Design of Masonry Structures

According Eurocode 6

Prof. em. Dr.-Ing. Wieland Ramm

Technical University of Kaiserslautern

0 Introduction

Definition of masonry:

Structural components consisting of masonry units laid in a bonding arrangement . Masonry can consist of artificial or natural units, which are normally laid with mortar.

(Masonry without mortar is not dealt with in EC 6)

Masonry is normally used for components subjected to compressive loading:

- walls
- columns

have to bear in vertical direction

- arches
- vaults
- domes

span across spaces and rooms

Masonry walls also have a limited capacity to support horizontal loads and bending moments.

Masonry is not only used for pure masonry buildings, but often and successfully in mixed structures.

During the last decades the efficiency of masonry has considerably improved by

- higher allowable stresses,
- refined possibilities of design.

This requires:

- more precision in analysis,
- more exact constructions,
- more exact production.

Therefore the design of masonry structures is today a task of civil engineering.

EC 6: Part of the Eurocode programme:

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.

These Structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

Objectives of the Eurocodes:

Harmonization of technical rules for the design
of building and civil engineering works.

Initiation by:

CEC = Commission of the European Communities

In 1990 the work was handed to:

CEN = European Committee for Standardisation

CEN members:

National standards bodies of:

Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

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

Establishing-procedure of an Eurocode:

– First CEN approves an European Prestandard (ENV) as a prospective standard for provisional application.

– CEN members are required to make the ENV available at national level.

– Members are requested to submit their comments.

– Finally and after necessary improvements the ENV will be converted into an European Standard (EN).

Main advantages:

- harmonization of building standards in Europe
- standardization of the basic requirements
  and of the design concept for the different types of construction
- equalization of the safety levels in respect of:
  - the different combinations of actions
  - the different types of buildings and building elements
- higher allowable stresses in some cases
- more flexibility in the design practice

On the other hand:

Full use of new possibilities demands:

- a higher level of knowledge and engineering education
- an increasing amount of personal work
- the availability of adequate software

Indicative values:

– certain safety elements, identified by ☐ (“boxed values”)

– may be substituted by national authorities
for use in national application

National Application Documents (NAD`s) :

– additional rules to be met in conjunction with the Eurocodes

– define the alternitive values, if there are national changes
with indicative values

– give substituting definitions,
if supporting European or international standards
are not available by the time.

Distinction between principles and application rules, depending on the character of the individual clauses:

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.

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

The application rules are generally recognised rules which follow the principles and satisfy their requirements.

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.

The application rules are all clauses not indicated as being principles.

1 General

1.1 Parts of Eurocode 6 (ENV 1996)

Design of masonry structures

Part 1-1:	General rules for buildings – Rules for reinforced and unreinforced masonry.
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 Scope

1.2.1 Scope of Eurocode 6

– design of building and civil engineering works in
  unreinforced,
  reinforced,
  prestressed,
  confined masonry

– concerned only with the requirements for
  resistance,
  serviceability,
  durability of structures

– not concerned with other requirements,
  so for thermal or sound insulation

– does not cover the special requirements
  of seismic design (given in Eurocode 8)

1.2.2 Scope of Part 1-1 of Eurocode 6

- **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,
  - autoclaved aerated concrete units,
  - manufactured stone units,
  - dimensioned natural stone units.

- **Detailed rules which are mainly applicable to ordinary buildings**

  subjects dealt with in Part 1-1:

  - Section 1: General.
  - Section 2: Basis of design.

  common to all Eurocodes, with the exception of some additional clauses which are required for masonry.

  - Section 3: Materials.
  - Section 4: Design of masonry.
  - Section 5: Structural detailing.
  - Section 6: Construction.

1.3 Special terms used in ENV 1996-1-1

1.3.1 Masonry

Masonry:

An assemblage of masonry units laid in a specified pattern and joined together with mortar.

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.

Prestressed masonry:

Masonry in which internal compressive stresses have been intentionally induced by tensioned reinforcement.

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).

Masonry bond:

Disposition of units in masonry in a regular pattern to achieve common action.

1.3.2 Strength of masonry

Characteristic strength of masonry:

The value of strength corresponding to a 5 % fractile of all strength measurements of the masonry.

Compressive strength of masonry:

The strength of masonry in compression without the effects of platten restraint, slenderness or eccentricity of loading.

Shear strength of masonry:

The strength of masonry subjected to shear forces.

Flexural strength of masonry:

The strength of masonry in pure bending.

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.

1.3.3 Masonry units

Masonry unit:

A preformed component, intended for use in masonry construction.

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.

Bed face:

The top or bottom surface of a masonry unit when laid as intended.

Frog:

A depression, formed during manufacture, in one or both bed faces of a masonry unit.

Hole:

A formed void which may or may not pass completely through a masonry unit.

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.

Web:

The solid material between the holes in a masonry unit.

Shell:

The peripheral material between a hole and the face of a masonry unit.

Gross area:

The area of a cross-section through the unit without reduction for the area of holes, voids and re-entrants.

Compressive strength of masonry units:

The mean compressive strength of a specified number of masonry units.

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.

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.

1.3.4 Mortar

Mortar:

A mixture of inorganic binders, aggregates and water, together with additions and admixtures if required.

General purpose mortar:

A mortar for use in joints with a thickness greater than 3 mm and in which only dense aggregates are used.

Thin layer mortar:

A designed mortar for use in joints between 1 mm and 3 mm in thickness.

Lightweight mortar:

A designed mortar with a dry hardened density lower than 1500 kg/m^3.

Designed mortar:

A mortar designed and manufactured to fulfil stated properties and subjected to test requirements.

Prescribed mortar:

A mortar made in predetermined proportions, the properties of which are assumed from the stated proportion of the constituents.

Factory made mortar:

A mortar batched and mixed in a factory and supplied to the building site.

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.

Site-made mortar:

A mortar composed of primary constituents batched and mixed on the building site.

Compressive strength of mortar:

The mean compressive strength of a specified number of mortar specimens after curing for 28 days.

1.3.5 Concrete infill

Concrete infill:

A concrete mix of suitable consistency and aggregate size to fill cavities or voids in masonry.

1.3.6 Reinforcement

Reinforcing steel:

Steel reinforcement for use in masonry.

Bed joint reinforcement:

Steel reinforcement that is prefabricated for building into a bed joint.

Prestressing steel:

Steel wires, bars or strands for use in masonry.

1.3.7 Ancillary components

Damp proof course:

A layer of sheeting, masonry units or other material used in masonry to resist the passage of water.

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.

Strap:

A device for connecting masonry members to other adjacent components, such as floors and roofs.

1.3.8 Mortar joints

Bed joint:

A mortar layer between the bed faces of masonry units.

Perpend joint:

A mortar joint perpendicular to the bed joint and to the face of wall.

Longitudinal joint:

A vertical mortar joint within the thickness of a wall, parallel to the face of the wall.

Thin layer joint:

A joint made with thin layer mortar
having a maximum thickness of 3 mm.

Movement joint:

A joint permitting free movement in the plane of the wall.

Jointing:

The process of finishing a mortar joint as the works proceeds.

Pointing:

The process of filling and finishing raked out mortar joints.

1.3.9 Wall types

Load-bearing wall:

A wall of plan area greater than 0,04 m^2, or one whole unit if Group 2a, Group 2b or Group 3 units of plan area greater than 0,04 m^2 are used, primarily designed to carry an imposed load in addition to its own weight.

Single-leaf wall:

A wall without a cavity or continuous vertical joint in its plane.

Cavity wall:

A wall consisting of two parallel single-leaf walls, effectively tied together with wall ties or bed joint reinforcement, with either one 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.

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.

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.

Faced Wall:

A wall with facing units bonded to backing units so as to result in common action under load.

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.

Veneer wall:

A wall used as a facing but not bonded or contributing to the strength of the backing wall or framed structure.

Shear wall:

A wall to resist lateral forces in its plane.

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.

Non-loadbearing wall:

A wall not considered to resist forces such that it can be removed without prejudicing the remaining integrity of the structure.

1.3.10 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.

1.4 Symbols used in ENV 1996-1-1

1.4.1 Particular material-independent symbols used are as follows:

F	action
G	permanent action
P	prestressing action
Q	variable action
A	accidental action
W	value of wind action
E	action effect
S	value of an internal action effect
R	resistance capacity
X	value of a material property
C	nominal value, or function, of certain properties of materials
a	value of geometrical data
γ	partial safety factor
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

1.4.2 Particular material-dependent symbols used for masonry are as follows:

A	area of a wall
I	second moment of area of a member
N	vertical load per unit length
M	moment
V	shear force
E	modulus of elasticity
G	shear modulus
e	eccentricity
t	thickness of a wall or leaf
f	compressive strength of masonry
f_v	shear strength of masonry
f_x	flexural strength of masonry
F	flexural strength class
f_b	normalized compressive strength of a masonry unit
f_m	mean compressive strength of mortar
M	mortar compressive strength grade

1.4.3 Indices

k	characteristic value
d	design value
inf	lower value
sup	upper value
nom	nominal value
ef	effectiv value
R	resistance
S	action, load

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.

(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,
- 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.

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: $\psi_o Q_k$,
- 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:

$$F_d = \gamma_F F_k$$

(2) Specific examples are:

$$G_d = \gamma_G G_k$$

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

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

$$P_d = \gamma_P P_k$$

where γ_F, γ_G, γ_Q, γ_A and γ_P are the partial safety factors for the action.

(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 then:

$$G_{d,sup} = \gamma_{G,sup} \quad G_k$$

$$G_{d,inf} = \gamma_{G,inf} \quad G_k$$

– where upper and lower characteristic values of permanent actions are used then:

$$G_{d,sup} = \gamma_{G,sup} \quad G_{k,sup}$$

$$G_{d,inf} = \gamma_{G,inf} \quad G_{k,inf}$$

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.

2.2.3.2 Design values

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

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

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:

$$R_d = R\ (X_d, a_d, \ldots)$$

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 situations and load cases shall be considered.

2.3.2 Ultimate limit states

2.3.2.1 Verification conditions

Limit state of static equilibrium

(or of gross displacements or deformations of the structure):

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

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

Limit state of rupture

(or excessive deformation of a section, member or connection):

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

S_d is the design value of an internal force or moment
(or of a respective vector of several internal forces or moments)

R_d is the corresponding design resistance.

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 the paragraph above.

2.3.2.2 Combinations of actions

– Persistent and transient design situations:

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

– Accidental design situations:

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

In both expressions prestressing and indirect actions shall be
introduced where relevant.

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.

(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.

(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 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

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)		Prestressing (γ_P)
		One with its characteristic value	Others with their combination value	
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).				

For accidental design situations the partial safety factor for variable actions is equal to $\boxed{1,0}$.

(3) By adopting the γ values given in table 2.2, the following simplified combinations may be used:

– considering only the most unfavourable variable action:

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

– considering all unfavourable variable actions:

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

whichever gives the larger value.

(4) Where 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

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

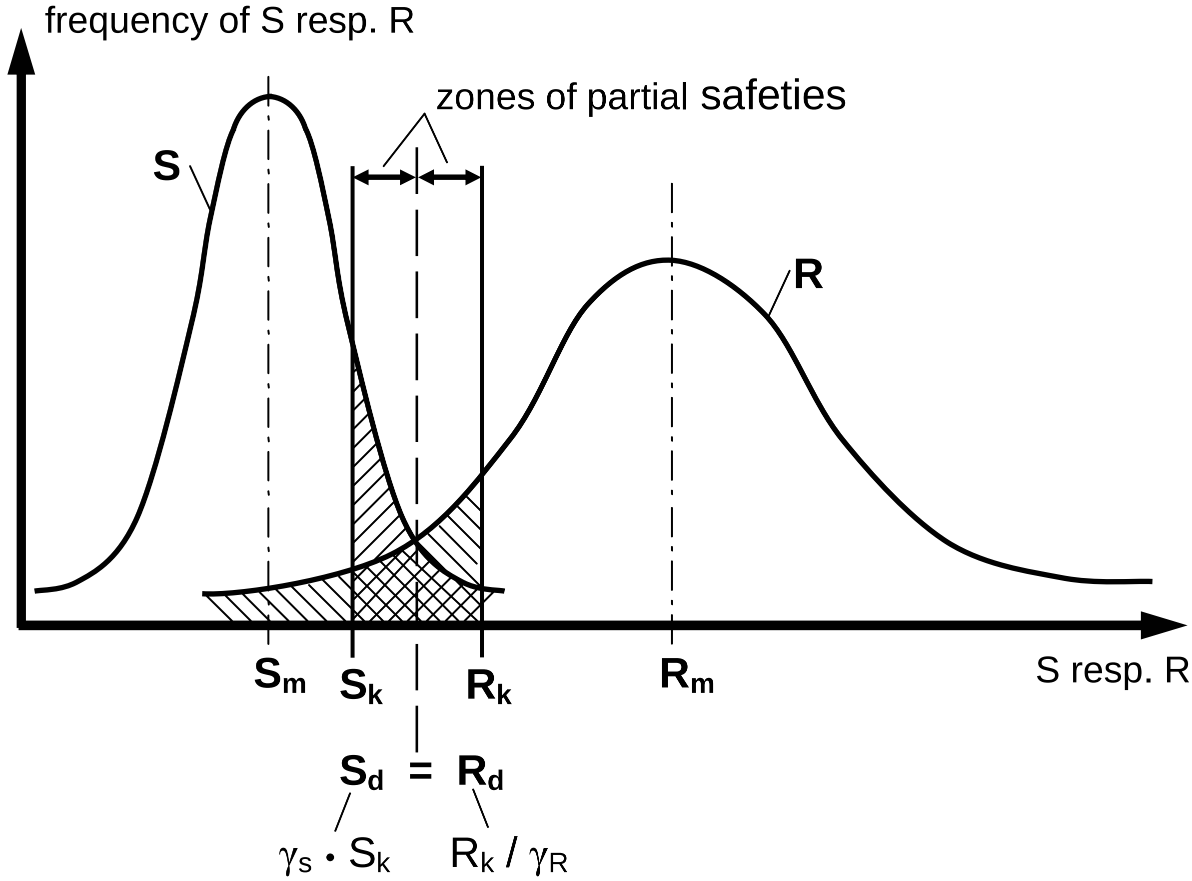
γ_M			Category of execution (see 6.9)		
			A	B	C
Masonry (see note)	Category of manufacturing control of masonry units (see 3.1)	I	$\boxed{1,7}$	$\boxed{2,2}$	$\boxed{2,7}$
		II	$\boxed{2,0}$	$\boxed{2,5}$	$\boxed{3,0}$
Anchorage and tensile and compressive resistance of wall ties and straps			$\boxed{2,5}$	$\boxed{2,5}$	$\boxed{2,5}$
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 **When verifying the stability in the case of accidental actions,**

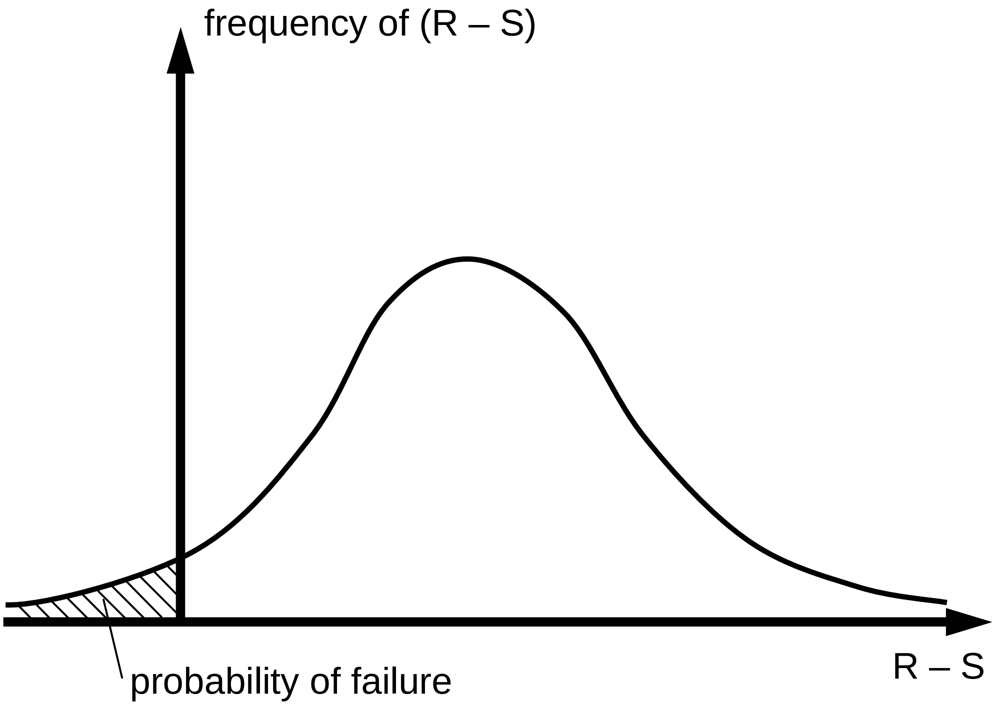
γ_M for masonry shall be taken as $\boxed{1,2}$, $\boxed{1,5}$ and $\boxed{1,8}$ for categories A, B and C of levels of execution respectively, γ_M for anchorage and tensile and compressive resistance of wall ties and straps, and for anchorage bond of reinforcing steel, shall be taken as given in table 2.3 and γ_s for steel shall be taken as $\boxed{1,0}$.

