RELEVANT ASPECTS OF THE NEW MEXICO CITY'S CODE FOR THE DESIGN AND CONSTRUCTION OF MASONRY STRUCTURES

J. J. Pérez Gavilán(1), A. Pérez G.(2), L. E. Flores(3), R. Jean(4), J. Cesin(5), O. Hernández(6)

(1) Researcher, Institute of Engineering, UNAM, jjpge@pumas.iingen.unam.mx
(2)Engineering Manager, Corporación Geo, aperez.smie@gmail.com
(3) Manager, CENAPRED lfc@cenapred.unam.mx
(4) Consultant, rjean@jeaningenieros.com
(5)Consultant, javcesin@yahoo.com.mx
(6) Consultant, oscheb@seguridadestructural.com.mx

Abstract

Mexico City's code for the design and construction of masonry structures was recently revised to include new and updated provisions related to a wide range of subjects, including an important thrust towards the formal determination of the material properties. The committee decided to drop the Simplified Method for analysis of masonry buildings as it was observed that the method was also used for structures which did not satisfy the prescribed requirements related to the applicability of the method. The lateral drift limits for different types of masonry structures were updated based on recent shaking table test results. A new minimum depth for grade beams which support structural walls has been prescribed. This is an important issue considering that many walls are supported on beams at base of a building to leave space for vehicle circulation and parking. A new procedure for estimating the shear strength of masonry walls has been prescribed. The procedure takes into account the aspect ratio of the walls and the interaction of the wall shear capacity with the overturning moment on top of the wall. Also, the provision for the shear strength contribution of the horizontal reinforcement was updated. In confined masonry walls embedded tie-columns require considerably wider masonry units thus the minimum required wall thickness was also increased. A new recommendation has been included related to the modelling using the Finite Element Method and the provision related to the modelling using the Wide Column Model has been updated.

Keywords: Masonry; masonry design code; shear strength; confined masonry; Mexico

1 Introduction

The current version of the Mexico City´s code for the design and construction of masonry buildings was released in 2004. It uses the Limit States Design approach. The overall target performance for the building subjected to design loads is Collapse Prevention. The 2004 version of the code represented a huge improvement with regards to the previous version of the code. It was the first design code in México that included comprehensive graphical illustrations which explained many aspects of the detailing of the structure and its reinforcement, and as a result the 2004 code was easier to follow than its previous versions.

The Mexico City seismic code, also released in 2004, specifies the allowable drift limits for masonry structures, however some drift limits were also specified in the masonry code. The seismic reduction factor, known in many codes as R-factor, is denoted as Q in the Mexican code and it is called "the seismic behavior factor". This factor is also prescribed in the seismic code, however larger Q values are specified for structures with horizontal reinforcement and multi-perforated masonry units in the masonry code compared to the seismic code. Many practicing engineers in Mexico felt that some reorganization of the two codes and the provisions was necessary.

In this paper, a brief overview of some important aspects of the Mexican Code is presented first, followed by some of the most important revisions and additions which have been proposed for the new version of the code. Details of the Code are still being worked out as of October 2016 and hopefully the code will be finalized and published in 2017.

2 Overview of the Mexican Masonry Code

2.1 Masonry systems

Masonry structures in México are built using two basic systems: confined masonry and internally reinforced masonry. Confined masonry system includes external reinforced concrete (RC) tie-columns and tie-beams that surround masonry wall panels. The masonry panels are built first; the tie-columns are cast in place after the masonry wall is built. Tie-beams are normally cast simultaneously with the floor construction. Floors are usually built using solid RC slabs or prefabricated beams in combination with light-weight elements topped with a thin concrete finishing. The walls may contain horizontal reinforcement placed in the mortar joints. Horizontal reinforcement usually consists of small-size bars (up to 6.35 mm diameter) made of high strength steel, which are anchored into the tie-columns. Lapping of the bars should be avoided. The small dimeter of the bars ensure they can fit into the mortar bed joints which can be up to 12 mm thick. A variety of masonry units can be used for this type of construction. The traditional units are hand-made clay solid units, multi-perforated extruded clay bricks, concrete blocks, concrete multi-perforated units, etc. Outdoor surfaces are meant to be plastered or covered in some way to prevent water from infiltration and for architectural purposes. The value of seismic behavior factor for this type of construction may be 2 ($Q = 2$) when solid or hollow multi-perforated units with horizontal reinforcement are used. A general description of confined masonry may be found in [1].

The second type of construction is called internally reinforced masonry. It uses hollow units and the walls contain horizontal and vertical reinforcement within the mortar bed joints and vertical bars inside the masonry unit cells. Typically, only partial grouting is used, that is, only the hollow block cells with vertical reinforcement are filled with concrete. The maximum spacing for vertical reinforcement is prescribed by the code. This type of construction may be considered as confined masonry when tie-columns are embedded into walls and the spacing of tie-columns complies with the requirements prescribed for confined masonry walls. The current code allows the grouting of the cells with the same mortar as used for the masonry construction. Embedded tie-columns are difficult to inspect. It appears that in some field applications grout does not fill completely the cells in 10 or 12 cm thick masonry blocks. Although lab tests show that confined masonry with embedded tie-columns may be satisfactory, in practice it is questionable whether these tie-columns are adequate mainly due to extremely small cells. Internally reinforced masonry is used with masonry units which are intended to be waterproof and which are well finished with precise dimensions to provide an architectural finishing for exterior walls (façade). This system should not be confused with reinforced masonry technology which is used in developed countries. The main difference is in the size of the masonry units and the size of reinforcement. More importantly, the use of cementitious grout in reinforced masonry in developed countries make these walls behave more like reinforced concrete walls.

