed lime sand mortars and ready mixed concrete infill

(1)P Factory made mortars and pre-batched mortars, shall be used in accordance with the manufacturer's instructions, including mixing time and type of mixer.

(2) Pre-mixed lime sand mortars should be mixed with the binder according to 6.3.2.

(3) Ready-to-use factory made mortars should be used before the expiry of the period of use stated by the manufacturer; only any water that has evaporated should be replaced, and then only within the period of use stated by the manufacturer.

(4) Ready mixed concrete infill should be used according to the project specification.

6.3.4 Strength of mortar and concrete infill

6.3.4.1 Strength of mortar

(1) Where required by the Category of execution (see 6.9), specimens, should be prepared and tested in accordance with EN 1015-11.

6.3.4.2 Strength of concrete infill

(1) Where required by the Category of execution (see 6.9), specimens should be prepared and tested in accordance with the requirements of EN 206.

6.4 Construction of masonry

6.4.1 General

(1)P Masonry units shall be laid and bonded in accordance with the specification required by the designer.

(2) Masonry units should be cut neatly to suit dimensional requirements and to maintain a uniform appearance. Cutting of units should be kept to a minimum.

(3) Prior to being laid, masonry units should have a moisture content suitable for achieving the specified bond with the mortar. The units may be soaked in water to adjust the moisture content, when appropriate.

(4) The consistency of the mortar should be adjusted appropriately, taking into account the material properties of the units. Mortar with improved water retention may be used, when appropriate.

6.4.2 Mortar joints

6.4.2.1 General

(1) Joints shall be completed as specified.

(2) Joints should be of uniform appearance and thickness, unless specified otherwise.

Note: See ENV 1996-2 for advice concerning rain penetration.

(3)P Where perpend joints are specified to be unfilled, adjacent faces of the masonry units shall be closely abutted together.

(4) Where specified, joints may be left open, for example, for drainage or ventilation or shell bedding.

6.4.2.2 Thin layer joints

(1)P Where thin layer joints are specified, the masonry units shall be laid accurately in order to maintain uniform joints of the specified thickness.

6.4.2.3 Jointing

(1) When specified, the masonry surface should be finished by jointing. In jointing, the exposed surface of the mortar in the joints is worked while the mortar is still plastic to give a finished surface in order to achieve the durability and rain shedding characteristics of the wall.

(2)P Joints shall not be recessed to a depth greater than 5mm in walls of thickness less than 200mm without the consent of the designer.

6.4.2.4 Pointing

(1) When specified, the face joints should be raked out with clean sides, to a depth of at least 15mm, but no more than 15% of the wall thickness, and later refilled with mortar. The mortar used for pointing should have similar properties to the mortar used for bedding the units.

(2) Before pointing, loose material should be brushed out and, if necessary, the masonry should be wetted. When raking the joint, care should be taken to leave sufficient distance between any perforation and the surface mortar.

6.5 Connection of walls

(1)P Walls shall be bonded and connected together as specified.

(2)P Where elements of walls are required to act together, for example, in cavity walls, double-leaf walls or faced walls, they shall be bonded or connected together as specified.

(3) Wall ties for cavity walls should be placed so that they are adequately embedded, in accordance with EN 845-1, having regard to the design of the tie in both leaves and so that no water can be led from the facing to the backing leaf via the ties.

(4)P Veneer walls shall be connected to the backing structure as specified.

6.6 Fixing reinforcement

(1)P Reinforcement shall be fixed in accordance with the drawings, specification and tolerances

(2) Spacers and stirrups should be used where necessary to hold loose reinforcement in the required position to give the cover specificed to the reinforcement.

(3) Reinforcement should be lapped only where shown on the drawings.

(4) Where necessary, reinforcement should be tied together with wire to ensure that it does not move during mortar or concrete filling.

6.7 Protection of newly constructed masonry

6.7.1 General

(1) Newly constructed masonry should be protected against mechanical damage, (for example, shock) and the effects of weather.

(2) The tops of walls should be covered so as to prevent mortar being washed out of the joints by rain, to avoid efflorescence and lime bloom and to avoid damaging non-water resistance materials.

6.7.2 Curing of masonry

(1) Newly constructed masonry should not be allowed to dry out too rapidly. Suitable precautions should be taken to keep the masonry damp until it has adequate strength, particularly in unfavourable conditions such as low relative humidity, high temperature and/or strong air movements.

