INTERNATIONAL RECOMMENDATIONS FOR DESIGN AND ERECTION OF UNREINFORCED AND REINFORCED MASONRY STRUCTURES

with an Appendix on
RECOMMENDATIONS FOR SEISMIC DESIGN OF UNREINFORCED, CONFINED AND REINFORCED MASONRY STRUCTURES

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P.O.Box 20704
3001 JA Rotterdam
Netherlands

Wall Structures
INTERNATIONAL RECOMMENDATIONS FOR DESIGN AND ERECTION OF UNREINFORCED AND REINFORCED MASONRY STRUCTURES

Reporter: B.A. Haseltine (United Kingdom)

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INTRODUCTION

In 1980, Publication 58, "International Recommendations for Masonry Structures", was published by CIB. This was the result of discussions by the W23a commission between 1974 and 1979 under the chairmanship of Professor Lewicki.

Following publication and as a result of trial calculations it was decided to amend the recommendations and to expand them to include reinforced masonry. These Recommendations are the result of that decision and follow discussions of three drafts by the W23 committee in Munich (1981), Madrid (1982), Prague (1983), Warsaw (1984) and Copenhagen (1985).

The working group responsible for the preparation of these Recommendations was Mr. B.A. Haseltine (E) (Reporter), Dr. P. Funk (D), Prof. A.W. Hendry (Sc), Prof. B. Lewicki (P), Prof. G. Macchi (I), Prof. W. Mann (D), Mr. O. Pfeffermann (B), Dr. D. Pume (C), Prof. Sahlin (Sw), Mr. H.W.H. West (E) and Prof. C. Zelger (D). Meetings were held in Brussels (1981), Gottenburg (1981), Munich (1982), Berlin (1983), Prague (1983) and Warsaw (1984).

The Appendix, "Recommendations for Seismic Design of Unreinforced, Confined and Reinforced Masonry", is the result of work by a working group co-ordinated by Prof. G. Macchi.
PART 0 - PRELIMINARY

0.1 Introduction

The present recommendations are part of a set of 6 volumes forming an International System of Technical Recommendations for the Design and Execution of Structures. This set has been prepared under the general coordination of the Joint Committee on Structural Safety which includes the following organisations:

- CEB Comité Européen du Béton
- CECM Convention Européenne de la Construction Métallique
- CIB Conseil International du Bâtiment
- FIP Fédération Internationale de la Précontrainte
- IABSE International Association for Bridge and Structural Engineering
- RILEM Réunion Internationale des Laboratoires d'Essais des Matériaux

Volume I Common Unified Rules for different types of construction and materials

II Model Recommendations for concrete structures

III Model Recommendations for steel structures

IV Model Recommendations for mixed structures (concrete-steel)

V Model Recommendations for timber structures

VI Model Recommendations for masonry structures
Simplified rules may be adopted in National Codes for small masonry structures, provided they do not conflict with this document. 

In these recommendations the use of conventional mortars based on cement and sand, possibly with lime, is envisaged. Mortars made with lightweight materials may not give results consistent with these recommendations. Some concrete blocks, particularly lightweight ones, are laid with thin joints; this code may be used for such construction insofar as the masonry strength has been confirmed by tests and experience.
The aim of this set of volumes is to give a common basis for the designer and the builder of structures and to allow different countries to draw on them in drafting national standards.

The present volume VI "International recommendations for Design and Erection of Reinforced and Unreinforced Masonry Structures" is based on volume I as far as the general principles for the Safety and the control of structures, independent of the basic materials (concrete, steel, masonry or timber) are concerned. Anything mentioned in the present volume is specifically applicable to masonry. These recommendations are specially intended for the use of code-drafters but where desired may be used by individuals.

0.2 Object and field of application

The present recommendations are applicable to the design and execution of reinforced and unreinforced masonry structures or their elements, where an engineering approach is needed. Masonry is an assemblage of structural units, either laid insitu or constructed as prefabricated panels, in which the units are bonded and solidly put together with mortar or grout.

These recommendations apply to masonry made with solid or vertically perforated units (bricks or blocks); the recommendations may be used also with horizontally perforated or non-rectangular units but in some circumstances special consideration will be required.
Generally perforations should be less than 50%; where they exceed 50% special consideration will be required.
Units may be of types defined as follows:

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid</td>
<td>units with no perforations. Units with indentations or frogs, in the bed faces that are filled with mortar during construction, can be considered to be solid.</td>
</tr>
<tr>
<td>Equivalent solid</td>
<td>units with less than 25% holes or cavities by volume and having similar behaviour to solid units.</td>
</tr>
<tr>
<td>Perforated</td>
<td>units with more than 25% small holes.</td>
</tr>
<tr>
<td>Hollow</td>
<td>units with more than 25% large holes or cavities through the units.</td>
</tr>
<tr>
<td>Cellular</td>
<td>units with more than 25% cavities which do not effectively pass through the units.</td>
</tr>
<tr>
<td>Horizontally perforated</td>
<td>units with horizontal perforations.</td>
</tr>
</tbody>
</table>

The percentage of perforation is defined as that percentage within the rectangular envelope of the unit.

These recommendations may be valid for non-traditional masonry, made with special mortar or units, when proved by testing or experience.

Masonry is considered to be a material whose strength can be defined by tests on appropriate specimens taking into account the size of the component elements.
Reinforcement can be introduced into masonry to provide additional strength in various ways. For example:

(a) Horizontal reinforcement can be spread evenly throughout the masonry within the mortar.

(b) Vertical reinforcement can be placed in pockets or voids resulting from the shape of the units, the pockets or voids being filled with concrete, cement mortar or grout.
Reinforced masonry is defined as masonry, made in accordance with these Recommendations, acting in combination with reinforcement or prestressing steel to resist the applied loads.
(c) Reinforcement can be placed in pockets or voids formed in the masonry at wide spacing, the pockets or voids being filled with concrete.

(d) Reinforcement can be placed in the cavity between two walls and the cavity filled with concrete.

(e) Reinforcement can be placed to strengthen a wall locally to form for example a beam or lintol.
(f) Reinforcement as described in (a) to (e) can be combined in one panel or element.

(g) Although not strictly reinforced masonry, for some applications e.g. seismic, plain or lightly reinforced masonry can be confined within reinforced masonry; alternatively masonry can be confined between concrete beams and columns but this system is not covered by these recommendations.

These recommendations do not apply to the design of floors using bricks or blocks.
For parts of the structure, other than masonry, the general safety principles can be considered to be satisfied by use of the appropriate volumes, e.g. for reinforced concrete, Vol. II.

For special structures, such as garden walls, arches, chimneys etc... the provisions of these Recommendations should be used in so far as they are applicable.
The symbols used in these recommendations are as follows:-

A  area
E  modulus of elasticity
F  action in general
G  permanent action
M  bending moment
N  axial force
Q  variable action
R  resistance
S  load-effect
V  shear force

Subscript

b  brick or block (compressive strength)
blong  longitudinal compressive strength of brick or block
b°  bond
c  concrete or compression
d  design
f  action
k  characteristic value
m  material (here: masonry considered as a structural material like concrete, steel or timber)

\( \bar{m} \)  average (mean)
p  prestressed
r  resistance
s  steel
t  tension 
u  ultimate
v  shear
x  flexure
y  yield

d  effective depth
e  eccentricity
f  strength in general
g  permanent action
k  statistical constant
q  variable action
s  mean quadratic or standard deviation of sample
t  thickness
y  safety factor
\( \sigma \)  actual compressive stress
\( \psi \)  factor < 1 applied to the actions (combination factor)
\( \phi \)  reduction factor for slenderness and eccentricity of loading
e  strain
0.3 Symbols and units

The symbols used are in conformity with the draft ISO recommendations as presented in Volume I.
The units are those of the International System (SI) based on seven basic units, (metre, second, kilogramme, kelvin, candela, ampere, mole). However, for temperature, this draft continues to use the centigrade or Celsius scale (°C) rather than kelvin (K); the relation between these two scales is

\[ T_K = T^\circ \text{C} + 273 \]

\[ 1K = 1^\circ \text{C} \]

For force and stress, the following relations exist between the SI units and those which were used in the "metric" countries:

unit of force \( = 1 \) N (newton) \( = \frac{1}{9.81} \) kgf; \( 1 \text{kN} = 102 \) kgf

unit of stress \( = 1 \) MPa \( = 1 \text{MN/m}^2 = 1 \text{N/mm}^2 \)
The units are those of the International System (SI)
Paragraph 1.1 and many others in Part 1 are based on volume 1, which contains the general principles of safety, but with some additions specifically related to masonry.
PART 1 - DESIGN OF MASONRY

1.1 Safety principles

1.11 Aim of the design

The aim of design is the achievement of acceptable probabilities that the construction being designed will not become unfit for the use for which it is required during some reference period and having regard to its intended life.

A masonry structure, and each of its elements, should be designed in such a manner that during construction and in use, it can support the actions applied to it (loads, deformations, climatic influences, etc) with appropriate safety.

1.12 Limit states

A structure or structural element ceases to fulfil the function for which it was designed when it reaches a particular state, called a limit state, in which it infringes one of the criteria governing its performance or use.

Two groups of limit states need to be considered as defined below:

(a) ultimate limit state, when the maximum (load carrying) capacity is reached, for instance:

- rupture of critical sections
- loss of equilibrium of a part or of the whole structure considered as a rigid body
- instability, such as buckling
- transformation of the structure into a mechanism
A complete probability analysis requires a knowledge of the statistical nature of loads and other actions, of the variability of the mechanical properties of materials, and of other factors which influence the probability of attaining a given limit state. An analysis of this type is possible practically only in few simplified cases; usually all the relevant statistical data are not available.
(b) serviceability limit state, when criteria governing normal use and durability (under instantaneous or long term loadings) are not satisfied, for instance
- deformation of any part of the structure, so that the appearance or efficiency of the structure, or the behaviour of attached finishings or equipment, is adversely affected
- excessive cracking of any part of the structure
- local spalling
- the effects of concentrated loads, of floor rotation or of local weakness.

1.13 Design method
The object of the design is to keep the probability of a limit state being reached below a certain value, accepted as being satisfactory, based on previous experience for the type of construction. The accepted probability depends on the limit state being considered and the degree of danger it represents.

In this volume measures to avoid a limit state being reached are based on level 1 (see volume I) of the design process. This is a semi-probabilistic approach in which the statistical aspects are treated specifically in defining the characteristic values of actions and strengths of materials; the characteristic values are then associated with partial factors, the values of
$\gamma_m$ is not normally used directly by the designer except for seismic design; it is used as a modification of $\gamma_m$. Higher values of $\gamma_m$ should be adopted for brittle structures, or when human or economic consequences require a lower probability of reaching a limit state, but it is always greater than 1.
which, although stated explicitly, should be derived, whenever possible, from a consideration of probabilistic aspects.

The variability of the actions on a structure is taken into account by defining characteristic values on a statistical basis, but where sufficient data are not available, the values are obtained from an appraisal or experience.

The variability of the strengths and other properties of the structural materials is treated by defining characteristic values related to standard test specimens and procedures, on a statistical basis.

Several types of partial safety factors are introduced:

\( \gamma_m \) for the strength of materials and elements

\( \gamma_n \) a modification of \( \gamma_m \)

\( \gamma_f \) for the actions or action effects

These partial safety factors vary depending upon the materials, the type of action, the nature of the structure, its use, and the importance of the consequences, both in human and economic terms, of the limit state being considered.

The design strength is the characteristic strength divided by \( \gamma_m \), and \( \gamma_n \), when used. The design load is the characteristic value of the action or load effect multiplied by \( \gamma_f \). For variable actions a factor \( \psi \) is also applied.
Depending on the nature of the problem and the method of the analysis, this may be written as:

\[
\text{function } (y_{f_1}, y, y_{f_3}, F_k) \leq \text{function } \left( \frac{f_k}{y_m} \right)
\]

Physical properties - such as thermal, moisture and freezing expansion, creep and shrinkage - may have to be taken into account.
Thus, the effect of the design load must be less than the design resistance:

Design load effect \( \leq \) Design resistance

\( S_d \leq R_d \)

1.2 **Strength of Masonry**

1.2.1 The distinction is made between

- the masonry itself, considered as an assembly of bricks or blocks and mortar, which has intrinsic mechanical properties
- the structural masonry element (e.g. wall or column), the strength of which depends on the strength of the masonry, the geometry of the element, the interaction of adjacent parts, the reinforcement or the prestressing force, etc.

The intrinsic mechanical properties of the masonry are essentially

- the compressive strength
- the shear strength
- the tensile strength
- the flexural tensile strength
- the stress-strain relationship
The characteristic compressive strength of masonry in bending is usually greater than the characteristic direct compressive strength.

For the determination of $f_{mk}$, ideally a large number of tests on specimens which represent the intrinsic qualities of the masonry should be performed. This is possible only in laboratory research or on a very large building site as

- tests on large specimens are expensive
- there are many parameters influencing the mechanical properties of the masonry: the nature of mortar and blocks or bricks, bond, joints and geometry of the specimen.

A simple masonry specimen is desirable, therefore, so that the compressive strength $f_{mk}$ can be determined, either directly, or by correlation between the test results and masonry strength. Suitable tests are described in Appendix 1.

For the direct determination of $f_{mk}$, the specimen must be representative of the masonry considered as a composite material. For industrial products (bricks, blocks) and materials with defined forms (cut stone), this implies that the length of the specimen be such that the vertical joints between
1.22 Characteristic compressive strength of masonry

1.22.1 Definition
The characteristic compressive strength of masonry is that strength which has a probability (accepted a priori) of being attained. The characteristic value is that value below which not more than 5% of test results will fall.

1.22.2 Determination of the characteristic compressive strength

$(f_{mk})$ experimentally
The value of $f_{mk}$ refers to the compressive strength of the masonry, excluding any reinforcement, in the direction of loading; the values in different directions may not be the same. $f_{mk}$ may be determined experimentally, having regard to the size of the bricks and blocks. If $f_{mk}$ is determined from small test pieces, the test must be correlated with the strength of representative sections of masonry and be tested in the direction of the compressive forces. The number of tests must be large enough to enable $f_{mk}$ to be determined satisfactorily by appropriate statistical procedures.
units and their disposition differ little from the ones in normal masonry, and
that the height of the specimen must be such that the effect of platen
restraint is negligible.

Table 1 is derived from a large number of tests on storey height walls
constructed with units laid in the normally accepted way with the load
applied normal to the bed joints, parallel to the perforations if any, and on
similar small walls loaded to failure, in laboratories over the world but
particularly in Germany, G.B., U.S. and Switzerland. Wall tests on the units
and mortars to be used in a particular building may be carried out to
determine a value of \( f_{mk} \) appropriate to the building (see Appendix 1); such
tests may give different results than those in Table 1 below.
1.22.3 Determination of $f_{mk}$ from the compressive strength of the bricks or blocks and the mortar

The characteristic compressive strength of masonry, excluding any reinforcement, may be determined from a knowledge of the compressive strength of the bricks or blocks, and from the strength of mortar used (Appendix 2). National standards should provide a table and/or mathematical relationship of $f_{mk}$ related to the compressive strength of bricks or blocks and mortar normally used in that country. The strength of the bricks or blocks should be determined to correspond to the directions of loading for which $f_{mk}$ is given.
<table>
<thead>
<tr>
<th>Characteristic Compressive Strength of units $f_{bk}$ (N/mm²)</th>
<th>Characteristic Compressive Strength of Masonry, $f_{mk}$ (N/mm²) for mortar type (table 2)</th>
<th>Mean Compressive Strength of units* $f_{bmm}$ (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M2</td>
<td>M5</td>
</tr>
<tr>
<td>2.0</td>
<td>1.3</td>
<td>1.4</td>
</tr>
<tr>
<td>5.0</td>
<td>2.9</td>
<td>3.3</td>
</tr>
<tr>
<td>7.5</td>
<td>3.5</td>
<td>4.1</td>
</tr>
<tr>
<td>10.0</td>
<td>4.1</td>
<td>4.7</td>
</tr>
<tr>
<td>15.0</td>
<td>5.1</td>
<td>5.9</td>
</tr>
<tr>
<td>20.0</td>
<td>6.1</td>
<td>6.9</td>
</tr>
<tr>
<td>30.0</td>
<td>7.2</td>
<td>8.6</td>
</tr>
<tr>
<td>40.0</td>
<td>8.1</td>
<td>10.4</td>
</tr>
<tr>
<td>60.0</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* Mean strengths correspond to characteristic unit strengths when the coefficient of variation is 15%.

**NOTE**

1 $f_b$ is the compressive strength of the unit having a height to width ratio in the region of 1.0 in the direction in which it is to be loaded.

2 Linear interpolation is permissible for units of intermediate strength.

3 The compressive strength of the units must be obtained from tests and using control procedures given in 2.11 and 2.2.

4 When the height to width ratio of masonry units is very different from 1.0, the strength of the units may need to be modified to allow for their different apparent compressive strength.