Both systems have proved economical and adequate for earthquake-resistant construction. The system uses thin walls that require small size confining elements, since in Mexico the building enclosures are not required to protect occupants against extreme weather conditions. Cavity walls and veneers as used in many countries are not considered. Attempts to use confined masonry with thicker walls produce larger size confining elements thus leading to a less cost-effective system [2].

2.2 Materials

A chapter describing the materials (masonry units and mortar) outlines the required testing procedures for estimating the mechanical properties of masonry, including the compressive strength and the diagonal tension strength which is used to estimate the shear strength due to tension. Mortar mix proportions of cement, sand and lime and their correlation to mortar compressive strength was well established in the past [3]. The code permits three types of mortar (Type I to III) with decreasing compressive strengths. Type III mortar with 4 MPa compressive strength is now considered a very poor quality mortar. The code also provides the so called index values for the mechanical properties mentioned above, as a function of the compressive strength of the masonry units and mortar type. When material testing is not possible, compressive strength of masonry units can also be determined from a table depending on the type of the masonry unit. There are no restrictions regarding the type of structure that can be designed with those index values. Although the prescribed index values are conservative (low), there is no guarantee whatsoever regarding the quality of the materials.

2.3 Analysis

The 2004 version of the masonry code allows the use of Simplified Method for the analysis of masonry structures which distributes the inter-storey shear force, due to a seismic event, into the resisting walls solely based on their relative lateral stiffnesses. The lateral stiffness of a wall is estimated as proportional to its cross-sectional area. The method is applicable to regular structures with regards to elevation and plan, which are not slender and up to five-storey high. For a more formal, computer-based structural analysis, the code provides useful guidelines for modelling masonry structures using the Wide Column Model.

2.4 Design

Separate chapters of the code describe the design of confined and internally reinforced masonry structures, although there are several similar or even identical aspects. The code prescribes the spacing of RC tie-columns in confined masonry walls and vertical reinforcement in internally reinforced walls, reinforcement of openings, etc.

It is important to understand that the shear strength of walls without horizontal reinforcement is set in the Mexican code to be equal to the shear cracking strength of the wall. Although it is well understood that the maximum shear strength is usually larger, it was previously considered that the maximum value depends on too many parameters and is therefore unreliable to predict. However, when the wall contains horizontal reinforcement, the code permits the use of maximum shear strength of the wall, since the overall behavior of the wall is more stable after cracking

The 2004 version of the code contain equations that take into account the contribution of horizontal reinforcement to the shear strength of the walls. Those equations were based on limited experimental investigations [7-9]. There was also evidence regarding the performance of retrofitting methodology using welded steel wire mesh. The efficiency factor of the reinforcement with regards to increasing the shear strength of the wall is the key concept. This factor rapidly decreases as a function of the amount of horizontal reinforcement, starting from a fixed amount of steel. The test results showed that an increase in the amount of horizontal reinforcement eventually changes the failure mode from diagonal tension to crushing; this limits the shear strength which can be obtained as a result of an increase in the amount of horizontal reinforcement.

2.5 Evaluation of new systems

For the first time, the 2004 version of the code included an Appendix which described the acceptance requirements for any new system of masonry construction. It specified the tests and the required evaluation procedures. This was a remarkable feature of the code which provided opportunities for private entrepreneurs to develop new masonry systems.

3 A new classification of the structures

Masonry structures are the structural system of choice for individual housing, from non-engineered construction to luxury residences, apartment buildings up to five-storey high and housing projects which range from a complex of few houses to hundreds of individual houses and/or large sets of buildings. The 2004 code recognized the necessity to distinguish large projects from small ones for inspection purposes. The new committee responsible for revising the code brought this idea further. A relaxed specification can be applied to smaller structures which are not part of a large development and *vice versa*. A new classification of the structures as Type I or Type II is now included in Chapter 1 of the code which covers general aspects. Type I structures have a built-up area up to 250 m^2, are up to two-storey high, and they are not a part of a housing project with more than ten units. Structures which do not meet the requirements of Type I structures are referred to as Type II structures.

4 Material properties

The use of index values for estimating the mechanical properties of masonry is now proposed to be limited to Type I structures. For Type II structures, material testing should be carried out. Alternatively, masonry units may be procured from manufacturers with established procedures for testing and quality control in compliance with the code.

Premixed industrialized mortar has controlled proportions of sand and cement, and the granulometry of the sand is also well controlled, thus usual problems of mortar contraction due to excessive content of fine particles are minimized. To recognize this fact, design strength of industrialized mortar may be estimated using a smaller variance coefficient of 0.1 - as opposed to variance coefficient of 0.2 for mortars prepared at the construction site.

