COMPLEMENTARY TECHNICAL NORMS
FOR DESIGN AND CONSTRUCTION
OF MASONRY STRUCTURES
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Complementary Technical Norms for Design and Construction of Masonry Structures

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Complementary Technical Norms for the Design and Construction of Masonry Structures

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$</td>
<td>total area of longitudinal reinforcing steel placed on each of a end tie-columns of a wall in confined masonry; area of vertical reinforcing steel in internally reinforced masonry, $\text{mm}^2$ ($\text{cm}^2$)</td>
</tr>
<tr>
<td>$A_{sc}$</td>
<td>area of transversal reinforcing steel of tie-columns placed at a spacing $s$, $\text{mm}^2$ ($\text{cm}^2$)</td>
</tr>
<tr>
<td>$A_{sh}$</td>
<td>area of horizontal reinforcement placed at a spacing $s_h$, $\text{mm}^2$ ($\text{cm}^2$)</td>
</tr>
<tr>
<td>$A_{st}$</td>
<td>area of steel devices or connectors, placed at a spacing $s$, necessary for continuity to transverse walls with flushed end, $\text{mm}^2$ ($\text{cm}^2$)</td>
</tr>
<tr>
<td>$A_{sv}$</td>
<td>area of vertical reinforcing steel placed at a spacing $s_v$, $\text{mm}^2$ ($\text{cm}^2$)</td>
</tr>
<tr>
<td>$A_T$</td>
<td>gross area of cross section of wall or wall segment, including tie-columns, $\text{mm}^2$ ($\text{cm}^2$)</td>
</tr>
<tr>
<td>$B$</td>
<td>floor dimension, measured parallel to static torsional eccentricity, $e_s$, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$b$</td>
<td>bearing length of slab supported by wall, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$c_j$</td>
<td>coefficient of variation of the compressive strength of fill mortar or grout</td>
</tr>
<tr>
<td>$c_m$</td>
<td>coefficient of variation of the compressive strength of masonry prismy</td>
</tr>
<tr>
<td>$c_p$</td>
<td>coefficient of variation of the compressive strength of units</td>
</tr>
<tr>
<td>$c_v$</td>
<td>coefficient of variation of the diagonal compressive strength of masonry specimen walls</td>
</tr>
<tr>
<td>$c_z$</td>
<td>coefficient of variation of the strength of interest of samples</td>
</tr>
<tr>
<td>$d$</td>
<td>distance between centroid of tensile steel and extreme fiber under compression, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$d'$</td>
<td>distance between centroids of steel placed at both ends of a wall, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$d_h$</td>
<td>diameter of reinforcing bars, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$E_m$</td>
<td>modulus of elasticity of masonry for compressive stresses perpenicular to joints, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$E_s$</td>
<td>modulus of elasticity of reinforcing steel, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$e$</td>
<td>eccentricity with which the load acts on natural-stone masonry, including effects of lateral pressure, if any, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$e_c$</td>
<td>eccentricity with which the load is transmitted from slab to end walls, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$e_s$</td>
<td>static torsional eccentricity, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$e'$</td>
<td>eccentricity calculated for obtaining the reduction factor for eccentricity and slenderness, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$F_{AE}$</td>
<td>factor of effective area of load bearing walls,</td>
</tr>
<tr>
<td>$F_E$</td>
<td>reduction factor due to eccentricity and slenderness effects</td>
</tr>
<tr>
<td>$F_R$</td>
<td>Strength reduction factor</td>
</tr>
<tr>
<td>$f_{c'}$</td>
<td>specified compressive strength of concrete, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$\overline{f_c}$</td>
<td>mean compressive strength of mortar cubes or grout cylinders, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$f_{c'}^*$</td>
<td>design compressive strength of grout, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$\overline{f_m}$</td>
<td>mean compressive strength of masonry prism, corrected due to its height-to-thickness ratio and referred to gross area, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$f_{m'}$</td>
<td>design compressive strength of masonry, referred to gross area, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$\overline{f_p}$</td>
<td>mean compressive strength of units, referred to gross area, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$f_{p'}$</td>
<td>design compressive strength of units, referred to the gross area, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$f_y$</td>
<td>specified yield stress of reinforcing steel, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$f_{yh}$</td>
<td>specified yield stress of horizontal reinforcing steel or welded wire mesh, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$G_m$</td>
<td>modulus of shear of masonry, $\text{MPa (kg/cm}^2\text{)}$</td>
</tr>
<tr>
<td>$H$</td>
<td>free height of wall between bracing element, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$H_o$</td>
<td>minimum length, measured at the ends of tie-columns, over which stirrups at closer spacing less must be placed, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$h_c$</td>
<td>sectional dimension of tie-columns or bond beam that provide in-plane confinement to the wall, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$k$</td>
<td>wall effective height factor</td>
</tr>
<tr>
<td>$L$</td>
<td>wall effective length (mm (cm))</td>
</tr>
<tr>
<td>$L'$</td>
<td>spacing of elements that brace the wall in the transverse direction, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$L_d$</td>
<td>development length of a straight reinforcing bar under tension, $\text{mm (cm)}$</td>
</tr>
<tr>
<td>$M_R$</td>
<td>in-plane design flexural moment strength, on a wall subjected to axial and bending moment, $\text{N-mm (kg-cm)}$</td>
</tr>
<tr>
<td>$M_o$</td>
<td>in-plane design flexural moment strength, on a wall subjected to pure bending, $\text{N-mm (kg-cm)}$</td>
</tr>
</tbody>
</table>
1. GENERAL CONSIDERATIONS

1.1 Scope

These Norms contain minimum requirements for the analysis, design, and construction of masonry structures.

Chapters 2 through 10 of these provisions apply to the analysis, design, construction and inspection of masonry structures made with walls constructed with prismatic units of fabricated solid or hollow units, or with natural stones joined with a binding mortar. They include walls reinforced with internal reinforcement, tie-columns, bond beams, or buttresses.

Chapters 4 through 7 refer to different construction systems based on masonry with fabricated units. While the behavior of construction systems is, generally, similar to each other, a division is established in chapters to ease analysis and design processes.

Chapter 8 applies to the design of structures made with natural stones.

Chapters 9 and 10 refer to the construction, inspection and quality control of the job.

Chapter 11 applies to the evaluation and rehabilitation of masonry structures.

In the Normative Appendix A, acceptance criteria for construction systems based on masonry designed for earthquakes are presented.

1.2 Measurement units

Provisions in these Norms are stated in the International System (SI) units, and in parenthesis in the usual metric system (whose basic units are meter, kilogram, and second).

The corresponding values in the two systems are not exactly equivalent, for this reason, each system must be used independently from each other, without combining them.

1.3 Other types of masonry units and other reinforcement and wall construction layout

Any other type of units, reinforcement or construction procedure based on masonry, different from those referred to here, shall be evaluated according to the Code and the Normative Appendix A of these Norms.

2. MATERIALS FOR MASONRY

2.1 Masonry units

2.1.1 Type of masonry units

Masonry units used in structural elements of masonry must comply with NMX-C-404-ONNCCE, except as stated for the lower limit of the net area of hollow units referred to in 2.1.1.2 (fig. 2.1)

The minimum net unit weight of the masonry units, in dry state, shall be as indicated in table 2.1.

In Chapter 5 of Complementary Technical Norms for Seismic Design, different seismic behavior factors, Q, are stated, as a function, among others, of the type of units used in the wall.
2.1.1.1 Solid units

For masonry units purposes of Chapter 5 of the Complementary Technical Norms for Seismic Design, and of these Standards, solid units are those having in their most unfavorable cross section a net area at least 75% of the gross area, and whose exterior shells have a thickness no less than 20 mm.

2.1.1.2 Hollow masonry units

The hollow units referred to in these Norms and in Chapter 5 of the Complementary Technical Norms for Seismic Design are those having, in their most unfavorable cross section, a net area at least 50% the gross area; in addition, thickness of their outer faces is no less than 15 mm (fig. 2). For hollow units with two, up to four cells, minimum thickness of the inner nerves shall be 13 mm. For multi-perforated pieces, whose perforations are of the same dimension and even distribution, minimum thickness of the inner nerves shall be 7 mm. Multi-perforated units are those with more than seven perforations or cells (fig. 2).

For purposes of these norms, hollow units with cells or perforations orthogonal to bearing sides are permitted.

2.1.2 Compressive strength

The Compressive strength is determined for each type of unit according to the test method in NMX-C-036.

For design, strength, \( f_{p}^{*} \), shall be used, measured over gross area, which shall be determined as that reached by at least 98% of the units produced.

The design strength shall be determined based on the existing statistical information on the product or from sampling of units, whether in plant or in site. If sampling is chosen, at least three samples shall be obtained, each from ten units, from different batches of production. The 30 units shall be tested in a laboratory accredited by the recognized accreditation entity according to the Federal Law on Metrology and Standardization. The design strength shall be computed as

\[
f_{p}^{*} = \frac{\overline{f_{p}}}{1 + 25c_{p}}
\]

where

- \( \overline{f_{p}} \) mean compressive strength of units, referred to gross area; and
- \( c_{p} \) coefficient of variation of the compressive strength of units

The value of \( c_{p} \) shall not be taken less than 0.20 for units mechanical plants evidencing a quality control system like that required in the NMX-C-404-ONNCCE standard, nor 0.30 for units from industrialized fabrication, but not having a quality control system, nor 0.35 for units from hand-made production.

The quality control system refers to several documented procedures of the production line of interest, including routine tests and their records.

For purposes of these Norms, the minimum compressive strength of units of the Mexican Norms NMX-C-404-ONNCCE corresponds to strength \( f_{p}^{*} \).

2.2 Cementitious materials

2.2.1 Hydraulic cement

In the fabrication of concrete and mortar, any type of hydraulic cement complying with the specified requirements in NMX-C-414-ONNCCE shall be used.

2.2.2 Masonry cement

In the fabrication of mortars, masonry cement complying with requirements specified in NMX-C-021 may be used.
2.2.3 Hydrated lime

In dosification of mortars, hydrated lime complying with requirements specified in NMX-C-003-ONNCCE may be used.

2.3 Stone aggregates

Aggregates shall comply with specifications of NMX-C-111.

2.4 Mixing water

Mixing water for mortar or concrete shall comply with specifications of the NMX-C-122. Water shall be stored in clean and covered containers.

2.5 Mortars

2.5.1 Compressive Resistance

Compressive strength of mortar, whether for binding units or for lime grouts, shall be determined according to the test specified in the NMX-C-061-ONNCCE.

Compressive strength of coarse grouts shall be determined from test cylinders made, cured, and tested according to NMX-C-160 and NMX-C-083-ONNCCE.

For design strength value $f_{ij}^*$, determined as that reached by at least 98% of the samples, shall be used. Design strength shall be computed from mortar samples, from mortar used for joining masonry units or to fill cavities, or from the coarse grout to be used.

In case of mortar, at least three samples shall be obtained, each one from at least three cube specimens units. The nine test specimens shall be tested following NMX-C-061-ONNCCE.

In case of coarse grouts, at least three cylindrical test specimens shall be obtained. The test specimens shall be made, cured, and tested, according to the above mentioned standards.

The design strength shall be

$$f_{ij}^* = \frac{f_{ij}}{1 + 25c_{ij}} \quad (2.2)$$

where

- $f_{ij}$ mean compressive strength of mortar cubes or concrete cylinders for coarse grouts; and
- $c_{ij}$ coefficient of variation of the compressive strength of mortar or, in no case it shall be taken less than 0.2

2.5.2 Mortar for binding pieces

Mortars used in masonry structural elements shall comply with the following requirements:

a) Mortar the compressive strength shall be at least 4 MPa (40 kg/cm²)

b) Mortar they shall always contain cement with the minimum amount indicated in table 2.2.

c) Volumetric ratio between sand and the total of cementitious material shall be between 2.25 and 3. The sand volume shall be measured in dry condition.

d) The minimum amount of water resulting in an easily workable mortar shall be used.

If mortar includes masonry cement, the minimum quantity, to be used in combination with hydraulic cement, shall be as indicated in table 2.2

2.5.2 Fine and course grouts

Fine and course grouts used in masonry structural elements for filling cells in hollow pieces shall comply with the following requirements:

a) Compressive strength shall be at least 12.5 MPa (125 kg/cm²)

b) Minimum size of aggregate shall not be greater than 10 mm.

c) The minimum amount of water allowing the mixture to be fluid enough for filling cells and completely covering the vertical reinforcing bars in case of having internal reinforcement, shall be used. Admixtures that enhance workability shall be accepted.

### Table 2.2 Proportioning, in volume, recommended for fine and course grouts in structural elements

<table>
<thead>
<tr>
<th>Type of mortar</th>
<th>Parts or hydraulic cement</th>
<th>Parts of masonry cement</th>
<th>Parts of hydrated lime</th>
<th>Parts of sand</th>
<th>Nominal compressive strength, $f_{ij}^*$, MPa (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1</td>
<td>—</td>
<td>0 to ¼</td>
<td>—</td>
<td>12.5 (125)</td>
</tr>
<tr>
<td>II</td>
<td>1</td>
<td>0 ½</td>
<td>—</td>
<td>¼ to ½</td>
<td>7.5 (75)</td>
</tr>
<tr>
<td>III</td>
<td>1</td>
<td>½ to 1</td>
<td>—</td>
<td>½ to ¼</td>
<td>4.0 (40)</td>
</tr>
</tbody>
</table>

† The sand volume shall be measured in the bulk condition

**NOTATION**

- $f_{ij}^*$ mean compressive strength of mortar cubes or concrete cylinders for coarse grouts; and
- $c_{ij}$ coefficient of variation of the compressive strength of mortar or, in no case it shall be taken less than 0.2
d) In Table 2.3, nominal slumps recommended for fine and course grouts based on absorption masonry units, are included.

Table 2.3 Permissible slump for fine and course grouts, as a function of absorption of masonry units

<table>
<thead>
<tr>
<th>Absorption of masonry units, %</th>
<th>Nominal slump¹, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 to 10</td>
<td>150</td>
</tr>
<tr>
<td>10 to 15</td>
<td>175</td>
</tr>
<tr>
<td>15 to 20</td>
<td>200</td>
</tr>
</tbody>
</table>

¹ Slumps with tolerances of ± 25 mm.

Table 2.4 Proportioning, in volume, recommended for fine and course grouts in structural elements

<table>
<thead>
<tr>
<th>Type</th>
<th>Parts of hydraulic cement</th>
<th>Parts of hydrated lime</th>
<th>Parts of sand¹</th>
<th>Parts of gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar</td>
<td>1</td>
<td>0 to 0.25</td>
<td>2.25 to 3</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>1</td>
<td>0 to 0.1</td>
<td>2.25 to 3</td>
<td>1 to 2</td>
</tr>
</tbody>
</table>

¹ Volume of sand shall be measured in bulk state

The volumetric ratios recommended among the various components are shown in Table 2.4

2.6 Admixtures

During concrete fabrication, fine and course grouts, admixtures enhancing workability and complying with requirements specified in NMX-C-255 can be used. Set accelerating admixtures shall not be used.

2.7 Reinforcing steel

Reinforcement used in tie-columns, bond beams, elements placed inside the wall or and/or outside the wall, shall consist of deformed bars, steel mesh, cold laminated deformed wire, or by electrically welded trusses, made of steel wire for tie-columns and bond beams, complying with the corresponding Mexican Standards. Plain bars, such as wires, shall be permitted only in stirrups, in welded wire mesh or in connectors. The minimum diameter for wires to be used in stirrups is 5.5 mm. Other types of steel can be used as long as its efficiency as a structural reinforcement is demonstrated to the government satisfaction.

The modulus of elasticity for reinforcement steel, Eₘ, shall be assumed equal to 2x10⁵ MPa (2x10⁶ kg/cm²)

For design, the minimum yield stress, fₘ, established in the above mentioned Standards, shall be considered.

2.8 Masonry

2.8.1 Compressive strength

The design compressive strength of masonry, f_m*, based on gross area, shall be determined with one of the three procedures indicated in 2.8.1.1 through 2.8.1.3. Strength in this Norms is referred to 28 days. If it is considered that wall will be loaded with design actions before this age, strength of the time of loading shall be evaluated in following 2.8.1.1.