5 When there are more than about 50% small holes or there are large holes the value of $f_{mk}$ may need to be altered.

6 For reinforced masonry, except if nominally reinforced, it is recommended that only mortar types M20 and M10 are used.
Table 2  Mortar types – composition and strength

<table>
<thead>
<tr>
<th>Type of Mortar</th>
<th>Average compressive strength at 28 days (N/mm²)</th>
<th>Approximate composition in parts by volume</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cement</td>
</tr>
<tr>
<td>M20</td>
<td>20</td>
<td>1</td>
</tr>
<tr>
<td>M10</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>M5</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>M2</td>
<td>2.5</td>
<td>1</td>
</tr>
</tbody>
</table>

NOTE 1  The cement must be suitable for use in mortars for masonry and must be suitable for the conditions in which it is to be used.

2  The strength of mortar which can be obtained with a particular sand will sometimes not reach the values given in table 2; high air contents may also make this difficult. It is permitted to vary the proportions of sand to cement and lime, to achieve the required strengths, following the procedure given in Part II.

3  The compressive strengths quoted in the table relate to tests on mortar prisms 40 x 40 x 160 mm, tested in the laboratory (see Appendix 2).

4  Mortar M2 is not recommended for reinforced structural masonry elements, nor for masonry in contact with soil or exposed to severe weathering.

5  Other compositions may be used when justified by testing and experience. Laboratory tests should give strengths at least 20% higher than those in table 2, but other properties may be important.

6  For guidance, bulk density of cement: 1 200 kg/m³, hydrated lime: 600 kg/m³, dry sand: 1500 kg/m³.
The characteristic compressive strength of masonry ($f_{mk}$) when loaded in directions other than normal to the bed joints should be derived from suitable tests (see Appendices 1 and 2) except that for solid or equivalent solid masonry units, filled hollow blocks and where any indentations are filled, the strength may be taken as that normal to the bed joints from Table 1.

In some situations the shear strength of the bond between intersecting walls may have to be considered (1.5.3.2(e)) and this will depend on the strength of the units. Insufficient data is available on which to base figures for characteristic shear strength in this case.

### Table 3  Characteristic shear strength of masonry: values of $f_{vko}$ and limiting $f_{vk}$

<table>
<thead>
<tr>
<th>Structural Unit</th>
<th>Mortar</th>
<th>$f_{vko}$ (N/mm²)</th>
<th>Limiting $f_{vk}$* (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perforated, hollow and cellular units</td>
<td>M20, M10, M5 M2</td>
<td>0.2</td>
<td>0.8 but $&gt;fb_{long}$</td>
</tr>
<tr>
<td>Calcium silicate solid bricks</td>
<td>M20, M10, M5 M2</td>
<td>0.2</td>
<td>0.8</td>
</tr>
<tr>
<td>Solid or equivalent solid units with $fbk \leq 15$ N/mm²</td>
<td>M20, M10, M5 M2</td>
<td>0.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Solid or equivalent solid units with $fbk &gt; 15$ N/mm²</td>
<td>M20, M10, M5 M2</td>
<td>0.3</td>
<td>1.5</td>
</tr>
</tbody>
</table>

* $f_{vk}$ should not exceed 0.06 $fbk$
1.23 Characteristic shear strength of masonry ($f_{vk}$)

1.23.1 Definition
The characteristic shear strength of masonry ($f_{vk}$) is that strength which has a probability (accepted a priori) of being attained. The characteristic value is that value below which not more than 5% of tests will fall.

1.23.2 Determination of $f_{vk}$ for unreinforced masonry
The characteristic shear strength of unreinforced masonry, $f_{vk}$ may be obtained from appropriate tests or it may be taken as $f_{vk0} + 0.4a_n$ up to a limit depending on the type of unit and the degree of perforation.

where $f_{vk0}$ is the shear strength at zero $a_n$

$a_n$ is the design vertical compressive stress in the wall at the level under consideration. (In this case the load combination giving the least vertical load is appropriate.)
The strength criterion shown is consistent with a wide range of experimental results.

The flexural strength of unreinforced masonry as used in bending calculations involves tension. It should not, however, be used to resist direct tension.

The size and number of specimens to be used together with the method of test, are given in Appendix 3.
1.23.3 **Determination of $f_{vk}$ for reinforced masonry**

The characteristic shear strength of reinforced masonry may be taken as the appropriate value of $f_{vk}$ for unreinforced masonry obtained from Clause 1.23.2.

Where the reinforcement will contribute to the shear strength, the characteristic shear strength may be obtained from appropriate tests or Clause 1.56 may be used if:-

(a) the reinforcement is surrounded with mortar type M20 or M10 (see Table 2) - or

(b) the main reinforcement is placed within pockets, cores or cavities filled with concrete, as specified in Clause 1.25, and surrounded with concrete.

1.24 **Characteristic flexural tensile strength of unreinforced masonry** ($f_{xk}$)

The flexural strength should be used only in the design of masonry under actions normal to the plane of the wall, usually wind. Preferably the characteristic value should be determined from tests following the methods described in Appendix 3.
Typical values obtained from tests using a normal standard of workmanship are shown in Table 4. Test values higher than these may be obtained and can be used when the designer is satisfied that the specification of materials and workmanship are maintained by supervision and site quality control tests. In the absence of test results half the values given in Table 4 may be used.

Table 4  Values of characteristic flexural strength of masonry obtained from tests, $f_{xk}$, N/mm$^2$

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Plane of failure parallel to bed joints</th>
<th>Plane of failure perpendicular to bed joints</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M20</td>
<td>M10,M5</td>
</tr>
<tr>
<td>Clay bricks having a water absorption</td>
<td></td>
<td></td>
</tr>
<tr>
<td>of $&lt; 7%$</td>
<td>0.7</td>
<td>0.5</td>
</tr>
<tr>
<td>of $7% \leq 12%$</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>of $&gt; 12%$</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Calcium silicate bricks and concrete blocks with a compressive strength $&gt; 3.5$N/mm$^2$</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Concrete blocks or highly perforated units with a compressive strength $\leq 3.5$N/mm$^2$</td>
<td>0.25</td>
<td>0.25</td>
</tr>
</tbody>
</table>
The characteristic flexural tensile strength should be determined for the plane of failure parallel to the bed joints and perpendicular to the bed joints.
Recommended concrete mixes are given in Clause 2.14.

The normal stress-strain relationship for reinforcing and prestressing steel is assumed for design purposes to be as follows:-

Simplified stress-strain diagram for mild steel or steel cold worked by rolling or drawing.

Idealised stress-strain diagram for steel cold worked by torsion and/or tension.
1.25 **Characteristic compressive strength of concrete infill** ($f_{ck}$)

The characteristic compressive strength of concrete infill is that strength which has an accepted probability of being attained. Determination of the characteristic strength should be in accordance with the requirements of the appropriate national codes.

1.26 **Characteristic tensile strength of reinforcement** ($f_{sk}$)

The characteristic tensile strength of reinforcing and prestressing steel is that strength which has an accepted probability of being attained. Determination of the characteristic strength should be in accordance with requirements of the appropriate national codes.
For reinforcement embedded in concrete of sufficient size and confinement, the bond strength should be as given in the CEB Model Code. For reinforcement embedded in mortar or in a small concrete section or when the reinforcement is not properly confined, the bond strength should be as given in Table 5.

**Table 5**  Characteristic anchorage bond strength of reinforcement $f_{bok}$ (N/mm$^2$)

<table>
<thead>
<tr>
<th></th>
<th>$f_{bok}$ N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth bars</td>
<td>1.5</td>
</tr>
<tr>
<td>High-bond bars</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Valid for M10 and M20 Mortars
1.27 Characteristic anchorage bond strength of reinforcement ($f_{\text{g0}}$)

The characteristic anchorage bond strength depends on the type of reinforcement and the strength of the concrete or mortar in which the reinforcement is embedded.

For special reinforcement, such as truss or ladder type bars, the anchorage strength should be determined by means of appropriate tests.

1.28 Design strength of masonry

- in compression $f_{\text{md}}$
- in shear $f_{\text{vd}}$
- in flexure (tension) $f_{\text{xd}}$

The design strength of masonry is given by

- in compression $f_{\text{md}} = \frac{f_{\text{vk}}}{\gamma_{\text{mm}}}$
- in shear $f_{\text{vd}} = \frac{f_{\text{vk}}}{\gamma_{\text{mm}}}$
- in flexure $f_{\text{xd}} = \frac{f_{\text{vk}}}{\gamma_{\text{mm}}}$
The partial safety factor $Y_{mm}$ is considered to take account of

- the possibility of unfavourable deviations of the strength of the masonry from the characteristic value
- the possibility of differences between the actual strength of the masonry in the structure from that derived from test specimens (due to bond, workmanship, weather-conditions)
- the possible local weakness in the masonry arising from the construction process (filling of the joints, defective units)
- the possible differences of the masonry strength derived from dimensional and geometrical inaccuracy.

$Y_{mm}$ may be modified by another factor $Y_n$ which need only be considered in the ultimate limit state. $Y_n$ takes account of

- the nature of the structure and its behaviour, in particular the risk of brittle failure and of progressive collapse
- the loss of human lives and economic cost of damage and repair of the structure, furniture or equipment and consequential loss.

Masonry is a composite material whose strength depends on the strength of the structural units, the mortar and the way in which the units and mortar are put together; the strength of reinforced masonry depends additionally on the strength of the reinforcement and any concrete infill. The values of $Y_{mm}$, therefore, need to be chosen taking into account the quality of control exercised in the manufacture of the structural units and the quality of supervision on the site. The requirements for quality control of units and site work are given in Part 2, clause 2.2.
The value of $\gamma_{mm}$ may range from 2.0 to 3.5 (in some cases 1.7) and should be selected by each National Code to suit the type of structural units produced in that country, the manner of building and the level of control possible with traditional building methods. Values for $\gamma_{mm}$ should be selected after calibration with existing practice has been completed.
The values of $Y_{mm}$ given in table 6 include a brittle allowance for $Y_n$.

**Table 6** Values of $Y_{mm}$ for the ultimate limit state for masonry in bending, compression and shear

<table>
<thead>
<tr>
<th>Category of manufacturing control</th>
<th>Category of construction control</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>A</td>
<td>2.0*</td>
</tr>
<tr>
<td>B</td>
<td>-</td>
</tr>
</tbody>
</table>

* In some countries this figure may be reduced to 1.7 when the accuracy of the associated design methods enables National Standards to recognise such a figure.

It is not recommended that construction control C is used for reinforced masonry except when nominally reinforced e.g. some seismic cases.

Walls and columns of very small plan area are vulnerable to damage in use.

The designer should consider very carefully the advisability of using masonry having a plan area less than $0.04m^2$.

The value of $Y_{ms}$ is generally taken as 1.15

The value of $Y_{mb}$ is generally taken as 1.5
For columns and walls of small plan area (<0.2m²) the value of $\gamma_{mm}$ should be increased by dividing by $(0.70 + 1.5A)$ where $A$ is the plan area of the loaded wall or column in m².

The design strength of reinforcement in masonry is given by:

- in tension $f_{td} = \frac{f_{yk}}{\gamma_{ms}}$

- in bond $f_{b0d} = \frac{f_{b0k}}{\gamma_{mb}}$
There is a great variation of up to ± 50% in this value of $E$.

When it is necessary to know $E$ more precisely, it is recommended that it be determined experimentally.

For concrete blockwork masonry, the creep strain ratio may vary between 1 and 5, according to the thickness of the element (creep increases when thickness decreases), the atmospheric humidity (creep decreases when humidity increases) and the age of the concrete (creep decreases when age increases).

$$\varepsilon_{\text{total}} = \varepsilon_{\text{elastic}} + \varepsilon_{\text{creep}} + \text{any other strains}$$
When verifying the stability in the case of accidental actions, $\gamma_{mm}$ may be taken as 1.5 except for seismic design, for which values are given in the Seismic Appendix.

1.29 Deformation properties of masonry

1.29.1 Modulus of Elasticity $E$

In the absence of better information, the short term secant modulus of elasticity under service conditions for all masonry may be taken as 1000 $f_{mk}$, where $f_{mk}$ is the characteristic compressive strength of masonry (1.22).

The long term elastic modulus will be reduced by creep in the masonry.

1.29.2 Shear Modulus

The Shear Modulus may be taken as 0.4$E$.

1.29.3 Creep

In the absence of better information, the ratio of the maximum value of creep strain to the elastic strain of masonry, under the effect of permanent loads, may be taken as follows:

- brick masonry 1.0
- calcium silicate and dense concrete block masonry 2.0
- lightweight concrete block masonry 2.5
6 and $10 \times 10^{-6}/°C$ are mean values and can vary substantially both above and below.

After a series of freezing cycles, masonry may become permanently greater in size.
1.29.4 **Thermal expansion**

The coefficient of thermal expansion for clay brickwork may be taken as $6 \times 10^{-6}/\degree{C}$ and for calcium silicate brickwork and concrete blockwork $10 \times 10^{-6}/\degree{C}$.

1.29.5 **Reversible moisture movement**

The magnitude of the expansion of masonry from the dry to the saturated condition, and the similar shrinkage on re-drying may be taken as

- $+0.2$ to $-0.1$ mm/m, for brick masonry
- $-0.1$ to $-0.4$ mm/m, for calcium silicate and ordinary concrete block masonry
- $-0.2$ to $-0.6$ mm/m, for lightweight concrete block masonry

1.29.6 **Irreversible moisture movement**

The irreversible moisture expansion of the clay brickwork to be used should be determined experimentally.

1.3 **Loads and other actions**

1.3.1 **Definitions**

In order to define the representative values of actions and to determine their rules of combination, the actions are classified according to the importance of their variations and occurrence in time into permanent, variable and accidental.
\( \psi_0 \) is a reduction factor which takes into account the reduced probability of all the variable actions being considered reaching their characteristic value together.

\( \psi_1 \) is a reduction factor which takes into account that part of the characteristic action that can occur a large number of times in the life of the structure.

\( \psi_2 \) is a reduction factor which takes into account that part of the characteristic action that can be expected to act for long periods of time.

\( \psi_1 \) and \( \psi_2 \) are used in considering serviceability limit states which are governed by quantities or events that are sensitive to the duration of the actions, e.g. cracking, deflections etc.

Examples of \( F_{\text{acc}} \): impact forces, pressures due to gas-explosions, natural cataclysms (tornadoes, avalanches, earthquakes).
Permanent actions (G) for which variations are rare or slow, and negligible in relation to the mean value
- representative values: $G_{\text{max}}$, $G_{\text{min}}$ gathered into one value G

Variable actions (Q) for which variations are frequent or continuous, and not negligible in relation to the mean value
representative values of the variable actions are:
- characteristic value $Q_k$ (1.32.2)
- combination values $\psi_0 Q_k$ to be applied in the ultimate limit state 1.33.2(a)
- frequent values: $\psi_1 Q_k$ (1.33)
- quasi-permanent values $\psi_2 Q_k$ (1.33)

Maximum values and minimum values, which may be zero, are defined when appropriate.

Accidental actions ($F_{\text{acc}}$) for which the occurrence in any given structure is unlikely during the life-time of the construction but the magnitude of which would be important.
The mean densities of the materials used in a masonry structure are given in National Standards, or books of reference.
1.32 Characteristic values of loads and other actions

1.32.1 Permanent actions

(a) Self weight of the structure: The self weight of the structure should be calculated from the nominal dimensions of the structure and the mean densities of the relevant materials.

(b) Self weight of the non-structural parts: Generally two values, a maximum ($G_{\text{max}}$) and a minimum ($G_{\text{min}}$), should be calculated taking into account all predictable variations. The choice of $G_{\text{max}}$ and $G_{\text{min}}$ is determined by the most unfavourable cases of stress; in practice, $G_{\text{max}}$ is often a nominal value with $G_{\text{min}}$ taken equal to 0 (no load) when it can be so.

(c) Soil actions: The maximum value of active soil pressure and the minimum value of passive soil pressure should be considered.

1.32.2 Variable actions ($Q_k$)

(a) Imposed loads: The values are defined by national codes or regulations. When none are defined the most unfavourable characteristic loads (maximum or minimum) corresponding to a mean return period, defined in Vol I, should be used.

(b) Climatic actions: National codes or rules define the values of the climatic actions (due to temperature, wind, snow and rain) as a function of the regional characteristic of the climate. These values are based on statistical information resulting from meteorological observations, and generally correspond to return periods given in...
$\gamma_{f2}$ has been replaced by $\psi$ factors

It should be noted that the partial safety factor $\gamma_{f3}$ included in the combination rule $\alpha_1$ covers normal uncertainties in the determination of axial loads. The higher uncertainty connected with the determination of bending moments in walls is covered by an increase in $\gamma_{f3}$, e.g. for bending moments in the rigid or semi-rigid joints scheme of 1.52.1 or by the consideration of accidental eccentricities, e.g. in the hinged joints scheme of 1.52.1.
Volume I. Reduced values \( \varphi_0 \) (or \( \varphi_1 \) or \( \varphi_2 \)) \( Q_k \) may be used for different combinations of climatic actions.

(e) **Construction loads:** Loads generated by the proposed method of construction should be considered in the design.

1.32.3 **Accidental actions (F_{acc})**

In general, accidental actions are given nominal values in each country's structural codes. This action is applied only in the accidental combination of actions.

1.33 **Design values of loads and other actions**

1.33.1 **Partial factors to be applied to actions: \( \gamma_f \) and \( \psi \)**

The partial safety factor \( \gamma_f \) should be considered as a function of two factors \( \gamma_{f1} \) and \( \gamma_{f3} \), where

\( \gamma_{f1} \) takes account of the possibility of unfavourable deviations of the actions from the characteristic values,

\( \gamma_{f3} \) takes account of possible inaccurate assessment of the action effects and their significance on safety and variations in the dimensional accuracy achieved in construction, as they affect the action effects.