6.7.3 Protection against frost

(1)P Suitable precautions shall be taken to avoid damage to newly constructed masonry by frost action.

6.7.4 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.

6.8 Permissible deviations in masonry

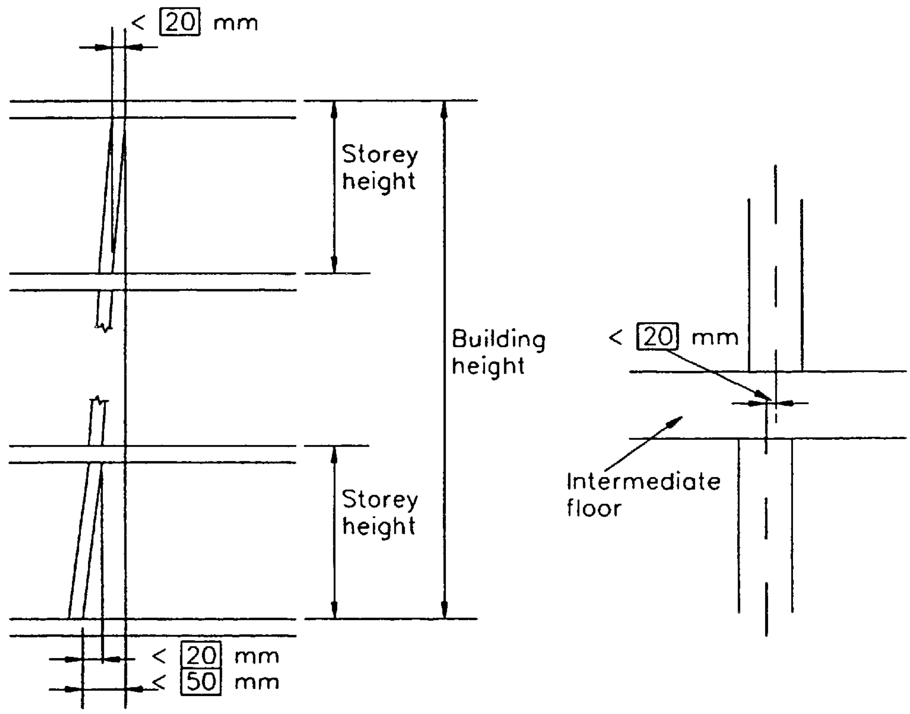
(1) Masonry should be built plumb and level with bed joints horizontal unless detailed otherwise by the designer.

(2) Maximum deviations that have been taken into account in this ENV 1996-1-1 are:

- **verticality**: $\boxed{20}$ mm in the height of a storey, or $\boxed{50}$ mm in the building height (see figure 6.1(a)), whichever is less;

- **vertical alignment**: $\boxed{20}$ mm maximum horizontal distance between the centre lines of walls above and below an intervening floor (see figure 6.1(b));

- **straightness**: $\boxed{5}$ mm per metre, with a maximum of $\boxed{20}$ mm per 10m.



a) Verticality b) Vertical alignment

Figure 6.1 : Maximum vertical deviations.

6.9 Category of execution

(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) Appropriately qualified and experienced personnel should be employed by the contractor for the supervision of the work.

(4) The specification should include requirements for workmanship no less onerous than the recommendations in this ENV 1996-1-1.

(5) The level of execution should be specified, in descending order of category, as being Category A, Category B or Category C.

Note: Guidance on the points that need to be considered in categorising execution is given in Annex G for those countries wishing to adopt more than one category of execution.

(6) Category C level of execution should not be used for reinforced or prestressed masonry, with the following exceptions:

- composite lintel construction, where the performance of the details of the construction have been proved by experience or testing and the mortar is mechanically mixed;

- masonry containing prefabricated bed joint reinforcement used solely to control cracking.

6.10 Other construction matters

6.10.1 Movement joints

(1)P Movement joints shall be formed as specified.

(2) The effective movement joint width should be sufficiently large to allow for the expected movement; the joint should be free of solid material, the outer surface being pointed with a flexible sealant if necessary.