2.8.1.1 Testing of prism constructed with units and mortars to be used in the work.

The prisms (Fig. 2.2) shall be made by at least three stack-bonded units. The height-to-thickness ratio of the pile shall be between two and five; the piles shall be tested at the age of 28 days. In the fabrication, curing, transportation, storage, capping, and testing procedure of specimens the corresponding Mexican Standards shall be followed.

Figure 2.2 Prism for compressive strength test method

Strength determination shall be made on a minimum of nine prisms in total, made up with masonry units coming from at least three different batches of the same product.

The average stress obtained, computed over the gross area, shall be corrected multiplying by the factors given in Table 2.5

Table 2.5 Corrective factors for strength of piles with different height-to-thickness ratios

<table>
<thead>
<tr>
<th>Height–thickness ratio¹</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Correction factor</td>
<td>0.75</td>
<td>0.90</td>
<td>1.00</td>
<td>1.05</td>
</tr>
</tbody>
</table>

¹ For intermediate height-to-thickness ratios correction factor shall be linearly interpolated

The design compressive strength shall be computed as

\[
f_m^* = \frac{f_m}{1 + 25c_m} \quad (2.3)
\]

where

f_m mean compressive strength of prism, corrected due to its height-to-thickness ratio and referred to gross area; and

c_m coefficient of variation of compressive strength of masonry prismatic, in no case shall be less than 0.15

2.8.1.2 From design strength of masonry units and mortar

Masonry units and mortar shall comply with quality requirements specified in 2.1 and 2.5, respectively.
a) For concrete blocks and bricks with height-to-thickness ratio no less than 0.5, and with $f_{m}^{*} = 10$ MPa (100 kg/cm²), the design compressive strength may be that indicated in table 2.6.

Table 2.6 Design compressive strength of concrete masonry units ($f_{m}^{*}$, over gross area)

<table>
<thead>
<tr>
<th>$f_{p}^{*}$, MPa (kg/cm²)</th>
<th>$f_{m}^{*}$, MPa (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (100)</td>
<td>5 (50)</td>
</tr>
<tr>
<td>15 (150)</td>
<td>7 (75)</td>
</tr>
<tr>
<td>10 (100)</td>
<td>20 (200)</td>
</tr>
<tr>
<td>15 (150)</td>
<td>30 (300)</td>
</tr>
<tr>
<td>20 (200)</td>
<td>40 (400)</td>
</tr>
<tr>
<td>30 (300)</td>
<td>60 (600)</td>
</tr>
<tr>
<td>≥ 50 (500)</td>
<td>100 (1000)</td>
</tr>
</tbody>
</table>

1 For intermediate values of $f_{p}^{*}$, linearly interpolate for the same type of mortar.

The values of $f_{m}^{*}$ of this table are valid for masonry units complying with $f_{p}^{*}$ stated in the table and with 2.1, and for masonry with horizontal joint thickness between 10 and 12 mm if the pieces are industrialized, or 15 mm if they are hand-made. For other cases, the strength must be determined according to 2.8.1.1.

b) For clay masonry units with height-to-thickness ratio no less than 0.5, the design compressive strength may be that obtained from table 2.7.

Table 2.7 Design compressive strength of clay masonry units ($f_{m}^{*}$, over gross area)

<table>
<thead>
<tr>
<th>$f_{p}^{*}$, MPa (kg/cm²)</th>
<th>$f_{m}^{*}$, MPa (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 (60)</td>
<td>2 (20)</td>
</tr>
<tr>
<td>7 (75)</td>
<td>3 (30)</td>
</tr>
<tr>
<td>10 (100)</td>
<td>4 (40)</td>
</tr>
<tr>
<td>15 (150)</td>
<td>6 (60)</td>
</tr>
<tr>
<td>20 (200)</td>
<td>8 (80)</td>
</tr>
<tr>
<td>30 (300)</td>
<td>12 (120)</td>
</tr>
<tr>
<td>40 (400)</td>
<td>14 (140)</td>
</tr>
<tr>
<td>≥ 50 (500)</td>
<td>16 (160)</td>
</tr>
</tbody>
</table>

1 For intermediate values of $f_{p}^{*}$, linearly interpolate for the same type of mortar.

The values of $f_{m}^{*}$ of this table are valid for masonry units complying with strength $f_{p}^{*}$ stated in it and with 2.1, and for masonry with horizontal joints between 10 and 12 mm if the pieces are industrialized, or 15 mm if they are hand-made. For other cases, the strength must be determined according to 2.8.1.1.

2.8.1.3 Indicative values

If experimental determinations are not made, values of $f_{m}^{*}$ shown in table 2.8 for different types of pieces and mortars may be used.

The values of $f_{m}^{*}$ in this table are valid for masonry units complying with strength $f_{p}^{*}$ shown in the table and with 2.1, and for masonry with horizontal joint thickness between 10 and 12 mm if the masonry units are industrialized, or 15 mm if they are hand-made. For other cases, the strength must be determined according to 2.8.1.1.

Table 2.8 Design compressive strength of masonry, $f_{m}^{*}$, for some types of masonry units, over gross area.

<table>
<thead>
<tr>
<th>Masonry unit</th>
<th>$f_{m}^{*}$, MPa (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burnt clay brick</td>
<td>1.5 (15)</td>
</tr>
<tr>
<td>Clay brick with vertical voids</td>
<td>4 (40)</td>
</tr>
<tr>
<td>Concrete block (heavyweight¹)</td>
<td>2 (20)</td>
</tr>
<tr>
<td>Concrete brick (heavyweight¹)</td>
<td>2 (20)</td>
</tr>
</tbody>
</table>

¹ With net unit weight, in dry condition, no less than 20 kN/m³ (2 000 kg/m³)

2.8.2 Diagonal compressive strength

The design diagonal compressive strength of masonry, $v_{m}^{*}$, over gross area of diagonal, shall be determined with one of the two procedures indicated in sections 2.8.2.1 and 2.8.2.2. The value of strength in this Standard is referred at 28 days. If it is considered that wall will be loaded with design actions prior to this age, strength must be calculated at this age according to section 2.8.2.1.

2.8.2.1 Testing of walls specimens made of masonry units and mortars that will be used in the job.

The walls specimens (fig. 2.3) shall have a length at least one and a half times the length of the masonry unit and the number of courses needed so that the height is about the same as the length. The walls specimens shall be tested subjecting them to a monotonic compression along its diagonal and the mean shear strength shall be determined dividing the maximum load by the gross area of the wall specimens measured along the same diagonal.

Walls specimens shall be tested at 28 days of age. In the fabrication, curing, transportation, storage, capping, and test procedures of specimens, the corresponding Mexican Standard shall be followed.

The determination shall be made on a minimum of nine walls made of masonry units from at least three different lots.

The design diagonal compressive strength, $v_{m}^{*}$, shall be equal to
\[ v_m^* = \frac{v_m}{1 + 2.5c_v} \]  
(2.4)

where

- \( v_m \) mean diagonal compressive strength of walls specimens, over gross area measured along the diagonal parallel to load; and
- \( c_v \) coefficient of variation of diagonal compressive strength of walls specimens, which in no case shall be less than 0.20.

![Figure 2.3 Wall specimen for diagonal compression testing](image)

For walls having some reinforcing system whose contribution to strength in required to be evaluated, or having characteristics that cannot be represented in the size of the wall specimen, the diagonal compressive tests described above shall be made in square walls of at least 2 m side.

### Table 2.9 Design diagonal compressive strength for some types of masonry, over gross area

<table>
<thead>
<tr>
<th>Masonry unit</th>
<th>Type of mortar</th>
<th>( v_m^* ) (MPa) (kg/cm²)</th>
<th>( v_m^* ) (MPa) (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burnt clay brick ( f_p^* )</td>
<td>I</td>
<td>0.35 (3.5)</td>
<td>0.35 (3.5)</td>
</tr>
<tr>
<td>= 6 Mpa, 60 kg/cm²</td>
<td>II</td>
<td>0.25 (3.5)</td>
<td>0.25 (3.5)</td>
</tr>
<tr>
<td>Clay brick with vertical voids</td>
<td>I</td>
<td>0.3 (3)</td>
<td>0.3 (3)</td>
</tr>
<tr>
<td>( f_p^* = 12 \text{ Mpa}, 120) kg/cm²</td>
<td>II</td>
<td>0.2 (2)</td>
<td>0.2 (2)</td>
</tr>
<tr>
<td>Concrete block</td>
<td>I</td>
<td>0.35 (3.5)</td>
<td>0.35 (3.5)</td>
</tr>
<tr>
<td>(heavyweight (^2) )</td>
<td>II</td>
<td>0.25 (2.5)</td>
<td>0.25 (2.5)</td>
</tr>
<tr>
<td>Concrete bricks ( f_p^* )</td>
<td>I</td>
<td>0.3 (3)</td>
<td>0.3 (3)</td>
</tr>
<tr>
<td>= 10 \text{ Mpa}, 100 kg/cm²</td>
<td>II</td>
<td>0.2 (2)</td>
<td>0.2 (2)</td>
</tr>
</tbody>
</table>

1. When the value in the table is greater than 0.25 \( \sqrt{f_m^*} \), in MPa (0.8 \( \sqrt{f_m^*} \), in kg/cm²) the latter value shall be taken as \( v_m^* \).
2. With net unit weight, in dry condition, no less than 20 kN/m³ (2 000 kg/m³)

#### 2.8.2.2 Indicative values

If testing of walls is not performed, design diagonal compressive strength shall be that indicated in table 2.9. Specimen hollow units referred to in the table shall comply with section 2.1.1.

The values \( v_m^* \) in this table are valid for masonry units complying with strength \( f_p^* \) shown in the table and in section 2.1, and for masonry with horizontal joint thickness between 10 and 12 mm. For other cases, the strength shall be determined according to section 2.8.2.1.

#### 2.8.3 Bearing strength

When a concentrated load is transmitted directly to masonry, bearing strength shall not exceed 0.6 \( f_m^* \).

#### 2.8.4 Tensile strength

Masonry tensile strength perpendicular to joints shall be considered equal to zero. When this strength is required, the necessary reinforcing steel shall be supplied.

#### 2.8.5 Modulus of elasticity

The modulus of elasticity of masonry, \( E_m \), shall be determined with one of the procedures indicated in sections 2.8.5.1 and 2.8.5.2.

**2.8.5.1 Testing of prisms made of masonry units and mortar that will be used in the job.**

Prism of the type, age, and number indicated in 2.8.1.1 shall be tested. The modulus of elasticity for short term loads shall be determined according to the corresponding Mexican Standard.

For obtaining the modulus of elasticity for sustained loads, long-term deformations due to plastic flow of masonry units and mortar shall be considered. Optionally, the modulus of elasticity for short term loads obtained from prismy testing can be divided by 2.3 if concrete unit are used, and 1.7 if clay units or any other material different from concrete is used.

**2.8.5.2 Determination from design of masonry.**

\( E_m = 800 \ f_m^* \) for short term loads
\( E_m = 350 \ f_m^* \) for sustained loads

**2.8.6 Shear modulus**

The masonry shear modulus, \( G_m \), shall be determined with one of the procedures indicated in sections 2.8.6.1 and 2.8.6.2. Section 2.8.6.2 shall be applied if the modulus of elasticity is determined according to section 2.8.5.2.

**2.8.6.1 Testing of walls specimens made of masonry units and mortar to be used in the job.**
Wall specimens of the type, age, and number shown in section 2.8.2.1 shall be tested. The shear modulus shall be determined according to the corresponding Mexican Standard.

2.8.6.2 Determination from modulus of elasticity of masonry

If section 2.8.5.2 is chosen to be used for determining the modulus of elasticity of masonry, the shear modulus of masonry may be taken as

\[ G_m = 0.4 \cdot E_m \]  \hspace{1cm} (2.9)

### 3. GENERAL SPECIFICATIONS FOR ANALYSIS AND DESIGN

#### 3.1 Design criteria

Dimensioning and detailing of structural elements shall satisfy according to the criteria related to ultimate and serviceability limit states established in the Title Six of the Code and these Standards, or by some optional procedure complying with the requirements of the title six. In addition, structures shall be designed for durability.

Internal forces and moments produced by the actions to which the structures are subjected shall be determined according to criteria prescribed in section 3.2.

##### 3.1.1 Ultimate limit states

According to ultimate limit state criteria, structures and structural elements shall be dimensioned and detailed in such a way that design strength in any section shall be at least equal to the design value of the internal force or moment.

Design strengths shall include the corresponding strength reduction factor, \( F_R \), prescribed in section 3.1.4.

Design internal forces and moments are obtained by multiplying by the corresponding load factor, the values of such internal forces and moments computed under actions specified in the Title Six of the Code Regulations and the Complementary Technical Norms on Criteria and Actions for Structural Design of Buildings.

##### 3.1.2 Servicability limit states

It must be verified that the structure responses (settlement, strains, cracking, vibrations, etc.) shall be limited to values such that operations in service conditions is satisfactory.

##### 3.1.3 Design for durability

Structures shall be designed and detailed for durability/or an inspected useful lifetime of the structure is 50 years.

The minimum requirements in these Standards are applicable for elements exposed to non-aggressive environments, both indoors or outdoors, and corresponding to exposition A1 and A2, according to Complementary Technical Norms of Design and Construction of Concrete Structures.

If the element will be exposed to more aggressive environments, the design criteria for durability of concrete structures shall apply.

#### 3.1.4 Strength reduction factors

Stresses shall be reduced by a strength reduction factor, \( F_R \). It is acceptable to apply these values in those construction and reinforcing schemes whose experimental behavior has been evaluated and satisfies Normative Appendix A. The values of the strength reduction factor shall be the following.

##### 3.1.4.1 In walls subjected to axial compression

\[ F_R = 0.6 \]  for confined walls (Chapter 5) or internally reinforced walls (Cap. 6). 

\[ F_R = 0.3 \]  for unconfined masonry without reinforcement (Chapter 7).

##### 3.1.4.2 In walls subjected to in plane or out-of-plan, axial force and bending.

For confined walls (Chapter 5) or internally reinforced (Chapter 6).

\[ F_R = 0.8 \quad \text{if} \quad P_p \leq \frac{P_u}{3} \]

\[ F_R = 0.6 \quad \text{if} \quad P_p > \frac{P_u}{3} \]

For unconfined masonry without reinforcement (Chapter 7).

\[ F_R = 0.3 \]

##### 3.1.4.3 In walls subjected to shear force

\[ F_R = 0.7 \]  for infill walls (Chapter 4), confined walls (Chap. 5) and walls with interior reinforcement (Chap. 6).

\[ F_R = 0.4 \]  unconfined masonry without reinforcement (Chap. 7).

#### 3.1.5 Contribution of reinforcement to vertical load strength

The contribution to strength under vertical load of tie-columns and bond beams (Chap. 5) or of internal reinforcement (Chap. 6) shall be considered according to sections 5.3.1 and 6.3.1.

#### 3.1.6 Hypothesis for obtaining design flexural strength

The determination of strength of any form of section subjected to flexure, axial load or a combination of both, shall be made following the criteria specified for reinforced concrete, and based on the following hypothesis:

a) Masonry behaves like a homogeneous material.

b) The distribution of longitudinal strains in any cross section of an element is plane.
c) Tensile strengths are resisted only by the reinforcing steel.

d) There is a perfect bond between the vertical reinforcing steel and surrounding grout.

e) Section fails when, in masonry, a maximum compressive strain, taken equal to 0.003 is attained.

f) Unless tests in prism allow a better determination of the stress-strain curve for masonry, a linear until failure shall be assumed.

In walls with hollow masonry units where not all cells are filled with grout, the value of $f_{m}^*$ of hollow masonry units without grout in the compression zone shall be considered.

Walls subjected to out-of-plane flexural moments, may be confined o internally reinforced. In the latter case, the bending strength may be determined taking into account the vertical reinforcement of the wall, when its spacing does not exceeds six times the masonry wall thickness.

3.1.7 Strength of masonry to lateral loads

The shear force resisted by masonry, according to systems described in Chapters 4 through 8, is based on the design shear strength stress which, in these Standards, is taken equal to diagonal compressive strength, $v_{m}^*$.

3.1.8 Factor of seismic behavior

For seismic design, the seismic behavior factor $Q$, indicated in the Complementary Technical Norms for Seismic Design and these Standards shall be used. The seismic behavior factor depends on the type of masonry unit used in the walls (2.1.1), the reinforcing scheme (Chapters 5 through 8), as well as the structural layout of the building.