\( \psi \)-factors (\( \varphi_0, \varphi_1, \varphi_2 \), see 1.31) are applied to the characteristic values of the variable actions (\( Q_k \)) for several cases of the combination of actions.
Combinations of actions are assemblies of representative values which, including the factors \( \gamma_f \) and \( \psi \), lead to design actions, covering with an acceptable probability the actions resulting from combinations of real values. The further application of \( \gamma_f \), similarly leads to acceptable action effects.

The symbol \( \Sigma \) means the contemporary application of loads for normal buildings.

Suggested values for \( \psi_0, \psi_1, \psi_2 \),

<table>
<thead>
<tr>
<th>Action</th>
<th>( \psi_0 )</th>
<th>( \psi_1 )</th>
<th>( \psi_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imposed loads, in domestic buildings and office buildings</td>
<td>0.5</td>
<td>0.8</td>
<td>0.4</td>
</tr>
<tr>
<td>Wind, snow loads</td>
<td>0.6</td>
<td>0.2</td>
<td>0</td>
</tr>
</tbody>
</table>

Normally two load combinations need to be considered:

1. Dead + live + wind load

\[
F_0 = 1.35 \left\{ \frac{G_k + 1.5 Q_{1k} + 1.5 \times 0.5 \sum_{i=2}^{n} Q_{ik} + 1.5 \times 0.6 Q_{(n+1)k}}{0.9} \right\}
\]

\( Q_{ik} - nk \) = live loads on the floors; \( Q_{(n+1)k} \) = wind load (can be 0)

2. Dead + wind + live load

\[
F_0 = 1.35 \left\{ \frac{G_k + 1.5 Q_{1k} + 1.5 \times 0.5 \sum_{i=2}^{n+1} Q_{ik}}{0.9} \right\}
\]

where

\( Q_{1k} \) = wind load and

\( Q_{2k} \rightarrow (n+1)k = \) live load on the floors (can be 0)

\( n = \) number of storeys
1.33.2 Combination of actions

The following combinations of actions are in accordance with Volume I of the "Joint Committee on Structural Safety".

(a) Ultimate limit state (ULS)

a1) Fundamental combination

\[ S_U = S \left\{ 1.35 \sum G_{\text{max}} + 0.9 \sum G_{\text{min}} + 1.2 P + 1.5 (Q_{ik} + \sum_{i=2}^{n} \psi_{oi} Q_{ik}) \right\} \quad (1) \]

where

- \( S_U \) is the design action effect
- \( Q_1 \) is the basic action in the combination
- \( \psi_{oi} Q_{ik} \) is the combination value of the accompanying action
- \( P \) is the prestressing force

This combination includes \( \gamma_{f3} \)
This combination \((a_2)\) covers

- either an accidental action \((F_{\text{acc}})\) in both permanent and transient situations

- or permanent and variable actions (excluding accidental action) in accidental situations: for instance after a gas explosion, a structural member (floor or wall or column), has been put out of service; this is an accidental situation for which the remaining structure has to be checked under the accidental combination of actions \((a_2)\), with \(F_{\text{acc}} = 0\).

Model Code Vol I considers other SLS, under infrequent or quasi-permanent combinations of loads; these are cases particularly applicable to reinforced masonry and concrete structures.
a2) Accidental combination

\[ S_{acc} = S (F_{acc} + P + \sum G_{max} + \psi_{1} Q_{1k} + \sum_{i=2}^{n} \psi_{2i} Q_{ik}) \]  

where \( F_{acc} \) is the basic action in the combination or zero.

(b) Serviceability limit state (SLS)

For unreinforced masonry structures the SLS i.e. cracking, deformation or durability is rarely considered in design because the rigidity of the structure leads to very small deformations. Consideration of thermal and other movements in relation to other elements of the structure may be necessary.

For reinforced masonry structures it may be necessary to consider the SLS as the structure is, by design, ductile. It may be necessary to verify that deformation e.g. due to creep, and cracking in the SLS is sufficiently limited. In this case, the frequent combination is used:

\[ S = S (\sum G_{max} \text{ or } \sum G_{min} + P + \psi_{1} Q_{1k} + \sum_{i=2}^{n} \psi_{2i} Q_{ik}) \]  

(3)
1.4 Structural behaviour and overall stability

1.41 Structural behaviour under normal loads

Buildings constructed using reinforced or unreinforced masonry are usually complex three-dimensional structures of floors and walls with all the elements interacting to resist the applied loads.

Since there is little knowledge of the real behaviour of such structures, their detailed design is usually simplified. Design can be carried out by considering the structure as a three-dimensional layout of walls and floors, as a series of structural frames or as a number of independent elements; the latter case is the most usually adopted basis of design.

Detailed design is usually based on the independent analysis of the strength of:-

(a) members carrying primarily vertical loads.
(b) members carrying primarily horizontal loads.
(c) walls or columns carrying a combination of vertical and horizontal loads including shear walls.
(d) beams
(e) floors

Where the member to be designed forms part of an indeterminate structure the method of analysis employed to determine the forces applied to the member should be based on as accurate a representation of the behaviour of the structure as is practicable.
Details for the consideration of accidental damage in the design of buildings for residential or similar purposes are given in Appendix 4.
The general arrangement of the structure and the bonding or connection of its various elements should be such as to give an appropriate stability or robustness. When this stability or robustness cannot be seen to exist or does not implicitly result from the calculation relating to wind or seismic actions, it should be checked that, in the absence of any external lateral force, the structure can resist a horizontal force at any level equal to 1% of the vertical loads, considered in the combination given in 1.33.2 (b), above the point of application of the horizontal force.

1.42 Structural behaviour under accidental loads other than earthquake

The structure should be designed in such a way that there is a reasonable probability that it will not collapse catastrophically in the event of misuse or accident.

This requirement can be achieved by ensuring that either

(i) the structure is designed so that any primary local collapse under the effect of the accidental combination (a2) of actions causes only partial collapse of the structure, or

(ii) structures or structural members and their connections which do not satisfy (i) above are designed to withstand one or other of the following loads:

a) a pressure in any direction as specified by National Codes

b) a horizontal impact force appropriate to the circumstances
Account should be taken in the design for any holes or grooves which are required (see 2.32.4).

See ISO-draft on limitation of deformation in building.

If the floors are composed of juxtaposed precast elements, connections should be provided between units, e.g. by use of reinforcement in joints or an in-situ topping on a floor.
1.43 Structural behaviour under earthquake action

The specific design is covered by "Recommendations for Seismic Design of Unreinforced, Confined and Reinforced Masonry Structures", Appendix to the present recommendations.

1.5 Design of constituent structural members

Besides the design of the members in the ultimate limit-state, as described hereafter, the serviceability of the structure and its members must be checked, especially to avoid excessive cracks or deflections which might damage facing materials, partitions, finishings or technical equipment. In order to ensure satisfactory behaviour of the building under normal conditions it must be established that cracks and deflections do not exceed limit values, which depend on the nature of the building.

1.51 Bearing of floors on walls

1.51.1 Load distribution

Appropriate measures should be taken to ensure that loads on floors are distributed over a sufficient width, to avoid, as much as possible, their localized effects on the bearing walls (see 1.52.2 - 3).

1.51.2 Support conditions

Floors or other structural elements should have sufficient bearing on walls, depending on how they are constructed, to ensure that loads will be satisfactorily transferred to the walls.
Figure 1:
Statistical Scheme

$e_u$ - Eccentricity of loading from wall above
$e_l$ - Eccentricity of reaction from left slab
$e_r$ - Eccentricity of reaction from right slab
1.52 Bearing walls

1.52.1 Statical schemes

The interaction between bearing walls and floors in terms of transferred forces and bending moments depends on the materials used or the details of the construction and on the joint characteristics. A scheme, close to the actual behaviour of the structure, should take into account the non-linear laws of deformation of the single members (floor, wall, joints) due mainly to cracking and non-elastic strains, both at the ultimate limit state and at the serviceability limit state.

In practical cases one of the following statical schemes may be assumed as representing the actual behaviour (Figure 1).

Rigid joints (1°) In this scheme it is assumed that the rotation of the wall and floor slab centre lines, at the joint between the wall and floor slab, is the same. In estimating the interactive forces, the tensile strength of the masonry is neglected and the wall stiffnesses are modified to allow for cracking. This scheme is only valid within the moment capacity of the joint and this must be checked for the limit state under examination; in reinforced masonry, this capacity may be more easily provided.

Semi-rigid joints (2°) In this scheme rotations are possible in the joint at a reasonably constant value of the end moment of the slab and wall. This scheme is only valid within the rotation capacity of the joint and this must be checked for the limit state under examination.
Hinged joints (3°) In this scheme eccentricities are assumed within which the wall to floor joints are considered to behave as hinges; each storey height of wall is considered to be hinged at its bottom and above the upper floor where the wall above rests on the floor.

In principle, there is no objection to the adoption of one scheme for one part of a building and another scheme for the remaining part. Similarly, for the serviceability limit-state a scheme different from that adopted for the ultimate limit-state could be used.

Generally, the scheme with hinged joints is conservative using the eccentricities indicated in this document. The scheme with rigid joints may be used when there is justification; it should be used for exterior walls when thermal gradients can be significant.

1.52.2 Design of bearing walls

1.52.2-1 General

For the analysis of a bearing wall, the load effects should be determined by applying design loads and other actions (see 1.33) to the chosen statical scheme.

The axial load can be determined by the traditional simplified methods, taking into account the direction of span of the floors and appropriate angle of distribution for concentrated loads (1.52.2-3). The bending moments should be derived from an analysis carried out on the selected statical scheme, neglecting second order deformations.
Shear walls and horizontal diaphragms must provide sufficient lateral stiffness to justify this assumption in respect of sway deflections.

1.52.2-2 Determination of load effects

2.1 Rigid and semi-rigid joints scheme.

The effects of vertical and horizontal loads, thermal effects, continuity etc. will have been allowed for in the calculation of moments. An allowance for dimensional and other inaccuracies is to be made by multiplying the moments so derived by $Y_f = 1.3$. For the design of a wall the loading at the top and bottom may be expressed in terms of a vertical load and an equivalent eccentricity. The calculation of the strength response to correspond to the load effects should be consistent with the assumptions made in the analysis. However, the eccentricities calculated may be used in the hinged joint scheme, below, if desired.

2.2 Hinged joints scheme

If this scheme is adopted, structural actions, and actions due to effects other than vertical loads (wind, temperature and dimensional inaccuracies) must be taken into account in terms of eccentricities.

The virtual eccentricity of load on a wall arises from several causes as below; the combination indicated in e) should be used in design.
In a suitably braced structure it can generally be assumed that the load, arising from a wall above the level under consideration, is applied concentrically, (apart from $e_a$, the accidental eccentricity).

Fig. 2. The resultant of the load of the floor simply resting on the wall is situated at $a/2$ or $a/3$ from the side of the wall, according to whether the support is flexible and levelled (mortar) or dry. Generally, this applies for the part of the loads due to the dead weight of the floor. The live loads have an eccentricity between 0 and $e$ according to the degree of fixing of the floor-wall junction. ($o = $ perfect fixing; $e =$ free rotation).

Fig. 3: $N$ great with regard to $g + q$. $e = 0$ in the case of a continuous floor having similar spans on either side of the wall.

Fig. 4: Structural eccentricity in the case of a discontinuous floor above the wall support (dead weight $g +$ variable load $q$ of the floor applied in the mean mechanical plane of the wall).

Fig. 5: Structural eccentricity in the case of a floor practically fixed at the support: the eccentricity of the applied load $q$ ($q_1$ or $q_p$) of the floor is between 0 and $e$ (or $e_q$ $e_p$).

Total load for wall design $N = N + g + q$
a) Structural eccentricity

- eccentric bearing of floors on walls
- bending moments due to continuity between floors and walls
- eccentricity of loading coming from the upper part of the construction, if the section of transmission at the floor-level is not superimposed exactly over the wall-section.

The structural eccentricity, therefore, only results from the effect of the floor on the level considered, as illustrated by figures 2, 3, 4 & 5 on which \( g \) represents the dead weight of the floor and \( q \) the variable load on it.
Appendix 6 gives some guidance on the choice of $e_a$.

In Appendix 6, $\rho$ and $\lambda_e$ and $t_{ef}$ are discussed.
b) **Accidental eccentricities** \((e_a)\)

The effect of the inclusion of units of different characteristics within a wall and of dimensional inaccuracies in the plan or elevation may lead to additional eccentricities, \(e_a\). The designer must assess the appropriate value to be used in design from his knowledge of the materials and workmanship; values may be given in National Codes.

c) **Eccentricities due to slenderness of the wall** \((e_s)\)

Second order deformations due to the slenderness of the wall should be considered. These should be based on the geometric slenderness of the wall which is taken as

\[
\frac{\rho l_e}{t_{ef}}
\]

(4)

where \(l_e = \) effective height of the wall  
\(t_{ef} = \) effective thickness of the wall  
\(\rho = \) a factor which takes into account the boundary conditions of the vertical edges of the wall

For unreinforced walls the slenderness ratio should not exceed 25 and for reinforced walls it should not exceed 40.

d) **Eccentricities due to the environment** \((e_w)\)

Allowance should be made for eccentricities due to the effect of wind and thermal gradients on external walls.
An increase in strength under bearings can be assumed, but care must be taken that bearings are not too small or too close to the edge of the wall, thus enabling a wedge of masonry to be sheared off the wall. The guidance in Appendix 5 is to avoid this happening: the angle $\alpha$ is usually taken between $45^\circ$ and $60^\circ$. 
e) **Resultant eccentricity (e)**

The eccentricities under a), b), c) and d) should be combined in order to give the most unfavourable eccentricity for the wall.

From the resultant eccentricity in the top and bottom sections of the wall, the calculated virtual eccentricity for verification of the buckling stability of the wall is given by

\[(0.6 \cdot e_1 + 0.4 \cdot e_2) + e_a + e_w + e_s \]  

(\(+ e_s \) if not allowed for in a reduction factor \( \varphi \), see 1.52.2-4)

where \( e_1 \) is numerically greater than \( e_2 \). When they are of opposite sign, \( e_2 \) should be taken as zero.

1.52.2-3 **Determination of the vertical load**

The vertical load to be considered at any one level is the sum of the appropriate design loads, including the vertical effect of wind resisted by walls.

a) **Concentrated loads under bearings**

When a beam or other structural member imposes a concentrated load on a wall, the design compressive strength of the masonry may be assumed to be up to one half greater than the value from 1.22 depending on the arrangement of the load on the wall. Some suggestions for the calculation of the increase are given in Appendix 5. The load under a bearing may be assumed to disperse at an angle \( \alpha^\circ \) from the edges of the bearing, but the wall must be checked for the load spreading from the bearing and any other
The angle $\alpha$ is usually taken as $60^\circ$. All loads within the triangle or wedge, including those from floors or beams, must be considered.
uniformly distributed load in the wall at a depth of 0.4 times the height of the wall below the bearing.

b) **Loads in lintels**

The loads to be considered on lintels should be calculated by an appropriate theory but as an approximation may be assumed to be those contained within a triangle having sides at $\alpha^0$ to the horizontal or between the end of a wall and a line at $\alpha^0$ to the horizontal (Figure 6).

![Figure 6: Loads on lintel to be considered](image)

**1.52.2-4 Design of wall for vertical load**

For the ultimate limit-state, at every horizontal section, the "design vertical load", $N_d$ per unit length of the wall, should not exceed the "design strength of the wall". The "design strength of the wall" is the strength of any section based upon the plan area under consideration, the design strength of the masonry $(f_k)/\gamma_m$, the design strength of any
A linear distribution of stress will usually be more conservative than non-linear.

In Appendix 6 a means of calculating $N'$ is given.

The ability of the floors to act in the way assumed should be verified.
reinforcement present and a reduction factor $\phi$ which allows for the slenderness of the wall and the resultant eccentricity of the load.

The factor $\phi$ takes account of the distribution of stresses in the section (non-linear, as a rule) due to bending, but neglects the tensile strength of the masonry; it also takes into account the deflection of the wall (theory of second order) allowing for the effect of long term strains (creep). National Standards should give values for $\phi$ taking into account the method of arriving at $e_a$, $\rho$, $l_e$ and $t_{ef}$.

Thus, $N'_d < \phi f_d t + \text{value of any reinforcement present.}$ \hspace{1cm} (6)

where $t$ is the thickness of the wall.

The section to be checked is dependent on the level of the worst virtual eccentricity ($1.52.2-2.2e$). If horizontal forces are acting it will also be necessary to check the strength in shear.

1.53 Shear walls

1.53.1 Statical schemes

Resistance to horizontal actions is generally provided by a system formed by the floors and the shear walls (transverse walls); this can be idealized as follows:

- the actions are applied to the floors, or transmitted to them by the front walls:
This is not normally allowed for seismic actions
the floors, acting as horizontal diaphragms, distribute the actions to
the shear walls:

- the shear walls act as vertical cantilevers (in high rise buildings).

Openings in shear walls can considerably affect their behaviour. The
presence of openings should be taken into account when they occur.