6.10.2 Construction height

(1) The height of the masonry to be built in one day should be limited so as to avoid instability and overstressing the fresh mortar; the wall thickness, the type of mortar, the shape and density of the units and the degree of exposure to the wind should be taken into consideration in determining an appropriate limit.

6.10.3 Reinforced concrete filled cavity walls

(1) Where the cavity in a cavity wall is to be reinforced and filled with concrete, the cavity should be cleared of mortar droppings and debris before filling commences. Filling should be carried out in layers ensuring that all voids are filled and the concrete does not segregate. The sequence of operations should be such that the masonry has adequate strength to resist the pressure of the plastic concrete.

6.10.4 Reinforced walls incorporating pockets

(1) Where a wall uses an arrangement of units to provide pockets and is reinforced, the main reinforcement should be fixed sufficiently in advance of the masonry construction to allow it to proceed without hindrance. The cavities formed around the reinforcement, by the bonding pattern, should be filled with mortar or concrete as the works progress.

6.11 Prestressing steel and accessories

6.11.1 Storage of tendons

(1)P Tendons, sheaths and anchorage devices shall be protected from harmful influences during storage and also when placed in the structure, until after concreting has taken place.

(2) During storage of the tendons, the following should be avoided:

- chemical, electro-chemical or biological attack liable to cause corrosion;

- damage to tendons;

- contamination liable to affect the durability or bond properties of the tendons;

- deformation of the tendons, not provided for in the design;

- any unprotected storage, exposure to rain or contact with the ground;

- welding in the vicinity of tendons without the provision of special protection (from splashes).

(3) For sheaths, the following should be taken into consideration:

- local damage and corrosion inside should be avoided;

- water-tightness should be ensured.

6.11.2 Fabrication of tendons

(1)P The devices used in anchoring and coupling tendons shall be as specified in European Standards. The tendons shall be assembled and placed in position in accordance with these Standards. Sheaths shall be as specified in the project design documents.

(2) Particular consideration should be given to:

- maintaining identification marks on all materials;

- appropriate methods of cutting;

- straight entry into anchorages and couplers;

- when lifting by crane, avoiding any local crushing or bending of the tendons.

6.11.3 Placing of tendons

(1)P Placing of tendons shall be carried out in compliance with the criteria relating to:

- the concrete cover and the spacing of the tendons;

- the specified tolerances in respect of the position of the tendons, anchorages and couplers;

- the ease with which the surrounding concrete can be cast.

(2)P The tolerances required for the placing of the tendons shall be stated in the contract documents.

(3) Where sheaths are used, they should be fixed carefully according to the designer's specification of dimensions, spacers and supports. After placing sheaths in position, vents should be provided at both ends and at their high points, as well as at all points where air or water may accumulate. Such sheaths should be protected from penetration of extraneous materials until the completion of grouting.

6.11.4 Tensioning of tendons

(1)P Prestressing shall be in accordance with a pre-arranged programme.

(2)P Written instructions shall be provided on the prestressing procedure to be followed.

(3)P Workmen and staff engaged in prestressing shall be skilled and have had special training.

(4) During prestressing, suitable safety measures should be taken.

Annex A (informative)

A.1 Derivation of the value of the reduction factor for slenderness and eccentricity within the middle height of a wall

(1) In the middle one fifth of the wall height, by using a simplification of the general principles given in 4.4.1, 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 4.4.3, from:

$$\Phi_m = A_1 \, e^{-\frac{u^2}{2}} \tag{A.1}$$

where:

$$A_1 = 1 - 2 \frac{e_{mk}}{t} \tag{A.2}$$

$$u = \frac{\dfrac{h_{ef}}{t_{ef}} - 2}{23 - 37 \dfrac{e_{mk}}{t}} \tag{A.3}$$

and e_{mk} , h_{ef} , t and t_{ef} are as defined in 4.4.3, and *e* is the base of natural logarithms.

(2) The values of Φ_m derived from equation (A.1) are given in table A.1 for different eccentricities and are represented in graphical form in figure 4.2.