When combination of structural systems is used, i.e. concrete or steel at the ground story frames and load-bearing walls (as in the case of low rise buildings with frames supporting masonry walls), the smallest seismic behavior factor in each direction of analysis shall be used. In addition, the requirements indicated in the Complementary Technical Norms for Seismic Design shall be satisfied.

3.1.9 Design of foundations

Foundation of masonry structures shall be dimensioned and detailed according to the requirements specified in the Title Six of the Code, the Complementary Technical Norms on Criteria and Actions for Structural Design of Buildings, the Complementary Technical Norms for Design and Construction of Foundations, the Complementary Technical Norms for Design and Construction of Concrete Structures, and in section 8.4 of these Standards, as pertinent.

Foundation elements shall be designed to resist the design pres and moment and soil reactions, without exceeding the soil strength. Maximum allowable settlements shall be revised.

Vertical reinforcement of walls and other elements shall extend into footings, whether of concrete or masonry, or into the foundation slab, and shall be anchored so that the specified tension yield stress may be developed. Anchorage shall be checked according to section 5.1 of the Complementary Technical Norms for Design and Construction of Concrete Structures. The vertical reinforcement shall be terminated with at 90-degrees-hooks near the bottom of foundation, with hook extension oriented towards the interior of the vertical element.

Reinforced concrete foundation slabs shall be designed as diaphragms, according to in section 6.6 of the Complementary Technical Norms for Design and Construction of Concrete Structures.

3.1.10 Design of floor and roof systems

Floor and roof systems of masonry structures shall be dimensioned and detailed according to the ultimate and servability limit states criteria, as well as durability, established in the Title Six of the Code. Also, the applicable requirements of the corresponding Complementary Technical Norms shall be applied, depending up on to the material used.

In any case, the transfer of forces and moments between walls and floor and roof systems shall not depend on friction between elements.

If it is the case, reinforcing bars of resisting elements in floors and roof shall be anchored into the walls, in such that the specified tension yield stress may be developed.

If the floor or roof systems are to transfer lateral forces in its own plane, to or between elements resisting lateral forces such as those induced by earthquakes, requirements for diaphragms shall be met, depending on the material used.

If floor and roof systems are made of panels, the requirements in NMX-C-405-ONNCCE shall be met.

If joist and vault systems are used, requirements of NMX-C-406-ONNCCE shall be met. When vaults are supported on walls parallel to joists, bearing length shall be at least 50 mm. In no case, the vaults and joists shall obstruct passage of bond beams.

3.2 Methods of analysis

3.2.1 General

Determination of internal forces and moments in walls shall be made, in general, by means of first order elastic analysis. When determining the elastic properties of walls, it shall be considered that masonry does not resist tensions in a direction perpendicular to joints and, therefore, cracked and transformed sections properties shall be used, when such tensions occur.

The modulus of elasticity of steel reinforcement and masonry, and the shear modulus of masonry, shall be taken as indicated in sections 2.7, 2.8.5 and 2.8.6, respectively. For concrete, the value supposed in section 1.5.1.4 of Complementary Technical Norms for Design and Construction of Concrete Structures shall be used.
c) Tensile strengths are resisted only by the reinforcing steel.

d) There is a perfect bond between the vertical reinforcing steel and surrounding grout.

e) Section fails when, in masonry, a maximum compressive strain, taken equal to 0.003 is attained.

f) Unless tests in prism allow a better determination of the stress-strain curve for masonry, a linear until failure shall be assumed.

In walls with hollow masonry units where not all cells are filled with grout, the value of $f_m^*$ of hollow masonry units without grout in the compression zone shall be considered.

Walls subjected to out-of-plane flexural moments, may be confined or internally reinforced. In the latter case, the bending strength may be determined taking into account the vertical reinforcement of the wall, when its spacing does not exceed six times the masonry wall thickness.

### 3.1.7 **Strength of masonry to lateral loads**

The shear force resisted by masonry, according to systems described in Chapters 4 through 8, is based on the design shear strength which, in these Standards, is taken equal to diagonal compressive strength, $v_m^*$.

### 3.1.8 **Factor of seismic behavior**

For seismic design, the seismic behavior factor $Q$, indicated in the Complementary Technical Norms for Seismic Design and these Standards shall be used. The seismic behavior factor depends on the type of masonry unit used in the walls (2.1.1), the reinforcing scheme (Chapters 5 through 8), as well as the structural layout of the building.

When combination of structural systems is used, i.e. concrete or steel at the ground story frames and load-bearing walls (as in the case of low rise buildings with frames supporting masonry walls), the smallest seismic behavior factor in each direction of analysis shall be used. In addition, the requirements indicated in the Complementary Technical Norms for Seismic Design shall be satisfied.

### 3.1.9 **Design of foundations**

Foundation of masonry structures shall be dimensioned and detailed according to the requirements specified in the Title Six of the Code, the Complementary Technical Norms on Criteria and Actions for Structural Design of Buildings, the Complementary Technical Norms for Design and Construction of Foundations, the Complementary Technical Norms for Design and Construction of Concrete Structures, and in section 8.4 of these Standards, as pertinent.

Foundation elements shall be designed to resist the design pres and moment and soil reactions, without exceeding the soil strength. Maximum allowable settlements shall be revised.

Vertical reinforcement of walls and other elements shall extend into footings, whether of concrete or masonry, or into the foundation slab, and shall be anchored so that the specified tension yield stress may be developed. Anchorage shall be checked according to section 5.1 of the Complementary Technical Norms for Design and Construction of Concrete Structures. The vertical reinforcement shall be terminated with at 90-degrees-hooks near the bottom of foundation, with hook extension oriented towards the interior of the vertical element.

Reinforced concrete foundation slabs shall be designed as diaphragms, according to in section 6.6 of the Complementary Technical Norms for Design and Construction of Concrete Structures.

#### 3.1.10 Design of floor and roof systems

Floor and roof systems of masonry structures shall be dimensioned and detailed according to the ultimate and serviceability limit states criteria, as well as durability, established in the Title Six of the Code. Also, the applicable requirements of the corresponding Complementary Technical Norms shall be applied, depending on to the material used.

In any case, the transfer of forces and moments between walls and floor and roof systems shall not depend on friction between elements.

If it is the case, reinforcing bars of resisting elements in floors and roof shall be anchored into the walls, in such that the specified tension yield stress may be developed.

If the floor or roof systems are to transfer lateral forces in its own plane, to or between elements resisting lateral forces such as those induced by earthquakes, requirements for diaphragms shall be met, depending on the material used.

If floor and roof systems are made of panels, the requirements in NMX-C-405-ONNCE shall be met.

If joist and vault systems are used, requirements of NMX-C-406-ONNCCE shall be met. When vaults are supported on walls parallel to joists, bearing length shall be at least 50 mm. In no case, the vaults and joists shall obstruct passage of bond beams.

### 3.2 **Methods of analysis**

#### 3.2.1 General

Determination of internal forces and moments in walls shall be made, in general, by means of first order elastic analysis. When determining the elastic properties of walls, it shall be considered that masonry does not resist tensions in a direction perpendicular to joints and, therefore, cracked and transformed sections properties shall be used, when such tensions occur.

The modulus of elasticity of steel reinforcement and masonry, and the shear modulus of masonry, shall be taken as indicated in sections 2.7, 2.8.5 and 2.8.6, respectively. For concrete, the value supposed in section 1.5.1.4 of Complementary Technical Norms for Design and Construction of Concrete Structures shall be used.
3.2.2 Analysis under vertical loads

3.2.2.1 Basic criteria

For analysis under vertical loads local rotations due to mortar crushing in wall and floor element joints, shall be taken into consideration. Therefore, for walls supporting monolithic or precast concrete slabs, it may be assumed that the joint has enough rotation capacity so that, for effects of moments distribution in the wall-slab connection, the out-of-plane wall flexural stiffness is zero and that walls are axially loaded only.

In the analysis interaction among soil, foundation and walls shall be considered. When long term effects are considered, the modulus of elasticity and modulus shear for restrained loads in sections 2.8.5 and 2.8.6 shall be used.

3.2.2.2 Design forces and moments

Vertical loads acting on each wall may be obtained from analyses based on tributary areas.

For design, the following flexural moments shall be considered.

a) Flexural moments that must be resisted for static conditions and that fixed to a cannot be re-distributed through joint rotation, as those in a cantilever fixed to a wall and those induced by wind or earthquake loads, perpendicular to the wall.

b) Flexural moments due to eccentricity with which the load of the floor immediately above is transferred to end walls. Such eccentricity, e, shall be taken as

\[ e = \frac{t - b}{2} \]

where t is the wall masonry thickness and b is the bearing length of slab supported by wall (fig. 3.1).

![Figure 3.1 Eccentricity of vertical load](image)

3.2.2.3 Reduction factor by eccentricity and slenderness effects

Eccentricity and slenderness effects shall be considered in design. Optionally effects may be considered the approximate values of the reduction factor \( F_E \).

a) \( F_E \) is equal to 0.7 for interior walls supporting adjacent span not differing more than 50%. \( F_E \) can be taken equal to 0.6 for end walls or with adjacent span differing more than 50%, as well as for cases where the ratio of design live loads to dead loads exceeds 1.0. For both cases, it shall be simultaneously satisfied that:

1) Strains at the top and bottom ends of a wall in a perpendicular direction to the wall plane shall be constrained by the floor system, bond beams, or by other elements.

2) Eccentricity in the applied axial load is less than or equal to \( t/6 \) and there are no significant forces acting perpendicular to the wall plane; and

3) The free height-to-thickness ratio of wall masonry, \( H/t \), does not exceed 20.

b) When conditions of 3.2.2.3.a are not satisfied, the reduction factor by eccentricity and slenderness shall be determined as the smaller of that specified in 3.2.2.3.a, and that obtained from the following equation

\[ F_E = \left(1 - \frac{2e'}{t} \right) \left(1 - \frac{kH}{30t} \right) \quad (3.2) \]

where

- \( H \) free height of a wall between elements capable of providing wall lateral support;
- \( e' \) eccentricity calculated for vertical load plus an accidental eccentricity that shall be taken equal to \( t/24 \);
- \( k \) wall effective height factor to be equal to:
  - \( k = 2 \) for walls without restraint lateral displacement in its upper edge;
  - \( k = 1 \) for end walls that support slabs; and
  - \( k = 0.8 \) on spanning on both sides of the wall.

3.2.2.4 Effect of restraints to lateral displacements

Where wall is connected to transverse walls, buttresses, columns or tie-columns (satisfying section 5.1) to restrain its lateral displacement, the factor \( F_E \) shall be computed as

\[ F_E = \left(1 - \frac{2e'}{t} \right) \left(1 - \frac{kH}{30t} \right) \left(1 - \frac{H}{L'} \right) + \frac{H}{L'} \leq 0.9 \quad (3.3) \]

where \( L' \) is the spacing between transverse elements restraining wall displacement (fig. 3.2).

3.2.3 Analysis for lateral loads

3.2.3.1 Basic criteria

For determining the internal forces and moments acting on the walls, masonry structures may be analyzed applying
dynamic or static methods (section 3.2.3.2), or by using the simplified method of analysis described in section 3.2.3.3. The effect of openings in lateral stiffness and strength shall be considered.

![Figure 3.2 Restraint to lateral displacement](image)

3.2.3.2 Methods for dynamic and static analysis

Analysis applying dynamic or static methods complying with Chapter 2 of the Complementary Technical Norms for Seismic Design shall be accepted.

Lateral load effects induced by earthquake shall be determined based on the relative stiffness of the different walls and wall segments. These shall be determined taking into account the deformations by shear and flexion. For revision of the ultimate limit state and for evaluating shear deformations, cracked properties in the cross sections of the most demanded walls or segments. When evaluating flexural strains, the cracked cross section of the wall or segment shall be considered when net vertical loads in tension occur.

Restraints on wall rotations, stiffness of floor and roof systems, as well as those of lintels and parapets shall be taken into account.

In structures designed as confined masonry or internally reinforced masonry, the walls and segments without openings may be modeled as wide columns (fig. 3.3), with moments of inertia moments and shear areas equal to those of the actual wall or segment. In long walls, as those with intermediate tie-columns, the expected behavior shall be evaluated to decide if, for analysis purposes, the wall is divided into segments, to each of which the corresponding moment inertia and shear area shall be assigned.

The wide columns shall be coupled by beams with the slab moment of inertia of corresponding from an effective width, to which the moment of inertia the lintel and parapet shall be added (fig. 3.4).

![Figure 3.3 Model of a wide column](image)

Modulus of elasticity and shear modulus of masonry, $E_m$ and $G_m$, for short term loads shall be used, (sections 2.8.5 and 2.8.6). These values shall reflect the axial and shear stiffness expected to be obtained from the masonry in the job. The values used in the analysis shall be indicated in the drawings (section 9.1).

For calculation the slab flexural stiffness, with or without parapets, consider a width four times the slab depth at each side of the girder or bond beam, or three times the slab depth when there is no girder or bond beam, or when the bond beam is included in the slab depth (fig. 3.4).

In the plane frame analysis, when calculation the flexural stiffness of wall with flanges, a width of compressive flange an each side of the web not exceeding six times the flange thickness shall be considered (fig. 3.5)

In the case of walls with openings, openings may be modeled as equivalent wide columns, only if the opening pattern is regular in elevation (fig. 3.3), in which case solid segments of the wall shall be modeled as wide columns and these shall be coupled by means of beams, as established above. If opening
distribution is irregular or complex in elevation, more refined methods for modeling such walls shall be used. Finite element method, the strut and tie method or other similar analytical procedures aiming to adequately model openings distribution in walls and its impact on stiffness, strains, and stress distribution along the length and height of walls shall be permitted.

Figure 3.5 Effective width of flange in compression of walls.

The infill walls may be modeled as equivalent diagonals or as panels joined at corners to beams and columns of the surrounding frame.

If both masonry and concrete walls are used, differences in the mechanical properties of both materials shall be considered.

Inelastic lateral drift angles, i.e. equal to the one computed through the set of reduced horizontal forces, and multiplied by the seismic behavior factor Q, shall not exceed the following values:

0.006 for infill walls.
0.0035 for load bearing walls of confined masonry made of solid masonry units with horizontal reinforcement or welded wire mesh (Chap. 5)
0.0025 in load bearing walls made of:
  a) confined masonry of solid masonry units (Chap. 5)
  b) masonry of hollow masonry units confined and horizontally reinforced (Chap. 5); or
  c) masonry of hollow masonry units confined and reinforced with mesh (Chap. 5)
0.002 in bearing walls of hollow masonry units with internal reinforcement (Chap. 6).
0.0015 in bearing masonry walls not complying with specifications for confined masonry or internally reinforced masonry (Chaps. 7 and 8)

3.2.3.3 Simplified method

It shall be permitted to consider that the shear force taken by each wall or segment is proportional to its cross area, to disregard the effects of torsion, the overturning moment, and flexibility of diaphragm, and to use the simplified method for seismic analysis specified in Chapter 7 of the Complementary Technical Norms for Seismic Design, when the requirements specified in Chapter 2 of such Norms are met. These are:

a) At each story including that supported by the foundation, at least 75% of the vertical loads are carried by continuous walls in elevation which are connected between them by means of monolithic reinforced slabs or by other floor systems sufficiently strong and stiff under shear. Such walls shall have story an approximate symmetrical distribution with respect to two orthogonal building axes. For that purpose, calculated state torsion eccentricity $e_T$ shall not exceed ten percent the dimension measured parallel to such eccentricity, $c$. Torsional eccentricity $e_T$ may be calculated as the ratio of the absolute value of the algebraic sum of the moments of the effective areas of walls with respect to the shear interstory center, and the total effective area of the walls in the direction of analysis (fig. 3.6). The effective area is the product of the gross area of the cross section of wall, $A_T$, and the factor $F_{AE}$, given by

$$F_{AE} = \begin{cases} 1; & \text{if } \frac{H}{L} \leq 133 \\ \left(133 \frac{L}{H}\right)^2; & \text{if } \frac{H}{L} > 133 \end{cases} \quad (3.4)$$

Where $H$ is the wall free height and $L$ is the effective length of the wall. In all stories, at least two parallel load bearing walls shall be placed in the perimeter, with total length at least equal to one half the story dimension in the direction of analysis (fig. 3.7)

b) The length-to-width ratio of the floor of the building does not exceed 2, unless that, for seismic analysis purposes, it may be assumed that such floor is divided in independent segments where the length-to-width ratio satisfies this restriction and those mentioned above (3.4), and each segments is independently checked for seismic strength.

c) The height-to-minimum dimension ratio at the base of building does not exceed 1.5 and the overall height of the building is not greater than 13 m.
3.2.4 Temperature analysis

Effects of temperature in deformation and forces shall be considered when differences in temperature may occur, or when the structure has a length greater than 40 m. Special attention should be given to mechanical properties of masonry when evaluating temperature effects.