A limited portion of an intersecting wall can act as a flange to a shear
wall, increasing the stiffness and strength; it can be taken into
account in the design, provided the connection of the web to the
flange is able to resist the corresponding shearing actions, and
provided the flange will not buckle within the length assumed.

Due to the lack of knowledge of the flexural characteristics of thin
masonry walls bent in their plane, for the above mentioned
distribution or horizontal actions only, the elastic rigidities of the
shear walls are considered. For tall walls, the effect of shear strains
on the stiffness can be neglected.

It is usually sufficient to consider the effect of horizontal actions on
the two principal axes.
If the floors can be idealised as infinitely rigid diaphragms (e.g. in the case of substantial, in-situ concrete slabs) a conservative procedure is to distribute the actions to the shear walls in proportion to their stiffnesses on the assumption that all deflect by the same amount, maintaining equilibrium, but other more refined analytical procedures may be used if appropriate. Where the plan arrangement of the shear walls is unsymmetrical, it will be necessary to consider the distribution of the actions between the shear walls resulting from rotation of the structure about its overall shear-centre (i.e. torsional effects). If the floors are relatively flexible when considered as horizontal diaphragms (e.g. partially inter-connected, pre-cast units), actions should be allocated to the shear walls to which the respective sections of the floors are directly connected.

1.53.2 Determination of the load effects

For the analysis of a shear wall, the design horizontal actions and the design (permanent) vertical loads should be applied to the wall.

The following load effects are used, with the assumption of linear elasticity for the force distribution and neglecting the tensile strength:

a) maximum axial load per unit length of the shear wall, due to vertical loads and, in the case of high rise structures, due to cantilever bending:
b) maximum axial load per unit length in the flanges:

c) maximum vertical shear per unit length of the connection between web and flange:

d) minimum axial load assisting the design shear resistance (see 1.33.2a) (neglecting the part of the wall subject to tension, and averaging the stress over the remaining part):

e) maximum horizontal shear per unit length of the shear wall (neglecting the part of the wall subject to tension, and averaging the stress over the remaining part).

1.53.3 Design of shear wall

For ultimate limit-state the shear wall and any flange should first be checked as indicated in 1.52.2-4 for bearing walls, by comparing the respective "design axial loads" $N_d$ with the "design strength of the wall". The wall should then be checked to see that the maximum shear stresses (see (d) and (e) above) do not exceed the "design shear strength", $f_{vd} \leq f_{vk}/\gamma_{mm}$.

1.54 Unreinforced laterally loaded walls

Unreinforced laterally loaded walls bent perpendicular to their plane are not capable of precise design. There are however two approximate methods which can be used.

a) as a panel supported on a number of sides with the edges being either simply supported or fully fixed.

b) as an arch spanning between suitable supports.
It has been found by experiment that a yield line theory, although not strictly theoretically correct, produces a suitable bending moment design for walls spanning two ways and subjected to lateral loading.

Calculation should be based on a simple three-pin arch. It can be assumed that the bearing at the supports and at the central hinge is 0.1 x the thickness of the wall, t.
1.54.1 **Walls spanning between supports**

Masonry walls are not isotropic and there is an orthogonal strength ratio, $\mu$, which can be derived from the characteristic flexural strengths with the plane of failure parallel and perpendicular to the bed joints (see Clause 1.24). Allowance can also be made for the increase in the flexural strength parallel to the bed joints due to applied vertical loads. The design moments can be obtained using a suitable bending theory and this should be compared with the flexural strength in the appropriate direction:

$$R_d > M_d$$

$$R_d = \frac{f_{xk}}{\gamma_{mm}} Z$$  \hspace{1cm} (7)

- $R_d$ = bending moment obtained by a suitable design theory
- $M_d$ = bending moment obtained by a suitable design theory
- $f_{xk}$ = characteristic flexural strength parallel to the bed joints
- $\gamma_{mm}$ = material modification factor
- $Z$ = section modulus $\frac{I}{0.5t}$
- $t$ = overall thickness of the wall
- $I$ = second moment of area of the wall

1.54.2 **Arching walls**

When a masonry wall is built solidly between supports or is continuous past a support, the wall may be designed by assuming that an arch develops within the thickness of the wall. The supporting structure
The arch rise is given by:

\[ t = \frac{t}{10} - \delta \]

where \( \delta \) is the lateral deflection under the design lateral load and can be taken as zero for walls with span/thickness less than 25.

For cases where the lateral deflection is small, the design resistance to lateral loading approximates to

\[ R_d = k \frac{f_{sk} t^2}{\gamma_{mm}} \]

where \( k \) is a constant depending on the effectiveness of the junction between the wall and the support and can be taken as 1.5 if the junction is filled solidly with mortar.

A small change in the length of a wall in arching can considerably reduce the arching resistance and care should be taken to select units that do not shrink in service.

Reference should be made to published interaction diagrams.
must be stiff enough to resist the arch thrust and be designed accordingly. The arch thrust has to be assessed from knowledge of the applied lateral load, the strength of the masonry in compression and the effectiveness of the junction between the wall and the support to resist the thrust or provide a tie. The effect of second order deformations should be considered if the arch spans vertically.

1.55 **Reinforced masonry members subject to bending and axial load**

The calculation of the resistance capacity to bending and/or axial load should be based on the following assumptions:-
Assumptions a, d and e are supplemented as follows:

for calculating the resisting load-effect, it is assumed that the strain diagram must pass through one of the three points A, B or C defined below:

![Strain Diagram](image)

**Figure 7: Strain Diagram**

For minimum percentages of reinforcement and detailing recommendations, refer to Part 2.
a) plane sections remain plane.
b) the reinforcement is subjected to the same variations in strain as the adjacent masonry or concrete infill.
c) the tensile strength of the masonry or concrete infill is zero.
d) the maximum compressive strain of the masonry or concrete infill is 0.0035 in bending and 0.002 in compression.
e) the maximum tensile strain in the reinforcement is 0.01.
f) the design stress block for masonry or concrete infill may be taken as either

\[(1)\]

![](image)

or (2) a rectangular stress block extending to 0.8 of the effective depth.
g) the design stress-strain diagram for ordinary reinforcing steel is deduced from the characteristic stress-strain diagram (see 1.26) by drawing a line parallel to the tangent at the origin, in the proportion of \(1/\gamma_{ms}\).

The slenderness of a wall or column should be allowed for as in 1.52.2-2.2c.
Consideration should also be given to the minimum spacing of shear reinforcement to control cracking to ensure that there is a reserve of strength after cracking and before failure. Refer to Part 2, Section 2.38.
1.56 Reinforced masonry members subjected to shear forces

This section deals with members subjected to shear forces, e.g. walls and columns resisting horizontal actions and beams or lintels resisting vertical actions.

For the ultimate limit state no shear reinforcement is required if the design shear load, \( V_d \), does not exceed the design shear strength of the masonry and concrete infill, \( V_{rd1} \) i.e. \( V_d < V_{rd1} \).

where \( V_{rd1} = f_v b d \left(1+50 \frac{A_{sl}}{bd}\right) \)  

\( f_v \) = design shear strength of masonry (see 1.23 and 1.28) 
\( b \) = effective width of section 
\( d \) = effective depth of section 
\( A_{sl} \) = area of effectively anchored longitudinal reinforcement 

and \( \frac{A_{sl}}{bd} \geq 0.02 \)

\( f_v b d \left(1+50 \frac{A_{sl}}{bd}\right) \geq 0.7 \text{ N/mm}^2 \)

For simply supported reinforced beams or cantilever retaining walls the design shear strength may be increased to:

\[ V_{rd1} = f_v b d \left(2.5 - 0.25 \frac{a}{d}\right) \]

where \( a \) = ratio of maximum design bending moment to maximum design shear force with a maximum value of 6.

and \( f_v \left(2.5 - 0.25 \frac{a}{d}\right) > 1.75 \text{ N/mm}^2 \)

However, the designer should consider the use of nominal links in beams to avoid sudden failure.
The following figure gives the order in which the various losses of prestress occur for the case of pre-tensioned tendons.
If the design shear load, \( V_d \), exceeds the design shear strength of the masonry and concrete infill, shear reinforcement should be provided such that

\[
V_d \leq V_{rd2} \quad \text{where} \quad V_{rd2} = 0.30 f_{md} b d
\]

and
\[
V_d \leq V_{rd3} \quad \text{where} \quad V_{rd3} = V_{rd4} + V_{rd5}
\]

where
\[
V_{rd4} = \frac{A_{sn}}{s} 0.9d f_{yd} (1 + \cot \alpha) \sin \alpha
\]

and
\[
V_{rd5} = 2.5 f_{yd} b d
\]

where
\[
A_{sn} = \text{area of shear reinforcement}
\]
\[
s = \text{spacing of shear reinforcement}
\]
\[
\alpha = \text{angle of shear reinforcement to the horizontal.}
\]

1.57 **Members prestressed with steel**

Prestressing forces should be applied and transferred to the masonry in accordance with national codes. Prestressing will be applied after hardening of the concrete and mortar (post-tensioning), when the tendons are placed in sheathing with anchorage devices at their ends, or before constructing the masonry (pre-tensioning), when the tendons are anchored to the masonry through bond.

The stresses in masonry due to prestress should be limited at the various stages of prestressing. When calculating the design strength, allowance must be made for the losses in the prestressing forces as follows.
This paragraph applies to statically indeterminate structures

Hyperstatic =  statically indeterminate
Isostatic =  statically determinate
- losses due to the instantaneous deformation of the masonry.
- time dependent losses due to shrinkage and creep of the masonry and relaxation of the steel.

For most cases it is sufficient to take account of the values of prestress at two stages.
- at the instant of the application of the prestress to the masonry.
- after a long period (or "long term").

Allowance in the design for the prestress should be such that
- before bond has developed between the tendons and the masonry, the effects of the prestressing are taken into account as an element of the applied load effects.
- after the bond has developed, the isostatic effects of the prestressing are taken into account as an element of the resisting load effects, when the elongation of the prestressing tendons is not less than $\varepsilon_{pk}$, and as an element of the applied load effects when the elongation does not exceed $\varepsilon_{pk}$.

The hyperstatic effects of the prestress are always taken into account as an element of the applied load effects.

Prestressing can be used in members to resist bending, axial loads and shear and any combinations thereof. Refer to Sections 1.52, 1.55 and 1.56 as appropriate.
The designer should ensure that deflections are not excessive, with regard to the requirements of the particular structure.

In any calculation of deflections the design loads should be those recommended for the serviceability limit state in 1.33.2b.
**1.6 Serviceability limit states**

1.61 Deflection

The deflection of the structure or any part of it should not adversely affect the performance of the structure or any applied finishes.

1.62 Cracking

Cracking should not be such as to affect adversely the appearance or durability of the structure; the flexural or tensile strength of the masonry $f_x$, must be exceeded for a crack to occur. The effects of temperature, creep and shrinkage may, unless otherwise catered for, require the provision of movement joints.

Recommendations for controlling cracking and maintaining durability are given in Part 2.
See 2.32.2 concerning resin mortars.

Sand should be of mineral origin.

The content of harmful elements such as organic materials should be limited, the fraction of very small grains should be limited and the dimensions of the sand grains should be limited in respect of the joint thickness.
PART 2 - MATERIAL SPECIFICATION AND ERECTION OF MASONRY STRUCTURES

2.1 Material specification

2.11 Structural units

All structural units should comply with the appropriate national standards provided that the compressive strength $f_{bk}$ is determined according to the method given in Appendix 2. Quality of manufacturing control should be determined according to 2.2.

2.12 Mortars

2.12.1 Classification

Mortars can be classified as follows according to the type of binding material.

- traditional mortars, where the binder is composed of cement, lime or a mixture of cement and lime;

- resin mortars, containing synthetic resins;

Mortars are characterised according to their strength, weathering, and workability (see 2.12.5).

2.12.2 Materials

Cement: should conform to national standards

Sand: should conform to national standards
When the quality of the water is doubtful, comparative tests should be made with distilled water on the binder which will be used.

The use of chlorides may have harmful effects on embedded metal and may lead to efflorescence.

Air entraining plasticisers may lead to over entrainment of air.

Pre-mixed mortars, requiring only the addition of cement or water on the site can also be used. Mortar should be homogeneous and the aggregate well covered with the binder.

Ready mixed mortar will usually contain a retarding agent. The designer should be aware of its effects.

In normal atmospheric conditions, mortar should not be used more than 2½ hours after mixing.
**Lime:** should conform to national standards

**Water:** In mixing water the content of harmful materials, such as oil, organic materials or harmful salts (chlorides, sulphates, etc.) should be very limited.

**Admixtures:** Admixtures may only be used according to the instructions of the supplier and national standards with respect to their compatibility with cement, with regard to efflorescence and reduction of strength.

The use of chlorides in admixtures or in mortar is not advised.

2.12.3 **Mixing**

Preferably mortar should be mixed mechanically. The mortar can also be supplied to the building site ready-mixed. In this case the mortar must have, at the moment of use, the qualities of a fresh mortar. For resin mortars, it is important to follow the instructions of the resin producer.

2.12.4 **Mortar use**

Mortar should be used before it begins to harden. Mortar should be deposited on a clean surface isolated from the ground.

It is important:
- to protect the prepared mortar against wind, rain, sun and extraneous matter.
- not to re-use or remix mortar which has started to harden. All mortar containers should be emptied and cleaned after use particularly at the end of each working day.
Some mix proportions and corresponding compressive strengths are given in Table 2, Part 1.

The moisture absorption of the masonry units and mortar joints will reduce the water content of the grout and effectively reduce the water/cement ratio.

There are two types of grout:
- fine grout, for filling spaces with a minimum dimension of 50mm
- coarse grout, for filling spaces with a minimum dimension of 100mm
2.12.5 Characteristics of mortars
Mortar for masonry, when having a suitable consistency, should have good cohesion as well as water retentivity power to resist suction by the materials on which it is applied.

Preferably mortar should be specified by mix proportion and compressive strength and should conform to national specifications.

Mortar to be used for reinforced masonry should have low porosity.

2.12.6 Specification of mortars
The mix proportions and strength to be obtained should be given in National Standards.

2.12.7 Determination of mortar strength
Tests for the determination of mortar strength are given in Appendix 2.

2.13 Grout
Grout should consist of a mixture of cementitious material and aggregate. Grout should be preferably proportioned by mass and should have sufficient water added to produce a consistency for pouring without segregation. Aggregate should conform to the requirements of Clause 2.14.2 and National Standards.
Concrete infill should be one of the following:

a) 1:1:3:2, cement:lime:sand:10mm aggregate proportioned by volume of dry materials achieving a 28 day cube strength of 25 N/mm².

b) A concrete mix with a minimum characteristic 28 days cube strength of 25 N/mm² with a nominal maximum size of aggregate of 10mm.

c) A concrete mix with a minimum characteristic 28 days cube strength of 25 N/mm² with a nominal maximum size of aggregate of 20mm.

Each of these mixes should have a slump between 75mm and 175mm.

Mixes (a) and (b) should be used for filling spaces with a minimum dimension of 50mm. Mix (c) may be used for filling spaces with a minimum dimension of 100mm.
2.14 Concrete infill

2.14.1 Classification
Concrete should comply with the appropriate National Standards.

2.14.2 Materials
Materials generally should be in accordance with Clause 2.12.2.
In addition:-
Aggregates should comply with National Standards. The maximum size of the aggregate is 10mm for filling spaces with a minimum dimension of 50mm, and maximum 20mm for filling spaces with a minimum dimension of 100mm.

2.14.3 Mixing
The concrete should be mixed mechanically.
In the composition of the concrete, account should be taken that the moisture absorption of the masonry units and mortar joints will reduce the water content. The concrete must also have a sufficient consistency (workability) to fill perfectly the spaces without segregation.
Concrete should have suitable workability.

2.14.4 Determination of the strength of concrete
The strength of concrete should be determined by samples in accordance with National Standards.

2.15 Reinforcing steel
Reinforcing steel should conform to National Standards and the requirements of Clause 2.38.
It is recommended that materials be transported in packages, rather than in bulk.

There are many different packaging methods. The dimensions of the packages should be chosen in respect of the capacity of the lifting and transporting machines and of the strength of scaffolds.

It is important to comply with the standardized sizes of the packages when masonry units are to be used in countries other than those in which they were manufactured.

The value of $\gamma_m$ adopted in Part 1 should be commensurate with the degree of control exercised during the manufacture of the structural units and also on the site supervision and the quality of the mortar used during construction.
2.16 Materials supply

2.16.1 It is recommended that for transport of masonry units, unloading, stacking and bringing on site, the units or packages of units should be manipulated carefully in order to avoid breakage, or cracking detrimental to their function in the masonry.

2.16.2 Masonry units should be stacked on the site in a ventilated place protected against water and ground moisture. Cement should be fresh and kept in a ventilated place. Materials should not be placed on concrete floors which have not attained sufficient strength. Reinforcing steel must be stored on the site to prevent accidental kinking or bending; it must be kept free of dirt, mud, oil, snow, ice or other foreign matter detrimental to bond.

2.2 Quality control

2.21 Recommendations for tests before and during construction

In order to construct masonry to the required standards it is necessary to control the manufacture, construction and erection of the masonry in accordance with the following procedures.