(3) For any modulus of elasticity E and characteristic compressive strength of unreinforced masonry f_k equations (A.1) and (A.2) may also be applied, however, with:

$$u = \frac{\lambda - 0{,}063}{0{,}73 - 1{,}17 \dfrac{e_{mk}}{t}} \tag{A.4}$$

where:

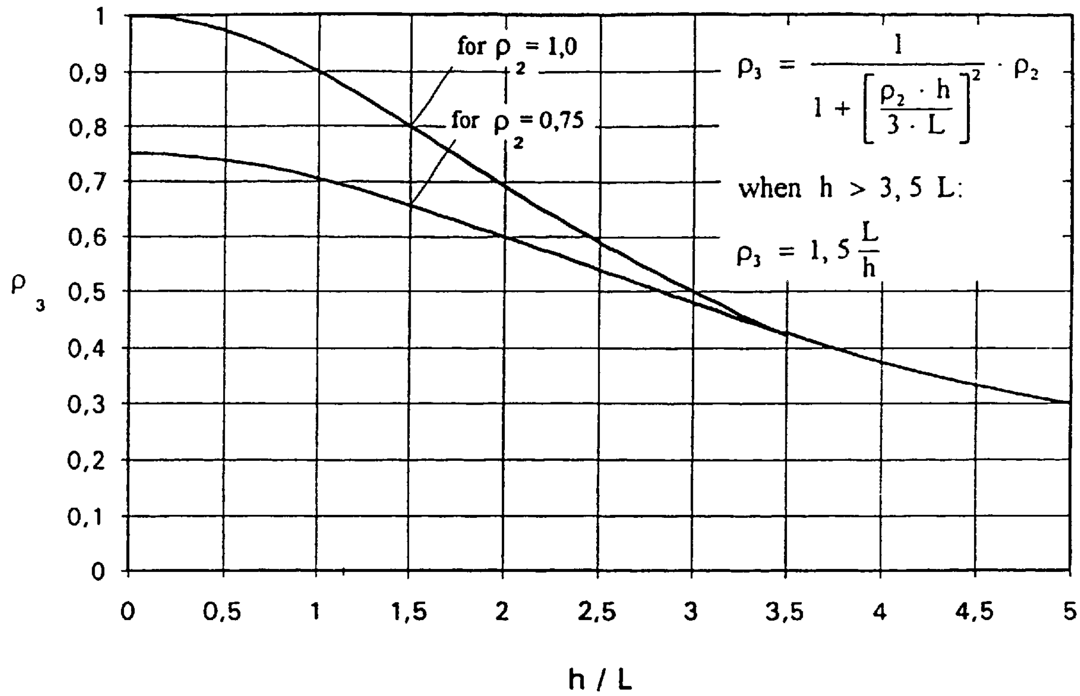
$$\lambda = \frac{h_{ef}}{t_{ef}} \sqrt{\frac{f_k}{E}} \tag{A.5}$$

Table A.1 : Capacity reduction factor, Φ_m , for E = 1 000 f_k

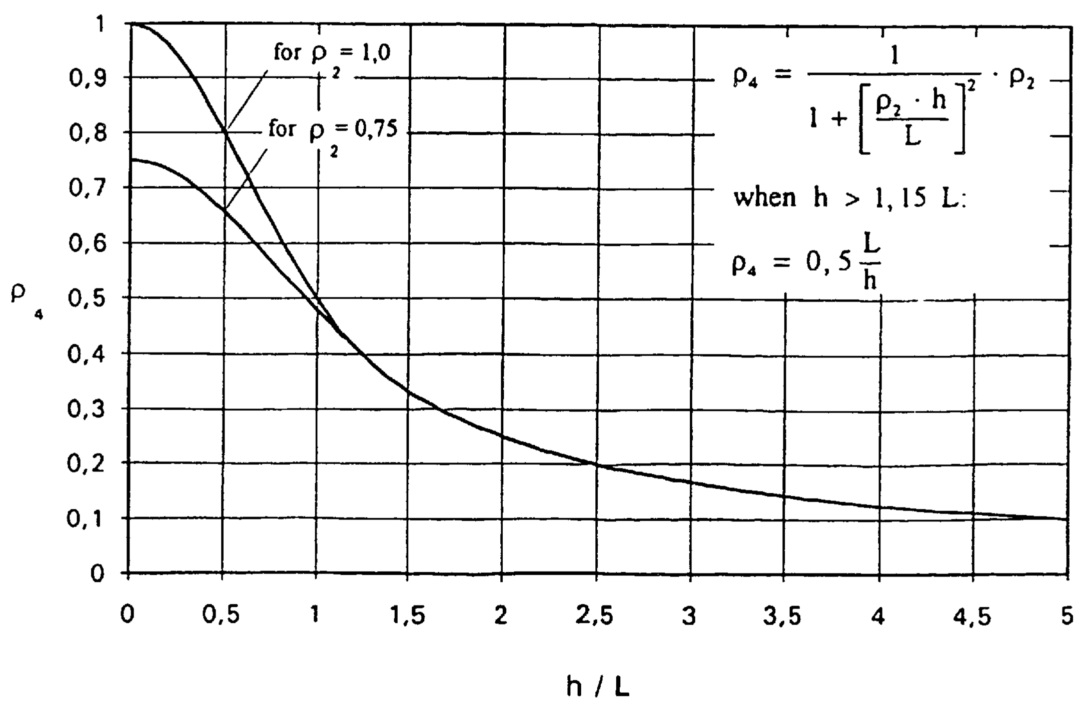
Slenderness ratio h_{ef}/t_{ef}	Eccentricity e_{mk}						
	0,05 t	**0,10 t**	**0,15 t**	**0,20t**	**0,25t**	**0,30t**	**0,33t**
0	0,90	0,80	0,70	0,60	0,50	0,40	0,34
1	0,90	0,80	0,70	0,60	0,50	0,40	0,34
2	0,90	0,80	0,70	0,60	0,50	0,40	0,34
3	0,90	0,80	0,70	0,60	0,50	0,40	0,34
4	0,90	0,80	0,70	0,60	0,49	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	o,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 B (informative)