Insertion:

On the background of the semi probabilistic safety concept, looking at only one action S and one resistant value R:



The difference R – S in an actual case indicates the actual margin of safety.
As the distributions of S and R are overlapping,
it is possible, that R – S becomes < 0,
which means failure of the structure



The safety factors have to be chosen such
that the probability of failure is small enough to be tolerated.

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 below.

(2)P Three combinations of actions for serviceability limit states are defined:

– Rare combination:

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

– Frequent combination:

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

– Quasi-permanent combination:

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

(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:

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

– considering all unfavourable variable actions:

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

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.

3 Materials

3.1 Masonry Units

3.1.1 Types of masonry units

– Clay units – Calcium silicate units – Aggregate concrete units (dense and lightweight aggregate) – Autoclaved aerated concrete units – Manufactured stone units – Dimensioned natural stone units	in accordance with EN 771, Parts 1-6

Classification in terms of manufacturing control:

- Category I:
  - specified mean compressive strength,
  - probability of failing is not exceeding $\boxed{5}$ %,
  - tested in accordance with EN 771 and EN 772-1.
- Category II:
  - mean compressive strength complies with the declaration in accordance with EN 771,
  - additional requirements for category I are not met.
- Natural stone units should be considered as Category II units.

Masonry units should be grouped as Group 1, Group 2a, Group 2b or Group 3:

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.

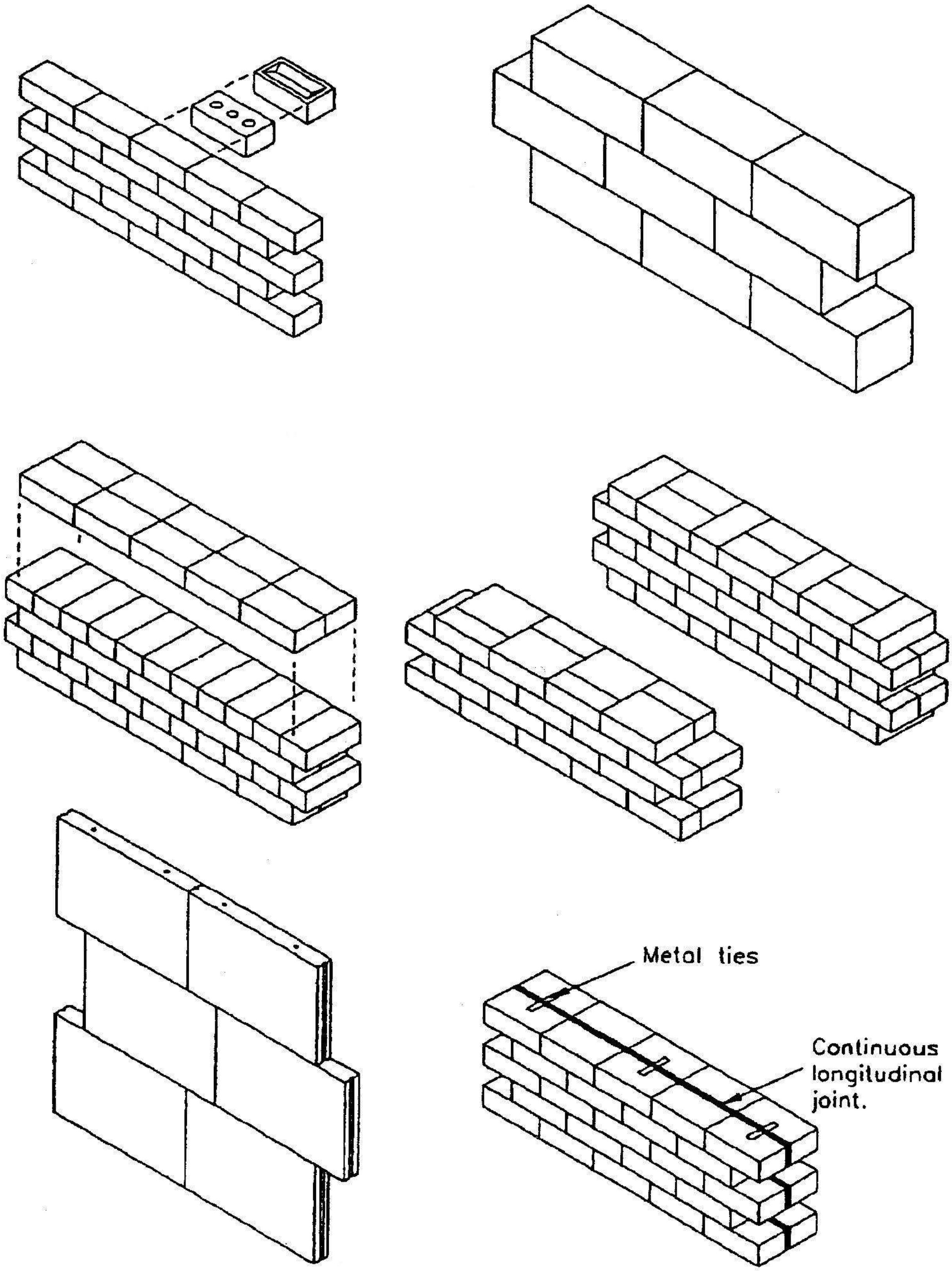


Figure 5.8: Examples of bonding arrangements using Group 1 masonry units.

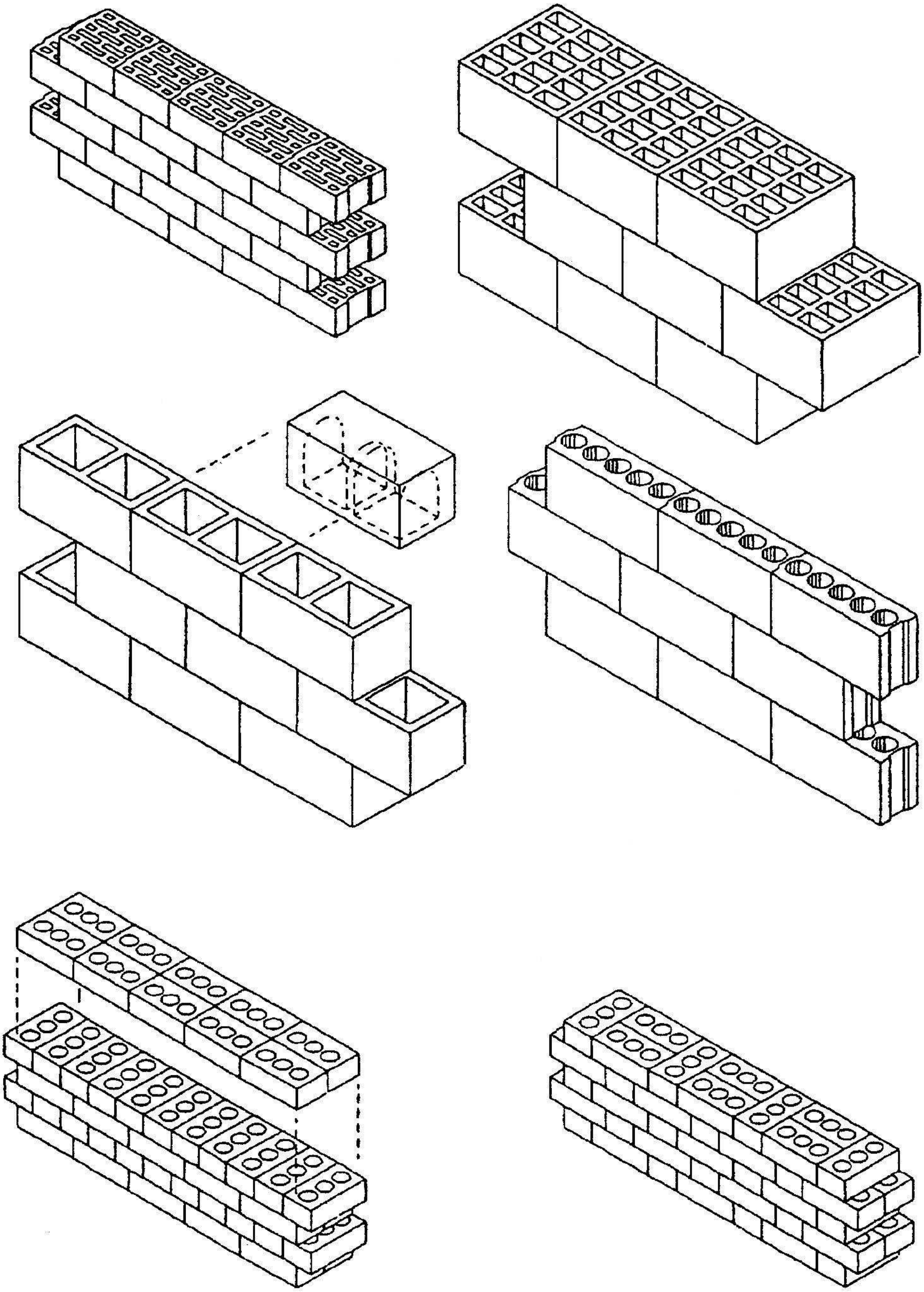


Figure 5.9: Examples of bonding arrangements using Group 2a and Group 2b masonry units.

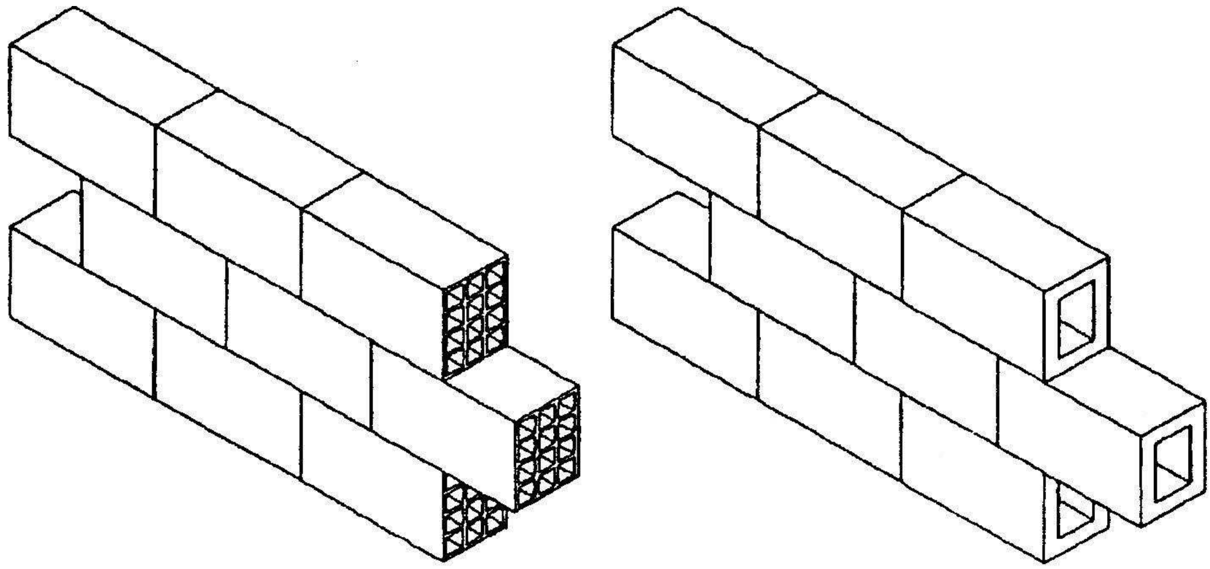


Figure 5.10: Examples of bonding arrangements using Group 3 masonry units.

5.1.3 Minimum thickness of walls

(1) The thickness of load bearing 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:

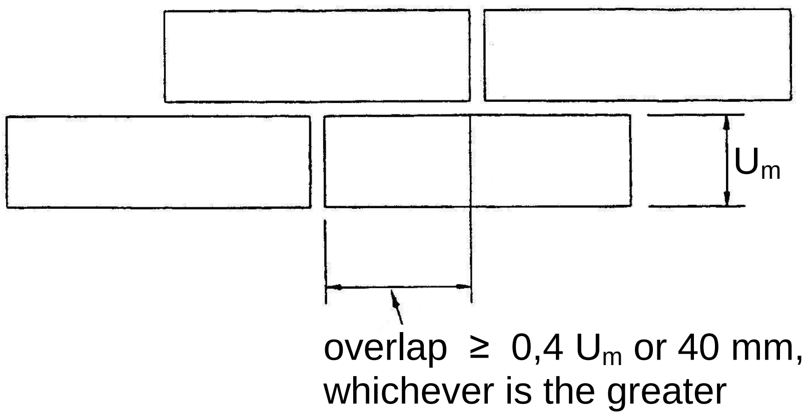


Figure 5.7: Overlap of masonry units.

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.

3.1.2 Properties of masonry units

3.1.2.1 Compressive strength of masonry units

- The normalized compressive strength f_b shall be used in design.
- Compressive strength is tested in accordance with EN 772-1:
  - The tests are carried out with a certain number of single units,
  - When quoted as the mean strength,
    it should be converted to f_b by multiplying by the factor δ
    to allow for the height and width of the units.
    (δ is a form factor, as the test results depend on the relation of the height to the horizontal dimension of the units).
  - When quoted as the characteristic strength, it should be converted first to the mean strength, using a conversion factor based on the coefficient of variation.

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.

3.2 Mortar

3.2.1 Types of mortar

Different kinds of preparation:

- factory made mortar
- pre-batched mortar
- side mixed mortar

Classified types of mortar:

- general purpose mortar
- thin layer mortar:

  use for bed joints with a nominal thickness
  of 1 mm to 3 mm

- lightweight mortar:

  made by using perlite, pumice, expanded clay,
  expanded shale or expanded glass

Classification according to their designed compressive strength:

for example: M5
↑
compressive strength [N/mm^2]

3.2.2 Properties of mortar

3.2.2.1 Compressive strength of mortar

Symbol: f_m

Specification of mortars:

- General purpose mortars:
  - by designed mixes, which achieve the specified compression strength f_m in accordance with EN 1015-11
  - by prescribed mixes, manufactured from specified proportions of constituents, for example:
    1:1:5 cement : lime : sand,
    which may be assumed to achieve the relevant value of f_m.

- Thin layer mortars and lightweight mortars:
  - specification always by designed mixes,
  - M5 or stronger.

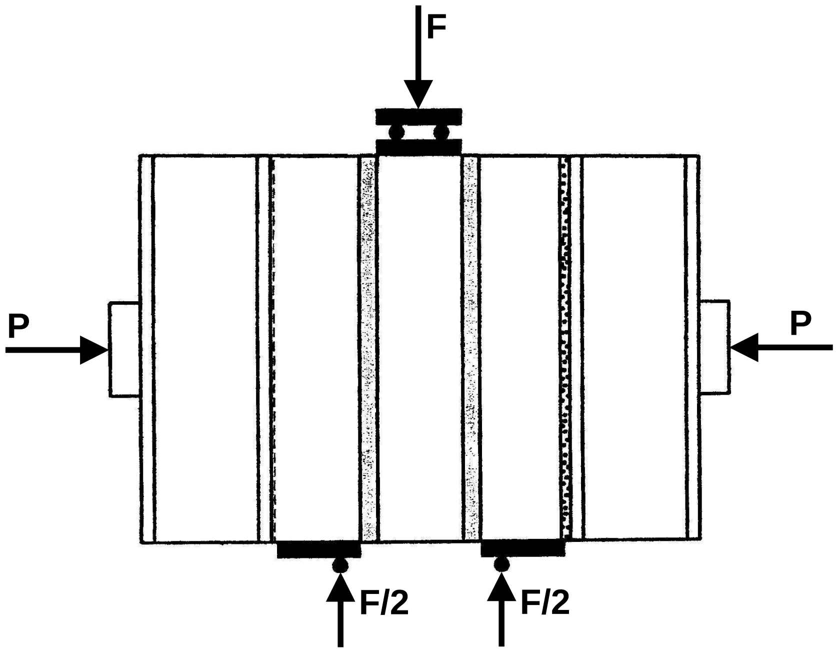
3.2.2.2 Adhesion between units and mortar

Adequate adhesion will normally be obtained with mortars manufactured in accordance with the relevant regulations.

In other cases shear tests should be carried out to check that the shear strength f_{vko} is not less than that for general purpose mortar.

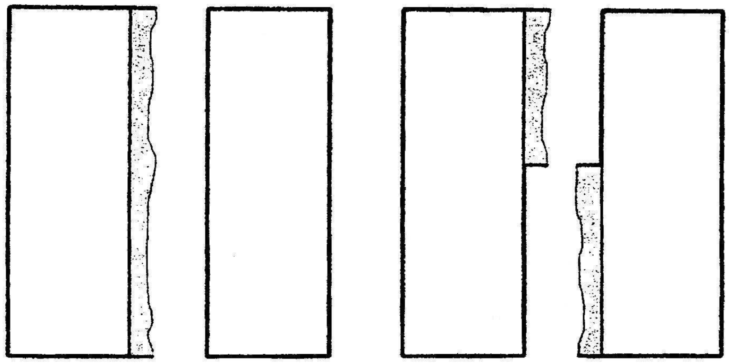
Shear tests according EN 1052-3:

- test set-up:

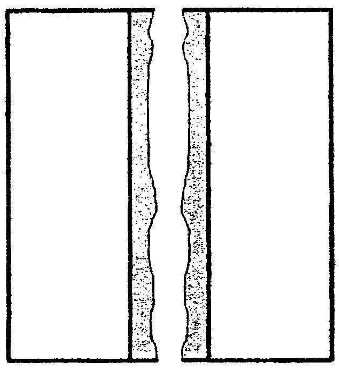


Tests have to be carried out with different preloads P and also with P = 0.