Two levels of control are recognized for control in the factory and three for control on site as follows:
These requirements for construction control assume that there will be supervision of the masonry work as this is mandatory if Table 6 is to be used.
-1- Control in the factory

Category A

This may be assumed when the manufacturer agrees to supply units of the specified characteristic compressive strength, when tested in accordance with Appendix 2, and is operating an established quality control scheme, the results of which he agrees to make available to prove to the satisfaction of an independent authority that the characteristic strength is consistently being met.

Category B

This category should be assumed when the supplier meets the characteristic compressive strength of the units, but does not meet the requirements for category A above.

-2- Construction control

Category A

This category of construction control may be assumed where:

(a) the specification, supervision and control ensure that the construction is compatible with the use of the relevant partial safety factor $\gamma_m$ given in Part 1 of these recommendations.

(b) preliminary compressive strength tests, carried out on the mortar, on the infilling concrete and on the units to be used, indicate compliance with the strength requirements of these recommendations; regular testing of the mortar and concrete used on site, shows that compliance with the strength requirements is being maintained.
Category C should only be used when the masonry is not highly stressed.

In some countries an independent certificate is provided by the manufacturers to the user. Such a certificate will usually obviate the need for testing by the designer.
Category B

This category may be assumed when intermittent control is made by the designer or his representative, and testing of the mortar, of the concrete and of the units used on the site shows that compliance with the strength requirements is being achieved.

Category C

This category may be assumed when the site control by the designer is not frequent or control is made only by the contractor.

2.22 Control of mortar strength

The size of the specimens and the testing method should be in accordance with the procedures given in Appendix 2.

2.23 Control of the strength of bricks or blocks

Tests for the compressive strength of bricks and blocks should be in accordance with Appendix 2 and should be carried out at regular intervals during construction.

2.24 Control of the strength of concrete

Tests for the compressive strength of concrete should be in accordance with National Standards.
A study of several specifications gives the following values of the permissible deviation (with category B construction control).

<table>
<thead>
<tr>
<th>Type of Error</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall plumb per storey</td>
<td>± 10 mm in 3 m for $f_d &lt; 3$ N/mm$^2$</td>
</tr>
<tr>
<td></td>
<td>± 5 mm in 3 m for $f_d &gt; 3$ N/mm$^2$</td>
</tr>
<tr>
<td>Vertical alignment (overall height)</td>
<td>± 20 mm</td>
</tr>
<tr>
<td>Wall spacing (at floor level)</td>
<td>± 20 mm</td>
</tr>
<tr>
<td>Horizontality (level)</td>
<td>± 10 mm in 5 m</td>
</tr>
<tr>
<td>Linearity (straightness)</td>
<td>± 7 mm in 3 m</td>
</tr>
<tr>
<td>Wall length</td>
<td>± 10 mm in 5 m</td>
</tr>
<tr>
<td>Wall height</td>
<td>± 20 mm in 3 m</td>
</tr>
</tbody>
</table>

The surface of the unit should appear moist to dry after wetting. However, some masonry materials are not recommended by the manufacturers to be wetted; other materials with very high rates of suction need to be laid in accordance with the manufacturers instructions.
2.25 Control of the geometry of the wall

Masonry should be built "plumb and level". Bed joints should be horizontal. For the design recommendations in Part 1 to apply the permissible deviations for masonry should conform to national recommendations.

2.3 Erection of masonry

2.31 Preparation of bricks and blocks

2.31.1 The use of cracked bricks or blocks should be avoided in such elements as angles, pilasters and columns, in the vicinity of concentrated loads and openings and generally in those parts where the actions in serviceability limit states are near to the limit strength of the masonry.

2.31.2 Clay masonry units having an initial rate of suction of more than 1.5 kg/m² per minute should be moistened before laying except when a suitable water retaining additive is used in the mortar.
In some masonry, partially filled joints are a feature of the design. In such cases, the characteristics of the masonry should be determined by tests or should be given in National Standards.

Experience may show that all joints need not be filled.

If the wall is raked out for later pointing the decrease in the bearing area of the masonry should be taken into consideration.
2.31.3 Prior to laying the first course of masonry, the concrete surfaces should be clean and free of laitance, loose aggregate, grease or anything that will prevent bond between the concrete, mortar and the masonry units. The first course of masonry should be laid on a full bed of mortar except over the areas where concrete is subsequently placed in pockets. Mortar should not be laid where concrete is required to have direct contact with the foundation or slab on which the masonry is laid.

2.31.4 The dimensions, locations of openings, positions of reinforcement, mortar and concrete mixes, unit strengths, bonding and general sequence of operations should be as shown on the detailed drawings.

2.32 Execution of masonry

2.32.1 Mortar joints

- The thickness of the joints depends on the modular system of the elements and the surface irregularity of the units. The thickness should be not more than 15mm unless wall tests are carried out. (see Appendix 1)

- All horizontal and vertical joints should be filled with mortar, except in the cavities of hollow or perforated units.

- Masonry can be pointed as the building of the wall proceeds, or pointed afterwards (see Clause 2.37).

- It is advisable for load bearing walls to be jointed as building work proceeds.
There is not sufficient experience in the use of this type of joint, and its use must be considered as experimental.

The mean thickness may sometimes be different from 3mm.
- Care should be taken to ensure that the detailed cover to any reinforcement is maintained. Where bed joint reinforcement crosses voids or pockets that contain reinforcement and are to be filled with concrete, it should be positioned such that it does not prevent compaction of the infill concrete.

2.32.2 Resin mortar joints
- The joints should have a mean thickness of 3mm; this requires the use of units with very high dimensional accuracy.
- The thickness of the joint must be adjusted to take account of the dimensional accuracy of the units.
- It is essential for the first course of units to be level in order to facilitate the placing of the following courses.
- If reinforcement is to be placed in the joints, the reinforcement may have to be adapted, e.g. by flattening, to make it possible to provide the correct cover, or the blocks grooved.

2.32.3 Masonry bonding
The drawings should indicate the chosen bonding pattern taking into account the dimensions of the materials.

The bond should meet the following requirements:
- vertical courses alternate. If for any reason (aesthetic for instance) this condition is not satisfied, the stability of the wall should be assured e.g. by reinforcing the masonry.
When this is not possible, mechanical tools should be used, making sure the masonry is weakened as little as possible. Account should be taken of any such holes in the design.

The maximum height of a wall to be built in a day should not exceed 3 m, but may need to be less depending on the unit type and mortar mix.
- mixing of materials within the same piece of masonry is avoided.
- there is continuity of the bonding at wall junctions or crossings;
- portions of units (⅛ or ¼) are specially manufactured or obtained by sawing from whole units.

2.32.4 Holes, grooves

Holes and grooves for pipes should be detailed on the plans. This is especially important for fair-faced walls and load-bearing walls. Preferably special grooved units, or bonding arrangements should be used.

Horizontal or diagonal grooves are normally not permitted; when the designer has taken them into account in his calculations they may be used only with his written approval.

2.33 Curing and protection of freshly executed masonry

Freshly executed masonry should be protected against mechanical damage (shocks) and weathering.

During hot and dry weather, freshly executed masonry should be protected from rapid drying out by the use of sheeting.

During rainy weather and at the end of each working day, the upper part of freshly executed masonry should be covered. The covering should give
Bricks or blocks with insufficient frost resistance should not be used below ground without protection.

It may be sometimes desirable to render the surface of masonry below ground, even in non-aggressive soils.
protection over at least the top 0.60m of the wall. When masonry is built during cold weather, the recommendations of 2.36 should be adopted.

2.34 Loading of walls

Walls may not be loaded before they have reached sufficient strength. Non-bearing walls may not be used as supports for formwork for structural elements (floors, beams, columns, etc.), without the approval of the engineer. It is not advisable to build non-bearing walls before removing the supports from adjacent structural elements as deflection could cause load to be transferred to the non-bearing walls.

2.35 Masonry underground

2.35.1 For masonry below ground, it is recommended that bricks and blocks without cracks or breaks are used.

2.35.2 Back-filling against walls should not be carried out until the wall is capable of resisting the loads resulting from the filling.

2.35.3 Protection of masonry in contact with earth depends on whether the soil is aggressive or non-aggressive.

When masonry is built in contact with non-aggressive earth, no protection is needed.
When the earth is likely to contain chemicals which might be harmful to the masonry, it should be protected in such a way that the aggressive chemicals cannot be transmitted into it by water or any other means.

When ground water is present, the protection may need to be increased.

2.35.4 Cellar walls
- when the soil does not contain aggressive substances, masonry which will come into contact with earth may be rendered over the total height with a layer of mortar of the same composition as that used in the masonry:
- when the soil contains aggressive substances, additional precautions will be needed depending on the nature of the problem.

2.35.5 Lightweight aggregate concrete block walls should be protected on the outside against water penetration; this protection should be increased when ground water pressure is expected.

2.36 Masonry work during cold weather
The following precautions may be taken to ensure that the temperature of the masonry, in particular of the mortar and the concrete, does not fall below 0°C during the early stages of hardening.
Rapid hardening cement should be used.
The proportion of cement should be increased.
Plasticising agents maintain workability at low water contents: accelerators (chlorides) are not recommended because they are not very efficient and may lead to efflorescence and corrosion. Air-entrainers are recommended.
Water and sand may be heated.

a) \[0^\circ C < t_{\text{air}} < 4^\circ C\]: heat water
b) \[-10^\circ C > t_{\text{air}} > 0^\circ C\]: heat water and sand

if \(t_{\text{air}} < -10^\circ C\), special measures are to be taken (heated environment, etc.).

The means of protection are:
insulating mats or plastic sheets (water protection) covering the wall.
draught screens around the construction area.
temporary protection by means of a plastic tent (external walls) or by closing openings (interior walls) and heating.
2.36.1 Supply and storage of materials
During storage all materials should be protected against rain water and ground humidity.

Frozen materials or materials containing ice should not be used.

2.36.2 Mortars
The mortar should be made and laid in such a way that it hardens satisfactorily.

2.36.3 Protection
Freshly executed masonry should be protected adequately against cold, snow and water. External surfaces of the joints are particularly liable to frost damage.
Occasionally despite those precautions some frost damage may occur. If this is limited to slight surface spalling, the wall may still be acceptable, if the damaged areas are raked out and repointed.
2.36.4 Execution of masonry

During winter-time, masonry should be executed with particular care: it should be laid quickly and without interruption; the mortar should be spread in small quantities and the bricks or blocks laid as soon as possible.

Masonry units should not be wetted to adjust the initial rate of suction.

2.37 Jointing and Pointing

The durability of the mortar and the rain-shedding capacity of the wall are improved by tooling the joints to compact the mortar. After mortar joints at the face of the wall have been cut off they may be left flush, pointed as the work proceeds or raked out for subsequent pointing, rendering or plastering. In exceptional cases raked joints may be left unpointed, unrendered or unplastered. In the case of reinforced masonry, and unreinforced masonry exposed to wind driven rain, the masonry must be jointed, rendered or plastered.

2.37.1 Jointing

In jointing, the faces of the mortar joints are worked while still green to give a finished surface. Where possible jointing should be used because it leaves the bedding mortar undisturbed.
2.37.2 **Pointing**

In pointing, the face joints are raked out to a depth of from 13mm to 20mm and later refilled with mortar. Pointing should be used whenever the bedding mortar is a different colour from that required on the face. Care should be taken in the following details:

- The type of finish for facing work should be carefully chosen in relation to colour, texture, form and durability of the units used and the conditions of exposure. Recessed joints which expose a greater surface to the attack of water should not be used.

- As work proceeds and as soon as the mortar has developed sufficient hardness, joints should be raked to an even depth, the excess mortar removed and joint sides raked clean.

- When raking the joint, attention should be paid to leaving a sufficient distance (at least 5mm) between any perforation and the surface of the mortar.

- It is recommended that mortars having similar properties to the original bedding mortar should be used, in particular when pointing load-bearing walls. In no case should the pointing mortar mix be stronger than the original mortar mix.

- Before pointing, loose materials should be brushed out and if necessary the brickwork wetted.

2.38 **Reinforcement details**

There are two types of reinforced masonry:
2.38.1 **Nominally reinforced masonry**, when the reinforcement is used to help control cracking of walls or to provide ductility to avoid cyclic degradation.

The minimum amount of reinforcement should be 0.05% of the gross cross section of the wall in each direction.

2.38.2 **Fully reinforced masonry**, when the masonry as a whole is considered in the calculation and is reinforced in consequence. When only isolated elements are reinforced, the amount and spacing of reinforcement is that resulting from specific calculations.

2.38.2.1 **Minimum area of secondary reinforcement in walls**

In walls designed to span in one direction only, the area of secondary reinforcement provided should be not less than 0.05% based on the effective depth times the breadth of the section.

Secondary reinforcement may be omitted from pocket-type walls except where specifically required to tie the masonry to the infill concrete.

Some or all of the secondary reinforcement may be used to help control cracking due to shrinkage or expansion resulting from thermal and moisture movements.
In members subject to bending at least 50% of the reinforcement provided at mid-span should be taken through to the support and should be properly anchored.
2.38.2.2 Maximum size of reinforcement

The diameter of reinforcing bars used in reinforced masonry should not exceed 8mm when placed in joints or 25mm elsewhere, except in the case of pocket-type walls, where bar sizes up to 32mm may be used.

2.38.2.3 Minimum size of reinforcement

The reinforcement used in reinforced masonry should have a minimum size of 3mm.

2.38.2.4 Anchorage and laps

Anchorage and laps should be calculated as for reinforced concrete using the appropriate bond strengths (see Clause 1.27). Stress transfer may be by means of an appropriate mechanical device. The anchorage length can be justified experimentally by laboratory tests.

2.38.2.5 Spacing of horizontal and vertical reinforcement

The minimum spacing of reinforcement should allow the concrete to be placed and compacted.

The maximum spacing of main and secondary tension reinforcement should not exceed 600mm.

Where the main reinforcement is concentrated in cores or pockets, e.g. in pocket-type walls, the maximum spacing centre to centre between the concentrations of main reinforcement may exceed 600mm.
Alternative equivalent protection, e.g. plastic coating can be provided to the satisfaction of the engineer.

Exposure situations requiring special attention

Special consideration should be given to any feature that is likely to be subjected to more severe exposure than the remainder of the building or structure. In particular, parapets, sills, chimneys and the details around openings in external walls should be examined. Normally such situations should be considered equivalent to exposure situation E3.
Additional restrictions on the spacing and the amount of reinforcement may be required when movement of walls need to be controlled (see Clause 2.4.3).

Horizontal reinforcement in walls should be provided at the top and bottom of wall openings and at roof and floor levels.

Vertical reinforcement in walls should be provided on each side of openings and at intersections, ends and corners.

2.38.2.6 Resistance to corrosion of metal components

(a) General. The type of reinforcement and the minimum level of protective coating for reinforcement which should be used in various types of construction and site exposures is given in table 8. This table applies to low carbon steel, high yield steel, galvanised steel and austenitic stainless steel. In all cases, cover to reinforcement should be in accordance with Clause 2.38.2.7(c). Alternatively, carbon steel reinforcement may be used, provided that it is protected by concrete cover in accordance with table 9.

(b) Classification of exposure situations. Exposure situations are classified into the following four situations.
Effect of different masonry units

The protection against corrosion provided by brickwork tends to be improved if high strength, low water absorption bricks are used in strong mortar. Where bricks that have a greater water absorption than 10% or concrete blocks having a net density less than 1500 kg/m³ are used, the steel recommended for the next most severe exposure situation or, where appropriate, stainless steel should be used, unless protection to the reinforcement is to be provided by concrete cover in accordance with Clause 2.38.2.7(c).
Exposure situation E1  Internal work and the inner skin of ungrouted external cavity walls and behind surfaces protected by an impervious coating that can readily be inspected or external parts built where the exposure is sheltered.

Exposure situation E2  Buried masonry and masonry continually submerged in fresh water or external parts built where the exposure is moderate.

Exposure situation E3  Masonry exposed to freezing whilst wet, subjected to heavy condensation or exposed to cycles of wetting by fresh water and drying out or external parts built where the exposure is severe.

Exposure situation E4  Masonry exposed to salt or moorland water, corrosive fumes, abrasion or the salt used for de-icing.
Table 8  SELECTION OF REINFORCEMENT FOR DURABILITY WHEN ADEQUATE CONCRETE COVER IS NOT PROVIDED (see Table 9)

<table>
<thead>
<tr>
<th>Exposure situation</th>
<th>Minimum level of protection for reinforcement, excluding cover see Clause 2.38.2.7 (c)</th>
<th>Located in grouted cavity or Quetta bond construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Located in bed joints or special clay units</td>
<td>Carbon steel heavily galvanized or with equivalent protection (see note).</td>
<td>Carbon steel</td>
</tr>
<tr>
<td>E1</td>
<td>Carbon steel heavily galvanized or with equivalent protection</td>
<td>Carbon steel or, where mortar is used to fill the voids, carbon steel heavily galvanized or with equivalent protection</td>
</tr>
<tr>
<td>E2</td>
<td>Carbon steel or, where mortar is used to fill the voids, carbon steel coated with 1 mm of stainless steel or equivalent</td>
<td>Carbon steel or, where mortar is used to fill the voids, carbon steel coated with 1 mm of stainless steel or equivalent</td>
</tr>
<tr>
<td>E3</td>
<td>Carbon steel or, where mortar is used to fill the voids, carbon steel coated with 1 mm of stainless steel or equivalent</td>
<td>Carbon steel heavily galvanised or with equivalent protection</td>
</tr>
<tr>
<td>E4</td>
<td>Austenitic stainless steel or carbon steel coated with 1 mm of stainless steel or equivalent</td>
<td>Austenitic stainless steel or carbon steel coated with 1 mm of stainless steel or equivalent.</td>
</tr>
</tbody>
</table>

NOTE: In internal masonry other than the inner leaves of external cavity walls carbon steel reinforcement may be used.