B.1 Graph showing values of ρ_3 using equations 4.13 and 4.14



B.2 Graph showing values of ρ_4 using equations 4.15 and 4.16



Annex C (normative)

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. 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_2I_2/h_2 instead of E_1I_1/h_1 in the numerator.

$$M_1 = \frac{\dfrac{nE_1I_1}{h_1}}{\dfrac{nE_1I_1}{h_1}+\dfrac{nE_2I_2}{h_2}+\dfrac{nE_3I_3}{l_3}+\dfrac{nE_4I_4}{l_4}}\left[\frac{w_3 l_3^2}{12}-\frac{w_4 l_4^2}{12}\right] \tag{C.1}$$

where:

n is the member stiffness factor, 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 table 2.2, unfavourable effect;

w_4 is the design uniformly distributed load on member 4, using the partial safety factors from table 2.2, 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.

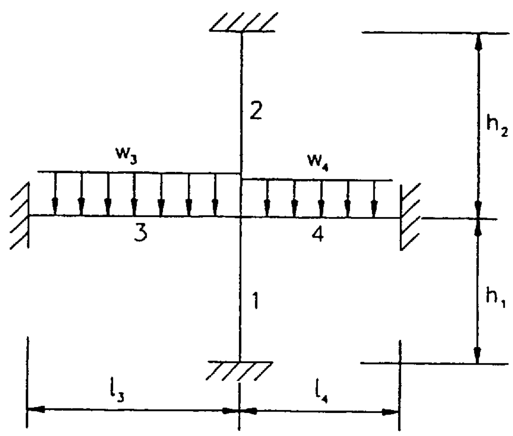


Figure C.1 : Simplified frame diagram

(2) 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 $(1 - k/4)$, 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 and k is not taken to be greater than 2;

where:

$$k = \frac{\dfrac{E_3\, I_3}{l_3} + \dfrac{E_4\, I_4}{l_4}}{\dfrac{E_1\, I_1}{h_1} + \dfrac{E_2\, I_2}{h_2}} \qquad \text{(C.2)}$$

(3) If the eccentricity calculated in accordance with paragraph (2) above is greater than 0,4 times the thickness of the wall, or the design vertical stress is 0,25N/mm² or less, the design may be based on paragraph (4) below.

(4) The eccentricity of loading to be used in design may be based on the design load being resisted by the minimum required bearing depth, but not based on a bearing depth of 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 clause 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.