3.3 Detailing of reinforcement

3.3.1 General

Construction drawings shall have figures or notes with reinforcement details (section 9.1). Every reinforcing bar shall be surrounded by mortar, or grout (fine or coarse) along the whole length, except horizontal reinforcing bars anchored according to section 3.3.6.4.

3.3.2 Sizes of reinforcing bars

3.3.2.1 Diameter of longitudinal reinforcing bars

Diameter of the longest reinforcing bar size shall not exceed half size of the smallest clear dimension of a cell. In the columns and bond beams, the diameter of the longest bar shall not exceed one sixth of the smallest dimension (fig. 3.8).

3.3.2.2 Diameter of the horizontal reinforcing bars

Diameters of the horizontal reinforcement shall not be smaller than 3.5 mm nor greater than three fourths the joint thickness (see section 9.2.2.1) (fig. 3.8).

3.3.3 Placement and spacing of longitudinal reinforcing bars

3.3.3.1 Clear space between bars

The clear space between parallel bars, bar splices, or between bars and splices shall not be smaller than the nominal diameter of the longest bar size, or 25 mm (fig. 3.8).

3.3.3.2 Bar bundles

A maximum two-bar bundles shall be accepted

3.3.3.3 Thickness of grout and reinforcement

Thickness of grout, between bars or splices and the face of the piece shall be at least 6 mm (fig. 3.8).

3.3.4 Protection of reinforcing steel

3.3.4.1 Cover in exterior tie-columns and bond beams

In confined walls with exterior tie-columns, longitudinal reinforcing bars of tie-columns and bond beams shall have a minimum concrete cover of 20 mm (fig. 3.8).

3.3.4.2 Cover in interior tie-columns and in walls with internal reinforcement

If the face of the wall is exposed to earth, cover shall be 35 mm for bars no larger than No. 5 (15.9 mm in diameter) or 50 mm for longer bars (fig. 3.8).

3.3.4.3 Cover of horizontal reinforcement

Minimum clear distance between horizontal reinforcing bar or welded wire mesh and the outside face of a wall shall be at least 10 mm or one-time the bar diameter (fig. 3.8).

3.3.5 Bends of reinforcement

3.3.5.1 In straight bars

Bars in torsion may be terminated with 90 or 180 degrees hooks. Hook extension shall not be less than 12 \( d_b \) for degree hooks, nor than 4 \( d_b \) for 180 degree hooks, where \( d_b \) is the bar diameter (fig. 3.9).

3.3.5.2 In stirrups

Stirrups shall be closed, made of one piece, and shall terminate in a corner with 135 degree hooks, followed by hook extension not smaller than 6 \( d_b \) long nor 35 mm (fig. 3.9).

3.3.5.3 In crossties

The crossties shall terminate with 180-degrees hooks, followed by hook extensions not smaller than 6 \( d_b \) long nor than 35 mm (fig. 3.9).
### 3.3.6 Bond and development

#### 3.3.6.1 General requirements

Tensile or compressive forces acting on the reinforcing steel at a critical section shall be developed on each side of the section considered, by means of bond within a sufficient length of the bar.

In general, requirements of the Complementary Technical Norms for Design and Construction of Concrete Structures shall apply.

#### 3.3.6.2 Straight bars under tension

Development length, \( L_d \), in which it is considered that a bar in tension is developed such that the specified yield stress is reached, shall be required for reinforced concrete.

#### 3.3.6.3 Bars with 90 or 180 degree hooks

Development of bars under tension shall satisfy requirements for reinforced concrete structures.

#### 3.3.6.4 Horizontal reinforcement in mortar joints

Horizontal reinforcement placed in mortar joints (5.4.3 and 6.4.3) shall be continuous along the wall, between two tie-columns if confined masonry is considered, or between two cells filled and reinforced with vertical bars in internally reinforced walls. If required, two or more bars or wires from coplanar or transverse walls may be anchored in the same tie-column or cell. Wire or horizontal reinforcing bars laps along walls shall not be permitted.

Horizontal reinforcement shall be developed in tie-columns, whether external or internal, or in grouted reinforced cells (fig. 3.10). Anchorage shall be made of 90-degree hooked bars placed inside the tie-columns or cells. The hook shall be placed vertically inside the tie-columns or grouted cell the farthest possible from the tie-column face from grouted cell face in contact with masonry.

If the design axial load, \( P_u \), acting on the wall is tensile or null, the anchorage length shall satisfy the requirements in Complementary Technical Norms for Design and Construction of Concrete Structures. For revising the

---

**Figure 3.8** Size, placement and protection of reinforcement
development length, the critical section shall be the face of the tie-column in contact with masonry or wall of the grout cell (fig. 3.10).

![Figure 3.9 Bends of reinforcement](image1)

3.3.6.5 Welded wire mesh

Welded wire meshes shall be anchored to masonry, as well as to tie-columns and beams if any, so that it can attain its specified yield stress may be reached (fig. 3.11). It may be accepted to embed the mesh in concrete; for that purpose, at least two vertical wires perpendicular to the direction of analysis shall be embedded, the shortest being at a distance no less than 50 mm from the critical section (fig. 3.11), powder-driven connectors or steel nails are used for attaching the welded wire mesh, maximum separation shall be 450 mm.

![Figure 3.10 Development of horizontal reinforcement](image2)

The mesh shall reinforced vertical edges of walls and edges of openings by surrounding the edges. If the mesh is placed on a face of the wall, that segment of the mesh surrounding the borders shall extend at least two times the transverse wire spacing. This segment edges shall be anchored so that the mesh specified yield stress is reached.

If the wire diameter of the mesh does not allow bending around, C-shape reinforcement made of mesh with wire gage not smaller than 10 (3.45 mm diameter) may be overlapped with the main mesh according to section 3.3.6.6.

It is permitted to attach the mesh directly in contact with masonry.

![Figure 3.11 Reinforcement with welded wire mesh and mortar cover](image3)

3.3.6.6 Bar splices

a) Bars under tension

Lap length of bars in concrete shall be determined according to specification for reinforced concrete. Welded connection shall not be accepted. If bars are lapped inside hollow masonry units, lap length shall be at least $50d_b$ in bars with specified yield stress up to 412 MPa (4,200 kg/cm²) and at least equal to $60d_b$ in bars and wire with larger specified yield stress; where $d_b$ is the diameter of the longest lapped bar size. Splice shall be located in the middle third of the wall height. Splices of more than 50% of the longitudinal steel of the element (tie-column, bond beam, wall) in the same section shall not be accepted.

Splices at both ends of tie-columns (whether external or internal) splices the first floor above grade along length $H_o$, defined in 5.1.1.h., shall not be accepted.
Splides in vertical reinforcement at the base of internally reinforced masonry walls along the calculated height of the flexural plastic hinge shall not be permitted.

b) Welded wire mesh

Welded wire meshes shall be continuous, without spli ces, along the wall. Mesh may be overlapped if wall height requires it. The overlap shall be placed in a zone where expected stresses in wires are low. Overlap measured between transverse end wires of the adjoining meshes shall not be less than twice the spacing between transverse wires plus 50 mm.

4. INFILL WALLS

4.1 Scope

Infill walls are those walls surrounded by beams and columns of a structural frame, to which they provide stiffness against lateral loads. Infills may be made of confined masonry (Chap. 5), internally reinforced (Chap. 6), without reinforcement (Chap. 7) or of natural stones (Chap. 8). Wall Thickness shall not be less than 100 mm.

Walls shall be constructed and inspected as indicated in chapters 9 and 10, respectively.

4.2 Design forces

In plane and out of plane design forces, shall be obtained from analysis under lateral loads including the corresponding load factor.

4.3 Strength to in-plane shear force

4.3.1 Shear force resisted by masonry

The design shear force resisted by masonry, \( V_{mr} \) shall be determined as follows:

\[
V_{mr} = F_R \left( 0.85 \nu_m * A_T \right)
\]  

(4.1)

where

\( A_T \) gross cross section of area of wall; and

\( F_R \) shall be taken equal to 0.7 (section 3.1.4.3).

4.3.2 Shear force resisted by horizontal reinforcing steel

If the infill wall is reinforced horizontally, whether with deformed bars or cold-drawn deformed wires in the mortar joints, or with welded wire mesh covered with mortar, the shear force taken by the horizontal reinforcement, \( V_{sr} \), shall be computed with equation 4.2.

\[
V_{sr} = F_R \eta \rho_h f_{yh} A_T
\]  

(4.2)

where \( \eta, \rho_h, \) and \( f_{yh} \) are the efficiency factor, the amount and the specified yield stress of the horizontal reinforcement, respectively.

The horizontal reinforcement shall be detailed as indicated in sections 3.3.2.2, 3.3.4.3, 3.3.5.1, and 3.3.6.4. The minimum and maximum amount, as well as the value for \( \eta \) shall be those indicated in Chapters 5 and 6, as it corresponds.

4.4 Overturning of infill wall

The possibility of infill wall out-of-plane overturning shall be prevented. For this, connections between frame and infill wall shall be designed and detailed, or else, the wall must be reinforced with tie-columns or internal reinforcement (fig. 4.1). The flexural strength out-of-plane shall be determined according to section 3.1.6.

4.5 In-plane frame-infill wall interaction

Each frame columns shall be designed to resist over a distance, equal to one fourth of its height measured from the horizontal side of the beam, a shear force equal to one-half the lateral load acting on the infill (fig. 4.2). The value of this load shall be at least equal to the in-plane sincerely bear strength of the infill.

If the infill wall is horizontally reinforced, to evaluate the effects on the columns, the shear force resisted by such reinforcement shall be calculated with eq. 4.2, but using an efficiency factor \( \eta = 1 \).
5. CONFINED MASONRY

5.1 Scope

Confined masonry is reinforced with tie-columns and bond beams. According to these Norms confined masonry, walls shall satisfy requirements of 5.1.1 through 5.1.4 (fig. 5.1 through 5.3). In confined masonry system, tie-columns or portions thereof are cast once the masonry or the corresponding part of it has been constructed.

For seismic design, Q = 2 shall be used when the units are solid; or when multi-perforated masonry units with horizontal reinforcement with the minimum amount and with external tie-columns. For any other case, Q = 1.5 shall be used.

Walls shall be constructed and inspected as indicated in Chapters 9 and 10, respectively.

5.1.1 External tie-columns and bond beams

Tie-columns and bond beams shall comply with the following (fig. 5.1 and 5.2)

a) Tie-columns at least at wall and at wall intersections, and at intermediate location with a spacing not greater than 1.5 H nor 4 m shall be provided. Parapets shall have tie-columns with a spacing not greater than 4 m.

b) Bond beam along the horizontal edge of the wall, unless the wall is connected to a reinforced concrete element with a minimum depth of 100 mm (fig. 5.2) shall be provided. Even in this case, longitudinal and transverse reinforcement shall be provided. In addition, bond beams within the wall spaced at not work not greater than 3 m and at the top of parapets whose height is greater than 500 mm shall be provided.

c) Tie-columns and bond beams shall have a minimum dimension equal to the masonry wall thickness t.

d) Concrete in tie-columns and bond beams shall have a compressive strength, \( f'_{cc} \), not less than 15 MPa (150 kg/cm²).

e) Longitudinal reinforcement tie-columns and bond beam shall be proportioned dimensioned to resist the corresponding vertical and horizontal components of the compressive on strut that develops in the masonry when resisting lateral and vertical loads. In any case reinforcement, shall be made of at least three bars, whose total area is at least equal to that obtained with eq. 5.1.

\[
A_s = \frac{0.2 f'_{cc} t^2}{f_y} \tag{5.1}
\]

where \( A_s \) is the total area of the longitudinal reinforcing steel in a tie-column or bond beam.

f) Longitudinal reinforcement in tie-columns or bond beams shall be developed in surrounding of the elements of the wall so that its specified yield stress developed.

g) Tie-columns and bond beams shall have transverse reinforcement made of by closed stirrups and with an area \( A_{sc} \), at least equal to the calculated with eq. 5.2

\[
A_{sc} = \frac{1000s}{f_y h_c} \tag{5.2}
\]

where \( h_c \), tie-column or bond beam dimension in the wall plane. Stirrup spacing, s, shall and exceed 1.5 t and 200 mm.

h) When the design diagonal compressive strength of masonry, \( v_m \), is greater than 0.6 MPa (6 kg/cm²), transverse reinforcement shall be provided, with an area equal to that computed with eq. 5.2 and with a spacing not less than one course within a distance \( H_o \) at each end of the tie-columns.

\( H_o \) shall be taken as the larger of H/6, 2 h_and 400 mm.

5.1.2 Walls with internal tie-columns

It is acceptable to consider walls as confined if internal tie-columns and bond beams comply with all items in 5.1.1, except 5.1.1.c. It shall be acceptable to used course grout as specified in section 2.5.3 with compressive strength not less than 12.5 MPa (125 kg/cm²). Stirrups and crossties shall be located at the ends of tie-columns as indicated in 5.1.1.h, regardless of \( v_m \). For seismic design, the seismic behavior factor Q, shall be equal to 1.5, regardless of the amount of horizontal reinforcement (section 5.4.3) or of the welded wire mesh (section 5.4.4).
5.1.3 Walls with openings

Reinforcing elements with the same characteristics as those for bond beams and columns shall be provided around the perimeter of every opening whose horizontal or vertical dimensions exceed one-fourth the wall length or one-fourth the tie-column spacing separation between columns, or 600 mm (fig. 5.3). Also, vertical and horizontal reinforcing elements shall be located in openings with a height equal to the wall height (fig. 5.1). In walls with internal tie-columns, it shall be accepted to substitute the bond beam at the lower edge of an opening with horizontal reinforcing steel bars anchored in the walls. The reinforcement shall consist of bars designed to resist a total load tension force of 29 kN (2,980 kg).

5.1.4 Thickness and height-to-thickness ratio of walls

Thickness of masonry wall, t, shall not be less than 100 mm and the free height-to-thickness ratio of masonry wall, H/t, shall not exceed 30.

5.2 Design forces and moments

The design forces and moments shall be obtained from the analysis procedures indicated in sections 3.2.2 and 3.2.3, using design loads including the corresponding load factor.

Strengths for vertical and lateral loads of a confined masonry wall shall be checked for axial load, shear force, in-plane flexural moments, and, when appropriate out-of-plane flexural moments. When designing for lateral loads, only the contribution of walls whose length is approximately parallel to the direction of analysis shall be considered.

Design for vertical loads shall be made according to section 3.2.2.

When the requirements of the simplified method for seismic analysis are applicable (section 3.2.3.3), design for lateral loads may be limited to the effects of shear force. When the building has more than three stories, in addition to the shear force, walls having a total height-to-largest length load ratio greater than 2.0 shall be checked for in-plane bending.

5.3 Walls subjected to axial and in-plane flexural moments

5.3.1 Compressive strength of confined walls

The resistant vertical load, \( P_R \), shall be computed as:

\[
P_R = F_R F_k (f_m A_t + \sum A_i f_i)
\]

where

- \( F_R \) is the factor of safety for the compressive strength of masonry.
- \( F_k \) is the factor of safety for the vertical load.
- \( f_m \) is the compressive strength of the masonry.
- \( A_t \) is the area of the tie-columns.
- \( A_i \) is the area of the reinforcing bars in the openings.
- \( f_i \) is the tensile strength of the reinforcing bars.

Figure 5.2  Columns and beams

Figure 5.3  Reinforcement in the perimeter of openings
$F_E$ is obtained according to section 3.2.2; and $F_R$ is taken equal to 0.6.