- Failure modes:

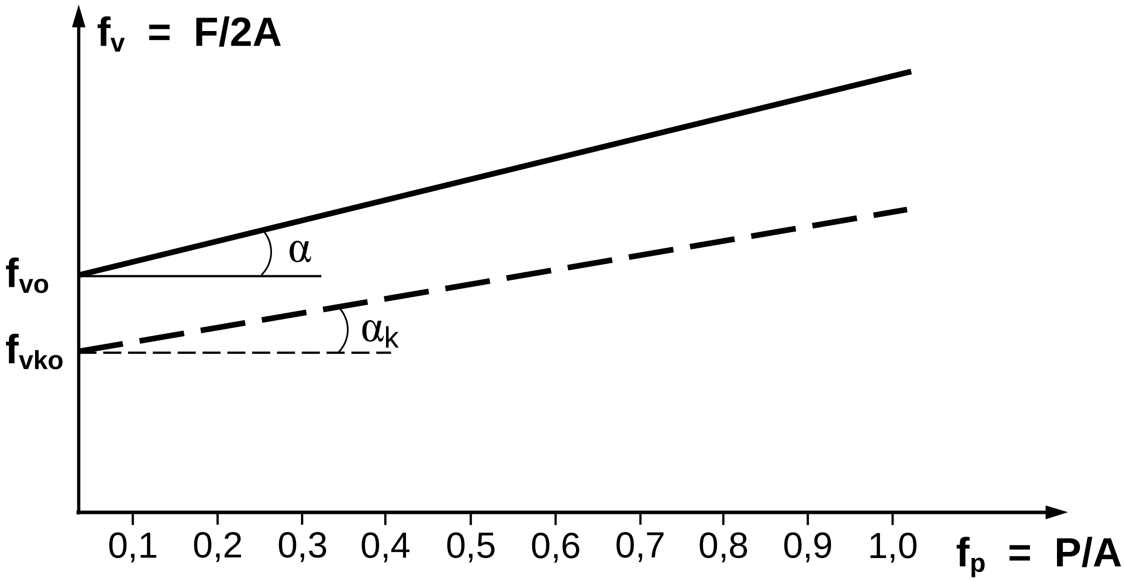


shear failure of the bond between mortar and units



shear failure in mortar only

- Test results:



—— test results: f_{vo}; α

– – – characteristic values:

$$f_{vko} = 0{,}8 \cdot f_{vo}$$

$$\tan \alpha_k = 0{,}8 \cdot \tan \alpha$$

3.3 Concrete infill

- Concrete used for infill shall be in accordance with EN 206.
- The characteristic shear strength of concrete infill, f_{cvk}, for the relevant concrete strength classes:

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

3.6 Mechanical properties of unreinforced masonry

3.6.1 General

(1) The distinction is made between:

- 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 depend on
  the intrinsic mechanical properties of the masonry,
- the geometry of the element,
- 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

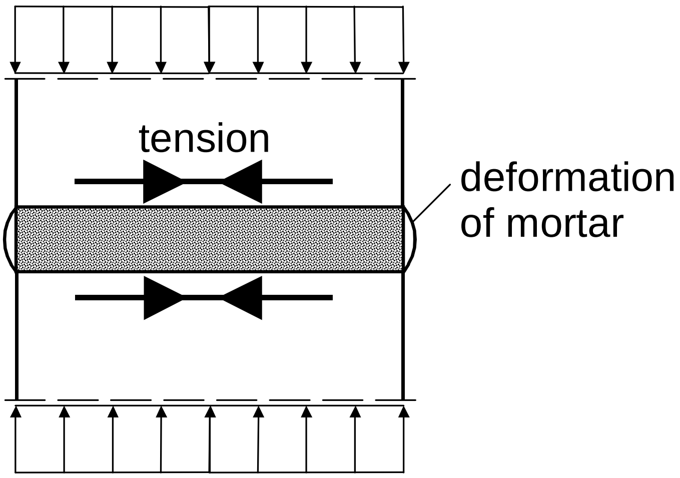
(1)P The characteristic compressive strength of unreinforced masonry, f_k, shall be determined from the results of tests on masonry.

(2) The characteristic compressive strength of unreinforced masonry

- may de 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.

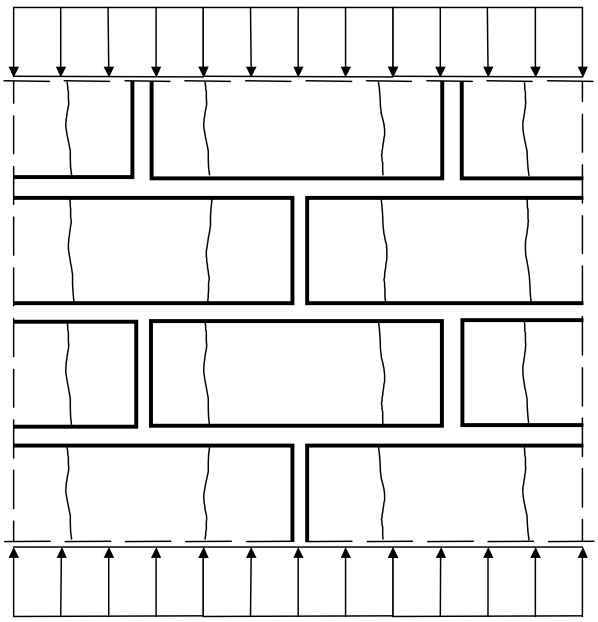
Failure mode of masonry:

In masonry under compressive load the transversal strain of the mortar in the bed joints is normally larger than that of the units.



This causes transversal tensile stresses in the units.

This leads to the effect that the compressive strength of masonry is limited by the tensile strength of the units.



When the compressive load is increased up to the bearing capacity, the units will crack normal to the mentioned tensile stresses.

So the compressive strength of masonry mainly depends on:

- the tensile strength of the masonry units
  (units with holes and also grip slots are disadvantageous
  in this regard),
- the compressive strength of mortar
  (as a higher strength of mortar reduces the transverse strain).

Note:
Otherwise certain deformability of mortar is advantageous,
so that masonry may accommodate induced deformations,
for example resulting from unequal settlements,
without cracking.

Additional parameters influencing on the strength of masonry:

- Masonry bond:

  Walls, in which every unit
  goes through the whole wall thickness are stronger
  than walls which are built up of several units,
  laying side by side over the wall thickness.
  In the latter case a sufficient number of through units
  is very important.

- Thickness of bed joints:

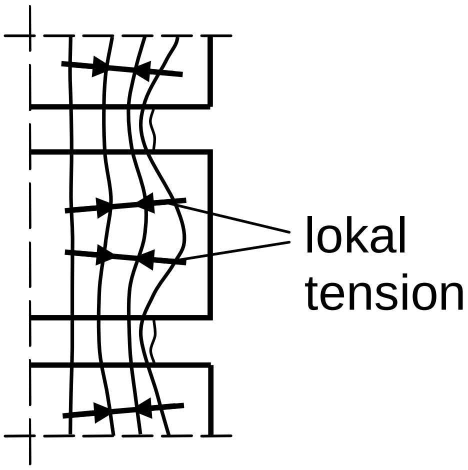
  Too thick bed joints are unfavourable.
  Therefore their thickness is limited
  (normally from 8 mm to 15 mm).

- Number of bed joints over the height of the wall:

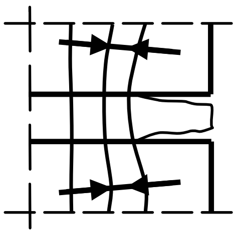
  Blocks are better than smaller units in this respect.

– Total filling of joints by the bricklayers:

Especially bed joints which are not completely filled reduce the supporting capacity:



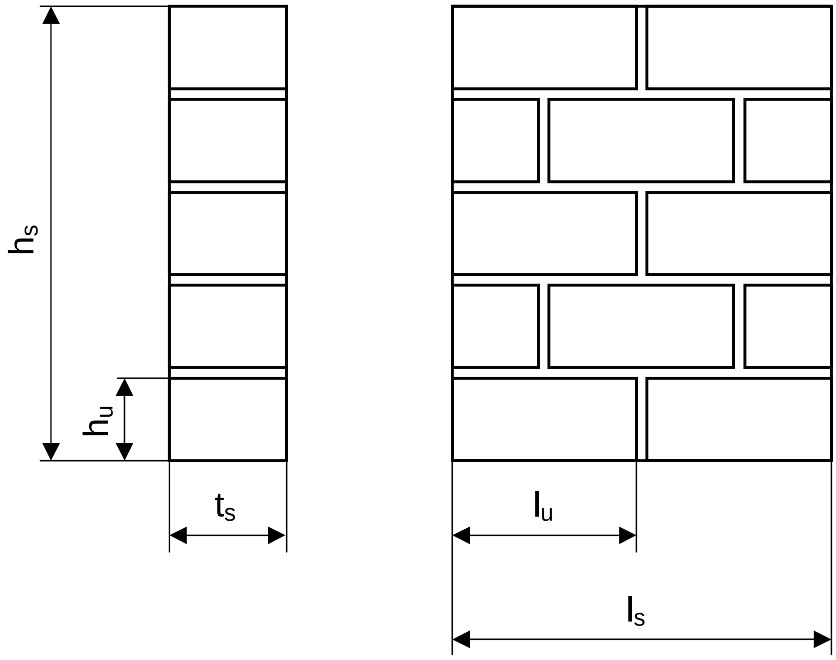
Not completely filled edges of the joints lead to an early spalling of stones near their surfaces.



A similar effect is caused by a settlement of the mortar
(due to a wrong consistency of the fresh mortar)
and an unequal shrinkage at the edges
(due to a too high water content)

Determination of the compressive strength by tests according EN 1052-1:

– Shape of the test-specimens:



<table>
<tr><td colspan="2">Dimensions of units</td><td colspan="4">Dimension of test-specimen</td></tr>
<tr><td>l_u
mm</td><td>h_u
mm</td><td>lenght
l_s</td><td colspan="2">height
h_s</td><td>width
t_s</td></tr>
<tr><td rowspan="2">≤ 300</td><td>≤ 150</td><td rowspan="2">$\geq (2 \bullet l_u)$</td><td>$\geq 5\ h_u$</td><td rowspan="4">$\geq 3\ t_s$ and
$\leq 15\ t_s$ and
$\geq l_s$</td><td rowspan="4">$\geq t_u$</td></tr>
<tr><td>> 150</td><td>$\geq 3\ h_u$</td></tr>
<tr><td rowspan="2">> 300</td><td>≤ 150</td><td rowspan="2">$\geq (1{,}5 \bullet l_u)$</td><td>$\geq 5\ h_u$</td></tr>
<tr><td>> 150</td><td>$\geq 3\ h_u$</td></tr>
</table>

– Number of specimens: ≥ 3

– Time of testing: When the compressive strength of the mortar has reached a value in the prescribed interval (see the following table)

Classi-fication of mortar	Minimum compressive strength (f_{md}) N/mm²	Mean value of compressive strength at the time of testing (f_m) N/mm²
M1	1,0	$1{,}0 \le f_m < 2{,}5$
M2,5	2,5	$2{,}5 \le f_m < 5{,}0$
M5	5,0	$5{,}0 \le f_m < 7{,}5$
M7,5	7,5	$7{,}5 \le f_m < 10{,}0$
M10	10,0	$10{,}0 \le f_m < 12{,}5$
M12,5	12,5	$12{,}5 \le f_m < 15{,}0$
M15	15,0	$15{,}0 \le f_m < 20{,}0$
M20	20,0	$20{,}0 \le f_m < 30{,}0$
M30	30,0	$30{,}0 \le f_m < 40{,}0$

– Transfer of the single test-results f_i, got on the basis of the actual values of the compressive strengths of the units f_b and of the mortar f_m to the normalized value f_d, belonging to the presented minimum strengths f_{bd} respective f_{md}:

$$f_{id} = f_i \cdot \left(\frac{f_{bd}}{f_b}\right)^{0,65} \cdot \left(\frac{f_{md}}{f_m}\right)^{0,25}$$

– Determination of the characteristic compressive strength:

$$f_k = \frac{f}{1{,}2} \quad \text{or} \quad f_k = f_{id,min}$$

(the smaller value is relevant)
f is the mean value of the single values f_{id}.

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

(3) The characteristic compressive strength of unreinforced masonry made with general purpose mortar, with all joints to be considered as filled, may be calculated using equation (3.1):

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

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

where:

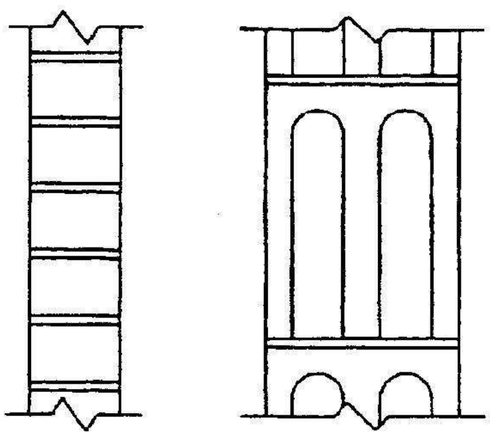
K is a constant in (N/mm^2)^{0,10} that may be taken as:

$\boxed{0,60}$ for Group 1 masonry units

$\boxed{0,55}$ for Group 2a masonry units

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

when the thickness of 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



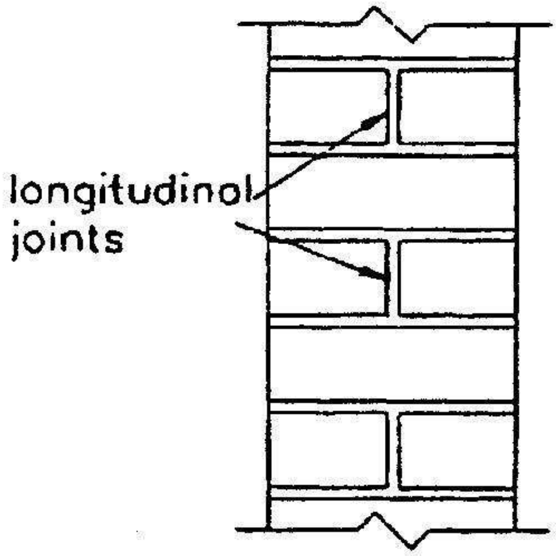
Walls without longitudinal joints

$\boxed{0{,}50}$ for Group 1 masonry units

$\boxed{0{,}45}$ for Group 2a masonry units

$\boxed{0{,}40}$ for Group 2b masonry units

when there is a longitudinal mortar joint through all or part of the length of the masonry



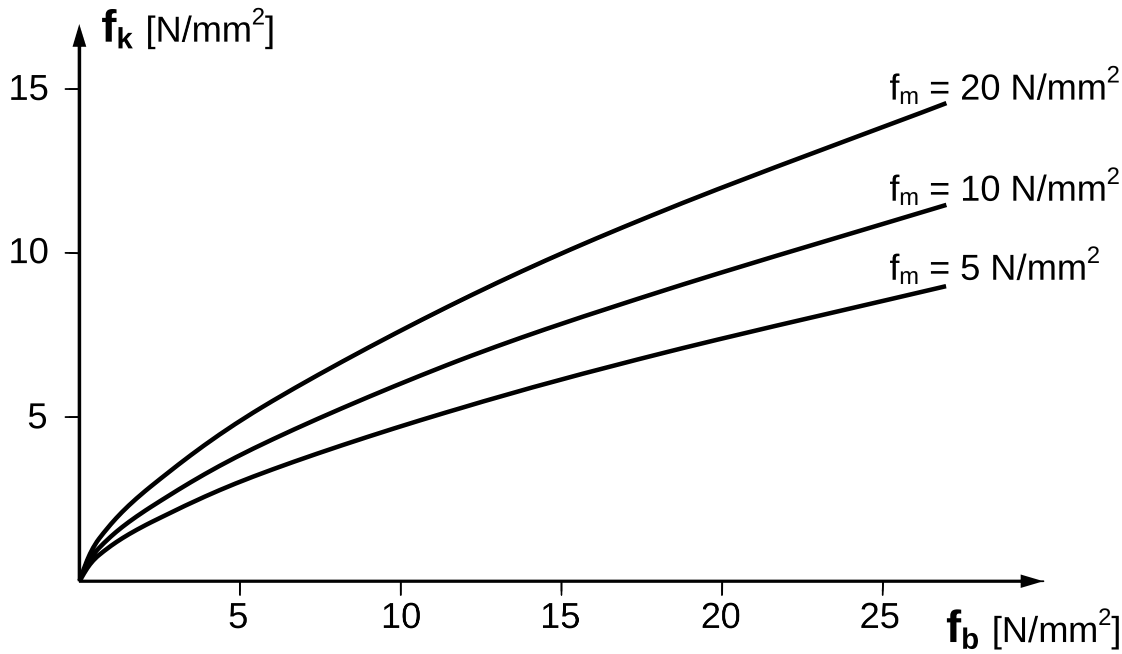
Walls with longitudinal joint

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

f_b is the normalized compressive strength of the masonry units in N/mm^2,

f_m is the specified compressive strength of the general purpose mortar in N/mm^2.