Shear modulus, G, is now proposed to be equal to $0.2\,E$, where E is the modulus of elasticity. It is well known that $0.1E < G < 0.3E$. This clearly violates the elastic relationship $G = E/[2(1+\nu)]$, since in this case it would imply that Poisson's ratio $\nu > 0.5$. In order to comply with this specification when using commercial structural analysis programs, it may be needed to alter the shear cross-sectional area. The reduction of G value results into smaller lateral forces to be resisted by squat walls and larger forces to be resisted by slender walls [11].

5 Analysis

The Simplified Method of analysis has been dropped from the code, because the experience has shown that the method has been used for structures which do not meet the requirements for its applicability. However, the Simplified Method is a very powerful tool that provides a good sense for the magnitude of forces and resistance involved, base shear, and the overall shear strength using simple calculations. Recognizing its usefulness, a global lateral strength check was left, reminiscent of that method, intended as a safeguard in case errors from a more sophisticated analysis may pass undetected. Design base shear may be calculated approximately from the following equation

$$V_u = \frac{a(T)}{Q'R} W \quad \text{where} \quad T = 2\pi \sqrt{\frac{\sum x_i^2 w_i}{g \sum F_i x_i}}$$

and the total strength is approximated with

$$V_R = F_R \,(0.5 v_m' + 0.3\sigma_i + \eta_s p_h f_{yh})\sum A_T$$
$$\sigma_i \le 3.33 v_m'$$

so that

$$V_R \geq 0.8V_u$$

Modelling provisions are contained in a new Appendix B. There are new recommendations regarding modelling using the Wide Column Model which complement the current provisions, recommendations regarding cross-sectional properties of walls for 3D analysis, and a new strategy for modelling using the Finite Element Method (FEM).

6 Walls supported on beams

The code contains a new provision that prescribes the minimum depth of beams that support structural walls. It is a common practice to suspend structural masonry walls above the foundation in order to provide space for vehicle circulation and parking in the underground levels or first floor at street level. The beams that support such walls are usually designed assuming linear elastic behavior, and the analysis is typically performed using the FEM. However, as a result, internal stresses in the supporting beam are very small; this is mainly due to the large stiffness of the wall. Less experienced engineers end up with designs characterized by very small beam depths and/or minimum reinforcement. However, even minor cracking of the walls causes a rapid transfer of the load from the wall to the beam, and increases the vertical displacement of the beam that again produces additional cracking. The gravity load is transferred to the beam through an arching mechanism, which causes stress concentration at the wall edges. Such stresses may cause crushing in the wall depending on the compressive strength of the masonry. The new provisions establish the minimum beam depth (or moment of inertia) based on the two criteria. The first criterion ensures that masonry can resist the compressive stresses, as follows [12]:

$$\begin{gathered} f_{CM} \leq F_R f'_m \\ f_{CM} = F_{CE} P_u / A_T \\ F_{CE} = (32.K - 7.8)p \\ p = \begin{cases} 0.7 & c/c_{max} = 0.0 \\ 1.0 & c/c_{max} = 0.5 \\ 0.5 & c/c_{max} = 1.0 \end{cases} \end{gathered} \qquad (1)$$

where p should be interpolated for intermediate values of c/c_{max} and

$$K = \sqrt[4]{\frac{E_m t L_T^3}{E_c I f_t}}$$

The notation is illustrated in Fig. 1. The stress concentration factor, F_{CE}, has been obtained from simple expressions, depending on the relative stiffness of the beam and the wall, K [13], the position of the wall within the beam ($c/c_{máx}$), and the normalized wall length ($a = L/L_T$). Small values of K indicate a rigid beam, which tends to decrease the concentration factor. The load applied on the beam affects the stresses. The effect of the load is to increase the beam displacement, i.e. it is like reducing the beam stiffness. The factor f_I reduce the flexural stiffness of the beam due to the load w; $f_I = R/(R + 5w)$ and $R = \sigma_m\, t(a^4 - 4a^3 + 8a)$.

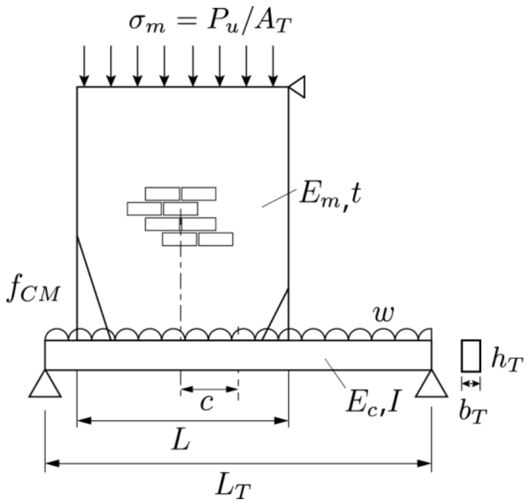

Fig. 1 – Structural wall supported on a beam

The second criterion defines the vertical displacement limit for the beam, which can be determined from Eq. 2

$$L_T/h_T \leq \left[\frac{1}{480}\frac{32E_c b_T}{5w+R}\right]^{\frac{1}{3}} \tag{2}$$