Alternatively, $P_R$ may be computed as

$$P_R = F_R F_w (f_m + 0.4) A_T; \text{ if MPa and mm}^2 \text{ are used (5.4)}$$

$$P_R = F_R F_e (f_m + 4) A_T; \text{ if kg/cm}^2 \text{ are used }$$

### 5.3.2 In-plane flexural moment strength

#### 5.3.2.1 General design method

Pure bending strength of a confined wall, either with external or internal tie-columns, subjected to axial loads and in-plane bending shall be computed based on the hypothesis of section 3.1.6. Design strength shall be obtained by reviewing the nominal strength with the strength reduction factor indicated in section 3.1.4.2.

#### 5.3.2.2 Optional method

For walls with longitudinal bars placed symmetrically in its end tie-columns, whether internal or external, the following simplified formulas (eqs. 5.5 and 5.6) give sufficiently approximate and conservative values of the design flexural moment strength.

The design flexural moment strength of the section, $M_{Rd}$, shall be computed according to equations (fig. 5.4)

$$M_R = F_R M_o + 0.3 P_u d; \text{ if } 0 \leq P_u \leq \frac{P_R}{3}$$

$$M_R = (1.5 F_R M_o + 0.15 P_R d) \left(1 - \frac{P_u}{P_R}\right); \text{ if } P_u > \frac{P_R}{3}$$

Where

- $M_o = A_s f_y d’$ pure bending strength of wall;
- $A_s$ total area of longitudinal reinforcing steel placed in each end tie-column;
- $d’$ distance between centroids of steel placed at both ends of the wall;
- $d$ distance between the centroid of tensile steel and the fiber under maximum compression;
- $P_u$ design compressive axial load, whose value shall be taken with positive sign in eqs. 5.5 and 5.6.
- $F_R$ shall be equal to 0.6, if $P_u P_R / 3$, and as 0.6 otherwise.

For tensile axial loads interpolation between strength under pure axial load in tension and $M_{Rd}$, affecting the result by $F_R = 0.8$.

### 5.4 Strength to lateral loads

#### 5.4.1 General considerations

Shear for strength shall be increased due to the effects of bond beams and tie-columns of confined walls according to section 5.1.

The strength to lateral loads shall be contributed by masonry (section 5.4.2). It shall be accepted that part of the shear force is resisted by horizontal reinforcing steel (section 5.4.3) or by welded wire mesh (section 5.4.4). When the vertical load acting on the wall is in tension, horizontal reinforcing steel or welded wire meshes shall resist the full lateral load.

When the simplified method of analysis is used (section 3.2.3.3), the shear force of walls (computed in sections 5.4.2, 5.4.3 and 5.4.4) shall be affected by the factor $F_{AE}$ defined by eq. 3.4.

The strength reduction factor, $F_{Rd}$, shall be taken equal to 0.7 (section 3.1.4.3).

#### 5.4.2 Shear force resisted by masonry

The design shear strength, $V_{mRd}$, shall be determined as follows:

$$V_{mRd} = F_{Rd} (0.5v_u + 0.3 P) \leq 1.5 F_{Rd} v_u A_T$$

where $P$ shall be taken positive in compression. In the area $A_T$ the tie-columns must be included, but without transforming the cross sectional area.

The vertical load P acting on the wall shall consider permanent actions, variable actions with its instantaneous intensity, and accidental actions leading to the least value and without multiplying by the load factor. If the vertical load P is tensile, the contribution of masonry $V_{mRd}$ may be ignored.

---

**Figure 5.4 Interaction diagram axial load–design flexural moment strength according to the optional method.**
The design diagonal compressive strength of masonry, $v_{cm}^*$, shall not exceed 0.6 MPa (6 kg/cm²), unless it is demonstrated by testing complying with section 2.8.2.1, that larger values may be attained. In addition, it shall be demonstrated that all applicable requirements of materials, analysis, design and construction are satisfied.

5.4.3 Shear force resisted by horizontal reinforcing steel

5.4.3.1 Types of reinforcing steel

It shall be permitted the use of horizontal reinforcing steel placed in the mortar joints for resisting shear force. The reinforcement shall consist of deformed bars or cold drawn deformed wire continuous along the wall.

The use of ladder-shape steel wire welded with by electrical resistance for resisting shear forces induced by earthquakes shall not be permitted.

Design specified yield stress, $f_{yh}$, shall not be larger than 600 MPa (6 000 kg/cm²).

Horizontal reinforcement shall be detailed as indicated in sections 3.3.2.2, 3.3.4.3, 3.3.5.1 and 3.3.6.4.

5.4.3.2 Spacing of horizontal reinforcing steel

Maximum spacing of horizontal reinforcement, $s_h$, shall not exceed six courses and 600 mm.

5.4.3.3 Minimum and maximum amounts of horizontal reinforcing steel

If horizontal reinforcing steel for resisting shear force is placed, the amount of horizontal reinforcing steel, $p_h$, shall not be less than $0.3 / f_{yh}$ if MPa is used ($3 / f_{yh}$, if kg/cm² is used), and the value resulting from the following expression

$$ P_h = \frac{V_{sh}}{f_{yh} A_T} $$

In no case $p_h$ shall be greater than $0.3 \frac{f_{cm}^*}{f_{yh}}$; and $1.2 / f_{yh}$ for solid masonry units, or $0.9 / f_{yh}$ for hollow masonry units if MPa is used ($12 / f_{yh}$ and $9 / f_{yh}$, respectively if kg/cm² is used).

5.4.3.4 Design of horizontal reinforcement

The shear force taken by the horizontal reinforcement, $V_{sr}$, shall be computed by

$$ V_{sr} = F_R \eta p_h f_{yh} A_T $$

The efficiency factor of horizontal reinforcement, $\eta$, shall be determined with the following criteria:

$$ \eta = \begin{cases} 0.6 & \text{if } P_h \leq 0.6 \text{ MPa (6 kg/cm²)} \\ 2 & \text{if } P_h \geq 0.9 \text{ MPa (9 kg/cm²)} \end{cases} $$

For values of $p_h f_{yh}$ between 0.6 and 0.9 MPa (6 and 9 kg/cm²), $\eta$ shall vary linearly (fig. 5.5).

5.4.4 Shear force resisted by welded wire mesh covered with mortar

5.4.4.1 Type of reinforcement and mortar

It shall be permitted the use welded wire meshes for resisting shear force. The mesh shall have, in both directions, the same reinforcement area per unit length.

The design yield stress, $f_{yh}$, shall not be greater than 500 MPa (5 000 kg/cm²).

The meshes shall be anchored and detailed as indicated in sections 3.3.4.3, 3.3.6.5, and 3.3.6.6.

The meshes shall be covered by a Type I mortar cover (table 2.2) with minimum thickness of 15 mm.

5.4.4.2 Minimum and maximum amounts of reinforcement

For calculation purposes, only the amounts of horizontal wire shall be considered. If the mesh is placed with the wires inclined, the horizontal components of wire amounts shall be considered.

For calculating the amount of reinforcement only the thickness of the masonry wall, $t$, shall be included.

The minimum and maximum amount shall be those prescribed in section 5.4.3.3.

5.4.4.3 Design of mesh

The shear force that the mesh will resist shall be obtained as indicated in section 5.4.3.4. The contribution to resistance by mortar shall not be considered.

6. INTERNALLY REINFORCED MASONRY

6.1 Scope

Internally reinforced masonry is that with walls reinforced with deformed steel bars or wires, horizontal or vertical, placed in the cells of masonry units, in ducts or in joints. The re-
inforcing steel, horizontal or vertical, shall be distributed along the height and length the wall. In order for wall to be considered as reinforced, the requirements of 6.1.1 through 6.1.9 shall be satisfied (fig. 6.1 through 6.3).

For seismic design, $Q = 1.5$ shall be used.

The walls shall be constructed and inspected as indicated in Chapters 9 and 10, respectively.

### 6.1.1 Amounts of horizontal and vertical reinforcing steel

a) The sum of the amounts of horizontal, $p_h$, and vertical, $p_v$, reinforcing steel, shall be no less than 0.002 and neither amount shall be less than 0.0007, i.e.:

$$p_h + p_v \geq 0.002$$

$$p_h \geq 0.0007; \quad p_v \geq 0.0007$$

(6.1)

where

$$p_h = \frac{A_{dh}}{s_h t}; \quad p_v = \frac{A_{dv}}{s_v t};$$

(6.2)

$A_{dh}$ area of the horizontal reinforcing steel that will be placed with a vertical spacing $s_h$ (fig. 6.1); and

$A_{dv}$ area of the vertical reinforcing steel that will be placed at a spacing $s_v$.

In eqs. 6.1 and 6.2 reinforcement of section 6.1.2.2 shall not be included.

b) When reinforcing steel with specified yield stress greater than 412 MPa (4200 kg/cm²) is used, the amounts of reinforcement calculated in 6.1.1.a may be reduced by multiplying them by $412 / f_y$, in MPa (4200 / $f_y$, in kg/cm²).

### 6.1.2 Size, placement and spacing of reinforcement

Applicable provisions of section 3.3 shall be satisfied.

#### 6.1.2.1 Vertical reinforcement

Vertical reinforcement inside the wall shall have a spacing not greater than six times its thickness, and 800 mm (fig. 6.1).

#### 6.1.2.2 Reinforcement at the ends of walls

a) A bond beam along the top edge of the wall shall be provided, unless provided the wall is connected to a reinforced concrete element with a minimum depth of 100 mm. Even in this case, longitudinal and transverse reinforcement shall be placed.

The longitudinal reinforcement of the beam shall be dimensioned for resisting the horizontal component of the compression shore that develops in the masonry for resisting lateral and vertical loads. In any case, it will be formed by at least three bars, whose total area is at least equal to that obtained by eq. 6.3.

$$A_{sh} = \frac{0.2 f_y t^2}{f_y}$$

(6.3)

The transversal reinforcement of the beam shall be formed by closed stirrups and an area $A_{sc}$, equal at least to the one computed with eq. 6.4

$$A_{sc} = \frac{10000 s}{f_y h_v}; \text{ if } \text{MPa and mm are used}$$

$$A_{sc} = \frac{1000 s}{f_y h_v}; \text{ if } \text{kg/cm² and cm are used}$$

(6.4)
where $h_c$ is the dimension of the beam in the plane of the wall. The separation of stirrups, $s$, shall not exceed 1.5 $t$ or 200 mm.

b) At least one No. 3 bar (9.5 mm in diameter) with specified yield stress of 412 MPa (4200 kg/cm²), or reinforcement with other characteristics with equivalent tensile stress, shall be placed in each of two consecutive cells, along all the end of the walls, at intersections between walls, or at every 3 m.

6.1.3 Filling grouts

For cells where vertical reinforcement is placed, grouts (fine or coarse) specified in section 2.5.3 may be used, or the same mortar used for joining masonry units, if it is Type I (section 2.5.2). The cells shall have a minimum dimension greater than 50 mm and an area no less than 3 000 mm².

6.1.4 Anchorage of horizontal and vertical reinforcement

Horizontal and vertical reinforcing bars shall comply with 3.3.6.

6.1.5 Transverse walls

When transverse walls are load-bearing and are built/wish, without overlapping of pieces, it shall be necessary to connect them by means of steel devices that assure continuity of the structure (fig. 6.2). The devices shall be able to resist 1.33 times the design shear strength of the transverse wall divided by the corresponding strength-reduction factor. In the shear strength, the shear force resisted by the masonry shall be included, and, if applies, that resisted by horizontal reinforcement.

Alternatively, the steel area of devices or connectors, $A_{st}$, placed at a spacing $s$ along the wall height, may be computed with the following expression

$$A_{st} = \frac{2.5(V_{mR} + V_{sR}) t_s}{F_R L f_y}$$

$$A_{st} = \frac{V_{mR} + V_{sR} t_s}{4F_R L f_y}$$

where $A_{st}$ is in mm² (cm²); $V_{mR}$ and $V_{sR}$, in N (kg), are the shear forces resisted by the masonry and the horizontal reinforcement, if applies; $F_R$ shall be taken equal to 0.7, $t$ and $L$ are the thickness and length of the transverse wall in mm (cm), and $f_y$ is the specified yield stress of the devices or connectors, in MPa (kg/cm²). The spacing $s$ shall not exceed 300 mm.

6.1.6 Walls with openings

There must be vertical and horizontal reinforcing elements in the perimeter of every opening whose dimension exceeds one fourth of the wall length, one fourth the distance between wall intersections or 600 mm, or else, in openings with a height equal to the height of the wall (fig. 6.3). The vertical and horizontal reinforcing elements shall be as stated in section 6.1.2.

6.1.7 Thickness and height-thickness ratio of walls

The thickness of the wall masonry, $t$, shall not be less than 100 mm and the height-thickness ratio of the wall masonry, $H/t$, shall not exceed 30.

6.1.8 Parapets

The parapets shall be internally reinforced with vertical reinforcing bars as those specified at 6.1.2.2.b. Horizontal reinforcement at the top of parapets whose height is over 500 mm high shall be provided according to section 6.1.6 (fig. 6.3).

6.1.9 Inspector

There must be a continuous inspector in the job to assure that reinforcement is placed according to construction drawings, and that cells where reinforcement is grouted are completely filled.

6.2 Design forces and moments

The design forces and moments shall be obtained from the analysis indicated in sections 3.2.2 and 3.2.3, using the design loads that include the corresponding load factor.

The resistance of vertical and lateral loads of an internally reinforced masonry wall shall be revised to check axial load, shear load, flexural moments in its plane, and when pertinent, also for flexural moments normal to its principal flexural plane. In the revision of lateral loads, only participation of walls
whose length is sensibly parallel to the direction of analysis shall be considered.

The revision of vertical loads shall be made according to established in section 3.2.2.

When the requirements of the seismic design simplified method are applicable (section 3.2.3.3), the revision of lateral loads can be limited to the effects of shear force. When the structure has more than three stories, additional to the shear force, the walls with a total height-length ratio greater than two shall be revised for flexion in the plane of the walls.

6.3 Resistance to compression and flexural compression in the plane of the wall

6.3.1 Compressive strength of masonry with interior reinforcement

The resistant vertical load, \( P_R \), shall be computed as:

\[
P_R = F_R F_e \left( f_m \ast A_T + \sum A_s f_s \right) \leq 125 F_R F_e f_m \ast A_T \tag{6.6}
\]

where

- \( F_e \) shall be obtained according to section 3.2.2; and
- \( F_R \) will be taken equal to 0.6.

Alternatively, \( P_R \) can be calculated with

\[
P_R = F_R F_e \left( f_m + 0.7 \right) A_T = 1.25 F_R F_e f_m \ast A_T \tag{6.7}
\]

if MPa and \( \text{mm}^2 \) are used

\[
\left( P_R = F_R F_e \left( f_m + 7 \right) A_T \leq 125 F_R F_e f_m \ast A_T \right)
\]

if \( \text{kg} / \text{cm}^2 \) and \( \text{cm}^2 \) are used

6.3.2 Flexural compressive strength in the plane of the wall

6.3.2.1 General method of design

The resistance to pure flexion or flexural compression in the plane of an externally or internally confined wall shall be calculated based on the hypothesis stipulated in section 3.1.6. The design resistance shall be obtained affecting the resistance by the resistance factor indicated in section 3.1.4.2.

6.3.2.2 Optional method

For walls with longitudinal bars symmetrically placed in its ends, the following simplified formulas (eqs. 6.8 and 6.9) give sufficiently approximate and conservative values of the design resistant flexural moment.

The design resistant flexural moment of the section, \( M_{R_d} \), is computed according to equations

\[
M_{R_d} = F_R M_o + 0.3 P_u d; \text{ if } 0 \leq P_u \leq \frac{P_R}{3} \tag{6.8}
\]

\[
M_{R_d} = \left( 1.5 F_R M_o + 0.15 P_R d \right) \left( 1 - \frac{P_p}{P_R} \right) \text{ if } P_p > \frac{P_R}{3} \tag{6.9}
\]

where

- \( M_o = A_p f_p d' \) resistance to pure flexion of the wall;
- \( A_s \) total area of the longitudinal reinforcing steel placed at the ends of the wall;
- \( d' \) distance between centroids of reinforcement placed at both ends of the wall;
- \( d \) distance between centroid of tensile steel and the maximum compression fiber;
- \( P_u \) design compressive axial, whose value will be taken with positive sign en eqs. 6.8 and 6.9; and
- \( F_R \) will be taken equal to 0.8, if \( P_u = \frac{P_R}{3} \) and equal to 0.6 if otherwise.