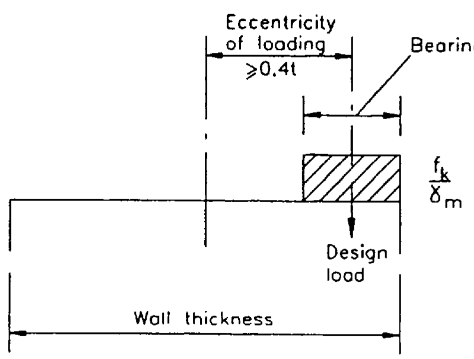
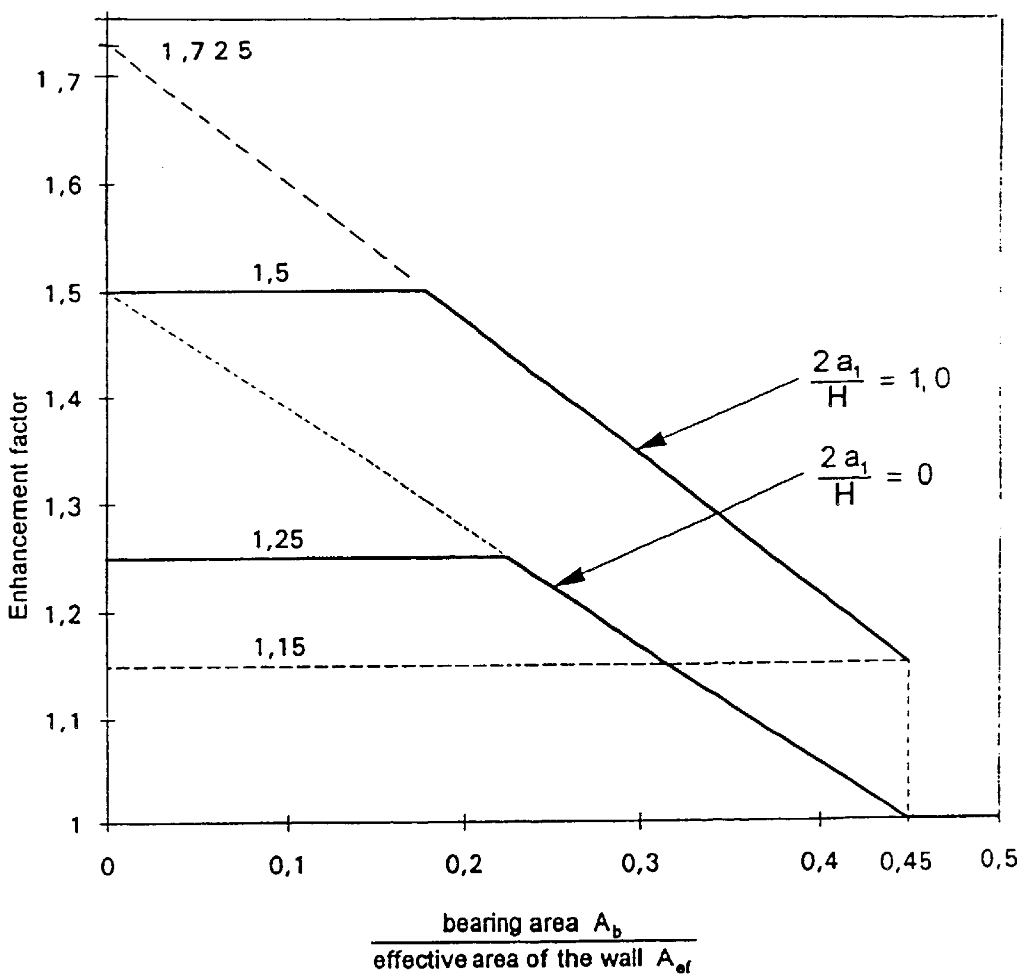


Figure C.2 : Eccentricity obtained from design load resisted by stress block.

Annex D (informative)

D.1 Graph showing the enhancement factor as given in 4.4.8 : Concentrated loads under bearings.