(c) Cover Where austenitic stainless steel, or steel coated with at least 1 mm of austenitic stainless steel, is used, there is no minimum cover required to ensure durability. However, some cover will be required for the full development of bond stress.

Where reinforcement is placed in bed joints, the minimum depth of mortar cover to the exposed face of the masonry should be 15 mm.
a) Bed joint reinforcement

b) Pocket type wall

c) Reinforced hollow block wall

Figure 9: Minimum concrete cover in pocket-type walls in reinforced hollow blockwork walls and for bed joint reinforcement.
For grouted-cavity or Quetta bond construction, the minimum cover for reinforcement selected using table 8 should be as follows:

(a) carbon steel reinforcement used in internal walls and exposure situation E1: 20mm mortar or concrete;
(b) carbon steel reinforcement used in exposure situation E2:20mm concrete;
(c) galvanized steel reinforcement: 20mm mortar or concrete;
(d) stainless steel reinforcement: not required for durability.

**TABLE 9. MINIMUM CONCRETE COVER FOR CARBON STEEL REINFORCEMENT**

<table>
<thead>
<tr>
<th>Exposure Situations</th>
<th>Thickness of concrete cover</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Concrete 28 day cube strength</td>
</tr>
<tr>
<td></td>
<td>25</td>
</tr>
<tr>
<td>Minimum cement content (kg/m3)</td>
<td>250</td>
</tr>
<tr>
<td>E1</td>
<td>mm</td>
</tr>
<tr>
<td>E2</td>
<td>20</td>
</tr>
<tr>
<td>E3</td>
<td>-</td>
</tr>
<tr>
<td>E4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>-</td>
</tr>
</tbody>
</table>

Table 9 shows the minimum concrete cover recommended for carbon steel reinforcement in pocket-type walls and in reinforced hollow blockwork walls.
Angles or apertures constitute places of load concentration and thus there is a danger of cracking (Figure 10).
The cut ends of all bars, except those of solid stainless steel, should have the same cover as that appropriate to carbon steel in the exposure situation being considered unless alternative means of protection are used.

(d) **Prestressing tendons** Where tendons are placed in pockets, cores or cavities that are filled with concrete or mortar, the recommendations given in clauses 2.38.2.6(a) & (c) should be followed.

Where carbon steel tendons or bars are installed in open cavities, pockets or ducts, they should be protected suitably, e.g. by heavy galvanising or an equivalent coating. Ducts for unbonded tendons should be suitably drained.

---

2.4 **Construction details**

2.4.1 **Openings in walls**

Masonry above openings should be supported by well-butressed arches or lintels formed from reinforced concrete, metal, or reinforced masonry. Lintels should bear on the wall at each end not less than 100mm.
Ties should conform to National Standards. The minimum spacing is 2.5 ties per m² but many countries require 5 ties per m². The wall ties should be distributed at a rate which takes into account the type of tie, width of cavity, thickness of leaves, disposition of ties and height of the wall.

Figure 11: Linking reinforcement in cavity walls
2.42  **Cavity walls**

2.42.1  Cavity walls should be of the overall thickness specified and should be formed of the two leaves of masonry united by staggered wall-ties, uniformly spaced. The two leaves may also be connected with continuous reinforcement (Figure 11). Additional ties should be provided within 150mm of the edge of openings, one for each 300mm of height. Close to the corners of buildings additional ties shall be provided to allow for high wind suctions and thermal movements. Ties should be placed as the work proceeds and set dead level or with a slight downward slope towards the outside leaf. The minimum embedment of ties in each leaf should be 50mm. As the wall is built one leaf should not, in general, rise higher than 450mm above the other. Where it is essential to increase this difference the wall-ties should be built into the first leaf in appropriate positions and not pushed in afterwards. The cavity should be kept clear of mortar or rubbish as the work proceeds.

Wall ties for high and low lift grouted cavity construction, (see Clause 2.47) should be galvanised mild steel, resin coated galvanised mild steel or austenitic stainless steel.

Mortar droppings reaching the base of the cavity should be removed daily through temporary openings. On completion the loose bricks should be properly bedded and jointed after finally cleaning the cavity.
When the cavity is not filled and where it is necessary to ventilate it, openings should be made by using special bricks or blocks, or by making some of the vertical joints of the outside wall wider and leaving them unfilled with mortar (Figure 12).

![Figure 12](image)

To allow water to escape at the foot of the wall, a band of bitumenised felt or similar material (Figure 13) should be placed at the base of the two leaves, or at ground level.

![Figure 13](image)

The distance between joints depends on the "e" value of the shrinkage, or expansion of the wall material and the wall thickness, as indicated in the table below:
2.42.2 If the cavity is to be ventilated by openings in the outside walls, the cross-sectional area of these openings should be not less than 10cm$^2$ per metre length of wall and per storey height.

Openings should be properly sized or screened against rats and insects where these may be harmful.

2.42.3 Special care should be taken at the base of a wall to ensure that water does not penetrate into the building.

2.43 Movement joints

Vertical movement joints in walls are provided to allow for dimensional variations due to temperature and humidity conditions and to separate the walls from the framework or other parts of the construction where to do otherwise might be harmful, e.g. to allow for differential settlement.
Shrinkage or expansion of wall material

<table>
<thead>
<tr>
<th>Shrinkage or expansion of wall material</th>
<th>Wall thickness ≥ 140 mm</th>
<th>Wall thickness ≤ 140 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>ε ≤ 0.1 mm/m</td>
<td>30 m (1)</td>
<td>30 m</td>
</tr>
<tr>
<td>0.1 &lt; ε ≤ 0.4</td>
<td>30 to 12 m* (2)</td>
<td>30 to 8 m*</td>
</tr>
<tr>
<td>0.4 &lt; ε ≤ 0.6</td>
<td>8 m (2)</td>
<td>6 m</td>
</tr>
</tbody>
</table>

* Linear interpolation is permitted

Note: (1) For interior walls in normal conditions.

For exterior walls it is advisable not to exceed 20 m

(2) For walls having no openings nor concentrated tension.

If there are openings or concentrated tensions, one movement joint at least should be provided every 8 m (0.1<ε<0.4) or every 5 m (0.4<ε<0.6).

(3) Where walls have been reinforced in the bed joints (see clause 2.38.2), these distances may be doubled.

The joint should be filled with a durable and elastic material which fits tightly (Figure 14).
In regions exposed to earthquake or to mine-settlement, special rules are required for movement joints.

Joints should penetrate the total thickness of the wall and should be 10 to 20mm wide depending on the movements envisaged and the material in the joints.
In angles there will be concentrations of tension. It is preferable to position movement joints near angles or corners (Figure 15).

**Figure 14: Movement Joints**

**Figure 15**
As a general rule movement joints are provided at places where the probability of cracking in the masonry is greatest, i.e. where changes or weakening of the section lead to concentration of loads.

2.44 Floor support

The width of bearing of floors on walls should not be less than half the floor thickness, with a minimum of 90mm. For concrete floors bearing on both sides of the wall, it is recommended that reinforcement be placed over the wall to provide continuity.

For timber floors, the part in contact with masonry should be protected against biological degradation.
Anchors should be of a strength equivalent to that provided by a bonded wall.

Bonding can lead to cracking due to differential movement between the load-bearing wall and the joining wall. There is also the possibility of the weaker bricks in the joining wall being used in place of the specified bricks in the load-bearing wall, resulting in reduction in strength.
2.45 **Bonding of intersecting walls**

2.45.1 **Bonding of loadbearing walls**

Simultaneous erection of the walls is desirable. The joint at the intersection should be either:

(a) bonded in true masonry bond, or

(b) provided with metal anchors or reinforcement

The metal anchors or reinforcement should conform to the following requirements:

- the metal should be corrosion-resistant (e.g. galvanised steel, stainless steel or non-ferrous metal).

- the anchors should extend into the masonry at least 0.50m on each side of the joint but where this is not possible because there is insufficient thickness of masonry to embed the anchors, equivalent anchorage should be provided by crosspins or other means.

2.45.2 **Bonding of loadbearing wall with partition wall**

Bonding of load-bearing walls with other walls and partitions is not recommended.
1. Metal, plastic or other profile fixed only to the floor.
2. Compressible fire resistant joint.
3. Reinforcement
4. Wall-support allowing separation

Figure 16
2.46 Bonding of partition walls to the structure where movement of
the structure can be expected

In such a case the wall should be allowed to move freely in relation to the
structure, whilst maintaining the stability. In some cases it is recommended
that the horizontal joints of the wall be reinforced, depending on the length
of the wall and the presence of openings.

2.47 Cavity wall grouting

The two leaves are built adequately tied together (see clause 2.42) and
congrete subsequently introduced into the cavity. This can be done by two
methods:

(a) Low lift grouting (see Figure 17):

The concrete should be placed as part of the process of the laying of
the units at a maximum vertical interval of 600mm. A sufficient time
should be allowed before grouting for the mortar to set and be able to
withstand the grout pressure. Any excess mortar and debris should be
removed before infilling. Concrete infill should be placed in layers to
within 50mm of the level of the last course laid. It should be
compacted immediately after.
The concrete should be placed after the two leaves have been built up to a maximum height of 3.0m. Clean out holes should be left along the base of one leaf to allow removal of mortar droppings and debris. These holes should be filled before placing the concrete infill. Concrete should not be poured until the mortar has set and able to withstand the grout pressure; at least 3 days should be allowed. The concrete should be placed in layers not exceeding 1.5m and compacted.
Figure 18: High lift grouting
The Appendices complement some of the principle recommendations. They concern only examples, test methods, constructive solutions and illustrations of an indicative nature and do not exclude other solutions, justified by experience or tradition.

**APPENDIX I: (ref. para. 1.22) - Reference specimen to obtain characteristic compressive strength of masonry $f_{mk}$**

(a) There are a number of ways of using tests on masonry to obtain $f_{mk}$

1°) from tests on small specimens (1.22-2) (refer to Appendix 1 (b) below).

$$f_{mk} = \text{function } (f_b, f_m) = \text{basic or intrinsic strength of masonry.}$$

A number of tests can be carried out and from the test-results the characteristic strength can be deduced on a statistical basis.
The slenderness of the specimen should satisfy the following requirement:

\[
3 \leq \frac{\text{height of specimen}}{\text{thickness of specimen}} \leq 5
\]

2°) from tests on full-scale masonry elements (e.g. one-storey high walls or columns)

\[
f_{\text{element}} = \text{function } (f_m, k, \lambda, e)
\]

3°) tests on very small specimens may be used for identification and control (e.g. on prisms of stack bonded units).
(b) Some reference specimens

Case 1°) above TBE-specimen
    Rilem BW24 (now 76 LUM)
Case 2°) above British Code using wall tests
Case 3°) above US - and Canadian Code

(c) Analysis of results

(i) Recognised statistical methods may be used;

The characteristic strength $f_{mk}$, is defined by:

$$f_{mk} = \bar{f} - ks$$

where $\bar{f}$ is the arithmetic mean of the test results,

$s$ is the (estimated) mean quadratic or standard deviation

It is permitted to use normal, log-normal or asymmetric distributions when calculating $f_{mk}$.

When the standard deviation is based on a very large number of test results, it can be considered as equal to the exact value of the standard deviation of the idealised distribution.

$k$ is a coefficient depending on the probability of obtaining test results less than $f_{mk}$. At the 95% confidence level $k = 1.64$, when there are sufficient results and the normal distribution is valid.
(ii) In the case of a very small number of results the mean value $f_m$ may be used for the judgement of acceptability with criterions such as:

$$f_m > f_{mk} + k_1$$

$$f_i > f_{mk} - k_2$$

where $f_m$ = mean value of test results

$f_i$ = individual value of test results

$f_{mk}$ = characteristic strength

$k_1, k_2$ = values fixed "a priori" according to the severity of the acceptance criterion.

(iii) Using a log-normal distribution, if the results obtained from ten replicates are:

$x_1, x_2, x_3, ... x_{10}$, the values

$y_1, y_2, y_3, ... y_{10}$, should be calculated

where

$y = \log x$ in each case

The mean ($\bar{y}$) and standard deviation ($s$) of $y$ should then be calculated as:

$$y = \frac{(y_1 + y_2 + y_3 + ... + y_{10})/10}{s} = \sqrt{\frac{(y_1^2 + y_2^2 + y_3^2 + ... + y_{10}^2) - (y_1 + y_2 + y_3 + ... + y_{10})^2/10}{9}}$$

The $y_c = y - 1.922s$

Characteristic value = antilog ($y_c$)
(d) Tests are usually carried out to determine $f_{mk}$ in a direction normal to the bed joints. Similar tests on small samples or full scale masonry elements can be carried out to determine $f_{mk}$ in directions other than normal to the bed joints using the method of analysis of the results stated in Appendix 1 (c) above.
APPENDIX 2: (ref. para. 1.22-3)

Compressive strength of units and test methods on units are described in references such as National or International Standards, but for correlation with table 1 the TBE-method is applicable as summarised below.

Compressive strength of structural units $f_b$ - TBE method

Compression tests shall be carried out with no less than 10 unbonded masonry units immediately after being stored in water for 24 hours. Due consideration shall be given to the effect of the brick format on its strength. Bed faces of the test specimens shall be plane and parallel. At least one platen of the testing machine shall be spherically seated to ensure uniform distribution of the load over the total area of compression. Test specimens shall be placed or clamped in a central position on the platen and loaded up to the point of failure at a constant rate (0.5 N/mm².s.).

The compressive strength of the specimen shall be calculated by dividing the ultimate load ($F_{max}$) recorded on the press by the gross cross sectional area ($A_o$) of the specimen.

$$f_b = \frac{F_{max}}{A_o} \text{ in N/mm}^2$$

Tests are normally carried out on masonry units in the direction in which they are laid i.e. normal to the bed joints. It is sometimes necessary to carry out tests on the units to find the compressive strength in the other two directions. This gives figures for use in reinforced masonry particularly for highly perforated units. The strength obtained from a test will need to be corrected to that corresponding to a height to width ratio in the region of 1.0.

Test methods on mortar should be in accordance with the RILEM specification.
APPENDIX 3: (ref. para. 1.24) - Reference specimen to determine flexural strength of masonry

The flexural strength of masonry can be obtained from the following testing methods using wallettes:

- **Vertical bending**
- **Horizontal bending**

Blocks should be immersed in water for 5 min to 6 min and allowed to drain prior to building the wallettes. The wallettes should be built within 1 h of the blocks being removed from the water. Bricks having a suction rate of more than 1.5 kg/m²/min may be docked or the water retentivity of the mortar may be adjusted. The method of conditioning the bricks should be recorded in the test report.

Immediately after building, each wallette should be precompressed with three courses of bricks laid dry, or by the equivalent uniformly distributed weight. The wallette should then be closely covered with a material which does not permit water vapour penetration and maintained undisturbed until testing.
The wallette should be tested in the vertical attitude under four-point loading. The equipment should accommodate variations of plane. Suitable precautions should be used at the contact area of the bearings to ensure that contact is provided over the full width of the masonry e.g. a hollow rubber bolster of 7mm wall thickness and 10mm bore containing 8mm diameter steel rod or a hydraulic bolster consisting of 12mm o.d. nylon tubing with appropriate pipe fittings, supported in 12mm angle irons set at 45° and filled with water and sealed.

The outer bearings should be about 50mm from the edge of the wallette. The spacing of the inner bearings may be varied to suit the format of the masonry but should be between 0.4 and 0.6 times the spacing of the outer bearings. The outer bearings should be located so that they are, as far as practicable, midway between the nearest mortar joints which are parallel to the bearings.

The base of each wallette should be free from frictional restraint e.g. by setting it on two layers of polytetrafluoroethylene (PTFE) or on ball, needle or roller bearings.

The rate of increase in flexural stress should be between 0.3 and 0.4 N/mm²/min.

The replicates should be tested in each format at an age of 28 ± 1 days.
Format of wallettes. For brickwork not more than 102.5mm thick, the format of wallette where the surface of failure will be at right angles to the bed joints should be four bricks long by four courses high. Where the surface of failure will be parallel to the bed joints, the format should be two bricks long by 10 courses high.

For brickwork between 102.5mm and 215mm thick, the length of the first format of wallette should be increased to five bricks long and the height of the second format of wallette should be increased to 14 courses high.

For blockwork, the format of wallette where the surface of failure will be at right angles to the bed joints should be 2½ blocks long by 4 block courses high. Where the surface of failure will be parallel to the bed joints, the format should be 1½ blocks long by 5 block courses high.
**APPENDIX 4: (ref. clause 1.42)**

**Consideration of accidental damage**

For buildings used for residential or similar purposes, primary local collapse may take the form of failure of two walls forming an angle. Generally, only those cases where one of the collapsed walls is an outer wall need be considered. It can be accepted that primary local collapse will cause severe damage in adjacent parts provided that the damage affects only the walls and supported floors of the storey above and the storey below. The collapsed length $l_{a}$, measured from the corner, is defined below:

a) In cases where the wall is unable to withstand a uniform pressure of 10 kN/m$^2$, the lengths $l_{a}$ can be assumed to be equal to the distance between lateral supports or between a lateral support and a free edge (Figure A4-1a).

b) In cases where the wall is capable of withstanding a uniform pressure of 10 kN/m$^2$, a strip is considered whose length from the corner is equal to 3.60m for a gable wall and 1.80m for an inside wall (Figure A4-1b).