For tensile axial loads it will be valid to interpolate between axial load resistant to pure tension and the resistant flexural moment \( M_{R_o} \), affecting the result by \( F_t = 0.8 \) (see fig. 5.4).

6.4 Resistance to lateral loads

6.4.1 General considerations

The resistance to lateral loads shall be supplied by masonry (section 6.4.2). It is accepted that part of the shear force be resisted by the horizontal reinforcing steel (section 6.4.3). When the vertical load acting on the wall is tensile, it will be accepted that the horizontal reinforcing steel resists the totality of the lateral load.

When the simplified method of analysis is used (section 3.2.3.3), the resistance to shear force of the walls (calculated in sections 6.4.2 and 6.4.3) shall be affected by the factor \( F_{AE} \) defined by eq. 3.4.
The resistance factor, $F_R$, shall be taken equal to 0.7 (section 3.1.4.3).

6.4.2 Shear force resisted by masonry.

The design resistant shear force, $V_{mR}$, shall be determined with the following:

$$V_{mR} = F_R \left( 0.5 \frac{m}{h} A_T + 0.3 \frac{P}{A_T} \right) \leq 1.5 F_R \frac{m}{h} A_T$$  \hspace{1cm} (6.10)

where $P$ shall be taken positive in compression.

The vertical load $P$ acting on the wall shall consider the permanent actions, the variable actions with instantaneous intensity, and accidental actions giving the least value without multiplying by the load factor. If the vertical load $P$ is tensile, the contribution of masonry $V_{mR}$ shall be discarded; so, the totality of the shear force shall be resisted by the horizontal reinforcement.

The diagonal compressive strength of masonry for design, $m_{th}$, shall not exceed 0.6 MPa (6 kg/cm²), unless it is demonstrated by tests satisfying section 2.8.2.1, that greater values can be attained. In addition, it shall be demonstrated that all the applicable requirements of materials, analysis, design and construction are met.

6.4.3 Shear force resisted by horizontal reinforcing steel

6.4.3.1 Types of reinforcing steel

It will be permitted the use of horizontal reinforcement placed in the mortar joints to resist shear force. The horizontal reinforcement shall consist of deformed bars and cold laminated deformed wires, continuous along the wall.

The use of “ladders” for resisting shear force induced by earthquake shall not be permitted.

The design yield force, $f_{sh}$, shall not be greater than 600 MPa (6000 kg/cm²).

The horizontal reinforcement shall be detailed as indicated in sections 3.3.2.2, 3.3.4.3, 3.3.5.1 and 3.3.6.4.

6.4.3.2 Separation of horizontal reinforcing steel

The maximum separation of horizontal reinforcement, $s_{th}$, shall not exceed six courses or 600 mm.

6.4.3.3 Minimum and maximum quantity of horizontal reinforcing steel

If horizontal reinforcing steel is placed for resisting shear force, the quantity of horizontal reinforcing steel, $p_h$, shall not be less than $0.3 / f_{sh}$ if MPa is used ($3/f_{sh}$, if kg/cm² is used) nor less than the value resulting from the following expression

$$p_h = \frac{V_{mR}}{F_R f_{sh} A_T}$$  \hspace{1cm} (6.11)

In no case $p_h$ shall be greater than $0.3 f_{m}^*$; or $1.2 / f_{sh}$ for solid pieces, or $0.9 / f_{sh}$ for hollow pieces if MPa is used ($12/f_{sh}$ and $9/f_{sh}$, respectively, if kg/cm² is used).

6.4.3.4 Design of horizontal reinforcement

The shear force taken by horizontal reinforcement, $V_{hr}$, shall be computed with

$$V_{hr} = F_R \frac{p_h f_{sh} A_T}{\eta}$$  \hspace{1cm} (6.12)

The efficiency factor of the horizontal reinforcement, $\eta$, shall be determined with the following criteria:

$$\eta = \begin{cases} 0.6; & \text{if } P_h f_{sh} \leq 0.6 \text{MPa} (6 \text{ kg/cm}^2) \\ 0.2; & \text{if } P_h f_{sh} \geq 0.9 \text{MPa} (9 \text{ kg/cm}^2) \end{cases}$$

For values of $p_h f_{sh}$ between 0.6 and 0.9 MPa (6 and 9 kg/cm²), $\eta$ must linearly change (see fig. 5.5).

7. UNCONFINED AND UNREINFORCED MASONRY

7.1 Scope

Unconfined and unreinforced masonry walls shall be considered those that, even having some type of internal reinforcement masonry or confinement (external or internal), do not have the necessary reinforcement to be included in the categories described in Chapters 5 and 6. The thickness of the masonry wall, $t$, shall not be less than 100 mm.

For seismic design, a seismic behavior factor $Q = 1$ shall be used.

The walls shall be constructed and inspected as indicated in Chapters 9 and 10, respectively.

7.2 Design forces and moments

The design forces and moments shall be obtained from the analysis indicated in sections 3.2.2 and 3.2.3, using the design loads that include the corresponding load factor.

The strength to vertical and lateral loads of an unconfined/unreinforced masonry wall shall be revised for the effect of axial load, shear force, in-plane flexural moments, and, when pertinent, also for out-of-plane flexural moments. In the revision of lateral loads, only the participation of walls whose length is sensibly parallel to the direction of analysis shall be considered.

The revision of vertical loads shall be made according to established in section 3.2.2.

When applicable, the requirements of the seismic design simplified method (section 3.2.3.3), the revision of lateral loads can be limited to the effects of the shear force, on condition that the structure does not exceed three stories and the total height-length of wall ratio does not exceed two. Otherwise, the effects of flexion on the plane of the wall and of shear force must be evaluated.
7.3 Reinforcement for structural integrity

With the purpose of enhancing redundancy and deformation capacity of the structure, integrity reinforcement with, the amounts and characteristic giving in section 7.3.1 through 7.3.3 shall be provided in all load bearing wall. Integrity reinforcement shall be placed in rectangular elements of reinforced concrete will minimal sectional dimension equal to 50 mm. Connections between walls and between walls and floor/roof systems that depend exclusively on gravity loads shall not be accepted.

Integrity reinforcement shall be to resist the horizontal and vertical components of a diagonal strut in compression in the masonry that is associated to the failure of masonry.

Optionally, in sections 7.3.1 through 7.3.3 may be satisfied.

7.3.1 Vertical reinforcement

Walls shall be reinforced at its ends, at wall intersections and at every 4 m with at least two bars or wires of reinforcing steel, continuous along the structure height. The total area of vertical reinforcement in the wall shall be calculated with the following expression (see fig. 7.1).

\[ A_s = \frac{2V_{nR}}{3F_R f_y} \]  

(7.1)

where \( V_{nR} \) and \( F_R \) will be taken from section 7.5.

The bars shall be adequately anchored for developing its specified yield stress, \( f_y \).

7.3.2 Horizontal reinforcement

At least two bars or wires of continuous reinforcing steel along the length of the walls placed at the joint between walls and floor or roof systems. The total area shall be calculated with eq. 7.1, multiplying the result by the free height of the wall, \( H \), and dividing it by the spacing of the vertical reinforcement, \( s_v \).

\[ A_s = \frac{2V_{nR}H}{3F_R f_y s_v} \]  

(7.2)

7.3.3 Transverse reinforcement

Transverse reinforcement shall be made of stirrups or cross-ties (fig. 7.1) with a maximum spacing of 200 mm and with a diameter of at least 3.4 mm.

7.4 Wall strength under axial load and in-plane flexural moment

7.4.1 Axial compressive strength

Strength to vertical load \( P_R \) shall be calculated as:

\[ P_R = F_R F_E f_m^* A_T \]  

(7.3)

where

\( F_R \) shall be obtained according to section 3.2.2; and

\( F_E \) shall be taken equal to 0.3.

7.4.2 Strength under and load and bending

In-plane strength shall be calculated, for walls without reinforcement, according to strength of material theory, assuming a linear distribution of stresses in the masonry. It shall be considered that masonry does not resist tensions and that the failure occurs when a compressive stress equal to \( f_m^* \) occurs at at the critical section. \( F_R \) shall be taken according to section 3.1.4.2.
7.5 Strength to lateral loads

When the simplified method of analysis is used (section 3.2.3.3), shear strength of walls shall be affected by the factor $F_{AE}$ defined by eq. 3.4.

The design shear strength of walls, $V_{mR}$, shall be determined as follows:

$$V_{mR} = F_R \left( 0.5 v_m^* A_T + 0.3 P \right) \leq 1.5 F_R v_m^* A_T \quad (7.4)$$

where

- $F_R$ shall be equal to 0.4 (section 3.1.4.3); and
- $P$ shall be taken as positive when in compression.

The vertical load $P$ acting on the wall shall consider permanent actions, variable actions with instantaneous intensity, and accidental actions leading to the smallest value and without multiplying it by the load factor. If the vertical load is tensile, $V_{mR} = 0$.

8. MASONRY OF NATURAL STONES

8.1 Scope

This section refers to the design and construction of foundations, retaining walls and other masonry structural elements known as of the third class, i.e. Made of untooled natural stones joined with cement.

8.2 Materials

8.2.1 Stones

Stones used in structural elements shall satisfy the following requirements:

a) The minimum compressive strength in the direction perpendicular to formation plane shall be 15 MPa (150 kg/cm²);

b) The minimum compressive strength in the direction parallel to the planes of formation shall be 10 MPa (100 kg/cm²);

c) The maximum absorption shall be 4%; and

d) The resistance to weathering, measured as the maximum weight loss after five cycles in a saturated sodium sulfate solution, shall be 10%.

The above properties shall be determined according to procedures indicated in Chapter CXVII of the General Construction Specifications of the Ministry of Public Works (1971).

Stones need not be tooled, but the use of rounded stones with rolled sides shall be avoided, as possible. At least, 70% of the element volume shall be constituted by stones with a minimum weight of 300 N (30 kg) each.

8.2.2 Mortars

Mortars used in natural stone masonry shall be at least of the Type III (table 2.2), such that the minimum compressive strength is 4 MPa (40 kg/cm²).

Strength shall be determined according to NMX-C-061-ONNCCE.

8.3 Design

8.4.3.1 Design stresses

Design stresses in compression, $f_m^*$, and shear, $v_m^*$, shall be taken as follows:

a) Masonry jointed with mortar of compressive strength not less than 5 MPa (50 kg/cm²).

$$F_R f_m^* = 2 \text{ MPa (20 kg/cm²)}$$

$$F_R v_m^* = 0.06 \text{ MPa (0.6 kg/cm²)}$$

b) Masonry jointed with mortar with compressive strength less than 5 MPa (50 kg/cm²)

$$F_R f_m^* = 1.5 \text{ MPa (15 kg/cm²)}$$

$$F_R v_m^* = 0.04 \text{ MPa (0.4 kg/cm²)}$$

Design stresses above already include a strength reduction factor, $F_R$, and therefore, shall not be considered again in the formulas for strength calculation.

8.3.2 Determination of strength

It shall be verified that, at each section, the design normal force does not exceed the design strength given by the expression

$$P_R = F_R f_m^* A_T \left( 1 - \frac{2e}{t} \right) \quad (8.1)$$

where $t$ is the thickness of the section and $e$ is the eccentricity with which the load acts and includes lateral pressure effects, if any. The above expression is valid when the height -to-average thickness ratio of the masonry element does not exceed five; when such ratio is between five and ten, the resistance shall be multiplied by 0.80; when the ratio exceeds 10, the effects slenderness shall be considered as indicated in section 3.2.2 for fabricated stone masonry.

The acting shear force shall not exceed the design stress strength multiplied by the cross sectional area of the most unfavorable section for shear strength, according to section 8.3.1.

8.4 Foundations

In natural stone foundations the slope of the inclined faces (batter) measured from the edge of the girder or wall, shall not be less than 1.5 (vertical) : 1 (horizontal) (fig. 8.1)

In masonry foundations of trapezoidal shapes with one vertical side and an inclined side, such as edge foundations, the stability of foundation under torsion shall be verified. If
these verifications are not made, perpendicular foundations at spacing not greater than those shown in table 8.1 shall be provided.

In table 8.1, the maximum allowable spacing refers to a distance between the axis of the perpendicular foundations, minus the average of the mean widths of the perpendicular foundations.

Table 8.1 Maximum spacing of perpendicular foundations to foundations where torsion stability is not revised.

<table>
<thead>
<tr>
<th>Soil contact pressure, kPa (kg/m²)</th>
<th>Maximum spacing, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 20 (2000)</td>
<td>10.0</td>
</tr>
<tr>
<td>over 20 (2000) up to 25 (2500)</td>
<td>9.0</td>
</tr>
<tr>
<td>over 25 (2500) up to 30 (3000)</td>
<td>7.5</td>
</tr>
<tr>
<td>over 30 (3000) up to 40 (4000)</td>
<td>6.0</td>
</tr>
<tr>
<td>over 40 (4000) up to 50 (5000)</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Reinforced concrete bond beams shall be built at all foundations subjected to overturning moments and the perpendicular foundations. The columns must beanchored into the foundations a distance not smaller than 400 mm.

The loss of resisting area due to intersection of foundations shall be considered in the design.

8.5 Retaining walls

In the design of retaining walls, the most unfavorable combination of lateral and vertical loads due to earth pressure self-weight of wall, other dead loads, and live loads leading to a lower safety factor against overturning or sliding, shall be considered.


9. CONSTRUCTION

Construction of masonry structures shall satisfy the conditions specified in title seventh of the code and the requirements in this Chapter.

9.1 Construction drawings

In addition to the requirements in the Code, construction drawings shall indicated show, at least:

a) The type, external and internal dimensions (if applies) and tolerances, design compressive strength, absorption, as well as the maximum and minimum unit weight of the masonry unit. If applicable, the name and brand of the masonry unit.

b) The type of cementitious materials to be used.

c) Characteristics and size of the aggregates

d) Proportioning and design compressive strength of mortar for joining masonry unit. The proportioning shall be expressed in volume, and must be stated so in the drawings. If applies, the retention, fluidity, and consumption of mortar shall be included.

e) Mixing and remixing indications

f) If applies, proportioning, compressive strength and slump of fine and course grouts. Proportioning must be expressed in volume. If admixtures are used such, as superplasticizers as the type and its proportioning shall be shown.

g) Type, diameter and grade of reinforcing steel bars.

h) Design compressive and diagonal compressive strength of masonry.

i) If applies, or if the structure was analyzed under lateral loads using of static or dynamic methods (section 3.2.3.2), the design modulus of elasticity and design shear modulus of masonry.

j) Reinforcement details by means of drawings and/or notes, including placement, anchorage, splices, bends.

k) Details of wall intersections and anchorages of facade elements.

l) Construction tolerances.

m) If applies, the type and frequency of sampling in mortars and masonry, as indicated in section 10.2.2.

9.2 Construction of fabricated stone masonry

9.2.1 Materials

9.2.1.1 Masonry units

Design expressions and procedures specified in these Standards are applicable for walls constructed with only one type masonry unit. If units, made of clay, concrete or natural stones are combined, the behavior of walls shall be obtained from testing on natural scale specimens.
The following requirements shall be met:

a) Condition of masonry unit. Masonry units shall be clean, without cracking.

b) Moisture of masonry units. All clay units shall be saturated at least 2 hours before placing. Concrete units shall be dry at the time of placement. A slight sprinkling of the surfaces on which the mortar will be applied is acceptable.

c) Orientation of hollow masonry units. Hollow pieces shall be placed so that their cells and perforations are orthogonal to the bearing side (2.1.1.2).

9.2.1.2 Mortars

The following shall be satisfied:

a) Mortar mixing. Dry mixing of solids, reaching a homogeneous color of the mix is acceptable but shall be used only within the first 24 hours after mixing. Materials shall be mixed in a non absorbent container, preferably by mechanical mixing means. Mixing time, once water is added, shall not be less than 4 minutes, nor the time necessary to reach 120 revolutions. Mortar consistency shall be adjusted trying to attain the minimum fluidity compatible with an easy placement.

b) Retempering. If mortar begins to harden, it can be retempered until it recovers the desired consistency adding some water if necessary. Only one retempering is acceptable.

c) Mortars based on ordinary portland cement shall be used in a term of 2.5 h counting from the time of initial mixing.

d) Slump of fine and course grout. Grouts shall be proportioned so that slump indicated in the construction drawing is reached. Slump and tolerance of section 2.5.3 shall be satisfied.