Example for the relation
between the compressive strength f_k of masonry
and the compressive strength f_b of the units
and f_m of the mortar
(according equation (3.1) and with K = 0,60):



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, with all joints to be considered as filled and using Group 1 calcium silicate units and autoclaved aerated concrete units may be calculated using equation (3.2):

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

provided that:

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

– the normalized compressive strength of masonry units, fb, is not taken to be greater than 50 N/mm^2;

– the thin layer mortar has a compressive strength of 5 N/mm^2 or more;

– 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 paragraph (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, with all joints to be considered as filled, may be calculated using equation (3.3):

$$f_k = K \cdot f_b^{0,65} \quad N/mm^2 \tag{3.3}$$

provided that f_b is not taken to be greater than 15 N/mm^2 and that there is no longitudinal mortar joint through all or part of the length of the wall.

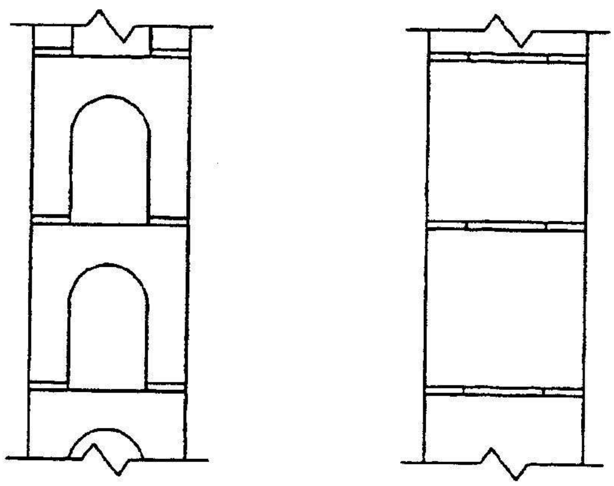
The value of K depends on:

– the density of the used lightweight mortar,

– the type of the masonry units.

Further regulations are given for:

- characteristic compressive strength of unreinforced masonry with unfilled vertical joints
- characteristic compressive strength of shell bedded unreinforced masonry



Example cross-sections trough a shell bedded wall

3.6.3 Characteristic shear strength of unreinforced masonry

The characteristic shear strength f_{vk} of unreinforced masonry can be determined

- from the results of tests on masonry,
- by calculation in the following way:

For general purpose mortar and when all joints may be considered as filled, f_{vk} will not fall below the least of the values described below:

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

or $= \boxed{0{,}065} \cdot f_b$, 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

σ_d is the design compressive stress perpendicular to the shear

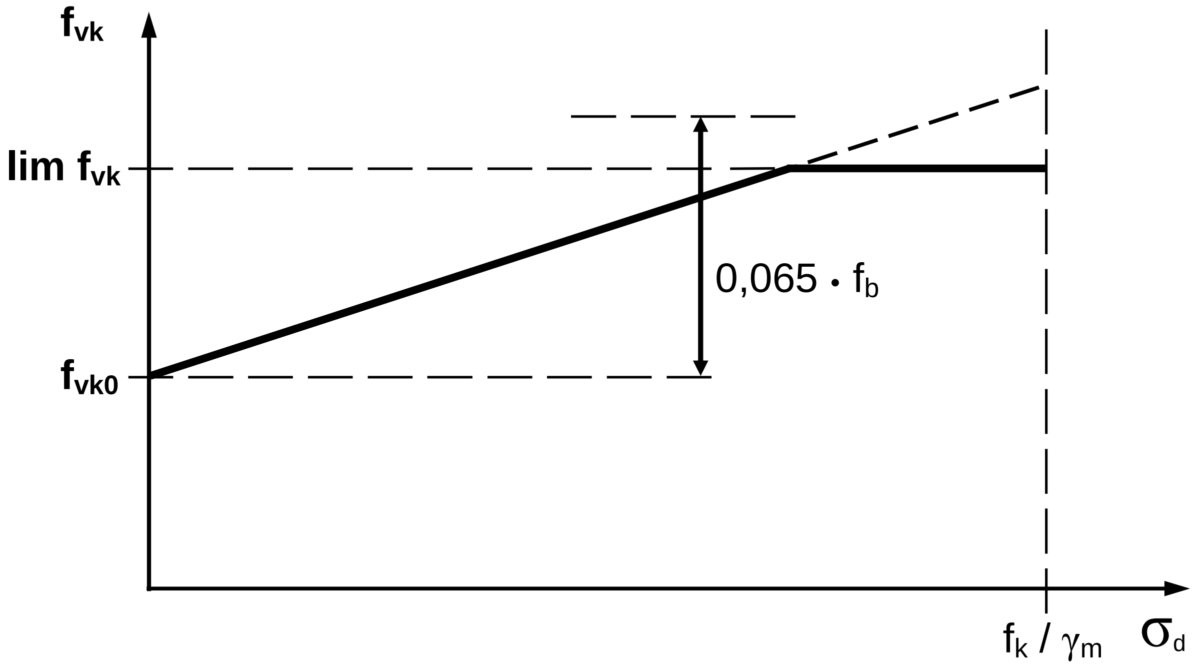
For:
- masonry with unfilled perpend joints,
- shell bedded masonry,
- thin layer mortar,
- lightweight mortar,

there are similar regulations.

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

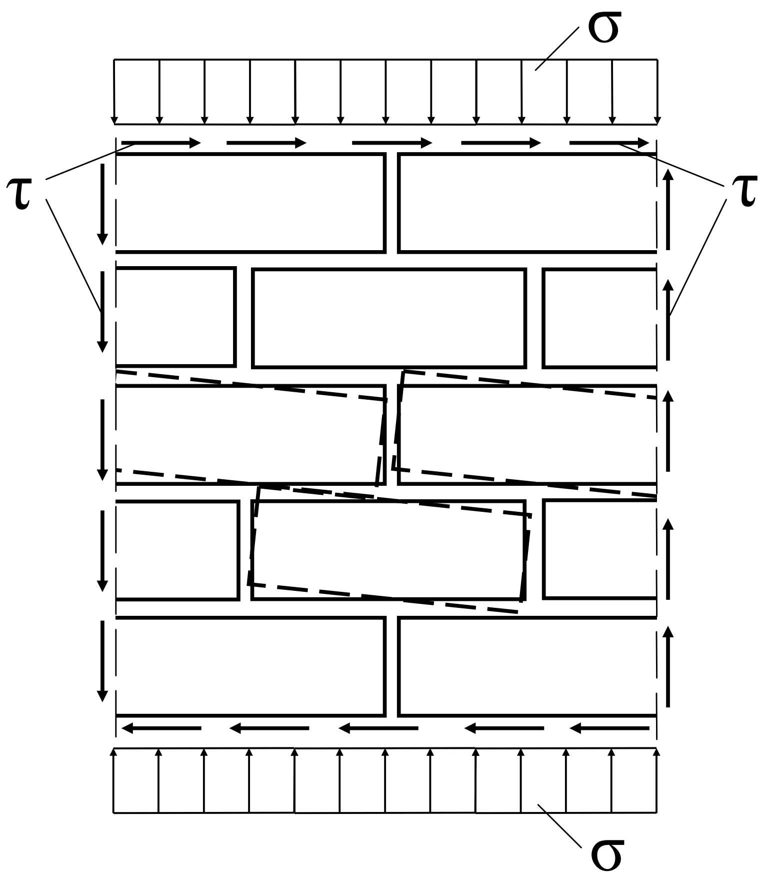
<table>
<tr><th>Masonry Unit</th><th>Mortar</th><th>f_{vko} (N/mm²)</th><th colspan="2">Limiting f_{vk} (N/mm²)</th></tr>
<tr><td rowspan="3">Group 1 clay units</td><td>M10 to M20</td><td>$\boxed{0,3}$</td><td colspan="2">$\boxed{1,7}$</td></tr>
<tr><td>M2,5 to M9</td><td>$\boxed{0,2}$</td><td colspan="2">$\boxed{1,5}$</td></tr>
<tr><td>M1 to M2</td><td>$\boxed{0,1}$</td><td colspan="2">$\boxed{1,2}$</td></tr>
<tr><td rowspan="3">Group 1 units other than clay and natural stone</td><td>M10 to M20</td><td>$\boxed{0,2}$</td><td colspan="2">$\boxed{1,7}$</td></tr>
<tr><td>M2,5 to M9</td><td>$\boxed{0,15}$</td><td colspan="2">$\boxed{1,5}$</td></tr>
<tr><td>M1 to M2</td><td>$\boxed{0,1}$</td><td colspan="2">$\boxed{1,2}$</td></tr>
<tr><td rowspan="2">Group 1 natural stone units</td><td>M2,5 to M9</td><td>$\boxed{0,15}$</td><td colspan="2">$\boxed{1,0}$</td></tr>
<tr><td>M1 to M2</td><td>$\boxed{0,1}$</td><td colspan="2">$\boxed{1,0}$</td></tr>
<tr><td rowspan="3">Group 2a clay units</td><td>M10 to M20</td><td>$\boxed{0,3}$</td><td rowspan="6">The lesser of longitudinal compressive strength (see note below) or</td><td>$\boxed{1,4}$</td></tr>
<tr><td>M2,5 to M9</td><td>$\boxed{0,2}$</td><td>$\boxed{1,2}$</td></tr>
<tr><td>M1 to M2</td><td>$\boxed{0,1}$</td><td>$\boxed{1,0}$</td></tr>
<tr><td rowspan="3">Group 2a and Group 2b units other than clay and Group 2b clay units</td><td>M10 to M20</td><td>$\boxed{0,2}$</td><td>$\boxed{1,4}$</td></tr>
<tr><td>M2,5 to M9</td><td>$\boxed{0,15}$</td><td>$\boxed{1,2}$</td></tr>
<tr><td>M1 to M2</td><td>$\boxed{0,1}$</td><td>$\boxed{1,0}$</td></tr>
<tr><td rowspan="3">Group 3 clay units</td><td>M10 to M20</td><td>$\boxed{0,3}$</td><td colspan="2" rowspan="3">No limits other than given by equation (3.4)</td></tr>
<tr><td>M2,5 to M9</td><td>$\boxed{0,2}$</td></tr>
<tr><td>M1 to M2</td><td>$\boxed{0,1}$</td></tr>
<tr><td colspan="5">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.</td></tr>
</table>

Diagram illustrating the dependency of f_{vk}:
(general purpose mortar)



Behaving of masonry under shear:

Element cut off a wall:

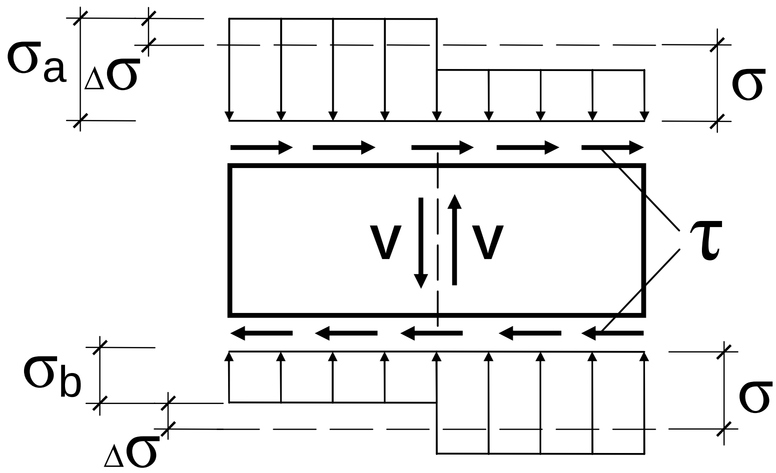


In the global system shear stresses τ act not only in horizontal but also in vertical direction (due to the equilibrium of moments at the element).

Locally in the perpend joints shear stresses cannot be transferred due to the following reasons:

- the surface of the unit heads are often very smooth,
- there are no normal stresses acting in the perpend joints, therefore there is no friction possible,
- the shrinkage of mortar reduces the possible adhesion,
- vertical joints often are not fully filled with mortar.

So if shear stresses only act in the bed joints, there must be a change in the distribution of the vertical normal stresses, as the equilibrium of a single stone shows. The stresses must become a stepped distribution due to the kinematics of deformations.

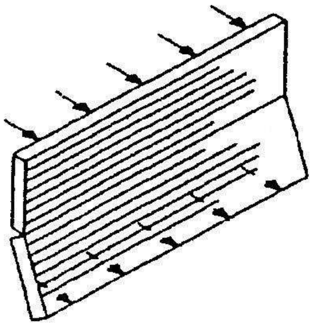


Three failure modes occur:

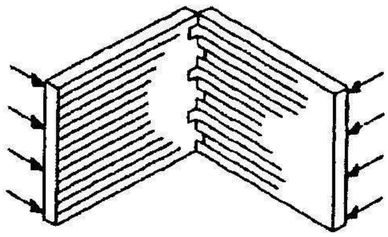
- small load σ: failure in the bed joint, due to τ under friction
- larger load σ: fracture of units, due to the principal tensile stress, deriving from σ and τ in the middle of units,
- very high load σ: failure of units, due to the pressure σ_a.

3.6.4 Characteristic flexural strength of unreinforced masonry

- symbol: f_{xk}
- determinated from the results of tests on masonry
- two different values:
  - f_{xk1}: failure parallel to the bed joints,
  - f_{xk2}: failure perpendicular to the bed joints.



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



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

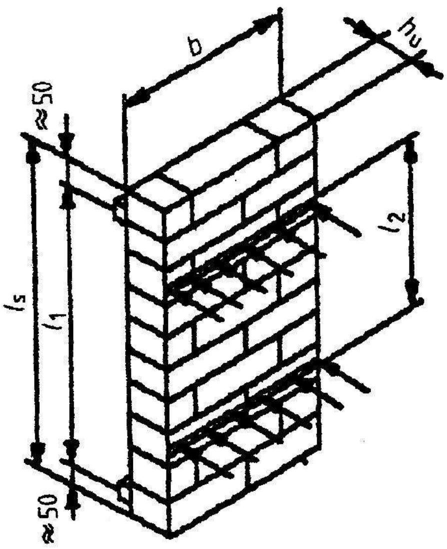
Flexural strengths f_{xk1} and f_{xk2}.

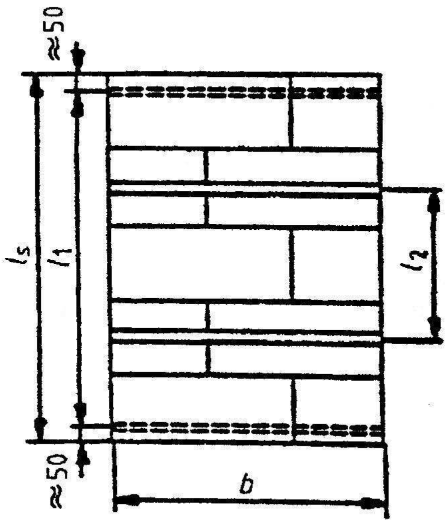
- use of f_{xk1}:
  - only for transient loads (for example wind)
  - $f_{xk1} = 0$, where failure of the wall would lead to a major collapse.
- f_{xk1} and f_{xk2} will be given in the NAD`s

Determination of the flexural strength by tests:

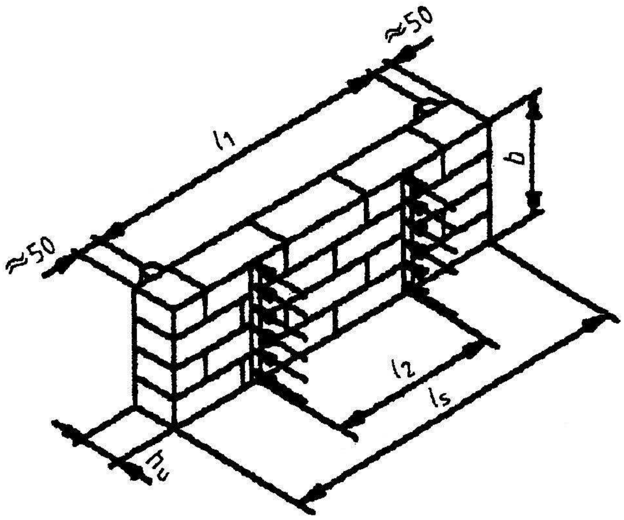
Examples of test set-ups and of typical test specimens:

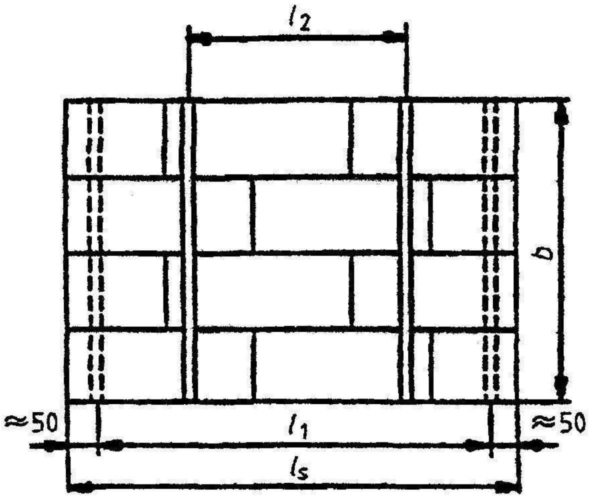
for f_{kx1}:





for f_{kx2}:





3.8 Deformation properties of masonry

3.8.1 Stress-strain relationship

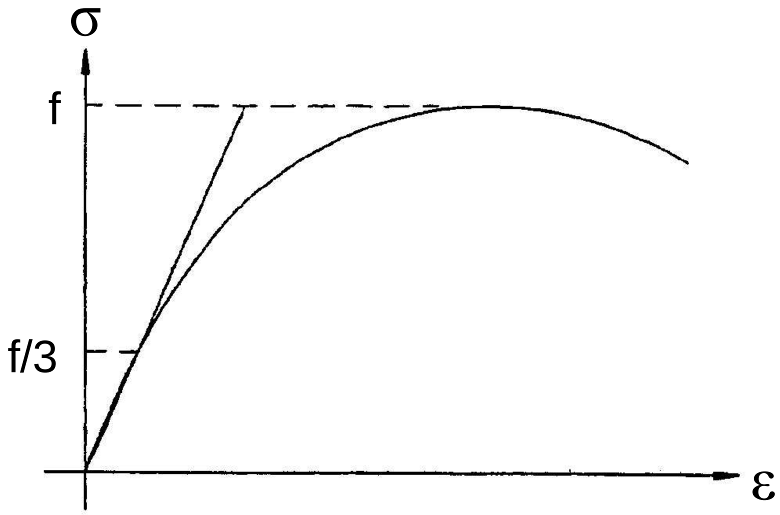


Figure 3.2: General shape of a stress-strain relationship for masonry.