![Figure A4-1: Extent of primary local collapse](image-url)

- **Figure A4-1:** Extent of primary local collapse
If there is a free edge beyond this band a distance "a" from the end of the strip, such that "a" is less than or equal to 1.20m, then (Figure A4-2):

- for a gable wall: $l_a = 3.60m + a$
- for an inside wall: $l_a = 1.80m + a$

If $a$ is greater than 1.20m then:

- for a gable wall: $l_a = 3.60m$
- for an inside wall: $l_a = 1.80m$

![Diagram](image)

**Figure A4-2: Definition of $l_a$ for inside walls with free edges**

A gable wall (an outside wall where $1 < 15m$) unstiffened by another wall or one whose length to the free edge is less than 2/3 of its total length, should be capable of withstanding the loads defined in clause 1.42 (ii).

The following definitions apply:

- **free edge:**
  - openings for doors or windows (if the height exceeds 1/3 of the storey height);
  - vertical joints between precast panels;
  - wall edges not joined effectively to a stiffening wall.
- gable wall: long outside wall unstiffened by other walls
- stiffening wall: a wall perpendicular to the gable wall provided that the junction between the two walls is capable of withstanding a force of 25 kN per linear metre in the vertical direction

No additional detailed recommendations are made in respect of buildings of 4 storeys and below. Residential or similar buildings of 5 storeys and above, with reinforced concrete floors, can be considered as satisfying the requirements mentioned in 1.42 (i) by compliance with the rules given in table A4-1. These rules are based on conservative calculation and consequently are less economical than the direct application of clause 1.42 (i).

**TABLE A4-1**

<table>
<thead>
<tr>
<th>Structural Requirements</th>
<th>Type of masonry bearing walls</th>
<th>Capable of withstanding 10 kN/m²</th>
<th>Incapable of withstanding 10 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reinforced concrete floors</td>
<td>Connecting reinforment over support capable of withstanding 30 kN/m²; equal connection of floors to the tie beam at the gable wall</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Horizontal tie-beams at masonry walls</td>
<td>at bearing wall resisting 80 kN, at other walls</td>
<td>at bearing walls resisting 120 kN, at other walls 40 kN</td>
</tr>
<tr>
<td></td>
<td>Vertical tie-beams</td>
<td>no requirements</td>
<td>at the crossing of the bearing wall with the external wall resisting 40 kN</td>
</tr>
<tr>
<td></td>
<td>Door lintels in bearing cross walls</td>
<td>Capable of withstanding shear force of 60 kN and moment ±kN/m</td>
<td></td>
</tr>
</tbody>
</table>
The reinforcement to connect the floor over the support should be made of mild steel

The resistance requirements given in table A4-1 are design values. Masonry walls 25cm thick, made with mortar M20 or M10 (clause 1.22-3, table 2) can be considered as capable of withstanding 10 kN/m².

Masonry walls 40cm thick, made with mortar M5 and below can be considered as capable of withstanding 25 kN/m².
APPENDIX 5 (ref. para. 1.52.2-3) - Concentrated loads under bearings

When a beam or other structural member imposes a concentrated load on a wall the design compressive strength $f_d$ of the masonry may be taken as

$$f_d = \frac{f_{mk}}{\gamma_{mm}} \left( 1 + 0.1 \frac{a_1}{l_1} \right) \leq 1.5 \frac{f_{mk}}{\gamma_{mm}}$$

The design compressive strength may only be varied from $\frac{f_{mk}}{\gamma_{mm}}$ if

$A_1 \leq 2t$ and $e \leq \frac{t}{6}$ (see figure)

Where

- $f_{mk}$ is the characteristic strength of the masonry (table 1)
- $\gamma_{mm}$ is the factor of safety (table 6)
- $a_1$ is the distance from the end of the wall to the edge of the bearing (see figure)
- $l_1$ is the length of the bearings (see figure)
- $A_1$ is the area of bearing
- $e$ is the eccentricity of the centroid of the load application
APPENDIX 6: Calculation of the design compressive strength of a wall

(Ref. para. 1.52.2-2.2 (b) accidental eccentricities
para. 1.52.2-2.2 (c) slenderness of wall
para. 1.52.2-4 design of wall for vertical load)

Apart from the characteristic compressive strength of masonry (para. 1.22) and the structural eccentricities (para. 1.52.2-2.2) the design compressive strength of a wall is dependent on the value of any accidental eccentricity (ea), and the reduction to be made for slenderness. In turn the latter will depend on the slenderness ratio which includes the factor $\rho$, the effective height ($l_e$) and the effective thickness ($t_{ef}$). Unless all of these parameters are considered in a consistent package, dangerous answers may be obtained.

It appears that each National Standard deals with these matters in different ways and so no values have been given in these recommendations, for fear that one value may be taken out of context with another. However, to enable the recommendations to be used, a conservative answer for $N_d'$ will be obtained when using this Appendix.

Values for individual items should not be extracted.

6.1 Accidental Eccentricities

Accidental eccentricities may occur because of:

- dispersion in the wall due to units having different properties ($ea_1$)
- dispersion due to lack of planeness or levelness in the construction of the wall ($ea_2$)
- lack of superposition of the wall between storeys (not to be considered for exterior walls) \( (e_{a3}) \)

No experimental evidence is available for the size of these eccentricities, but some authorities suggest

\[

e_{a1} = 0.02 \, t \quad \text{where} \quad t = \text{wall thickness} \\
e_{a2} = 0.002 \, H \quad \text{where} \quad H = \text{wall height} \\
e_{a3} = 5 \, \text{mm}
\]

These values are indicative and not necessarily cumulative.

\( (e_a \leq e_{a1} + e_{a2} + e_{a3}) \)

If execution of the masonry is carefully carried out and controlled, lower additional eccentricities may be used in the calculations, but not less than an allowance for \( e_a \) of \( 1e/300 \). Where execution and/or control are poor the additional eccentricities should be increased up to \( 2x \) the above mentioned values.

6.2 Slenderness of the wall - \( \rho \) - values

The figure below illustrates the general case of a wall with thickness \( t \), stiffened by floors (interdistance 1) and transverse return walls (interdistance \( a \)), preventing any horizontal displacement of the boundaries of the wall.
$\rho$-value as a function of the boundary conditions of the vertical edges of the wall and of the relation between the height $l$ and the length $a$ of the wall.
If the wall being designed and the return walls are loaded unequally or if the return wall and the main wall are not of materials of similar deformation characteristics, \( p \) must be assumed equal to 1.

This is conservative but should be taken in the general case.

Where return walls are loaded in such a manner that they are monolithic with the designed wall \( p \) can be graphically derived from the figure above as a function of the boundary-conditions of the vertical edges of the wall and of the ratio \( 1/a \). Because in practice \( 1/a \) is often smaller than 1/2, many national codes do not consider the \( p \)-factor (thus \( p = 1 \)), and define the slenderness by

\[
S = \frac{l_e}{t_{ef}}
\]

When 3 edges are supported \( p = 1 \) should be assumed for \( a/t_{ef} > 15 \)

When 4 edges are supported \( p = 1 \) should be assumed for \( a/t_{ef} > 30 \)

6.3 Reduction factor \( \phi \) for eccentricity and slenderness

The slenderness ratio is defined in 1.52.2-3 as

\[
\frac{\rho l_e}{t_{ef}}
\]

where \( \rho \) is taken from 6.2

\( l_e \) is the effective height

\( t_{ef} \) is the effective thickness

When using the appropriate reduction factors, \( \phi \) in the graphs below, the effective height \( l_e \) should be taken as the clear height of the wall between lateral supports; allowance has been made in the curves for any reduction in \( l_e \) below the full height as a result of the statical scheme adopted.
For solid walls, the effective thickness is the actual thickness.

For cavity walls (see 2.42) the effective thickness may be taken as the greater of

\[ \frac{2}{3} \text{ the sum of the thicknesses of the two leaves or the thickness of the thicker leaf.} \]

The reduction factor \( \phi \) can be defined as a function of

\[ \frac{\rho_l e}{t_{ef} \sqrt{\alpha}} \]

where \( \frac{\rho_l e}{t_{ef}} \) is the geometric slenderness,

and:

\[ a = \frac{800}{(1 + \beta \zeta)} \]

where \( \beta \), creep coefficient, = 

- 1 brick masonry
- 1.2 concrete block masonry
- 1.8 light-weight concrete masonry

\( \zeta \) ratio of long duration loading to total loading.
Hinged joints scheme

Rigid and semi-rigid joints scheme
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8.3 Refined testing methods

This Appendix is the result of work of a Group, made up of the following

G. Macchi (Italy) Coordinator
H. Bossenmayer (Germany)
E. Carvalho (Portugal)
A. Giuffre (Italy)
B. Haseltine (Gr. Britain)
C. Modena (Italy)
R. Meli (Mexico)
M. Tomazevic (Yugoslavia)
E. Vintzeleou (Greece)

Meetings were held in Rome in 1982, in Pavia in 1983 and again in Pavia in 1984.

The text takes into account most of the comments and proposals received. Nevertheless, on some points only the prevailing point of view has been reported.

The present text has been finalised after discussion in the plenary meeting of Commission W 23 in Copenhagen in 1985.
0. PRELIMINARY

0.1 Introduction

These Recommendations are an Appendix to the C.I.B. 'International Recommendations for Design and Erection of Unreinforced and Reinforced Masonry Structures', in order to cover seismic design.

0.2 Object and purpose

The Recommendations in this Appendix apply to the design and execution of masonry structures, built in seismic regions, made with units (bricks or blocks) complying with the specifications given in Section 4 and fulfilling the requirements for Masonry Systems given in Section 5, or checked according to the procedures illustrated in Section 8.

The purpose of this Appendix is to protect human life against catastrophic seismic events and to limit damage arising from minor seismic events.

0.3 Symbols and units

The symbols and units used in this Appendix are the same as those used in the main text of the 'International Recommendations'.

It has been necessary to introduce the following additional symbols. It has not been possible to avoid some symbols being used with different meanings, because symbols used internationally in seismic design have been introduced; the risk of confusion is very slight.
A design value of the ground acceleration normalised by g
B width of a building
C base shear coefficient
E seismic action; modulus of elasticity
G permanent load; shear modulus
I importance factor
K behaviour factor
R(T) normalised design spectrum
S type of soil
T fundamental period
W total vertical load

Subscript E, earthquake

h height
n number of floors
s spacing of horizontal reinforcing bars
β exponent of the normalised design spectrum
δ displacement
Limiting damage and making possible repairs should be achieved by suitable choice of unit type, structural scheme and detailing, according to these Recommendations or based on the results of experimental tests.

Unforeseen behaviour can be avoided by:

a) using an adequate structural model in design analysis, and including in the design documents a quality control plan;

b) having adequate organisation of control during execution, and verifying possible modifications which occur during execution using the same criteria adopted in the design.
1. GENERAL REQUIREMENTS

For the planning, design, and construction of structures in seismic regions, in addition to the general rules for non-seismic regions, the following requirements apply.

1.1 Safety against collapse

Masonry structures should be planned, designed and constructed such that they resist the seismic actions defined in Section 2 with an adequate degree of reliability.

An adequate degree of reliability against collapse is ensured if the detailing rules given in Section 6 are observed and the verifications of Section 7 are performed.

1.2 Limiting susceptibility to damage

Masonry structures should be planned, designed and constructed such that the damage caused by a moderate seismic action is limited to an acceptable extent and that repairs are possible.

1.3 Limitation of unforeseen behaviour

The structure should be planned, designed and constructed such that the risk of unforeseen behaviour is limited.
2. SEISMIC ACTION

The seismic actions are dynamic in nature, and have horizontal and vertical components.

For the sake of simplicity, the equivalent static forces approach has been adopted here, more sophisticated analyses should be performed only if supported by documented justification.

2.1 Base Shear

The seismic action is defined by the normalised design spectrum and by the value $A$ of the horizontal acceleration of the ground (related to the gravitational acceleration). With reference to the three soil profiles defined in Section 2.3, three different design spectra are provided. The total horizontal design seismic force $V$ on a structure is calculated as follows:

$$C = \frac{AR(T)}{K}$$

$$W = G + \psi_E Q_k$$

$$V = C W$$

where:

- $E, E'$ = every effect of seismic action
- $C$ = base shear coefficient
- $T$ = fundamental period (See Section 6.22)
- $A$ = design value of the ground acceleration normalised by $g$ according to Section 2.21
- $R(T)$ = normalised design spectrum according to Section 2.22
- $K$ = behaviour factor defined in Section 2.4
- $W$ = total vertical load
- $G$ = nominal value of total permanent load
- $\psi_E$ = combination factor to be applied to variable loads for calculating the inertia forces; it accounts for the probability of the joint occurrence of variable loads acting upon the structure as a whole; the following expression can be used:
In the absence of national specifications, the following values can be used:

**TABLE 2.21**

<table>
<thead>
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<th></th>
<th>Zone L</th>
<th>Zone M</th>
<th>Zone H</th>
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<tbody>
<tr>
<td>A</td>
<td>0.15</td>
<td>0.25</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Introducing the denominations of the zones L M H (Low, Moderate, High seismic activity).

In the absence of national specifications, the following values may be used:

**TABLE 2.22**

<table>
<thead>
<tr>
<th>SOIL</th>
<th>$T_0$</th>
<th>$\beta$</th>
<th>$R_0$</th>
</tr>
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<tbody>
<tr>
<td>S1</td>
<td>0.4</td>
<td>$2/3$</td>
<td>2.5</td>
</tr>
<tr>
<td>S2</td>
<td>0.6</td>
<td>$2/3$</td>
<td>2.5</td>
</tr>
<tr>
<td>S3</td>
<td>1.0</td>
<td>$2/3$</td>
<td>2.0</td>
</tr>
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</table>

Where the denominations of the soil profiles (Rock, stable deposits, soft-to-medium stiff deposits) are according to definitions given in Section 2.3.
\[ \psi_E = \psi_j \left( 0.8 + \frac{1}{n} \right) \geq 1 \]

\[ n = \text{number of floors on which variable loads act} \]
\[ \psi_j = \text{values given in Section 3.21} \]
\[ Q_k = \text{characteristic value of variable loads} \]
\[ V = \text{total horizontal design seismic force (base shear)} \]

2.2 Numerical values of the parameters regarding seismic action

2.21 Intensity parameter

Taking into account the information on the seismicity of each Country, the competent Authorities should define national zones, specifying the design values of the normalised ground acceleration A.

2.22 Normalised design spectra

The normalised design spectra for the three soil profiles defined in Section 2.3 are shown in Figure 2.22. The relevant values of the parameters are specified by the competent National Authorities.

\[ R(T) \]

\[ R(T) = R_c \]
\[ R(T) = R_c \left( \frac{T_c}{T} \right)^{\alpha} \]
\[ R(T) > 0.3 R_c \]

FIG 2.22 Normalised Design Spectra
2.3 Site effects

Local site conditions are allowed for by modifying the shape of the response spectrum according to Figure 2.22.

Each of the 3 curves refers to a soil profile type, as follows:

- Soil profile S1: Rock (shear wave velocity greater than 1000 m/sec) or stable deposits or unconsolidated minerals as for S2, with a depth of less than 50 m on a solid rock base.

- Soil profile S2: Stable deposits (compact sands and gravels or stiff clays) of depth exceeding 50 m on solid rock base.

- Soil profile S3: Soft-to-medium stiff deposits (sands, stiff clays) having a depth of 10 m or more.

If the site investigations do not enable any of the profiles to be used, then the most unfavourable of the 3 curves should be used.

2.4 Behaviour factor K

The capability of a structural system to attain deformations in the post-elastic range is allowed for by means of the behaviour factor $K$. It takes into account the energy dissipation capacity of the system, and should be evaluated according to Sections 5 and 6.
The seismic action is combined neither with any other accidental actions (unless these must be considered as caused by the earthquake or occurring simultaneously) nor with variable actions of short duration (e.g. wind). Values are assigned to variable actions of long duration (such as live loads) which are probable at the time of the earthquake event. These values are obtained by multiplying the characteristic or nominal values by the combination factor \( \psi \).

**Importance factor:**

Structures may be divided into 4 classes for considering the consequences of a seismic failure.

- Structure of which the seismic resistance is of vital importance for civil protection, e.g. hospitals, fire-stations, electricity plants. 
  \[ I = 1.4 \]
- Structures, the seismic resistance of which is of importance, e.g. schools, etc. 
  \[ I = 1.2 \]
- Structures which do not belong to either the above classes or the one below 
  \[ I = 1.0 \]
3. COMBINATION OF ACTIONS

3.1 Components of the seismic action

The two horizontal components of the seismic action can be considered separately.

The vertical component is usually disregarded, but it should be taken into account with its most unfavourable value, together with each horizontal component, in the following cases:
- constructions with high arching forces
- horizontal cantilever members.