7 Reinforcement position and lapping

The horizontal reinforcement may be placed at the center of wall and one bar is permitted (Fig 2). These clarifications were needed as the drawing used in the 2004 code were interpreted by many engineers in the

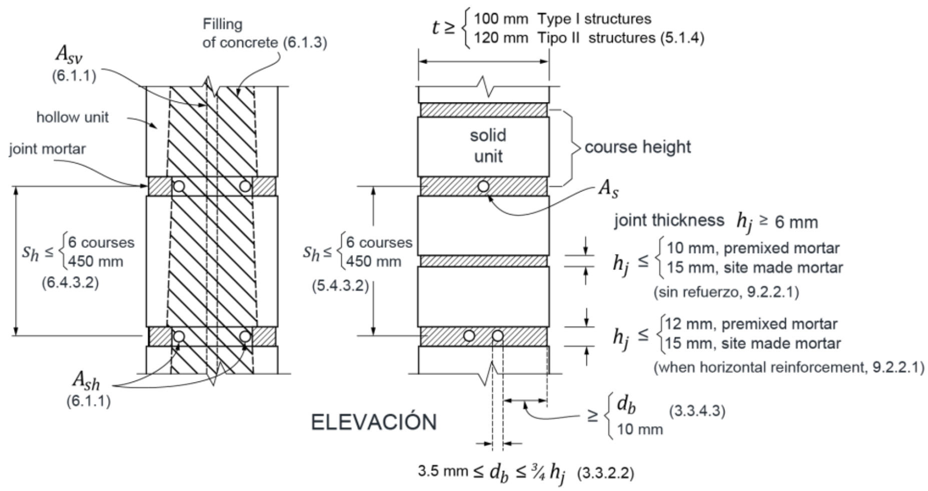

Fig. 2 Position of horizontal reinforcement in masonry walls

sense that two bars should be provided and that those bars should be located near the face of the wall. On the other hand, the location of the horizontal bars within internally reinforced walls should be located near the face of the wall but not on top of the solid part of the masonry units. In that case, the bars should be bent in order to ensure satisfactory anchorage into the cells.

It is now allowed to lap 100% of vertical reinforcing bars in tie-columns and vertical reinforcement in the walls, however lap length needs to be increased in that case. This provision was included because many contractors complained about the previous provision (contained in the 2004 code) where only 33% of the vertical bars could be lapped in one section. It is now possible to place masonry units around vertical reinforcing bars and provide the correct lapping length. Previously it was impractical to insert the masonry units when bars were too long for moving the lapping position, and the workers had a tendency to bend the vertical reinforcement.

Additional precautions regarding the appropriate concrete cover for tie-columns for exterior walls are also given. The required cover depends on the concrete compressive strength for tie-columns, whether concrete plaster has been used, and the type of plaster used (premixed or mixed on site).

8 Shear strength

The shear strength of a masonry wall, V_R, is provided by the masonry, V_{mR}, and the horizontal reinforcement, V_{sR}. It is considered that the vertical reinforcement in the tie-columns and the wall does not contribute to the lateral strength of the wall, that is,

$$V_R = V_{mR} + V_{sR} \tag{3}$$

The shear strength of a confined masonry wall without horizontal reinforcement is equal to the cracking strength (masonry shear strength at the onset of cracking), however the maximum shear strength is used for walls with horizontal reinforcement.

8.1 Cracking strength

Two new variables have been considered for estimating the cracking strength of the wall according to the new code: the height-to-length aspect ratio (H/L) and the value of the bending moment on top of the wall (M_a). The aspect ratio is well recognized as a parameter in reinforced masonry [4,5]. In confined masonry, a recent research study has shown that a decrease in the wall aspect ratio (e.g. squat walls) increases its shear strength (compared to the value obtained from the 2004 code which did not take into account the effect of aspect ratio) [14]. The bending moment on top of the wall may reduce its shear strength for single curvature bending and, in theory, may increase the shear strength for bending in double curvature. A simple model was proposed to estimate the reduction of the wall shear strength due to M_a. The model was verified through an experimental study in which the walls were bending in single curvature (cantilever walls). This effect is more pronounced in slender walls [15]. Since experimental evidence was not available for the case of double curvature it was decided that the eventual increase in strength will be disregarded in that case. The proposed cracking strength equation is as follows

$$V_a = (0.5v_m' A_T + 0.3P)\cdot f - \frac{M_a}{H_k} < 1.5 v_m' A_T \cdot f \tag{4}$$

$$f = \begin{cases} 1.6 & \dfrac{H}{L} < 0.2 \\ 1 & \dfrac{H}{L} > 1.0 \end{cases}$$

$$H_k = \frac{2}{3}\frac{k_f + k_v}{k_v}H \quad k_f = \frac{3EI}{H^3}, \quad k_v = \frac{GA_T}{\kappa H}$$

where f value should be interpolated for intermediate aspect ratio values. The importance of the shear-moment interaction effect was investigated in [16]. Several typical masonry structures were analyzed and it was found that most slender walls with $H/L > 1.5$ tend to bend in double curvature; consequently, the term containing M_a in Eq. 4 should not be considered to reduce the shear strength of the wall. Walls with smaller aspect ratios tend to bend in simple curvature; however, the effect of the moment on top of the wall decreases for squat walls. The results showed that walls with the aspect ratio in the range from 1.0 to 1.5 that bend in simple curvature and have a normalized moment $\beta = 2M_a/(VH)$ large enough to produce a reduction of the wall shear strength from 20 to 30% are not rare. The cracking strength proposal was presented in [17].