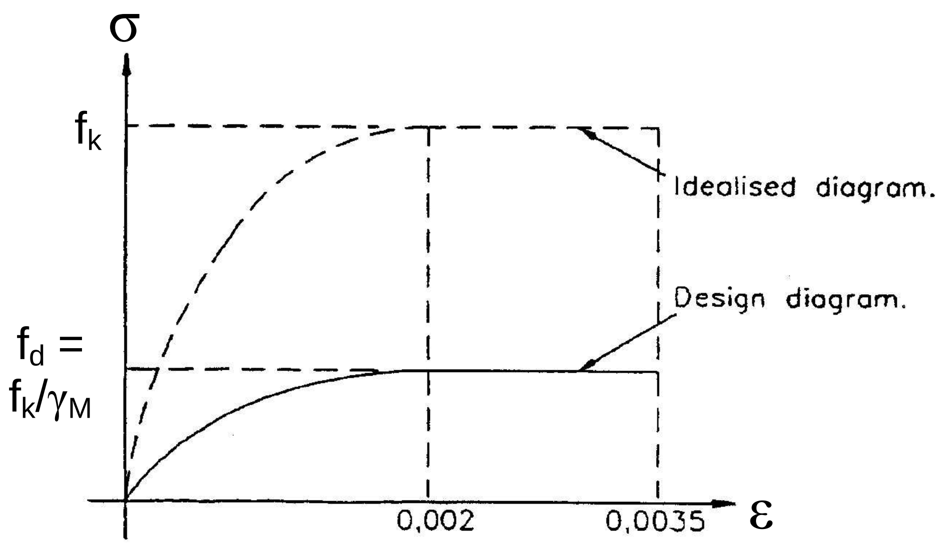


Figure 3.3: Stress-strain relationship for the design of masonry in bending and compression.

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.

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 $\boxed{1\ 000} \cdot f_k$.

(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

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.

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.

4 Design of masonry

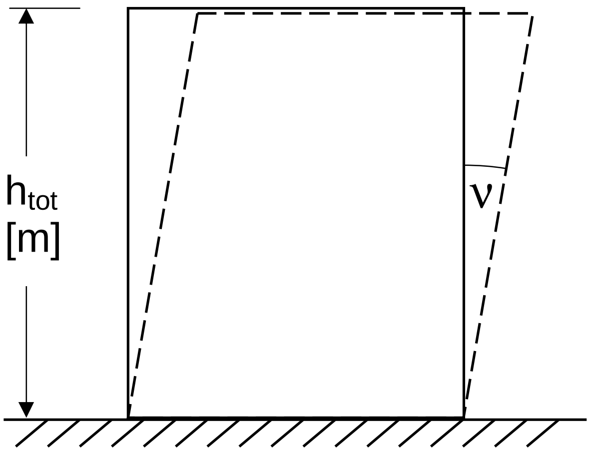
4.1 Structural behaviour and overall stability

4.1.1 Design models for structural behaviour

- 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.

- Structures incorporating masonry walls,
  should have their parts suitably braced together,
  so that sway of the structure will not occur.

- The possible effects of imperfections should be allowed for,
  by assuming that the structure is inclined at an angle ν



$$\nu = \frac{1}{100 \cdot \sqrt{h_{tot}}}$$

4.1.2 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, finishing's or technical equipment, or which might impair water-tightness, are avoided or minimised.

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

4.2 Design strength of masonry

The design strength of masonry is given by:

– in compression $$f_d = \frac{f_k}{\gamma_M}$$

– in shear $$f_{vd} = \frac{f_{vk}}{\gamma_M}$$

– in flexure $$f_{xd} = \frac{f_{xk}}{\gamma_M}$$

characteristic strength divided by the appropriate partial safety factor γ_M.

4.4 Unreinforced masonry walls subjected to vertical loading

4.4.1 General

(2) It may be assumed that:

- plane sections remaine 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) 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}:

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

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:

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

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;

f_k is the characteristic compressive strength of masonry;

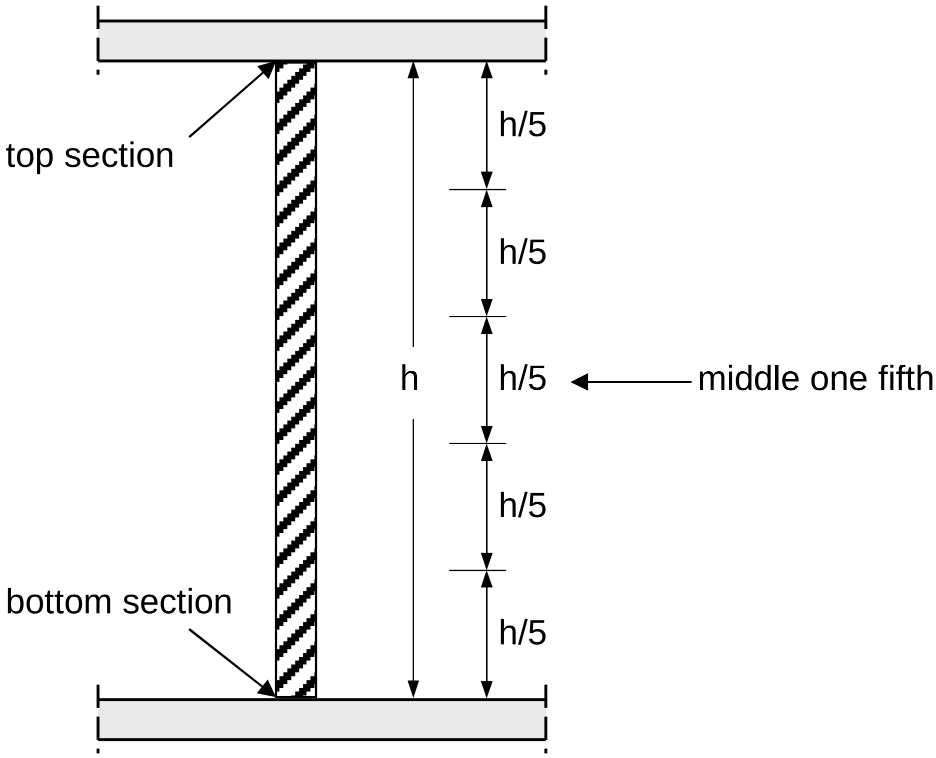
γ_M is the partial safety factor for the material;

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

(2) The design strength of a wall may be at its lowest:

– in the middle one fifth of the heigth, when Φ_m should be used,

– or at the top of the wall or bottom of the wall, when Φi should be used.



(3) Where the cross-sectional area A 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) A [m^2]

Comment: The masonry bond at the edges of walls
is of less quality
due to the use of smaller units or parts of units.
This leads to a lower local strength.

Cavity walls:

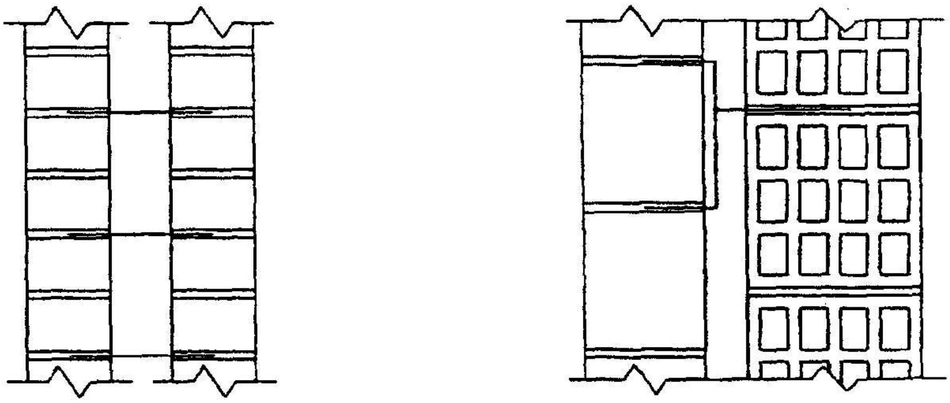


Figure 5.2: Example cross-sections through a cavity wall

– the load carried by each leaf should be assessed,

– the design vertical load resistance of each leaf, N_{Rd} , should be verified,

– when only one leaf is loaded:
  – the loadbearing capacity of that wall should be based on the horizontal cross-sectional area of that leaf alone,
  – but using the effective thickness for the purpose of determining the slenderness ratio (see later).

Faced walls:

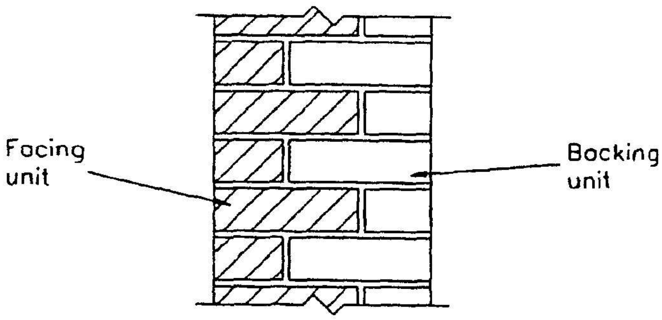


Figure 5.4: Example cross-section through a faced wall

- when bonded together as to result in common action under load
  may be designed in the same manner as a single-leaf wall
  constructed entirely of the weaker units,
  using the value K appropriate to a wall with a longitudinal
  mortar joint,
- when 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.

Double leaf walls:

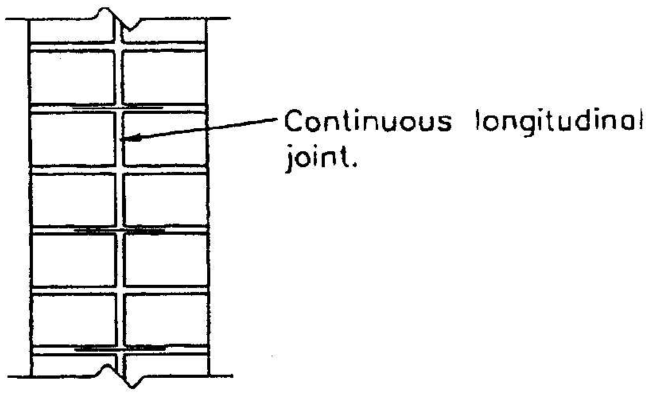


Figure 5.3: Example cross-section through a double-leaf wall

- 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.

Chases and recesses

- **General**

  Chases and recesses:

  – reduce the loadbearing capacity of a wall,

  – shall not impair the stability of the wall,

  – should not be allowed
  if their depth would be greater
  than half the thickness of the shell of the unit,
  unless the strength of the wall is verified by calculation.

- **Vertical chases and recesses**

  The reduction in the loadbearing capacity may be neglected if such vertical chases and recesses are kept within the limits given in table 5.3.

  If these limits are exceeded,
  the vertical load, shear and flexural resistance
  should be checked by calculation.

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.

- **Horizontal and inclined chases**
  - should preferably be avoided,
  - where it is not possible to avoid horizontal and inclined chases, they should be positioned within one eighth of the clear height of the wall, above or below the floor, and the total depth should be less than the maximum size as given in table 5.4,
  - if these limits are exceeded, the loadbearing capacity should be checked by calculation.

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.

- 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%.

4.4.3 Reduction factor for slenderness and eccentricity

Symbol: Φ

(I) At the top or bottom of the wall.

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

where:

e_i is the eccentricity at the top or the bottom of the wall:

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

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.

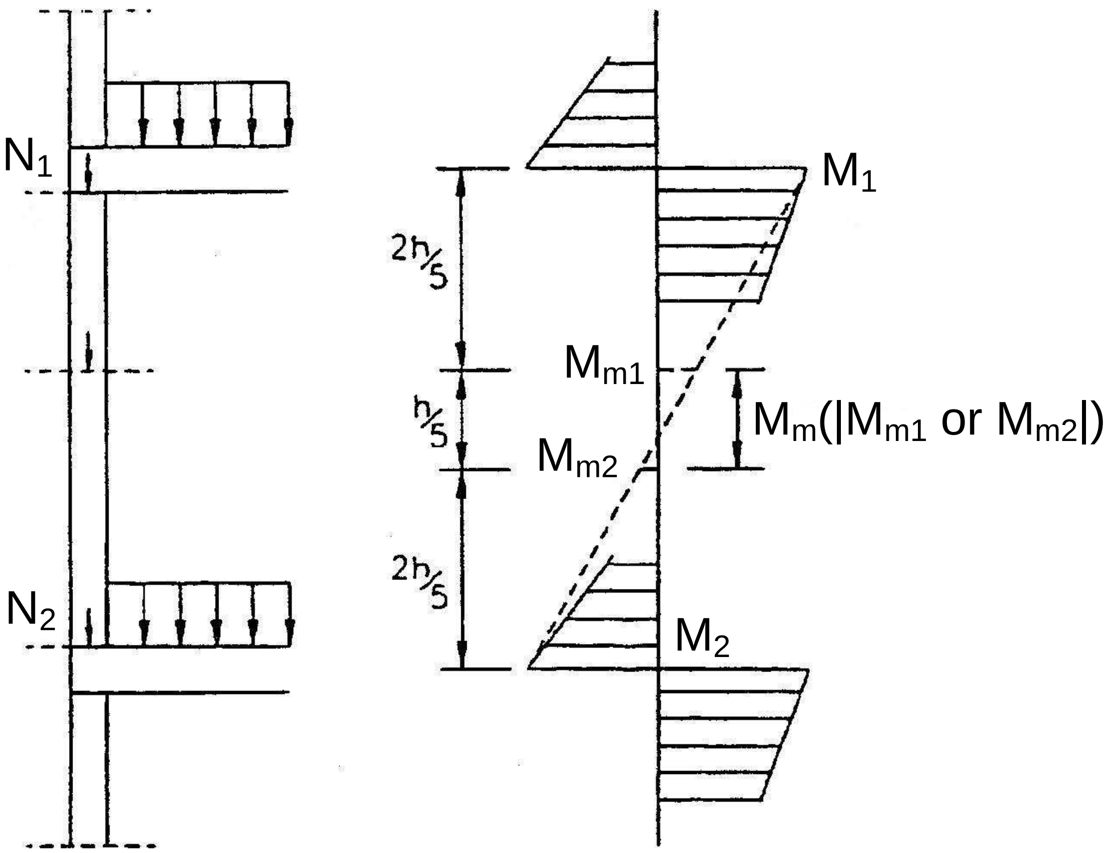


Figure 4.1: Moments from calculation of eccentricities.

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

Φ_m, may be determined from figure 4.2:

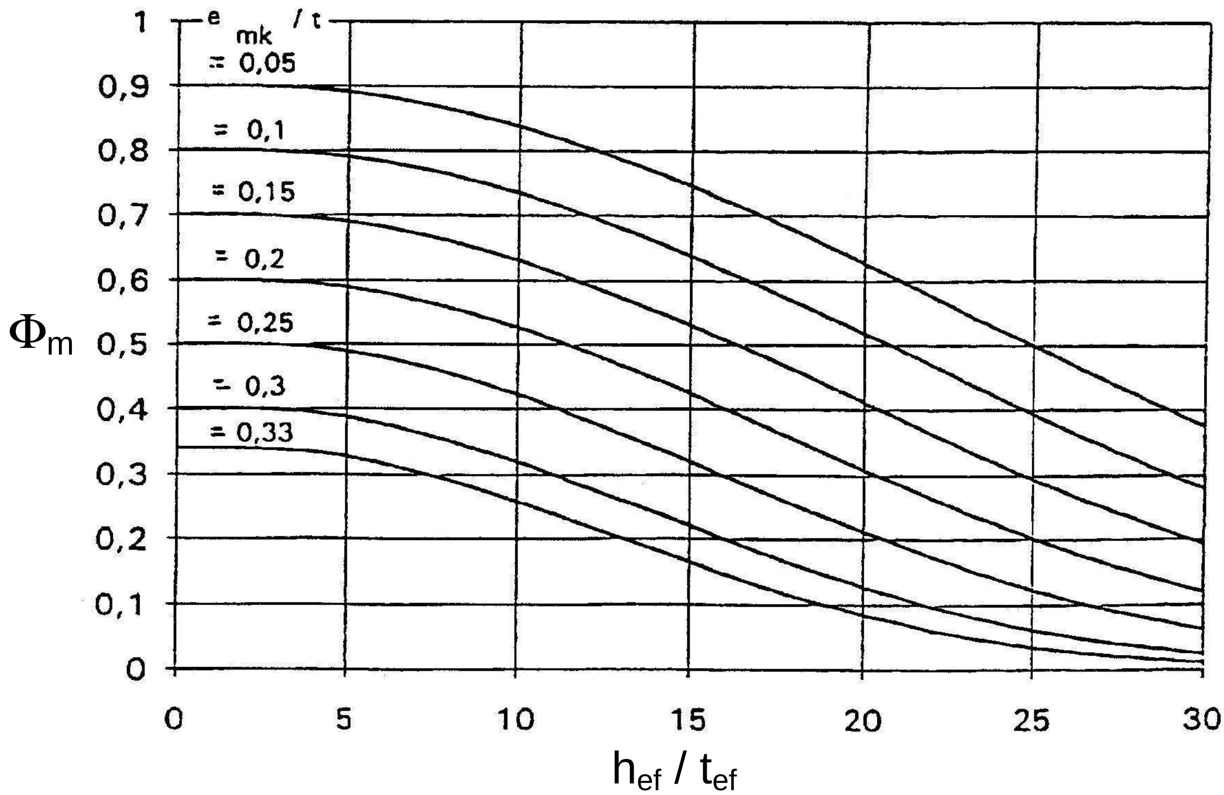


Figure 4.2: Graph showing values of Φ_m against slenderness ratio for different eccentricities.

e_{mk} is the eccentricity within the middle one fifth of the wall height:

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

with

$$e_m = \frac{M_m}{N_m} + e_{hm} \pm e_a$$

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:

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

ϕ_∞ is the final creep coefficient.