9.2.1.3 Concretes

Concretes for reinforcing elements, internal or external to the wall, shall have the necessary amount of water to assure a workable consistency without segregation of the constituent materials. The use of admixtures to enhance workability is accepted.

9.2.2 Construction procedures

9.2.2.1 Mortar joints

Joint mortar shall totally cover the horizontal and vertical sides of the masonry unit. Joint thickness shall be the minimum to get a uniform layer of mortar and good alignment of the units. If industrialized units are used, the thickness of the horizontal joints shall not exceed 12 mm if horizontal reinforcement is placed in the joints, and 10 mm without horizontal reinforcement. If hand made units are used, the joint thickness shall not exceed 15 mm. Minimum thickness shall be 6 mm.

9.2.2.2 Placement

The vertical interface of masonry with external tie-columns shall be detailed to transfer shear forces. It shall be accepted to indent the masonry or else, to place steel connectors or horizontal reinforcement. Casting of tie-columns shall be made once the masonry wall or the corresponding part has been constructed.

Design expresions and procedures specified in these Standards are applicable only if running bond is used (fig. 9.1); for other types, wall behavior shall be obtained from testing on full scale specimens.

9.2.2.3 Fine and course grouts

Cells shall be free from extraneous materials and joint mortar. In columns and inner voids grouts shall fill completely the cells. Grout compaction, without excessively vibrating the reinforcement, shall be accepted. Filling of internal vertical cavities shall be made in heights no greater than:

a) 500 mm, if the cell area is up to 8 000 mm²; or

b) 1.5 m, if the cell area is greater than 8 000 mm².

If for construction reasons wall erectum is interrupted, grout shall not be filled up to one half the height of the unit in the last course placed (fig. 9.1).

It necessary to totally fill the perforations of the multi-perforated units. In walls with hollow and multi-perforated units, only the cells former shall be filled (fig. 9.1).

Bending of reinforcement shall not be permitted once grout placement has begun.

Figure 9.1 Grouthing of masonry units
9.2.2.4 Reinforcement

Reinforcement shall be tightly secured forecasting. Cover, spacing, and splice length, as well as the horizontal reinforcement placed in the joints shall be as specified in section 3.3. Splices of reinforcing bars placed in horizontal joints, overlapping of welded wire mesh in a vertical section of the wall, or splicing vertical reinforcement of walls internally reinforced at the calculated height of the flexural plastic hinge, shall not be permitted.

9.2.2.5 Piping and ducts

Piping and ducts shall be installed without damaging the masonry. If solid or hollow units with total growth are used grooves in the wall to shall be permitted embed piping and ducts, but the following shall need be satisfied:

a) Groove depth shall not exceed one fourth the thickness of the masonry wall (t/4);

b) Groove shall be vertical; and

c) Groove shall not be longer than one half the free height of the wall (H/2).

If hollow units are used, pipes or ducts shall not be placed cells with reinforcement. Cells with pipes and ducts shall be grouted.

It shall not be permitted to place piping and ducts in tie-columns a structural function, whether external or internal or in cells with vertical reinforcement as indicated in Chapters 5 and 6, respectively.

9.2.2.6 Construction of walls

During construction, in addition to the requirements of the sections above, the following shall be met:

a) Thickness of a structural wall, other than facade walls shall not be less than 100 mm.

b) All walls touching or intersecting themselves shall be anchored or connected to each other (sections 5.1.1, 6.1.2.2, 6.1.5 and 7.3.1), unless measures to assure stability and good performance are taken.

c) Surfaces of construction joints shall be clean and rough. They shall be moistened in case clay units are used.

d) Facade walls made of natural or fabricated stone materials shall be adequately connected and anchored to the back wall.

e) During construction, all necessary caution to assure wall stability in the job, shall be taken, possible horizontal pressure and loads, including wind and earthquake shall be considered.

f) For walls reinforced with welded wire mesh and mortar covering, the surface shall be saturated and free of materials that may affect mortar bond to masonry.

9.2.2.7 Tolerances

a) Axis of structural walls shall not deviate more than 20 mm from that indicated in the construction drawings.

b) The deviation from the vertical of a wall shall not be greater than 0.004 times its height nor 15 mm.

9.3 Construction of masonry with natural stones

9.3.1 Natural stones

Natural stones shall be clean and without cracks. Stones in the form of flagstone shall not be used. Stones shall be moistened before using them.

9.3.2 Mortar

Mortar shall be mixed with the minimum amount of water necessary to obtain a workable paste. For mixing and retempering, the requirements of section 9.2.1.2 shall be satisfied.

9.3.3 Construction procedure

Masonry shall be placed over a flat mortar or concrete slab. In the first courses, stones with the larger sizes shall be placed, and the stones with the best sides shall be employed for aesthetics. When stones are sedimentary strata shall be placed perpendicular to the direction of the compressions force. Stones shall be moistened before placement and shall be placed to fit in the best manner possible the voids felt by other stones. Voids shall be completely filled with small stones and mortar. Stones perpendicular to face shall be used, occupying at least one fifth of the area of the surface and shall be distributed in a regular form. Defined planes of failure transverse to the element shall not exist. All other applicable requirements of section 9.2.2.6 shall be satisfied.

9.4 Construction of foundations

Foundations shall be constructed following Chapter 7 of the Complementary Technical Norms for Design and Construction of Foundations. If the foundation is made of concrete, Chapter 14 of the Complementary Technical Norms for Design and Construction of Concrete Structures shall be satisfied. If the foundation is made of natural stone masonry, section 9.3.3 of these Standards shall be followed.

10. INSPECTION AND QUALITY CONTROL OF THE WORK

10.1 Inspection

The Director Responsible of works shall check that construction provisions of Chapters 9 and 10 be full filled.
10.1.1 Before construction of masonry walls

It shall be verified that foundations have been constructed with the tolerances indicated in the Complementary Technical Norms for Design and Construction of Concrete Structures, if the foundation is made of concrete, or in section 8.4 of these Standards, if the foundation is made of masonry.

It shall be checked that longitudinal reinforcement of tie-columns, or vertical bars of walls, are anchored and in the position shown in the structural drawings. Provisions in 3.3.6.6.a shall be fulfilled.

10.1.2 During construction

It shall be revised, specially, that:

a) Masonry units are of the type and have the quality specified in the construction drawings.

b) Masonry units made of clay are submerged in water at least 2 h before placement.

c) Concrete masonry units are dry and are sprinkled with water immediately before placement.

d) Masonry units are free from dust, grease, oil or any other substance or element that reduces bond or makes difficult its placement.

e) Reinforcing bars are of the type, diameter, and grade indicated in the construction drawings.

f) Masonry is built with running bond.

g) Vertical edges of externally confined walls are indented or have connectors or horizontal reinforcement.

h) Longitudinal reinforcement of tie-columns or within the masonry wall is free from dust, grease, or any other substance that reduces bond, and that its design position is secured during casting.

i) No more than 50% of the longitudinal reinforcement in tie-columns, bond beams, and vertical reinforcement in the same section is spliced.

j) Horizontal reinforcement is continuous along wall, without splicing, and anchored at the ends with 90-degree hooks placed in the wall plane.

k) Mortar is not mixed in contact with soil or without control of the proportioning.

l) Grout vertical cavities in hollow masonry units up to four cells in the maximum height specified in the drawings.

m) Vertical and horizontal joints are totally filled with mortar.

n) If multi-perforated bricks are used, mortar shall penetrate in perforations the distance indicated in the drawings, but not less than 10 mm.

o) Joint thickness does not exceed the value indicated in the construction drawings.

p) Dat-of-plumbness of the wall shall not exceed 0.004H and 15 mm.

q) In internal tie-columns, grout has completely filled the cavities, leaving no voids.

r) Walls made of multi-perforated bricks and hollow masonry units (the latter for electric fixtures or internal tie-columns), the hollow masonry unit fully grouted.

s) In walls reinforced with welded wire mesh, mesh connectors are firmly attached to masonry and concrete, with the spacing indicated in the construction drawings.

t) Transverse load-bearing walls built flush to orthogonal walls are connected.

u) Wall openings, if so indicated in the drawings, are reinforced or confined at the edges.

v) Parapets have tie-columns and bond beams or internal reinforcement.

10.2 Quality Control of the work

10.2.1 Scope

Quality control provisions of the work are applicable to each building and every contractor participating in the work. The following cases are exempted:

a) Buildings simultaneously satisfying the condition of having a magnitude (built surface) no greater than 250 m², no more than two stories, including parking lot, and if belong to any of the following types: one family housing, services, industry, infrastructure, agricultural, cattle, or forest.

b) Buildings for multi-family housing with no more than ten families in a plot of land lot, including those existing, and no more than two stories, including parking lot. In addition, each house shall not have a magnitude (built surface) greater than 250 m².

10.2.2 Sampling and testing

10.2.2.1 Mortar for joining masonry units.

A minimum of six samples shall be taken for each lot of 3000 m² or fraction of the constructed wall. In cases of buildings not being part of housing units, at least two samples shall be of the first floor (ground story) in buildings of up to three stories, and of the first and second floors in buildings of larger number of stories.

Samples shall be taken during construction of the indicated lot. Each sample shall consist of three cubic test specimens. Fabrication, curing, testing, and strength determination of the test specimens shall be made according to NMX-C-061-ONNCCE. Samples shall be tested at 28 days. Testing shall be made in laboratories accredited by the accreditation entity recognized in the terms of the Federal Law on Metrology and Standardization.

10.2.2.2 Grout
At least three samples shall be taken for each lot of 3,000 m² or fraction of the constructed wall. In case of buildings not bearing part of housing units, at least one sample shall be of the first floor in buildings of up to three stories, and of the first and second floors in buildings larger number of stories.

Samples shall be taken during construction of the indicated lot. Each sample shall consist of three cube test specimens in the case of lime grout, and of three cylinders in the case of course grout. Fabrication, curing, testing, and strength determination of the specimens test lime grout shall be made according to NMX-C-061-ONNCCE standard. The fabrication, curing, and testing of the course growth cylinders shall be made according to NMX-C-160 and NMX-C-083-ONNCCE. Samples shall be tested at 28 days. Testing shall be made in laboratories accredited by the accreditation entity recognized in terms of the Federal Law on Metrology and Standardization.

10.2.2.3 Masonry

At least three samples shall be taken from each lot of 3,000 m² or fraction of the wall constructed with each type of masonry unit. In the case of buildings not forming part of housing units, at least one sample shall be from the first floor in buildings of up to three stories, and from the first and second floors if the building has larger number of stories. Samples shall be taken during construction of the indicated lot. Test specimen shall be made with the materials, mortar and masonry units, used in the construction of the lot. Each sample shall consist of a prism and a wall specimen. It is accepted to fabricate the test specimen in the laboratory using the masonry units, dry mix of the mortar, and the amount of water used in the construction of the lot. Fabrication, curing, transportation, and strength determination of the test specimens shall be made according to the corresponding Mexican Standards. Samples shall be tested at 28 days. Tests shall be made in laboratories accredited by the recognized accreditation entity in terms of the Federal Law in Metrology and Standardization.

10.2.2.4 Penetration of mortar in multi-perforated pieces

The application of any of the following procedures shall be accepted:

a) Mortar penetration. Mortar penetration shall be determined by removal of a multi-perforated masonry unit from a wall at the first floor if the building has up to three stories, or from the first and second floors if the building has larger number of stories.

b) Mortar consumption. Consumption of mortar that penetrates the perforations of units, in addition to that used in horizontal and vertical joints in all the walls from the first floor, shall be controlled if the building has up to three stories, or from the first and second floors if the building has larger number of stories.

10.2.3 Acceptance criteria

10.2.3.1 Mortar and masonry

The acceptance criterion are based on the fact that design strength, specified in the construction drawings, correspond to at least 98% of the test specimens. In other words, the following shall be satisfied

$$z^* \geq \frac{\bar{z}}{1 + 2.5 c_2}$$

where

- $z^*$ design strength of interest ($f_{ij}^*$ of mortar or of grout, $f_m^*$ and $v_m^*$ of masonry)
- $z$ mean strength of samples obtained according to section 10.2.2;
- $c_2$ variation coefficient of the strength of interest from samples, which in no case shall be smaller than 0.20 for the compressive strength of mortar or growth and than that indicated in sections 2.8.1.1 and 2.8.2.1 for prims and wall specimens, respectively.

10.2.3.2 Penetration of mortar in multi-perforated masonry units

If 10.2.2.4.a is selected, the mean penetration of mortar, both at the top and bottom of the joint of the piece, shall be 10 mm, unless the construction drawings specify other minimum values.

It shall be accepted that, applying 10.2.2.4.b, mortar consumption may vary between 0.8 and 1.2 times the consumption indicated in the construction drawings.

10.3 Inspection and quality control of the work in buildings under rehabilitation

Sections 10.1 and 10.2 shall be satisfied. In addition. Characteristics of materials used in the rehabilitation, including those from commercial products that specify those characteristics at the time of purchase shall be endorsed sampling and laboratory testing through.

The correct application of project solutions, as well as the load-carrying and formation capacities, of elements or components, such as connectors shall be verified.

Measurement of dynamic characteristics of a structure provides useful information for judging the effectiveness of rehabilitation, when this includes strengthening the addition or removal of structural elements.

11. EVALUATION AND REHABILITATION

11.1 Scope

These provisions are complementary to the Title Six of the Code.
11.2 Evaluation

11.2.1 Need for evaluation

Structural safety of a building shall be evaluated when some indication exists that structure has suffered some damage, when some servicability or durability problems exist, if it will undergo some modification, it will change type of occupancy, or else, when it is required to verify the level of safety established in the Title Six of the Code.

11.2.2 Process of evaluation

The process of evaluation shall include:

a) Investigation and gathering of documents about the structure, including damage caused by earthquake or other actions.

b) If applicable, classification of damage for each structural element of the building (structural and non structural) according to damage intensity and mode of behavior.

c) If applies, study of effects damage in structural elements on future building performance.

d) Determination of the need for rehabilitation.

11.2.3 Investigation and gathering of documents about of the building and of actions that caused the damage

11.2.3.1 Basic information

Basic information of the building and of the actions that caused damage shall be gathered; in particular, it shall be necessary to:

a) Compile design calculations, specifications, architectural and structural drawings, as well as reports and opinions available.

b) Inspect the building, as well as to determine the age and quality of construction.

c) Study regulations and construction standards valid at the time of design and construction of the building.

d) Determine material and soil properties.

e) Define type and intensity of damage.

f) Meet owners, tenants, as well as original contractors and designers.

g) Obtain information on the actions that caused the damage, such as magnitude, duration, direction, response spectrum or other relevant aspects.

At least, one on-site inspection shall be made conducted aimed to identifying the structural system, configuration and condition. If necessary, cover and other elements obstructing visual inspection shall be removed.

11.2.3.2 Determination of material properties

The Material properties may be obtained through non destructive or destructive procedures, with care that if the latter procedure is used, it shall not reduce the capacity of structural elements. When foundation is damaged or when foundation modifications one need to changes in the structure in the subsoil characteristics shall be verified by means of a geotechnical study.

11.2.4 Damage classification in building elements

11.2.4.1 Mode of behavior

Depending upon the mode of behavior of the structural and non-structural element, the type and intensity of damage shall be classified. The mode of behavior is defined by the type of prevalent damage in the element. The mode of behavior will depend on the relative strength of the element to the different forces and moments that act on it.

11.2.4.2 Damage

Damage intensity in structural elements may be classified in five levels:

a) Insignificant, not affecting in a relevant manner structural capacity (strength and deformation). The repair shall be cosmetic.

b) Light, when the structural capacity is lightly affected. Minor repair measures are required for most part of elements and modes of behavior.

c) Moderate, when structural capacity is moderately affects. Rehabilitation of damaged elements depends on the type of element and mode of behavior.

d) Severe, when damage significantly affects structural capacity. Rehabilitation implies an ample intervention, with replacement or strengthening of some elements.

e) Very severe, when damage has deteriorated the structure to the point it is no longer reliable. It includes total or partial collapse. Rehabilitation involves replacement or strengthening of most of the elements, or even total or partial demolition.