Annex E (normative)

E.1 An empirical method for designing basement walls subjected to lateral earth pressure

(1) When the following conditions are fulfilled, detailed verification of the design for a basement wall for the effect of soil pressure is not required:

- the clear height of the basement wall, $h \leq 2\,600$mm, and the wall thickness, $t \geq 200$mm;

- the floor over the basement acts as a diaphragm and is capable of withstanding the forces resulting from the soil pressure;

- the imposed load on the ground surface in the area of influence of the soil pressure on the basement wall, P_s , does not exceed 5kN/m² and no concentrated load within 1 500mm of the wall exceeds 15kN;

- the ground surface does not rise, and the depth of fill does not exceed the wall height;

- the vertical design load on the wall per unit length, N, which results from permanent loading at the mid-height of the fill, satisfies the following relationships (see also figure E.1):

(i) when $b_c \geq 2h$:

$$\frac{t\, f_k}{3\, \gamma_M} \geq N \geq \frac{\rho_e\, h\, h_e^2}{20\, t} \qquad \text{(E.1)}$$

where:

b_c is the distance apart of cross walls or other buttressing elements;

h is the clear height of the basement wall;

h_e is the depth of soil retained by the wall;

t is the wall thickness;

ρ_e is the bulk density of the soil;

f_k is the characteristic compressive strength of the masonry, obtained from 3.6.2;

γ_M is the partial safety factor for material obtained from 2.3.3.2;

(ii) when $b_c \leq h$:

$$\frac{t\ f_k}{3\ \gamma_M} \geq N \geq \frac{\rho_e\ h\ h_e^2}{40\ t} \qquad \text{(E.2)}$$

where the symbols are as defined in (i) above.

(iii) For values of b_c between h and 2h, linear interpolation between the values obtained from equations (E.1) and (E.2) is permitted;

- there is no hydrostatic pressure;

- no slip plane is created by a damp proof course.

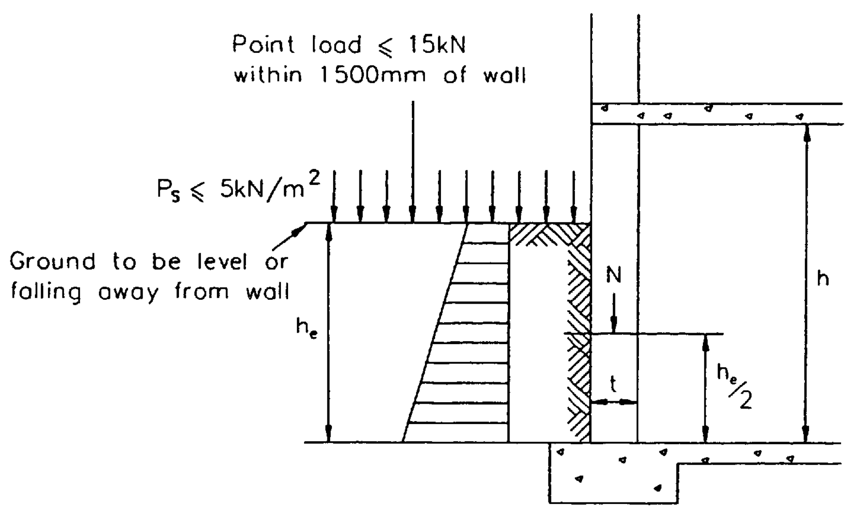


Figure E.1 : Design loads for basement walls.

Annex F (informative)

F.1 Verification of reinforced masonry cantilever walls subject to bending.

(1) The design of reinforced masonry cantilever walls subject to bending should be in accordance with the assumptions in 4.7.1.1 and the design method given in 4.7.1.6(2), (3), (4), (6) and (8).

(2) In determining the moment of resistance of the section, a rectangular stress block, as indicated in figure 4.10, may be assumed but with the design compressive strength f_k/γ_M taken over the full depth to the neutral axis, x , with x not to be taken greater than d/2. The symbols are as defined in paragraph (3) below.

(3) The design moment of resistance, M_{Rd} , calculated in accordance with paragraph (2) above, should not be taken to be greater than:

$$\frac{0{,}4\ f_k\ b\ d^2}{\gamma_M} \qquad \text{(F.1)}$$

where:

f_k is the characteristic compressive strength of masonry;

b is the width of the section;

d is the effective depth of the section;

γ_M is the partial safety factor for masonry.

(4) In the case of flanged members, the design should be in accordance with 4.7.1.6(7).

Annex G (informative)

G.1 Points to be considered in categorising execution

(1) In defining[16)] the relevant conditions to be used for the Categories of execution given in Table 2.3, the following points should be considered:

- appropriately qualified and experienced personnel, employed by the contractor, for supervision of the work;

- appropriately qualified and experienced personnel, independent of the contractor's staff, for 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.

[16)] The definitions of categories of execution may be given in the National Application Documents when considered necessary.