The creep eccentricity, e_k , may be taken as zero

– for all walls built with clay and natural stone units

– for walls having a slenderness ratio up to 15 constructed from other masonry units.

The values 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 load bearing 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.

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 between the walls
    for example, due to shrinkage, loading etc., is not expected,
- 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.

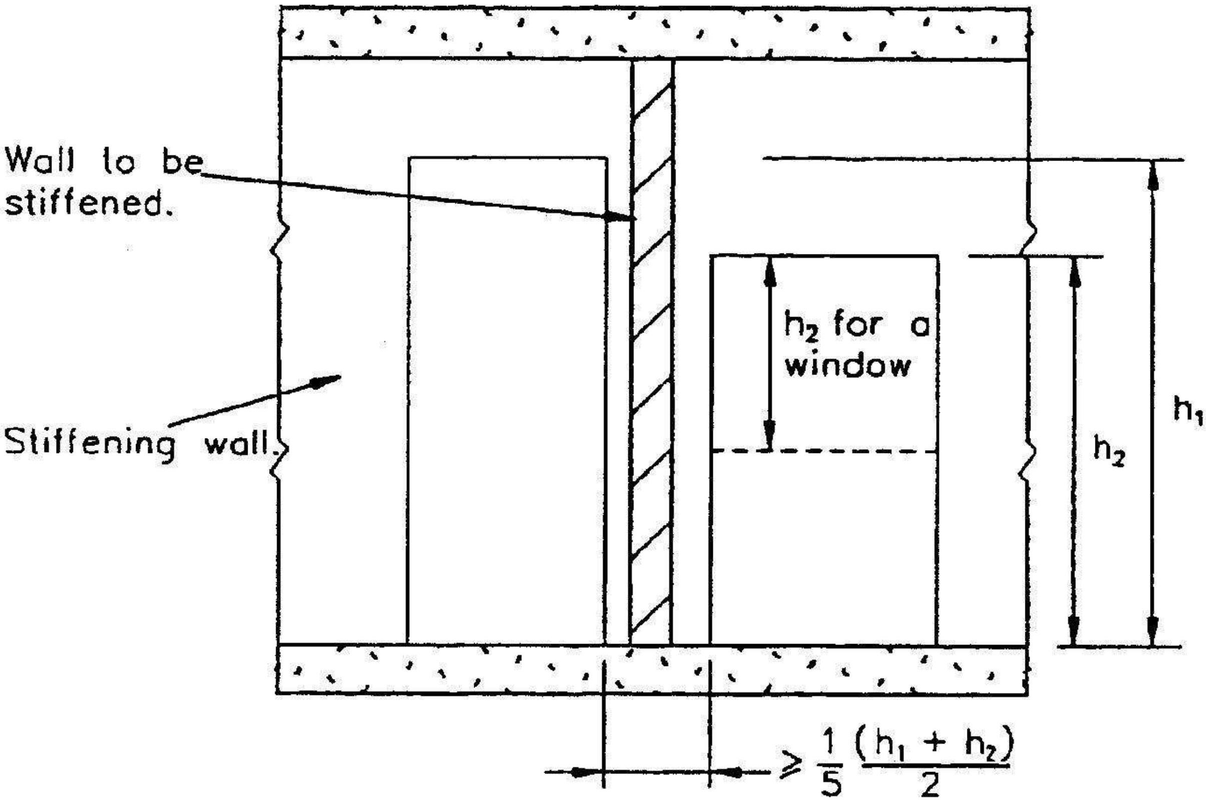


Figure 4.3: Minimum length of stiffening wall with openings.

(4) Alternatively, walls may be stiffened by members other than masonry walls, provided that they have the equivalent stiffness of the masonry stiffening wall, 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 h_{ef} can be taken as:

$$h_{ef} = \rho_n \; h$$

where:

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.

(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 85 mm:

$\rho_2 = \boxed{0,75}$ unless the eccentricity of the load at the top of the wall is greater than 0,25 times the thickness of the 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.

(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):

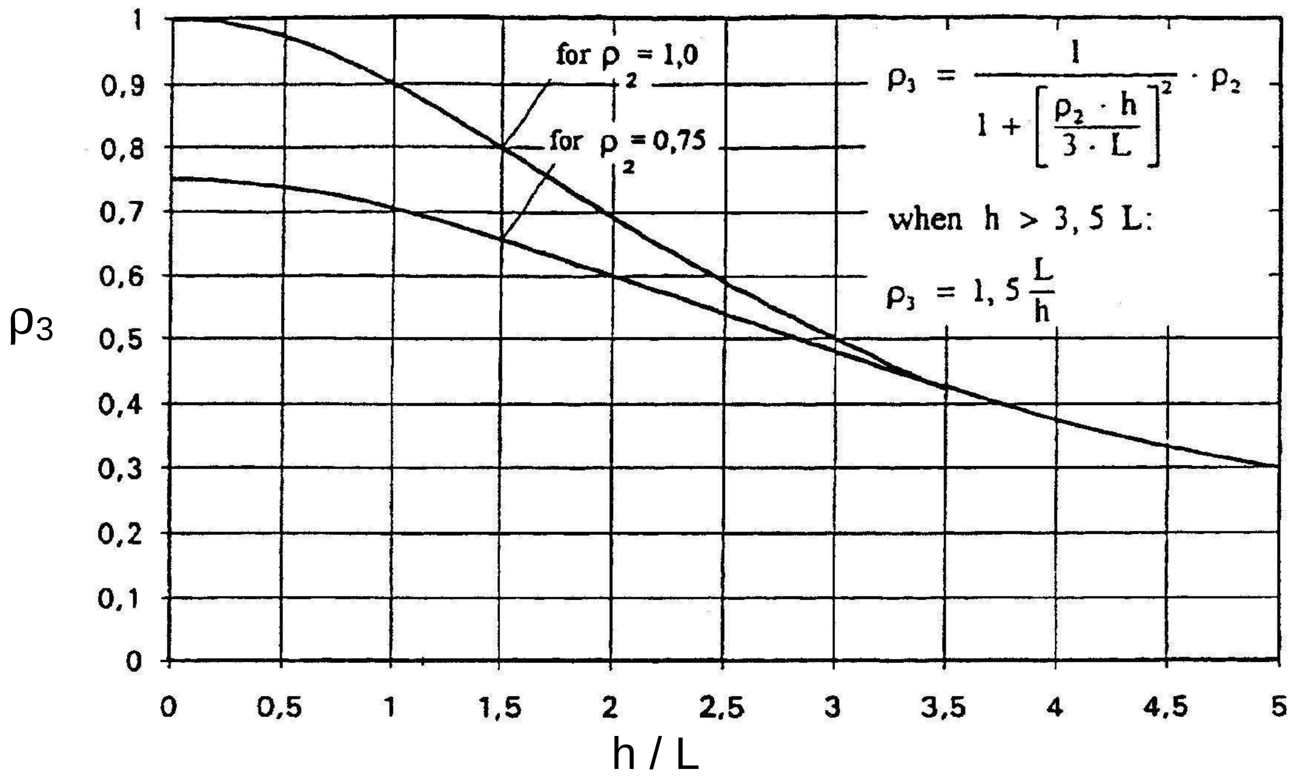
$$\rho_3 = \frac{1}{1+\left[\frac{\rho_2\ h}{3\ L}\right]^2}\ \rho_2 \quad > \quad 0{,}3$$

when h $\leq$ 3,5 L, with ρ_2 from (I), (II) or (III) whichever is appropriate, or

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

when h > 3,5 L,

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



B.1 Graph showing values of ρ_3

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

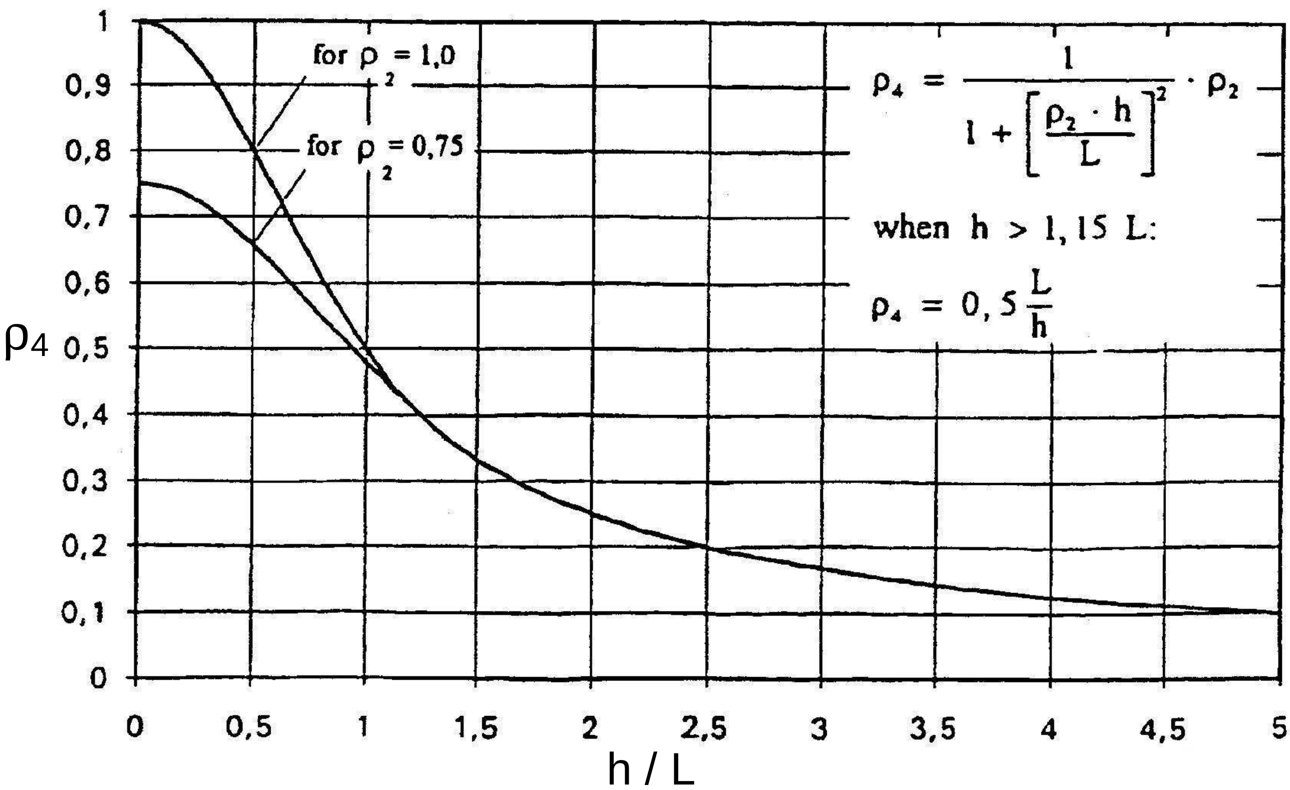
$$\rho_4 = \frac{1}{1+\left[\frac{\rho_2 \, h}{L}\right]^2} \rho_2$$

when h ≤ L, with ρ_2 from (I), (II) or (III) whichever is appropriate, or

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

when h > L,

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



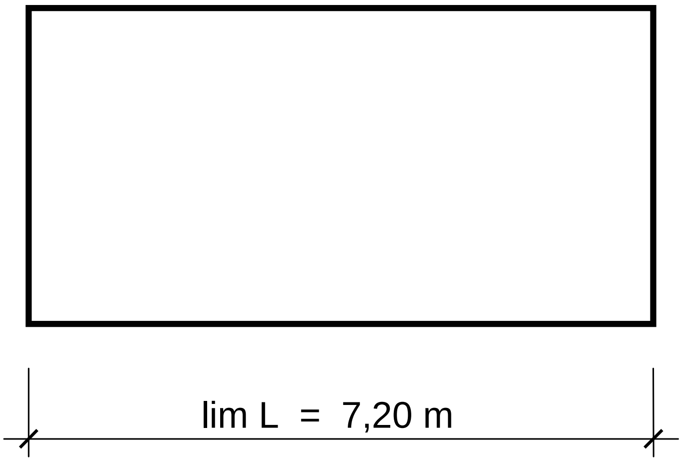
B.2: Graph showing values of ρ_4

(3) If $L \geq 30 \cdot t$, for walls stiffened on two vertical edges,
or if $L \geq 15 \cdot t$, 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.

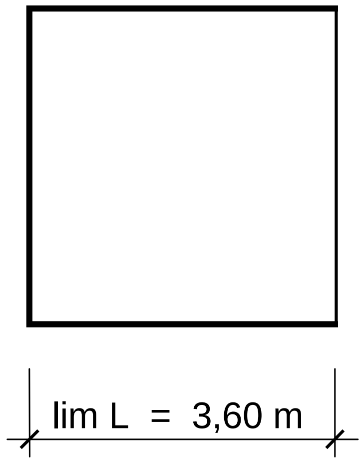
Examples:

$t = 24$ cm

Wall stiffened
on four edges:



Wall stiffened on
three edges:



4.4.4.4 Effects of openings, chases and recesses in walls

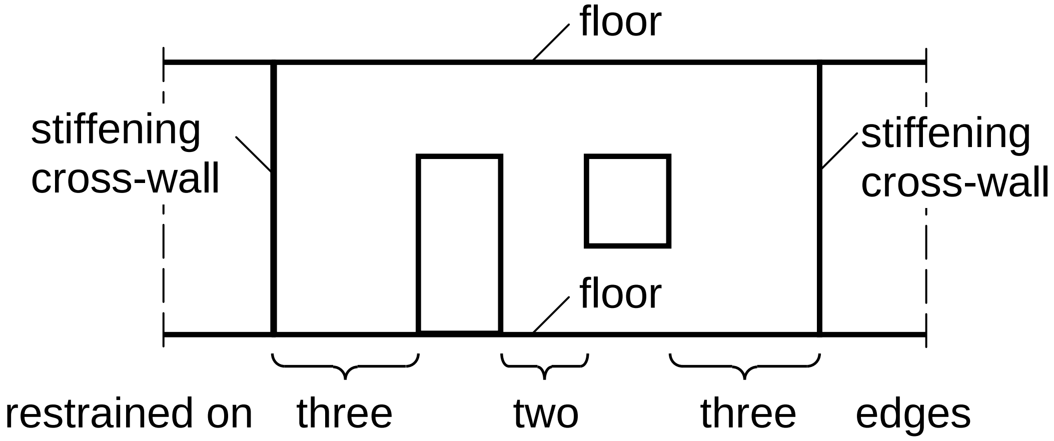
(1) If the stiffened wall is weakened by vertical chases and/or recesses, other than those allowed by table 5.3,

- 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.

Example:



4.4.5 Effective thickness of walls

- $t_{ef} = t$ (actual thickness) for:
  - single-leaf walls,
  - double-leaf walls,
  - faced walls,
  - shell bedded walls,
  - veneer walls,
  - grouted cavity walls.

- $t_{ef} = \sqrt[3]{t_1^{\,3} + t_2^{\,3}}$ (4.17) for cavity walls in which both leaves are connected with wall ties.

  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

The out-of-plane eccentricity of loading on walls:

- shall be assessed,
- may be calculated:
  - from the material properties given in Section 3,
  - the joint behaviour,
  - and from the principles of structural mechanics.