11.2.5 Impact evaluation of damaged elements in building performance

11.2.5.1 Damage impact

The effect of cracks or other evidence damage shall be assessed for the future performance of the building, as a function of possible modes of behavior of damaged elements, whether structural or non structural.

11.2.5.2 Building without structural damage

If the building does not present any structural damage, the different possible modes of behavior of the elements, and
their effect on the future performance of the building shall be studied.

11.2.5.3 Residual capacity

To evaluate structural safety of a building, the reduced capacity of each element for each possible or prevalent mode of behavior shall be determined. Such capacity shall be defined for when the element, from structure or foundation, achieve a first limit state either ultimate or at serviceability failure or service, depending on the type of check being conducted.

11.2.5.4 Calculation of structural capacity

For calculating the structural capacity, conventional methods of elastic analysis, as well as the applicable requirements and equations from these Norms and from other Complementary Technical Norms, may be used. If after and on-site inspection no structural damage is observed, it may be assumed that the original capacity of the structural element remains the same. In buildings with structural damage, the contribution of damaged elements, by reducing their individual capacity according to type and intensity of damage, shall be considered. In inclined buildings the effect of out-of-plumbness shall be included in the analysis.

11.2.5. Considerations for evaluating structural safety

For evaluating the structural safety of a building, its deformation capacity, defects and irregularities in the structure and foundation, related to its location, interaction with neighboring structures, quality of maintenance and the type of occupancy are some of the factors that shall be considered.

11.2.6 Determination of the need for rehabilitation

11.2.6.1 Minor damage

If as a result of the evaluation process of the structural safety it may be concluded that building satisfies the current norms and only exhibits insignificant or light structural damages, a rehabilitation project that considers restoration or repair of such elements shall be made.

11.2.6.2 Major damage

When current regulations are not satisfied, when structural damage is of moderate or higher intensity, or situations that endanger structural stability are detected, a rehabilitation project that considers not only repair of damaged elements, but also modification of the capacity of the whole structure shall be developed. Evaluation may also recommend total or partial demolition of the structure.

11.3 Rehabilitation

11.3.1 Shoring, temporary rehabilitation and demolition

11.3.1.1 Access Control

If structural damage that could endanger its stability is detected, access to the structure shall be controlled temporary rehabilitation while the evaluation is completed, shall be carried out. In cases where damage makes imminent the partial or total collapse, putting at risk neighboring constructions or roads, it shall be necessary to urgently demolish of the structure or the zone representing a risk.

11.3.1.2 Temporary rehabilitation

When the level of damage observed in a building requires it, it shall be necessary to temporarily rehabilitate or shore, the structure by providing provisional stiffness and strength necessary for the workers’ safety while working in the building, as well as for safety of neighbors and pedestrians in adjacent zones. Temporary rehabilitation shall also be necessary when modifications to a structure are made, and specially when modifications consist of a temporary reduction in stiffness or strength capacity of some structural elements.

11.3.1.3 Safety during rehabilitation

Temporary shoring or rehabilitation works must be sufficient to assure stability of the structure. Before beginning the rehabilitation works, it shall be demonstrated that the building has the capacity for simultaneously supporting the calculated vertical actions (dead and live loads) and 30% of the accidental actions obtained from the Complementary Technical Norms for Seismic Design with the permanent actions calculated to occur during the execution of the works. To achieve such capacity, it shall be necessary, in certain cases, to temporary stiffen some parts of the structure.

11.3.2 Connection between existing elements or new materials

Connections between existing elements and new materials or new elements shall be designed and built to achieve a monolithic behavior and to assure the transfer of forces among them. It shall be permitted to use, connectors, or adhesive or anchors power-driven.

11.3.3 Repair of elements

11.3.3.1 Scope

When restoration of the original capacity of an element is required, it shall be necessary to repair or restore them. Those damaged elements that will be additionally strengthened shall be firstly repaired.

It shall be noted that success of a repair, for example grouting of cracks, depends, among other factors, on damage intensity and quality of execution. The level of restoration of the structural capacity that is feasible to reach for the behavior mode, damage intensity and quality of the construction process shall be considered in the analysis and evaluation process.

11.3.3.2 Replacement of damaged masonry units, mortar, bars and concrete

In elements with severe and very serious damage it may be necessary to replace damaged materials with new materials, previously shoring the element to be repaired. A good bond between existing and the new materials shall be provided, as well as small volumetric changes due to the shrinkage. Mate-
rials of the same type and of strength at least equal to the strength of original materials shall be used.

11.3.3.3 Repair of cracks

a) Injection of fluids

Injection of resins or fluids based on polymers or hydraulic cements may be used. Injection by curing the vacuum method shall not be permitted.

Hydraulic cement based fluids (grout) shall be proportioned to assure flow through cracks and voids, but without increasing segregation, bleeding and plastic shrinkage.

Viscosity and type of epoxy resin shall be determined depending on cracks width and masonry units absorption.

When cracks have a significant width (about 5 mm), they may be repaired with stone pieces reference to as flagstones. Flagstones shall be adequately wedged and shall be joint with Type I mortar.

In all cases, wall finishes shall be removed at least in a distance equal to 300 mm adjacent to the crack.

b) Insertion of steel fixtures

It shall be permitted to insert plates, staples, bolts, or other steel fixtures that cross the cracks. Steel elements shall be anchored to the masonry or concrete, so that they may develop the design strength. Reinforcement shall be covered with waterproofing mortar to protect against weathering. If this technique is applied to repair damage caused by earthquake, precautions shall be taken to avoid bucking of staples during cycles of displacement.

Steel bars may be inserted into perforations previously made in the masonry and shall be bonded to masonry with grout injected into the voids. Perforation shall be made with equipment adequate for not damaging the masonry. Bars may be prestressed.

c) Mesh covered with mortar

Cracks may be repaired with bands made of welded wire mesh, connected to masonry and covered with a mortar cover of a few centimeters in thickness. Bands shall be anchored to the masonry so that the design strength may be developed.

11.3.3.4 Repair of damage due to corrosion

Cracked concrete or masonry shall be removed to fully exposed corroded and sound reinforcing bars inside the affected zone. To assure bond among new materials, reinforcing bars and old concrete or masonry, bars and surfaces of existing materials shall be thoroughly cleaned. If corroded bars have lost more than 25% of cross section, bars shall be replaced or supplemental bars adequately anchored shall be placed. Newly placed concrete or masonry shall have lower permeability than the existing materials. Protection against corrosion through active or passive measures shall be considered.

11.3.4 Strengthening

11.3.4.1 General

Strengthening necessary when the load-carrying capacity or the deformation capacity be modified. Strengthening of an element often causes stiffness changes which shall be taken into account in the structural analysis. It shall be checked that strengthening of elements does not cause other-non strength elements to prematurely reach a serviceability or ultimate limit states that could lead to unfavorable and unstable behavior. Structural analysis may be carried out assuming monolithic behavior of the original element and the strengthening scheme, if the design and execution of the connections assure such type of behavior.

11.3.4.2 Jacketing of concrete and masonry elements

Concrete and masonry elements may be rehabilitated by attaching steel or plastic mesh covered with mortar or by element jacketing ferrocement and with plastic materials bonded with resins.

In the design, detailing and construction of jacketing with mortar or ferrocement, the requirements of sections 3.3.6.5, 5.4.4 and Chapter 9 shall apply.

When a structural element is strengthened through elements jacketing of plastic fibers, the surface of the element must be smoothened and covers that impair bond of plastic materials and resins shall be removed. Edges of elements shall be rounded to avoid rupture of fibers. Compatibility of resins and fibers used shall be assured. Elements directly exposed to solar radiation and strengthened with resins vulnerable to UV radiation, a protecting material shall be placed.

11.3.4.3 Addition of confining elements of reinforced concrete

Additional confining elements may be built in those buildings without tie-columns or bond beams, or when the tie-columns or bond beams do not meet the requirements indicated in sections 3.3 and 5.1. In the design, detailing, and construction of the new tie-columns and bond beams, sections 3.3, 5.1 and Chapter 9 shall be followed. Longitudinal reinforcement shall be anchored to develop its specified tensile yield stress.

11.3.4.4 Addition and removal of walls

Walls may need to be added or removed when irregularities or defects in the structure layout, need to be corrected to strengthen the building as a whole or to modify part of the structure. In the design, care shall be taken that of joint stiffness of new elements is compatible with stiffness of the original structure. Design of connections between the new elements and the original structure requires special attention. Also, load transfer to foundation shall be revised, and this could often lead to the need of modification.

If masonry infill walls are built, requirements in Chapter 4 shall apply.
11.3.5 Construction, inspection and quality control

Rehabilitation works shall satisfy provisions of Chapter 9. Inspection and quality control shall meet requirements of Chapter 10.

NORMATIVE APPENDIX A – ACCEPTANCE CRITERIA FOR SEISMIC DESIGN OF MASONRY SYSTEMS

A.1 Definitions

Drift angle
Rotation of the wall vertical axis under lateral load, with respect to a vertical plane. It may be obtained by dividing the lateral displacement applied at the slab level, and measured in the mid-length of the wall, by the story height.

Specimen
Structure tested in the laboratory that represents common array of reinforcement and boundary conditions.

Strength
Maximum load carrying capacity in one cycle or at a certain drift angle. It may be measured or calculated.

Equivalent energy dissipated
Rates of the energy dissipated of the specimen subjected to reverse cyclic lateral displacement and the ideal energy dissipation capacity. It is calculated as the area contained within the hysteresis curve for a cycle divided by the area enclosed in parallelograms defined by the stiffness of the first cycle and the maximum load at the cycle for which the equivalent energy dissipated is calculated.

Cycle stiffness
Slope of the secant joining the points of maximum drift angle in the positive and negative direction, for the same cycle.

A.2 Notation

H     unrestrained height of the wall, mm (cm)
R     calculated lateral strength of the specimen, N (kg)
R_a   approximate lateral strength of the specimen, N (kg)
R_max strength (maximum lateral load) of the specimen measured in laboratory, N (kg)
Å     lateral displacement applied at the top of the specimen and measured at mid-length of the wall, mm (cm)
λ     over-strength factor for connections
θ     Drift angle

A.3 Scope

In this appendix, acceptance criteria for construction systems based on masonry walls for their design under earthquake-induced forces are established. Acceptance is supported on experimental evidence of the performance, as well as on mathematical analysis.

Behavior of the construction system to be evaluated shall be, at least, equal to the behavior exhibited of masonry walls designed and built according to the masonry systems of these Standards, either made of solid or hollow masonry units.

Lateral load strength, lateral displacement capacity, energy dissipation capacity and lateral stiffness shall be obtained through laboratory testing of specimens.

The test specimen shall maintain its structural integrity and its vertical load carrying capacity to drift angle at least equal to 0.006 for solid masonry units and 0.004 for hollow masonry units.

A.4 Design criteria of specimens

Before testing, the design process shall involve the non-linear behavior of materials, effect of connections and reinforcement, as well as the influence of reverse cyclic loads. If for the development of the design process preliminary tests are required, these tests shall not be a part of the test acceptance of this Appendix.

Figure A.1 History of load and lateral load-distortion curve
Specimens shall be designed with the design process of the system. The calculated lateral strength $R_s$ shall be determined from the specified geometry, specified yield stresses of steel, specified strength of masonry and concrete (if applies), from a deformation compatibility analysis and using a strength-reduction factor equal to 1.0.

Specimens shall be designed so that the lateral strength associated to failure of the weakest connection is $\lambda$ times the approximate lateral strength of the specimen, $R_s$. The term connection refers, for example, to the joint between transverse or oblique walls, to the connection of the specimen with the foundation, and with the floor or roof systems, or to joint the between elements providing strength, stiffness or confinement, as in case of tie columns in the confined masonry. The minimum value of the over strength factor for connections, $\lambda$, shall be 1.3. The approximate lateral strength of the specimen, $R_s$, shall be computed using the design process of the system, from the real (measured) geometric and mechanic properties, with a strength reduction factor equal to 1.0 and if applies, the effects of strain hardening of steel.

**A.5 Test specimens**

At least one specimen shall be tested for each characteristic configuration of the reinforcement, or boundary conditions. Specimens shall be designed and constructed at a scale that allows an accurate simulation of the load transfer phenomena, particularly in connections and edges. The smallest scale allowable shall be one-half. Boundary conditions (restraint to rotate or displace) of the configuration under study shall be reproduced.

**A.6 Laboratory**

Tests shall be conducted in a well-recognized laboratory with calibrated equipment. The experimental program and data analysis shall be reviewed by the Advisory Committee on Structural Safety of the Mexico City Government.

**A.7 Testing protocol**

Specimens shall be tested under a series of deformation-controlled cycles as shown in Figure A.1. Tests shall be conducted under a constant vertical load representing the permanent actions of the Code, consistent with the expected occupancy of the construction system, as well as with the magnitude (number of stories). At each drift angle, two cycles shall be applied. The first two cycles of cycles shall be load-controlled, and shall correspond to one-fourth and to one-half of the smallest of the calculated load at inclined cracking of the wall or at yielding of vertical reinforcement. The third couple of cycles shall correspond to first inclined cracking or to yielding of the wall, whichever occurs first. Form there on, drift angle of figure A.1 shall be applied until reaching, at least, a drift angle to 0.006 if solid masonry units are used, or 0.004 if hollow masonry units are used. Alternated cyclic lateral force shall be applied in such a way that its distribution is sensibly uniform along the wall. It shall be accepted to apply the lateral force at opposite top ends of the wall, according to the semi-cycle in question.

During testing, at least one graphic record shall be kept, defining the lateral load-drift angle curve, one photo of the specimen at the end of each couple of cycles at same drift angle, and a report with the date of the test, the operator and the information of the relevant events occurred during the test, such as cracking, spalling, fractures, noises, leak of oil, etc.

**A.8 Tests report**

Report of the tests shall contain, as a minimum, the following:

A.8.1 Theory used for calculating the strength (with a strength reduction factor equal to 1.0) and the predicted value. If more than one mode of failure is expected, associated theories and strengths shall be included.

A.8.2 Details of the specimens tested (dimensions, amount and detailing of reinforcement), as well as those of construction. Clear and illustrative figures shall be included.

A.8.3 Properties of materials, both those specified in design and those measured through test specimens in the laboratory.

A.8.4 Description of the load applied fixtures, with photographs and figures.

A.8.5 Type, location, and purpose of sensors used in the instrumentation. If applies, characteristics of the data acquisition system shall be included. Photographs and figures shall be presented.

A.8.6 Graph with the displacement history applied to the specimen.

A.8.7 Description of the performance observed during the experiments, with photos of the specimen immediately after a relevant event. At least photos corresponding to first inclined cracking to the formation of a stable cracking pattern, to the drift angle associated to measured strength, to the drift angle associated to a drop of 20% of the measured strength, and at the end of the test, shall be included.

A.8.8 Graphs of the lateral load-drift angle curve

A.8.9 Graph of the equivalent energy dissipated drift angle curve

A.8.10 Date of the test, name of the laboratory, operators and authors, supervisor (Co-responsible in Structural Safety) and sponsor.

**A.9 Acceptance criteria**

Performance of the specimen shall be considered satisfactory if all of the following criteria are satisfied in both directions of the cyclic behavior:

A.9.1 Specimen shall reach a strength, $R_{\text{max}}$, equal to or larger than the calculated strength, $R_s$ for a drift angle smaller
than or equal to 0.006 for solid masonry units and 0.004 for hollow masonry units (fig. A.2)

A.9.2 Measured strength, \( R_{\text{max}} \), is smaller than \( \lambda \), \( R \) (fig. A.2), where \( \lambda \) is the over strength factor for connections described in section A.4.

A.9.3 Characteristics of the cycle repetition to a drift angle to 0.006 for solid masonry units and 0.004 for hollow masonry units shall satisfy:

a) The load at the cycle of the repetition is at least equal to 0.8 \( R_{\text{max}} \) in the same load direction (fig. A.2).

b) The equivalent energy dissipated is not smaller than 0.15 (fig. A.3).

c) The cycle stiffness at drift angle of 0.006 for solid masonry units and 0.004 for hollow masonry units is not less than 0.1 and 0.05 times respectively the cycles stiffness, calculated from the first cycle applied in the experiment (fig. A.4).

If any of the specimens does not satisfy what is indicated or the failure is in the connections, the constructive system will be considered as not complying with the acceptance criteria.