8.2 Contribution of masonry to shear strength

As mentioned before, if horizontal reinforcement is not provided, the wall shear strength is provided by the masonry and is set to be equal to the cracking strength.

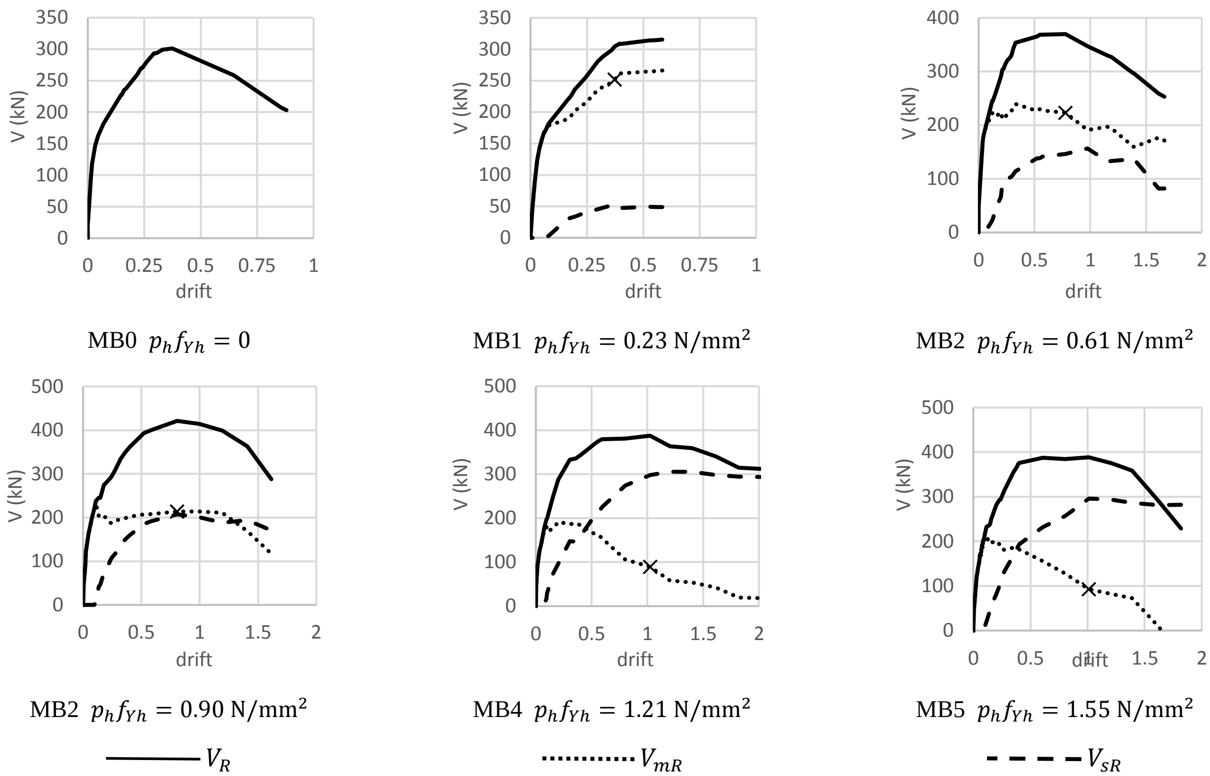

Fig. 3 Contribution of masonry and reinforcement to the wall shear strength [18]

When horizontal reinforcement is provided, the code uses the maximum shear strength of the wall. Recent tests of walls with varying amounts of horizontal reinforcement confirmed that the masonry shear strength decreases with an increase in the amount of horizontal reinforcement (Fig. 3).

The proposed contribution of masonry to the wall shear strength can be determined from the following equation [11]:

$$V_{mR} = F_R k_0 k_1 V_a$$

The maximum shear strength of the wall without horizontal reinforcement is $k_0 V_a$. It was recently found that this value depends on the wall aspect ratio [14]. For slender walls without horizontal reinforcement, the maximum masonry shear strength is equal to the cracking strength ($k_0 = 1$), while the cracking strength in square or squat walls may be on average 1.3 times the cracking strength of slender walls ($k_0 = 1.3$) [19]. For walls with horizontal reinforcement it was found that the masonry contribution to the wall strength decreases with the amount of horizontal reinforcement given by $q = p_h f_{yh}$ (Fig. 3) [18]. The rate at which the masonry contribution decreases is given by $k_1 = 1 - \alpha q_e$; α=0.45 mm^2/N. The value $q_e \leq q_\ell$ is the effective amount of reinforcement, and $q_\ell = \lambda f'_m$ is the limit amount of reinforcement, which is defined as the amount of reinforcement beyond which the steel contribution to the wall shear strength cannot be further increased. It is important to note that the q_ℓ value depends on the masonry compressive strength. The variation in the limit amount of reinforcement and the masonry compressive strength may give the λ value, which was estimated from experimental results to be $\lambda = 0.1$.