A simplified method is given in Annex C:

Assumptions:

- the joint between the wall and the floor may be simplified, by using uncracked cross sections;
- 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:

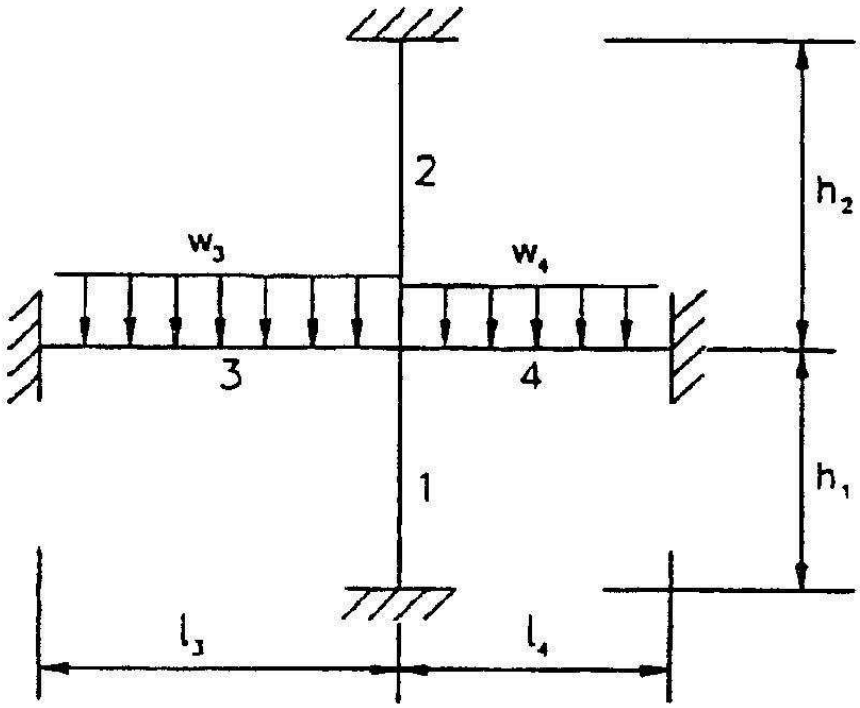


Figure C.1: Simplified frame diagram

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.

Note:
The simplified frame model is not considered to be appropriate, where timber floor joists are used.
(For such cases refer to paragraph (4) below).

The moment in member 1, M_1, may be calculated from equation (C.1):

(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)

$$M_1 = \frac{\dfrac{nE_1I_1}{h_1}}{\dfrac{nE_1I_1}{h_1} + \dfrac{nE_2I_2}{h_2} + \dfrac{nE_3I_3}{l_3} + \dfrac{nE_4I_4}{l_4}} \cdot \left[\frac{w_3 l_3^2}{12} - \frac{w_4 l_4^2}{12}\right] \qquad \text{(C.1)}$$

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

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.

(2) The results of such calculations will usually be conservative because to 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 above, by multiplying it by (1–k/4), provided that:

- the design vertical stress acting at the junction in question is greater than 0,25 N/mm^2 when averaged across the thickness of the wall
- and k is not taken to be greater than 2;

$$k = \frac{\dfrac{E_3 I_3}{l_3} + \dfrac{E_4 I_4}{l_4}}{\dfrac{E_1 I_1}{h_1} + \dfrac{E_2 I_2}{h_2}}$$

(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 0,25 N/mm^2 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 the roof.

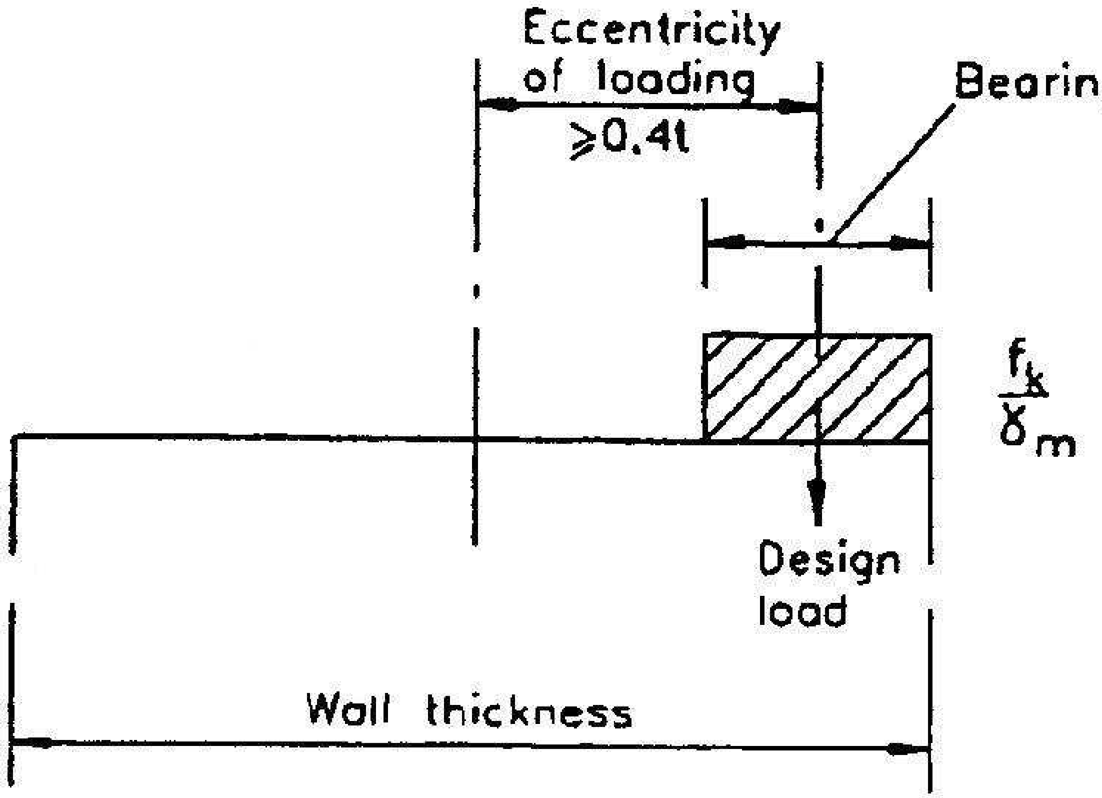


Figure C.2: Eccentricity obtained from design load resisted by stress block

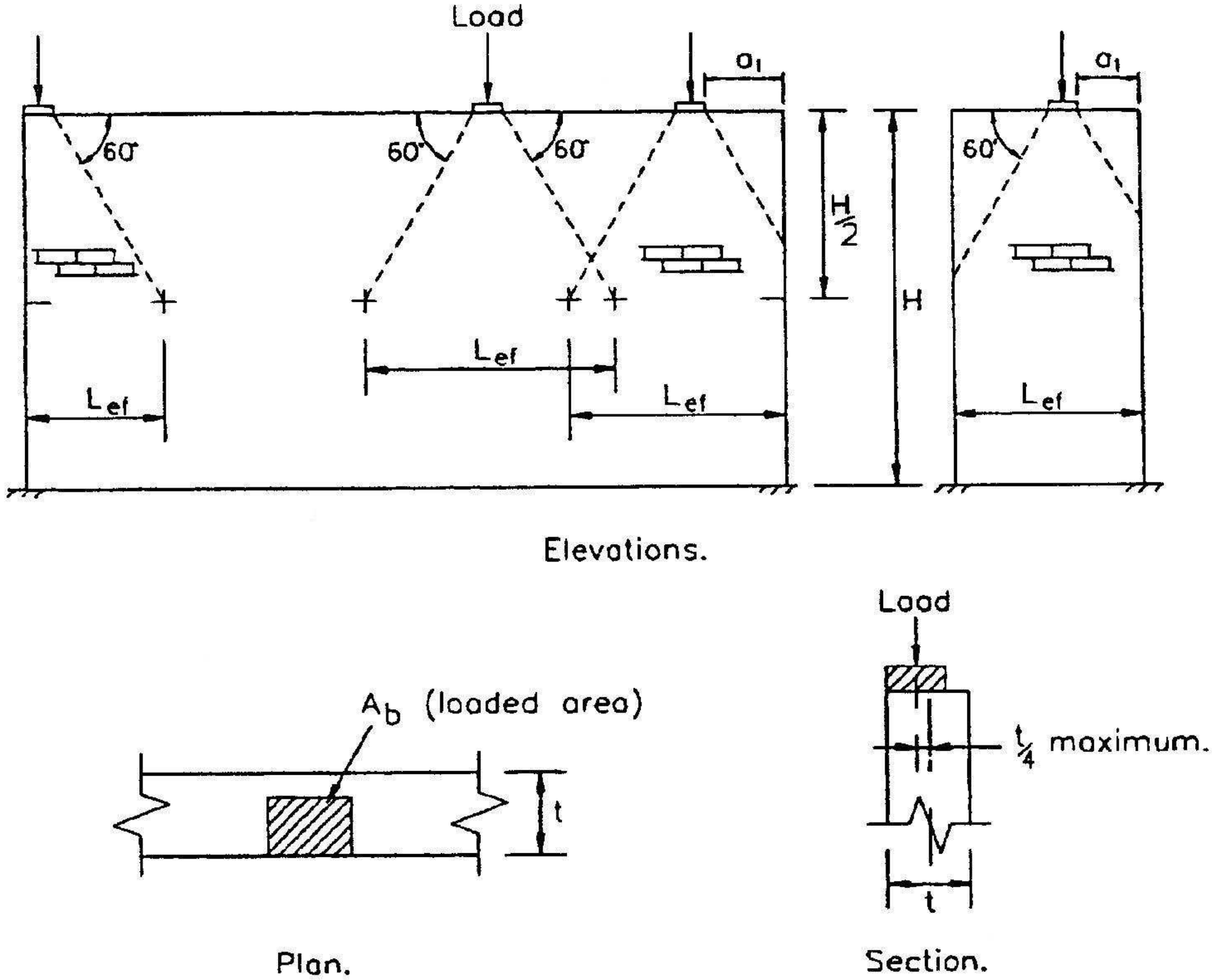
4.4.7.2 Allowance for imperfections

An accidental eccentricity, e_a,

– shall be assumed for the full height of the wall to allow for construction imperfections,

– may be assumed to be h_{ef} / $\boxed{450}$, where h_{ef} is the effective height of the wall.

4.4.8 Concentrated loads



Locally under the bearing of the concentrated load, the design compressive stress shall not exceed the following values:

- Walls built with Group 1 masonry units (not shell bedded):

$$\frac{f_k}{\gamma_M}\left[(1+0{,}15\,x)\cdot\left(1{,}5-1{,}1\frac{A_b}{A_{ef}}\right)\right] \geq \frac{f_k}{\gamma_M}$$

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

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

- All other cases:

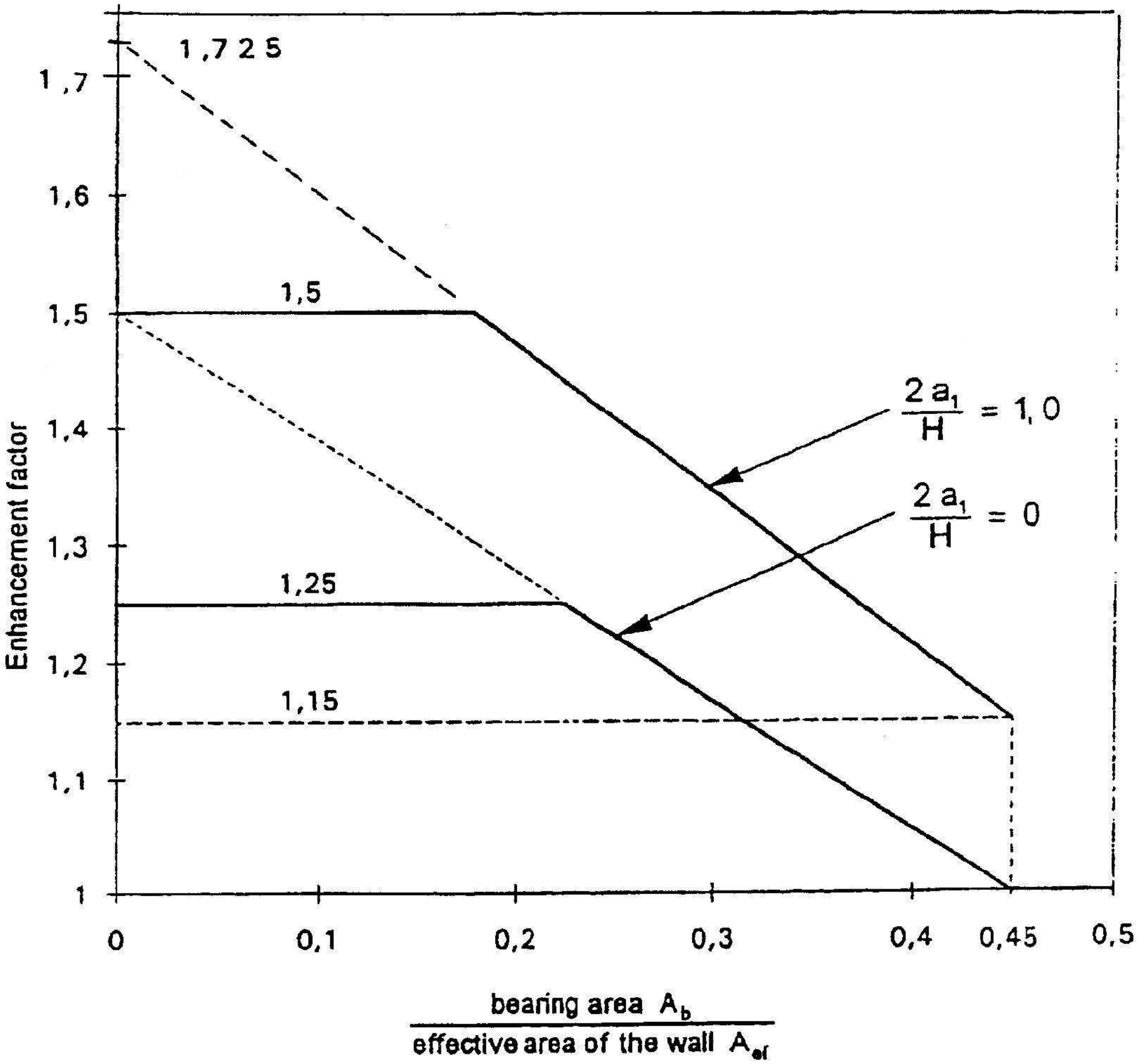
$$\frac{f_k}{\gamma_M}$$

where: $x = \dfrac{2a_1}{H}$

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).

D.1 Graph showing the enhancement factor as given in 4.4.8: Concentrated loads under bearings.



4.5 Unreinforced masonry shear walls

4.5.1 General

- Resistance to horizontal actions
  is generally provided by a system,
  formed by the floors and shear walls.

- Horizontal actions:
  - wind loads,
  - effects due to the imperfection to be assumed (angle $\nu = 1/100\sqrt{hm}$),
  - others in special cases.

- Openings in shear walls
  - can considerably affect their behaviour,
  - their presence should be taken into account.

- Chases and recesses reduce the shear capacity of a wall.

- A limited portion of an intersecting wall can act as a flange to a shear wall, increasing its stiffness and strength:

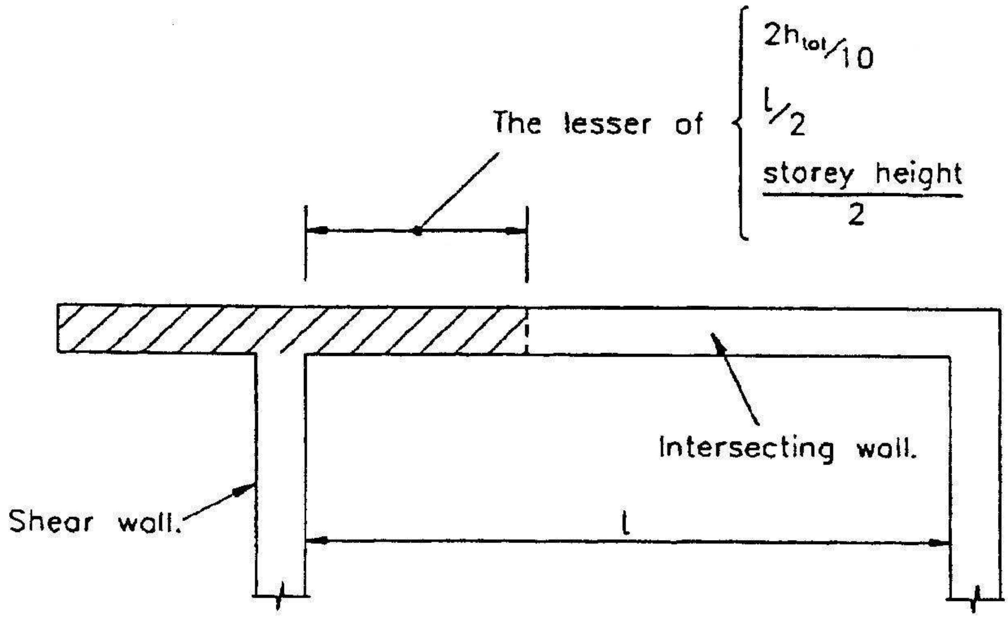


Figure 4.5: Flange widths that can be assumed for shear walls.

For the distribution of horizontal actions only,
the elastic stiffness of the shear walls,
including any flanges, should be used.

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.