3.2 Combination rules for the actions

For the determination of actions to be considered in the verification, the seismic situation is considered as an 'accidental situation'.

For calculating the action effects the following actions are, therefore considered as occurring jointly:

\[ \pm I E' + G \quad \text{or} \quad \pm I E + G + \sum \psi_k Q_k \]

where:

\( I \) = importance factor, a partial safety factor which is applied to the seismic action in order to adapt the degree of seismic protection to the social and economic significance of the relevant building category;

\( E \) = seismic action according to Section 2, calculated by taking account of the presence of masses which were multiplied by the joint occurrence factor \( \psi_E \).
- Structures of minor importance for public safety requirements, e.g. agricultural
  $I = 0.8$

<table>
<thead>
<tr>
<th>Combination factor $\psi_i$ for variable loads:</th>
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<td>Variable loads from persons and equipment</td>
<td>0.3</td>
</tr>
<tr>
<td>Variable loads from persons at places with the likelihood of a large number of occupants (halls)</td>
<td>0.5</td>
</tr>
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<td>Long term storage (warehouses, libraries)</td>
<td>0.9</td>
</tr>
<tr>
<td>Variable loads on staircases and corridors</td>
<td>1.0</td>
</tr>
<tr>
<td>Snow</td>
<td>0.5</td>
</tr>
<tr>
<td>Wind</td>
<td>---</td>
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</table>
E' = Seismic action without variable actions
G = Nominal value of the permanent loads
ψj = Combination factors for variable loads for local verifications (dimensioning of cross sections). They account for the probability that the structural member to be designed is simultaneously exposed to live loads and seismic action.

Qki = Characteristic values or nominal values of variable loads.

3.21 Numerical values for the coefficients appearing in the combination rule.

The values of the coefficients appearing in the combination rules should be defined by the National Authorities. In the absence of national values those given in the Commentary may be used.
4. MATERIALS

4.1 Units

The units used for masonry construction may be solid or perforated, and made of the following materials:

- fired clay (either normal or lightweight)
- concrete (made with lightweight or dense aggregate)
- autoclaved aerated concrete
- calcium silicate
- natural stone.

The characteristic compressive strength of units should not be less than $4 \text{ N/mm}^2$ in the vertical direction, and not less than $2 \text{ N/mm}^2$ in the horizontal direction (in the plane of the wall).

Perforated units to be used for aseismic construction should not have a percentage of perforation of more than 50%; the thickness of the face shells and cross webs should be at least 15mm.

4.2 Mortar

Mortar for structural masonry shall be cement or cement-lime, according to the definitions of the C.I.B. Recommendations.

4.3 Masonry strength (characteristic values)

Compressive strength and shear strength can be derived from appropriate standard tests, or assumed according to the C.I.B. Recommendations according to the properties of the units and of the mortar.

4.4 Steel

The reinforcing steel is defined by its nominal characteristic strength. Such strength should not be more than $450 \text{ N/mm}^2$. 
5. MASONRY SYSTEMS REQUIREMENTS

Several masonry systems may be used in seismic areas, as listed below, provided that they fulfil the requirements given in this section.

Other systems can be adopted if special experimental tests, as illustrated in Section 8, are successfully carried out.

5.1 Unreinforced masonry

Solid units and perforated units according to Section 4.1 may be used in unreinforced masonry but their use is limited to:
- 1 storey buildings in High seismicity zones H
- 2 storey building in zone M

Cavity walls may be used if at least one 6 mm steel horizontal tie for 0.4 m$^2$ of wall area is provided.

Horizontal tie-beams at every floor level should always be provided.
For unreinforced masonry buildings the behaviour factor $K$ should be taken equal to 1.5

5.2 Confined masonry

In this system masonry is confined within horizontal reinforced concrete tie-beams at every floor level and vertical columns placed at every intersection between walls, at both sides of each opening, and at 5 m maximum distance. Further reinforcement should be provided above and below every opening.

Tie-beams and columns should be built as R.C. members; their thickness and width should not be less than 15 cm.
Not less than 4 bars of diameter 10mm tied by stirrups, at 200mm max spacing, should be provided in each of these beams and columns.

Tie-beams and columns can be included into the masonry by means of special units. For confined masonry the behaviour factor K should be taken equal to 2.0

5.3 Reinforced masonry

Reinforced masonry is defined in Section 02 of the CIB Recommendations. For use in seismic areas, widespread reinforcement and concentrated reinforcement are both permitted.

A minimum reinforcement percentage should be provided in the horizontal and in the vertical direction, in order to provide sufficient ductility to the wall by avoiding brittle failure of the steel; the minimum value given in the Recommendations for Reinforced Masonry (see 2.38.2.1) can be insufficient and may need to be increased taking into account the area of the wall and the strength of the masonry.

The amount of reinforcement should also be limited so that brittle failure of the masonry is avoided.

For reinforced masonry buildings, the behaviour factor K may be taken as 2.0 without tests, and may be increased up to 3.0 according to the results of cyclic tests defined in Section 8.1
6. STRUCTURAL DESIGN

6.1 Criteria for planning

The following definitions and requirements should be taken into account in the design.

6.11 Structural systems

Three different structural systems can be defined:

a) the single cantilever shear wall system, in which inelastic deformation only occurs at the base of the walls;

b) the coupled shear walls system, in which firstly the coupling beams enter in the plastic range, and then base hinging of the walls occurs;

c) the perforated shear walls system, in which plastic deformations, by shear or bending, are concentrated in the piers of one, generally the lowest, storey.

For the structural system type c) a reduction of the behaviour factor assigned in Section 5 to the masonry system, or derived by testing as provided in Section 8, should be made. For the structural system type b) the effectiveness of the beams should be proved by test if \( K \) higher than 2 is to be used.

6.12 Definition of 'simple building'

(with reference to Section 6.21)

A building is considered a simple building if the following requirements are fulfilled.

- The height does not exceed 2 stories, each not higher than 3.5m.
- The length of a block does not exceed 4 times its width, and blocks are separated by joints of 40mm throughout the height above the base level.
- At least 75% of the vertical load is supported by bearing walls.
- The horizontal cross section of bearing walls in each of the orthogonal directions is not less than 5% of the gross plan area of the floor; piers having a width to height ratio less than 0.5 should not be considered.
- Bearing walls in one direction are connected with bearing walls in the orthogonal direction at a maximum distance of 7m.
- The difference in mass and in wall horizontal cross section between the stories does not exceed 20%.
- Bearing walls are symmetrical in each storey; if not, torsion can be balanced by at least two parallel walls, having a length at least 50% of the length of the building, placed near the periphery of the building.
- The height to thickness ratio of walls does not exceed 20 for solid units and 15 for perforated units.
- Floors are rigid in their own plane; if they are not monolithic, a reinforced concrete topping at least 40mm thick should be provided; otherwise, effective shear connectors can be used.
- Horizontal tie beams are provided at the floor level and at roof, continuously through all load bearing longitudinal and transverse walls; a beam consists of 2 longitudinal 12mm diameter steel bars with links or stirrups embedded in 75mm thick concrete; full continuity of bars should be provided at corners and junctions.
6.2 **Analytical models for static analysis**

The seismic effects and the effects of the other actions to be considered in the combination rule given in Section 3 should be determined from a static analysis on the basis of suitable models of the building as a whole.

For the analysis of the vertical load effects the behaviour of the wall-floor joint should be represented by the 'hinged joints' scheme.

The total horizontal seismic force $V$, evaluated according to Section 2, should be distributed over the height of the building, according to the following formula:

$$ F_x = V \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i} $$

where:

- $F_x =$ horizontal force at $x$-level
- $W_x =$ $G_x + \psi_E Q_x$, i.e. vertical load of the masses at $x$-level
- $h_x =$ distance of $x$-level from foundation level

The two following types of static analysis are envisaged:

1. Simplified analysis
2. Linear elastic analysis
In the absence of experimental information the following values may be taken for the elasticity modulus:

Modulus of Elasticity: \( E = 1000 f_k \)

where \( f_k \) is the characteristic compressive strength of masonry.

Shear modulus: \( G = 0.4 E \)

The fundamental period of the building can be estimated using the following formulae (period \( T_n \) in sec.):

\[
T_n = 0.055 n \quad \text{or} \quad T_n = 0.075 \frac{h}{B} \left( \frac{h}{h + B} \right) \quad \text{whichever is less.}
\]

where:

- \( n \) = number of floors
- \( h \) = height of the building (m)
- \( B \) = width of the structure, parallel to the direction under consideration (m).
6.21 Simplified analysis

A simplified analysis can be performed for 'simple buildings' as defined in Section 6.12. The total horizontal seismic force should be evaluated using the value $R_o$ for the normalised design spectrum (Section 2.22). It should be divided among the walls, parallel to its direction, proportionately to the gross area of their cross-sections. Walls vertically interrupted by openings should not be considered in the resisting cross-section.

6.22 Linear elastic analysis

A linear analysis should generally be used, related to the building as a whole, according to the following hypothesis:

a) the floors are perfectly rigid in their plane and have no out-of-plane stiffness;

b) flexural, shear and axial deformability are taken into account in evaluating the stiffness of the elements.

Axial force, bending moments and shear force should be evaluated for every element.

If the structural analysis takes into account the coupling beams, a frame analysis could be used for the determination of the effects in the vertical and horizontal structural elements. More refined methods (e.g. finite elements) may also be used. The distribution among the walls of the total base shear, as obtained by the linear analysis, can be modified, provided global equilibrium is assured and the action in any wall is neither reduced more than 30% nor increased more than 50%.
7. STRENGTH VERIFICATION AND DESIGN FOR SEISMIC COMBINATIONS OF ACTIONS

7.0 General criteria for strength verification

The verification for the seismic combination defined in Section 3.2 implies that the structure has been verified for the normal combinations of actions according to the CIB Recommendations and the partial safety coefficients \( \gamma_f \) and \( \gamma_m \) there suggested.

Generally, the strength verification involves a comparison between the load effect and the resistance, according to the design method presented in Section 6.3, and following the procedures given below.

In some cases, the strength verification can consist only in providing suitable details able to assure the effectiveness of the requirements listed in Section 7.3

7.1 Design method

The ultimate limit state design should be adopted. Strength verification should be performed as specified in the following Sections. Load effects should be evaluated according to the combination rule given in Section 3, with \( \gamma_f = 1 \).

The following verification rule should be used:

\[
S_d \leq \frac{R_d}{\gamma_{n1}}
\]

where:

- \( S_d \) is the design load effect evaluated according to the combination rule given in Section 3
\( \gamma_{n1} \) is a partial safety factor related to the type of verification and to the type of element (normally \( = 1 \)).

\( R_d \) is the design resistance evaluated with the characteristic values of the strength of materials (masonry and steel) divided by the corresponding partial safety factors \( \gamma_{m} \) and \( \gamma_{s} \).

Values of the \( \gamma_{m} \) factors should be given by the competent National Authorities. In the absence of a National specification the following values may be used for masonry (compression and shear strength) and for reinforcing steel. Reference is made to the Categories of Control in the factory and of Construction defined in the Recommendations (2.21).

- Category A of Control in factory and of Construction control  
  \( \gamma_{m} = 1.2 \)

- Category B of Control in factory and of Construction control  
  \( \gamma_{m} = 1.5 \)

- Category B of Control in factory and Category C of Construction control  
  \( \gamma_{m} = 1.8 \)

- Reinforcing steel  
  \( \gamma_{s} = 1.0 \)

### 7.2 Structural members

The resisting members in a masonry building are:

Bearing walls, coupling beams, wall-floor joints, horizontal diaphragms.
7.3 **Performance requirements**

All structural members should exhibit ductile behaviour. They should behave in such a way that the damage arising at the first attainment of the ultimate strength is easily repaired.

Horizontal diaphragms should be able to behave as rigid in their plane and to transmit the horizontal forces into the walls.

7.4 **Bearing walls**

Under seismic actions, all bearing walls should be verified under in-plane shear and bending and for vertical forces.

7.41 **Shear verification**

For unreinforced and confined masonry the shear carried by the design load effect should not exceed the design resistance $V_{Rd}$, given by the following formula:

$$V_{Rd} = \frac{f_{vk}}{\gamma_m} A$$

where:

- $f_{vk}$ is the shear strength of masonry given by the CIB Recommendations
- $\gamma_m$ is the partial safety factor given in Section 7.1
- $A$ is the gross cross-sectional area of the wall.
In the reinforced masonry, the steel is often used only to provide ductility (and therefore to increase the K factor) without giving a significant increase in shear strength; in this case the above procedure of verification is still applicable. If, on the other hand, the horizontal reinforcement is intended to carry the entire shear action, the design capacity of the wall is given by the following formula:

\[ V_{Rd} = 0.8 f_{yk} A_{sh} \frac{d}{s} \]

where:
- \( A_{sh} \) = area of each layer of horizontal reinforcement
- \( s \) = vertical spacing of horizontal bars
- \( d \) = effective depth of the wall
- \( f_{yk} \) = characteristic yield strength of steel.

The design capacity should not be greater than:

\[ V'_{Rd} = \frac{f_{vk1}}{\gamma_m} A \quad \text{or} \quad V'_{Rd} = \frac{1.5 f_{vk1}}{\gamma_m} A \]  

for walls with solid masonry units

where:
- \( f_{vk1} \) = limiting value for \( f_{vk} \), defined in CIB Recommendations, 1.23.

7.42 Verification under bending and axial force.

The verification suggested in the CIB Recommendations can be used with the slenderness reduction factor, \( \phi \) and the \( \gamma_m \) and \( \gamma_f \) factors defined in Section 7.1.
In order to avoid degradation of the wall under out-of-plane cyclic actions, it should be verified that under the action \( G + \phi_g Q_k \), the axial force in the wall, does not exceed 1/4 of the appropriate characteristic strength of masonry \( f_k \).

7.5 Coupling beams

7.51 Definition and geometric constraints

The connecting members in coupled cantilever shear walls are called coupling beams if the following requirements are fulfilled:

- they are bonded in true masonry bond
- the thickness is not less that of the connected walls
- in confined and reinforced masonry systems suitable reinforcement is placed at both upper and lower edges, fully anchored in adjoining masonry.

If the above requirements are not fulfilled, the beams should be considered as non-structural elements, and the walls considered as independent cantilevers.

7.52 Shear verification

The procedures for reinforced masonry should be followed.

7.6 Wall-floor joints

The connection between floor and both bearing wall and lateral walls should assure the transmission of actions.
7.7 **Horizontal diaphragms**

Diaphragms are, in general, monolithic reinforced concrete slabs and should have a thickness not less than 1/30 of their span. They may be ribbed or hollow slabs, provided that a layer of continuous concrete having a thickness of not less than 40 mm exists, or there are other means of assuring rigid diaphragm action.

Horizontal diaphragms should be reinforced in two orthogonal directions and the steel should be anchored in the perimeter tie-beams.

When the diaphragms are floor structures having parallel ribbing, further reinforcement should be placed in the upper layer of concrete orthogonally to the ribs.

Concentrated ties should be placed at distances not exceeding 5m.

Precast slabs should be reinforced in two orthogonal directions and connected with the support beams and with each other by shear connectors, so that the whole forms a rigid diaphragm.
prevailing Shear Test

Shear and Bending Test

Diagonal Compression Test
8. DESIGN BY TESTING

Experimental tests may be used for the determination of the available ductility of walls. (see Section 5.3).

8.1 Cyclic tests on walls or panels in reinforced masonry.

The aim of the tests is to determine experimentally the ductility level of a given technique, and so the behaviour factor $K$ (see Section 2.4).

The following types of test can be used:
- prevailing shear test
- shear and bending test
- diagonal compression test

according to the figures.

During the tests, the lateral displacements $\delta$ should be measured.

A complete reversed cycle should first be performed up to $1/3$ of the expected maximum capacity, followed by a series of cycles with increasing values of the displacement $\delta$.

A conventional value $\delta_y$ is obtained in the first cycle by extending up to the maximum load, the secant to the first cycle at 0.7 of the maximum load. (linear uncracked $\delta$).

$\delta_y$ represents the increment to be applied in each direction at each semi-cycle.

The test is concluded when in one direction the maximum force reached at the end of a semi-cycle is less than 0.7 the maximum capacity.
If the progressive stiffness deterioration does not allow the maximum load to be reached in each cycle before $2\delta_y$, $3\delta_y$, ......, the load should be increased up to the value of the horizontal tangent to the curve.

8.2 Evaluation of available ductility.

After the test described in Section 8.1, the behaviour factor $K$ (see Section 2.4) can be assumed to have the following values:

- $K = 2$ if the test ends at $5\delta_y$ or less
- $K = 2.5$ if the test exceeds $6.25\delta_y$
- $K = 3$ if the test exceeds $7.5\delta_y$. 

![Diagram showing cyclic loading and horizontal tangent](image-url)
8.3 Refined testing methods.

More complete information on the cyclic behaviour of a given technique can be obtained by performing several tests on similar specimens.

The following procedure is suggested:

a) 1 monotonic test on one specimen, reaching the permitted degradation $0.7 \ F_{\text{max}}$;

b) cyclic alternate tests until stabilisation under a single imposed displacement $\delta_i$, followed by a monotonic test as in a); for each $\delta_i$ the test should to repeated on at least 2 specimens; the values of $\delta_i$ should be increased up to the permitted strength degradation $0.7 \ F_{\text{max}}$. 