8.3 Strength due to horizontal reinforcement

The proposed equation for the shear strength due to the horizontal reinforcement is as follows

$$V_{sR} = F_R \eta_s q_e A_T \tag{5}$$

The main difference with regards to the 2004 code is that the efficiency factor η_s is now independent of the amount of horizontal reinforcement – instead it depends on the type of masonry which is characterized by the compressive strength.

8.4 Minimum and maximum amount of reinforcement

The minimum amount of horizontal reinforcement in the 2004 code was established so that the minimum amount of steel could sustain the wall cracking strength. This specification may lead to a large amount of steel, especially for masonry with a large diagonal compressive strength (v'_m). According to the proposed provision, the minimum amount of horizontal reinforcement is $q_{min} = 0.3$ N/mm^2. This value is similar to the minimum value specified for internally reinforced walls. In the 2004 code the maximum amount of horizontal reinforcement was set to the minimum of a fixed value (1.2 N/mm^2 for solid units and 0.9 N/mm^2 for hollow units) and 0.3 f'_m. The latter value governs only for masonry with very low compressive strength (less than 4 MPa). Consequently, the maximum amount of horizontal reinforcement may be considered fixed for most masonry walls. According to the new proposal, the maximum value is $q_{máx} = 0.15\ f'_m$ but there is a new geometrical limit: the maximum area of reinforcement within a joint cannot be larger than 5% of the joint area. This limit applies to walls with large amounts of steel reinforcement. Again, the important fact here is that both the effective and the maximum amount of reinforcement are dependent on the masonry compression strength. The 2004 code provisions occasionally gave inconsistent results which were due to the fact the both the efficiency and the maximum amount of reinforcement were dependent on the fixed rather than variable values dependent on the masonry´s compressive strength [10]. A similar procedure is proposed for the internally reinforced walls.

To preserve the design philosophy in which V_{mR} is independent on the amount of horizontal reinforcement the equations were reformulated as follows

$$V_R = V_{mR} + V_{sR}$$

$$V_{mR} = F_R V_a$$

$$V_{sR} = F_R q_e \psi A_T$$

$$\psi = \frac{V_a}{q_e A_T}(k_0 k_1 - 1) + \eta_s$$

This gives the same prediction of shear strength as formulas presented above. Finally, an optional simplified procedure was proposed for Type I structures where H/L and M_{au} are not considered and $\psi = (1 - 0.3q_e)$. This procedure gives very conservative shear strength values for squat walls.

8.5 Transverse reinforcement in tie-columns

There is no formal procedure for designing the amount and spacing of the transverse reinforcement in tie-columns. The evidence from many tests shows that the failure is initiated only after a shear crack develops in the tie-column. Based on that observation it was considered important to reduce the tie spacing in the upper and lower parts of the tie-columns, so that an inclined crack may cross the reinforcement like in RC beams and columns.

A new provision for ties was proposed according to which it is not required to anchor the tie with a 135^o hook towards the central part of the tie-columns; a 90^o hook will be sufficient, after one side of the stirrup is overlapped. Consequently, pouring of concrete into the confined core within a tie-column will be much easier.

9 Geometry of confined masonry walls

There was a consensus within the committee that 10 cm thick walls lead to very small tie-columns that are very difficult to pour. It was recognized, however, that this may be economically justified for small structures. The new provision specifies a minimum 12 cm wall thickness for Type II structures. Similar reasons gave way to a new minimum dimension of 15 cm for tie-columns parallel to the plane of the wall but only for type II structures. Tie-columns in Type I structures may still be equal to the minimum wall thickness (i.e. 10 cm).

Internally reinforced walls may be considered as confined masonry walls when the embedded tie-columns are provided at the spacing required for confined masonry walls. The committee was concerned regarding the feasibility of pouring and embedding tie-columns into the cells of hollow units for relatively thin walls (wall thickness 12 cm or less). Additionally, it is impossible to verify and inspect the embedded tie-columns. The new proposal established that embedded tie-columns may be considered only if poured into a 20 cm thick masonry unit. This provision is likely going to cause the market to abandon the concept of confined masonry walls in the form of internally reinforced masonry. The main difference in design seismic forces for confined and internally reinforced walls is due to the different seismic behavior factor Q. For confined masonry walls with solid units and walls with multi-perforated units with horizontal reinforcement a value $Q = 2$ may be specified, while for internally reinforced walls only a $Q = 1.5$ may be considered. These values may have a very significant effect on the design of the structure.

10 Allowable drifts and seismic reduction factor

A complete revision of the allowable drifts was carried out. The revisions were made based on the shaking table tests of scaled structures performed in Mexico which showed that recorded drifts from shaking table tests are much larger (more than twice) than those obtained from quasi-static tests [20]. Based on these results, the lateral displacements observed for different masonry systems corresponding to the maximum shear strength were scaled accordingly by a factor of 2. The selected drift limit corresponding to the maximum shear strength is considered consistent with the collapse prevention criterion, especially for structures without horizontal reinforcement as the wall behavior becomes unstable once the maximum shear strength has been reached.