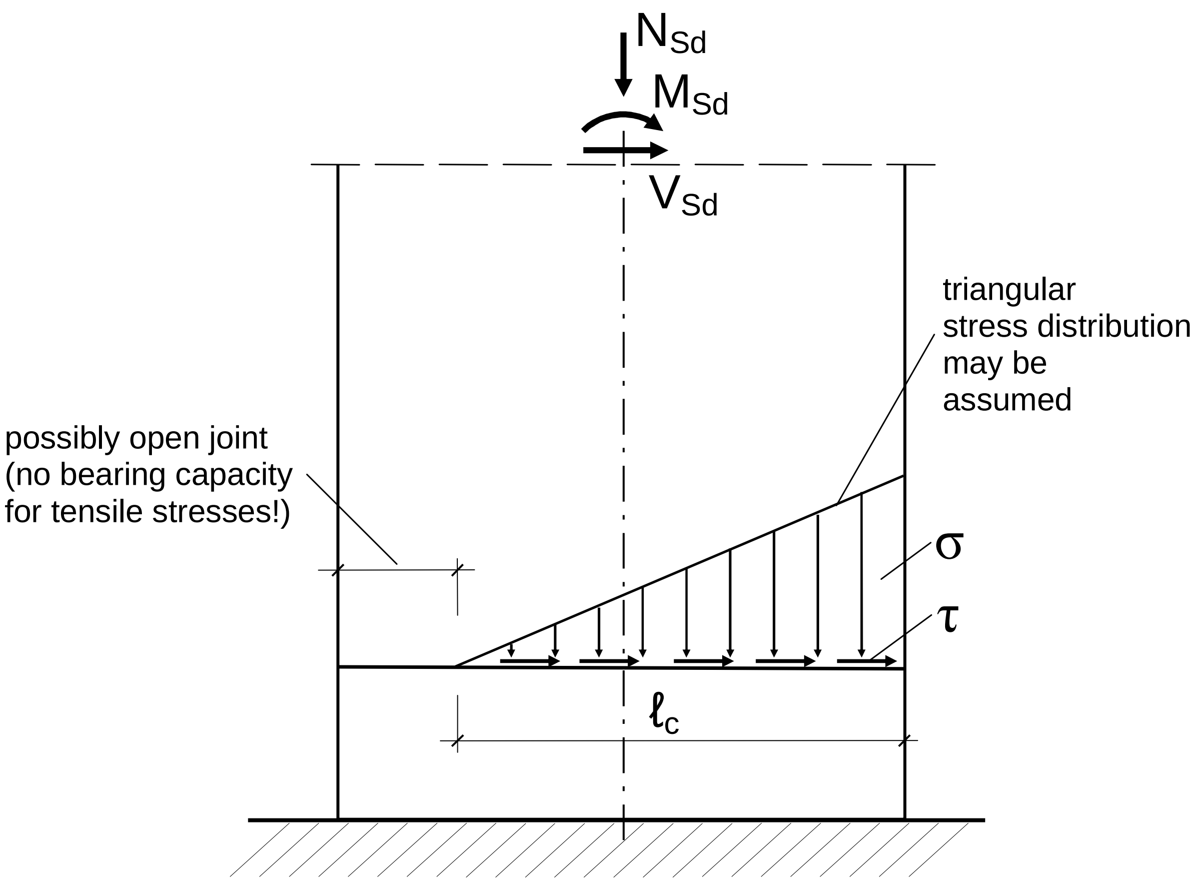
Where the plan arrangement of the shear walls is asymmetric,
or for any other reason the horizontal force is eccentric
to the overall stiffness centre of the structure,
account shall be taken
of the effect of the consequent rotation of the system
on the individual walls (torsional effects).

4.5.2 Analysis of shear walls

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

This causes the following situation of the individual shear wall:

Elevation of a shear wall:



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.

4.5.3 Verification of shear walls

The shear wall and any flange of an intersecting wall
shall be verified
for vertical loading and for shear loading:

$V_{sd} \leq V_{Rd}$

V_{sd}: design value of the applied shear load

V_{Rd}: design shear resistance

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

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.

(2) Chases and recesses reduce
the flexural strength of a wall
used in lateral load design.

(3) 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.

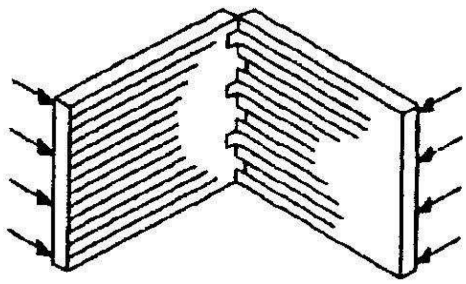
4.6.2.2 Method of design for a wall supported along edges

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

The calculation of the design moment, M_d,
should take this into account
and may be taken as either:

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

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

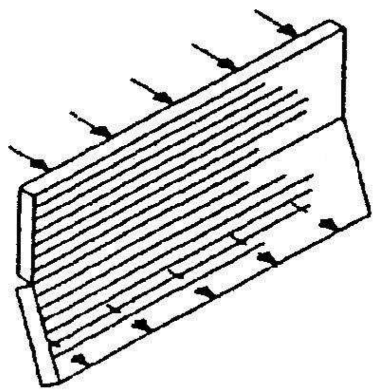


f_{xk2}: Plane of failure
perpendicular to bed joints

or:

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

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



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

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. (it is implicit that a suitable theory is given in the National Application Documents.)

γ_F is the partial safety factor for loads;

μ is the orthogonal ratio of the characteristic flexural strength of the masonry, fx_{k1}/f_{xk2};

L is the length of the panel between supports;

W_k is the characteristic wind load per unit area.

When a vertical load acts so as to increase the flexural strength f_{xk1}, the ortogonal strength ratio may be modified:

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

where:

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

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

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

where:

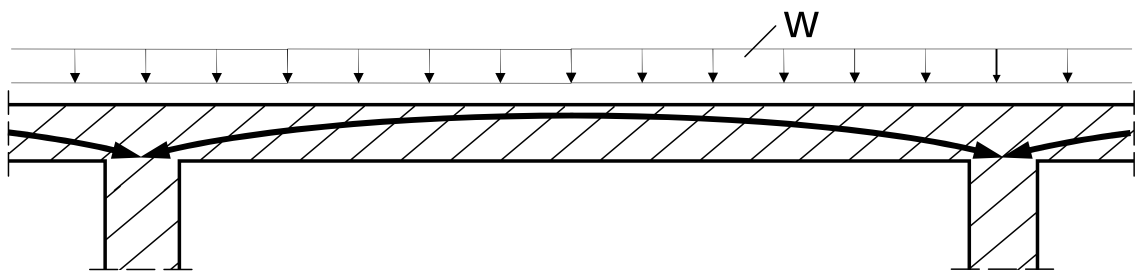
Z the section modulus of the wall.

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,
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.

Example:
Horizontal section:



(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.

(4) The arch rise is given by:

$$0{,}9\,t - d$$

where:

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:

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

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

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

where:

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

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.

Annex

E.1 An empirical method for designing basement walls subjected to lateral earth pressure

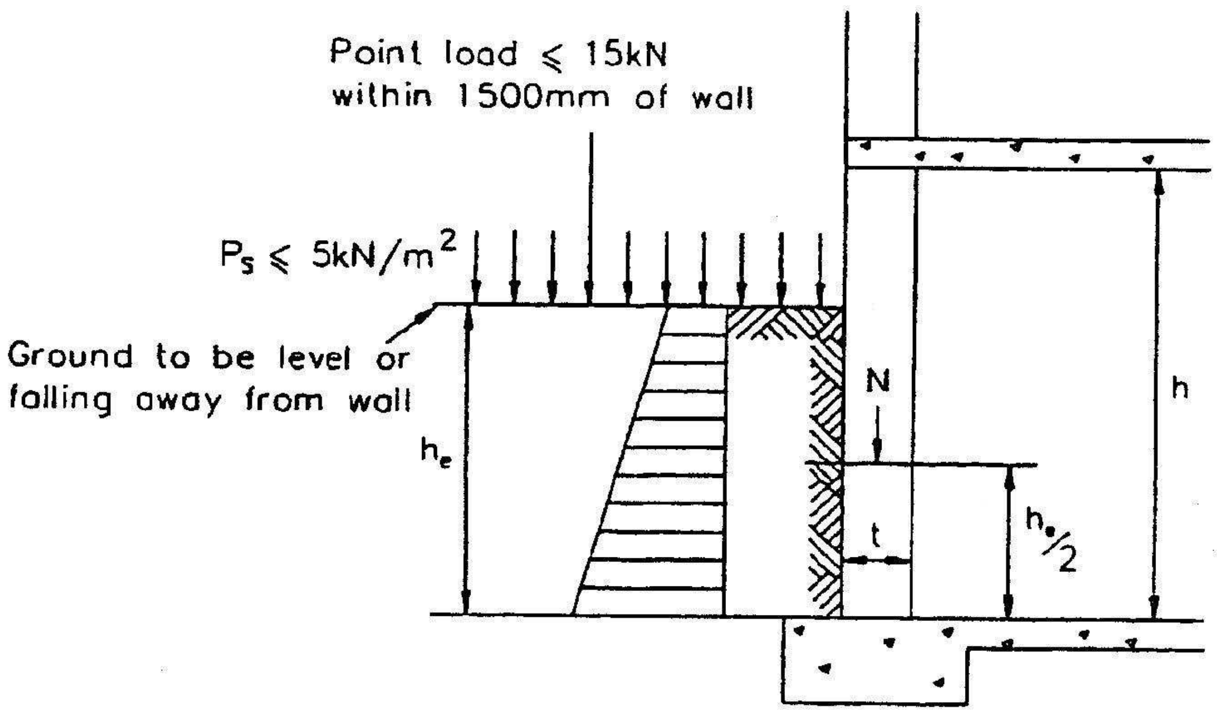


Figure E.1: Design loads for basement walls.

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 basement wall, $h \leq 2600$ 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_3, does not extend 5 kN/m^2
  and no concentrated load within 1500 mm of the wall
  exceeds 15 kN;

– 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 $b_c \geq 2h$:

$$\frac{t\, f_k}{3\, \gamma_M} \geq N \geq \frac{\rho_e\, h\, h_e^2}{20\, t}$$

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;
- ρ_e is the bulk density of the soil;

(II) when $b_c \leq h$:

$$\frac{t\, f_k}{3\, \gamma_M} \geq N \geq \frac{\rho_e\, h\, h_e^2}{40\, t}$$

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 is permitted;

– there is no hydrostatic pressure;

– no slip plane is created by a damp proof course.

Annex:

Structures for the global stiffening of masonry buildings

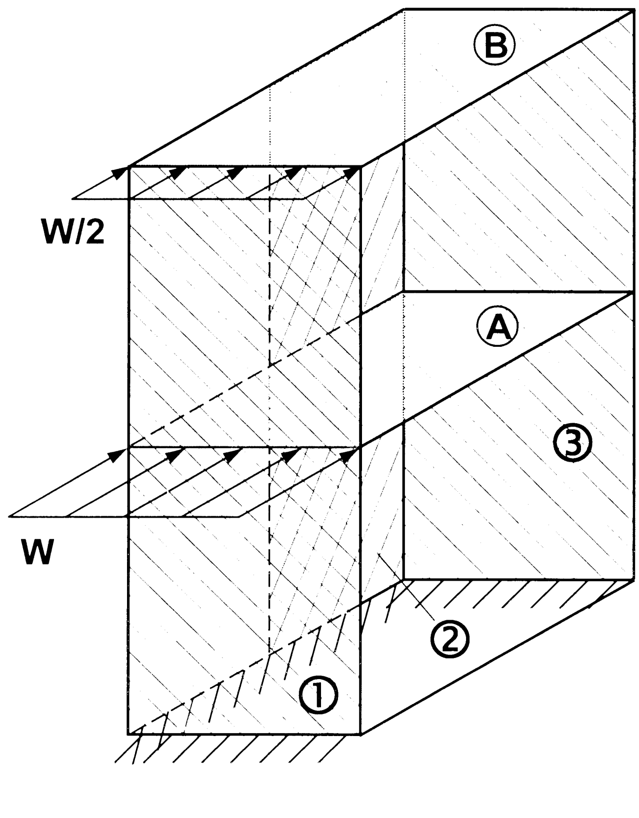
Scopes of the global stiffening system:

- stabilisation of the total building
- load bearing capacity for horizontal actions

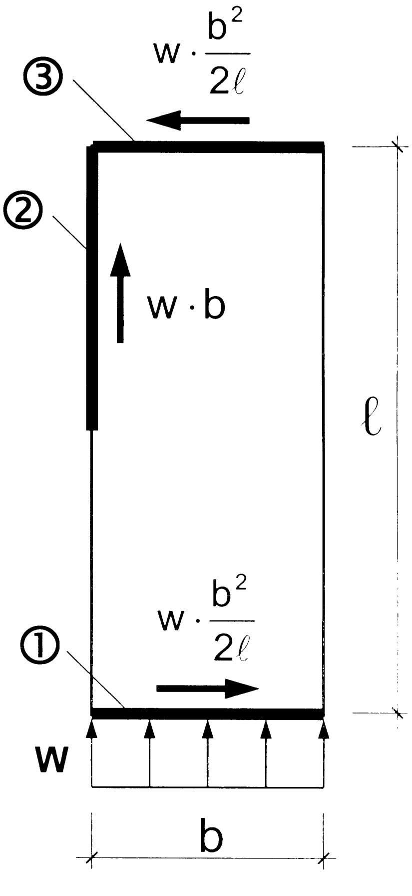
A. Stiffening system in the case of existing floor slabs

Floor slabs of reinforced concrete act as diaphragms, which distribute the horizontal actions onto the vertical shear walls.

Example:



floor plate Ⓐ :



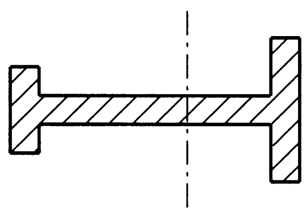
To guarantee a 3-dimensional stable system
at least 3 vertical shear walls are necessary,
the planes of which are not parallel and do not have a common axis.
The distribution of the horizontal actions then is statically defined.

In the case of more than 3 shear walls
the distribution of the horizontal actions is not statically determined.
It has to be calculated according the stiffnesses of the shear walls.

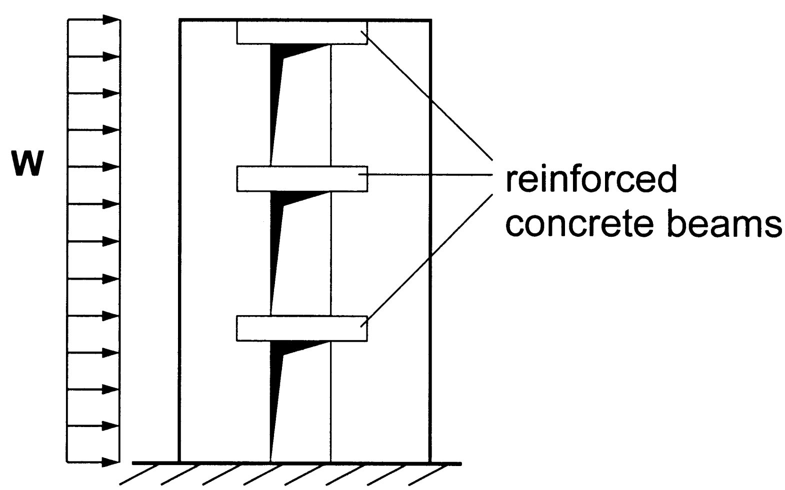
The exact distribution onto all existing walls need not to be calculated in detail if the bearing capacity of some walls already is sufficient.
As Stiffening components can be used:

- long walls
- composite cross sections

} important is a sufficient vertical load



If the bearing capacity of individual walls is not sufficient,
walls which are compartmented by openings can be used, too:



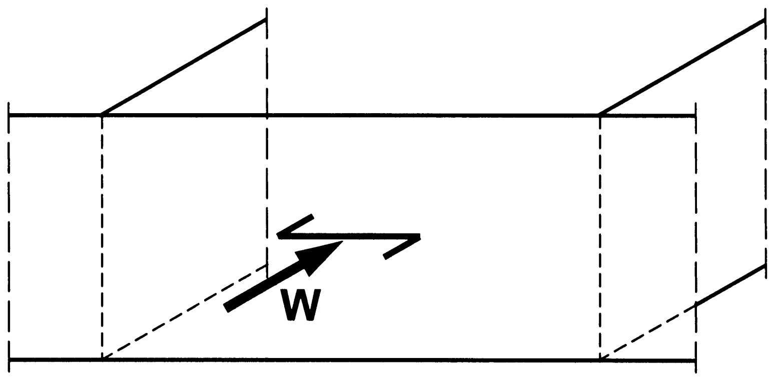
In such cases either a reduced effective stiffness is used
or a calculation of the framework is done.

B. Stiffening system in the case of missing floor slabs

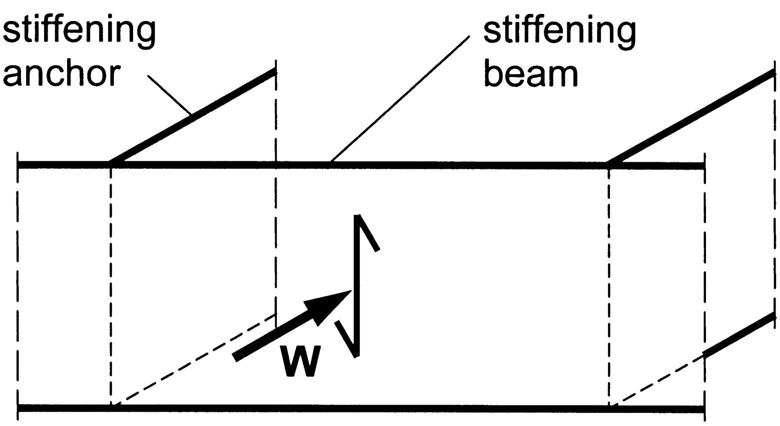
(girder- and timber joist floors)

The horizontal loads must be transmitted directly to the stiffening walls by the loaded walls acting as plates:

either:



or:



This creates a tightly compartmented system of cells which stiffen each other.

Stiffening anchor: components bearing normal forces, "reinforcement of the wall plates", executing following general construction rules.

Stiffening beam: components bearing bending moments, executing verified by calculations.