The proposed drift limits are summarized in Table 1. It can be argued that for walls with horizontal reinforcement is safe to consider a larger drift limit. The inelastic drift limits contained in the 2004 code were based on quasi-static tests and they were set to be consistent with a moderate damage level. The drift limits corresponded to the level between the onset of tension cracking and the maximum strength so they were much smaller compared to the proposed values [21].

Table 1 Allowable drift levels and seismic behavior factor Q

Structural System	Q [2]	$\gamma_{\text{li,máx}}$ [2]
Confined masonry walls with solid units and horizontal reinforcement [3]	2.0	0.01
Confined masonry walls with solid units	2.0	0.006
Confined masonry walls with hollow units and horizontal reinforcement [3]	2.0	0.008
Confined masonry walls with hollow units	1.5	0.004
Internally reinforced walls with hollow units	1.5	0.006
Infill walls	(4)	(5)
Masonry walls not confined nor internally reinforced [6]	1.0	0.002
Natural Stone walls	1.0	.002

[1] Highly irregular structures according to the seismic code (NTCS) are not allowed in seismic zones II and III (pending).

[2] The value of Q and $\gamma_{\text{li,máx}}$ may differ in the two orthogonal analysis directions.

[3] The structure must have horizontal reinforcement in all walls to be considered in this category.

[4] When the walls are part of frames that cannot resist, at least, 70% of the lateral force not considering the walls, the seismic behavior factor will be the one corresponding to the masonry structural system used in the infill wall; otherwise use $Q = 3$ or $Q = 4$ in accordance with the NTCS.

[5] Should correspond to the type of masonry system used.

[6] Only for the existing structures.

The values of seismic behavior factor, Q, remained unchanged compared to the 2004 code (this factor is known in other codes as the seismic reduction factor R). The committee considered the possibility of increasing the Q values, however at the end the current values were preserved. Based on the well-known assumption that plastic deformations are concentrated at the first floor level it was found that larger Q values might require very large drifts at the first floor level [22].

Equation 6 estimates the ductility required at the first floor level based on the global ductility ($\mu = Q$) and the number of floors (n). For example, if Q =2.5 is used and n =6 or more, it is required to have ductility larger than 6 at the first floor level. An ongoing shaking table research study may give new reliable information regarding the maximum first floor ductility that may be used for walls with horizontal reinforcement.

$$\mu_1 = 1 + \frac{2}{3} n(\mu - 1) \tag{6}$$

An effort was made to reorganize the masonry code so that all provisions related to the ductility and drift limits were gathered into the seismic code.

11 Other masonry structures (unconfined and without internal reinforcement)

The design of a new masonry structure without confinement and without internal reinforcement is no longer permitted. The basic definitions of strength were preserved for the revision of existing structures.

12 Concluding Remarks

The new Mexico City masonry code incorporated the findings of latest research studies conducted in Mexico over the last 10 years. The code contains a more rigorous policy regarding the need for reliable values for mechanical properties of masonry materials. It is expected that the new reinforcement lapping provisions will facilitate construction of reinforced masonry walls. New provisions regarding ties for RC tie-columns are intended to facilitate their construction, while the new spacing prescription is expected to result in larger displacement capacity (ductility) of the walls. The wall aspect ratio and shear-moment interaction are now taken into account while estimating its shear cracking strength. These new expressions are expected to be especially important for taller masonry buildings that are now being constructed in Mexico City. The provision regarding the contribution of the horizontal reinforcement to the wall shear strength was thoroughly revised while preserving the main format of the original equations. The new equations give consistent results for a large range of masonry compressive strengths. The new provisions also provide the framework needed to justify and promote the use of good quality materials by recognizing the benefit of larger strengths and displacement capacities.

Despite these comprehensive revisions, many aspects of the code are yet to be investigated. More rational design procedures that can guarantee adequate performance during intense seismic events for structures higher than five storeys are still needed. For example, a formal design procedure for transverse reinforcement (ties) in RC tie-columns is required. The new code still lacks specifications which take into account out-of-plane behavior of the walls. A major effort is required to develop a reliable design procedure for RC frames with infill walls, to mention just a few.

13 Acknowledgements

Many thanks are due to Drs. Roberto Meli, Sergio Alcocer and Arturo Tena, who assisted many of the committee's sessions and gave valuable input. The former Federal District Government, now Mexico City Government, provided partial or full funding for several research projects which led to some of the recommendations included in the new code. Many thanks are also due to Dr. Svetlana Brzev for her valuable input.

14 References

[1] Meli R, Brzev S, et al. (2011): Seismic design guide for low-rise confined masonry buildings, *Confined Masonry Network, EERI & IAEE*.

[2] Jain SK, et al. (2014): Application of confined masonry in a major project in India, *Tenth U.S. National Conference on Earthquake Engineering,* Anchorage Alaska.

[3] Meli R (1979): Comportamiento sísmico de muros de mampostería, segunda edición, *Instituto de Ingeniería No 352*, Mexico.

[4] MSJC SD (TMS 402/ACI 530/ASCE 5), (2013): Building Code Requirements and Specification for Masonry Structures, *Masonry Standards Joint Committee*.

[5] NZS (4230:2004) (2004): Design of Reinforced Concrete Masonry Structures, *Standards Association of New Zealand*. Wellington.

[6] E.070 (2006): Norma Técnica E.070 Albañilería, Perú.

[7] Alcocer S and Meli R (1995): Test Program on the Seismic Behavior of Confined Masonry Walls, *The Masonry Society Journal* (Boulder), **12**(2), 68-76.

[8] Aguilar GR. Meli R, Diaz R, Vazquez-del-Mercado (1996): Influence of Horizontal reinforcement on the behavior of confined masonry walls, *Eleventh World Conference on Earthquake Engineering*, paper 1380 Acapulco, Mexico.

[9] Alcocer SM, Zepeda JA (1999): Behavior of multi-perforated clay brick walls under earthquake-type loading, *8th North American Masonry Conference*, Austin, Texas, USA.

[10] Pérez Gavilán JJ, Cruz O AI (2016): Shear strength of confined masonry walls with transverse reinforcement, *16th International Brick and block Masonry Conference*, Padova, Italy.

[11] Tomazevic M (2009): Shear Resistance of Masonry Walls and Eurocode 6: Shear Versus Tensile Strength of Masonry, *Materials and Structures*, **42** 889-907.

[12] Pérez Gavilán JJ, Lizárraga P JF (2013): Peralte mínimo de trabes que soportan muros estructurales de mampostería, *XIX Congreso Nacional de Ingeniería Sísmica*, Veracruz México. (in Spanish)

[13] Stafford B S, Pradolin L, Riddington JR (1982): Composite design method for masonry walls on steel beams, *Can J Civ. Eng.* **9** 96-106.

[14] Pérez Gavilán JJ, Flores L and Alcocer S (2015): An experimental study of confined masonry walls with varying aspect ratio, *Earthquake Spectra*, **31** (2), 945-968.

[15] Manzano A, Pérez Gavilán JJ (2013): Shear moment interaction in confined masonry walls, *12th Canadian Masonry Symposium*, Vancouver, Canada.

[16] Pérez Gavilán JJ, Cardel O J (2015): Shear-moment interaction in confined masonry walls: is it worth considering?, *The bridge and Structural Engineer*, **45**(1) 19-28.

[17] Pérez Gavilán JJ, Flores LE, Manzano A (2014): A new shear strength design formula for confined masonry walls: proposal to the Mexican code, *Tenth U.S. National Conference on Earthquake Engineering*, Anchorage, Alaska, USA.

[18] Pérez Gavilán JJ Cruz O AI (2016): Shear strength of confined masonry walls with transverse reinforcement, *16th International Brick and Block Masonry Conference*, Padova, Italy.

[19] Riahi Z, Elwood KJ, Alcocer SM (2009): Backbone model for confined masonry walls for performance-based seismic design, *Journal of Structural Engineering*, **135** (6), 644-654.

[20] Alcocer SM, Arias JG, Vásquez A, (2004): Response assessment of Mexican confined masonry structures through shaking table tests, *13th world conference of Earthquake Engineering*, Vancouver, Canada.

[21] Alcocer SM, et al. (2003): The Mexico City Building Code Requirements For Design And Construction of Masonry Structures, *Ninth North American Masonry Conference*, Clemson South Carolina, USA

[22] Paulay T, Priestley M JN (1992): Seismic Design of Reinforced Concrete and Masonry Buildings, *John Wiley & Sons*, New York.

15 Symbols

$a(T)$	spectral acceleration
A_s	area of horizontal reinforcement
A_{sv}	area of vertical reinforcement
A_T	wall cross-sectional area
b_T	beam width
E_c	concreate modulus of elasticity
E_m	masonry's modulus of elasticity
f	aspect ratio factor
f'_m	masonry compressive strength
f_{CM}	maximum compressive stress
f_I	factor modifier of beam inertia
P	axial load
q	horizontal reinforcement $= p_h f_{yh}$
q_e	effective amount of horizontal reinforcement
q_ℓ	limit amount of reinforcement
Q	seismic behavior factor
Q'	reduced seismic behavior factor due to structure irregularities.
R	over-strength factor
R	characteristic vertical load per unit length
t	wall thickness
T	structure dominant period

F_{CE}	stress concentration factor
F_i	seismic lateral force
h_T	beam height
H	wall height
H_K	characteristic height
I	beam inertia
k_0	maximum to cracking strength factor
k_1	masonry strength reduction factor with amount of reinforcement
K	relative beam to wall stiffness
L	wall length
L_T	beam length
M_a	bending moment on top of the wall
p_h	percent of horizontal reinforcement
v'_m	diagonal compressive strength
V_a	cracking strength
V_{mR}	shear strength due to masonry
V_{sR}	shear strength due to reinforcement
V_R	shear strength
V_u	shear force
w	beam load
w_i	story weight
W	total weight of the structure
x_i	lateral displacements from the base
η_s	efficiency of horizontal reinforcement
σ_i	average vertical stress on resisting elements
σ_m	vertical stress load on top of a wall