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Review of Design Codes for Masonry Buildings

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1. INTRODUCTION

The most effective use of masonry construction is seen in load bearing structures wherein it performs a variety of functions, namely, supporting loads, subdividing space, providing thermal and acoustic insulation, as well as fire and weather protection, which normally in a framed building has to be accounted for separately. Until 1950's there were no engineering methods of designing masonry for buildings and thickness of walls was based on *Rule-of-Thumb* tables given in Building codes and Regulations. As a result walls used to be very thick and masonry structures were found to be very uneconomical beyond 3 or 4 stories. Since 1950's intensive theoretical and experimental research has been conducted on various aspects of masonry in advanced countries. Factors affecting strength, stability and performance of masonry structures have been identified, which need to be considered in design.

In India, there has not been much progress in the construction of tall load bearing masonry structures, mainly because of poor quality of masonry workmanship and materials such as clay bricks that are manufactured even today having nominal strength of only 7 to 10MPa. However, recently mechanized brick plants are producing brick units of strength 17.5 to 25N/mm^2 and therefore it is possible to construct 5 to 6 storeyed load bearing structures at costs less than those of RC framed structures.

Use of reinforcement in masonry can further improve its load carrying capacity and most importantly its flexure and shear behavior under earthquake loads. Masonry units are being manufactured in shapes and sizes that make reinforcement embedding in masonry less cumbersome. With these developments, structural design of load bearing masonry buildings has been undergoing considerable modification as evidenced by changes that are taking place in the masonry design codes throughout the world.

The objective of the report is to study different codes from a number of countries on the design of masonry structures. The focus has been on the comparative study of the codes with respect to the design philosophy, effect of reinforcement on masonry and design of masonry under compression, flexure, and shear.

2. MASONRY CODES REVIEWED IN THIS STUDY

A brief description and major highlights of the various codes that have been reviewed is presented below. A summary of key provisions of these codes related to design approach, member sizing and details are provided in Table 1.

2.1 Building Code Requirements For Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02)

This code is produced by the joint efforts of American Concrete Institute, the Structural Engineering Institute of the American Society of Civil Engineers and The Masonry Society. The Code covers the design and construction of masonry structures and is accompanied with a Commentary on the Building code requirements. The code provides minimum requirements for the structural design and construction of masonry units bedded in mortar using both allowable stress design as well as limit state design (strength design) for unreinforced as well as reinforced masonry. The topic on strength design is a new addition to the previous edition of this code (ACI 530-99/ASCE 5-99/TMS 402-99). In strength design, more emphasis is laid on reinforced masonry than unreinforced masonry. An empirical design method applicable to buildings meeting specific location and construction criteria is also included.

2.2 International Building Code 2000

The International Building Code 2000 (ICC 2000) is designed to meet the need for a modern, up-to-date building code addressing the design of building systems through requirements emphasizing performance. This model code encourages international consistency in the application of provisions and is available for adoption and use by jurisdictions internationally. The provisions of this code for the design of masonry members have been heavily borrowed from ACI 530-02/ASCE 5-02/TMS 402-02.

2.3 New Zealand Standard – Code of Practice for the Design of Concrete Masonry Structures (NZS 4230: Part 1:1990)

This Standard was prepared under the direction of the Building and Civil Engineering Divisional Committee for the Standards Council, established under the Standards Act 1988. The content of this Code is largely dictated by seismic considerations and is intended to provide a satisfactory structural performance for masonry structures during a major earthquake. Minimum reinforcing requirements for different structural systems and the reinforcing and separation of non-structural elements will limit non-structural damage during moderate earthquakes. The design philosophy adopted throughout this code is strength design using reinforced masonry only. The Code has been set out in two parts, Code and Commentary. This code contains cross-references to NZS 3101, the primary code for the seismic design of structure.

2.4 Eurocode 6: Design of Masonry Structures (DD ENV 1996-1-1: 1996)

This code was published by the European Committee for Standardization (CEN) and is to be used with the National Application Document (NAD) of member countries. This code specifies a general basis for the design of buildings and civil engineering works in unreinforced and reinforced masonry made with clay and concrete masonry units laid in mortar. Limit state design method has been adopted throughout this code. However, Eurocode 6 does not cover the special requirements of seismic design. Provisions related to such requirements are given in Eurocode 8: Design of structures in seismic regions. The designer should consider the relative contribution of concrete infill and masonry in resisting load and, where the concrete infill makes a much greater contribution to the load resistance than the masonry, Eurocode 2 should be used and the strength of masonry should be ignored.

2.5 Indian Standard – Code of Practice for Structural Use of Unreinforced Masonry (IS: 1905-1987)

The Indian Standard on masonry design was first published in 1960 and later on revised in 1969, 1980 and 1987. The current third version, published in

1987, was reaffirmed in 1998. The provisions of this code are very similar to those of BS 5628: Part 1:1978. A separate handbook to this code, SP 20(S&T): 1991, is also available. This Indian Standard provides recommendations for structural design aspect of load bearing and non-load bearing walls using unreinforced masonry only. Design procedure adopted throughout the code is allowable stress design, along with several empirical formulae. The code refers to IS: 4326 for strengthening unreinforced masonry building for seismic resistance and does not provide any calculation for the design of reinforcement.

3. DESIGN PHILOSOPHIES

In the following section, design philosophies of various codes have been compared with regard to their design assumptions and assumed factor of safety.

3.1 Empirical design

Empirical rules and formulae for the design of masonry structures were developed by experience and traditionally, they have been used as a procedure, not as a design analysis for sizing and proportioning masonry elements. This method of design predates any engineering analysis and the effect of any steel reinforcement, if used, is neglected. Empirical design method is still being continued in ACI 530-02 and with some changes in IBC 2000. However, this design procedure is applicable to very simple structure with severe limitations on building height proportions and horizontal loads such as due to wind and earthquake. Indian Standards also mixes empirical procedure with allowable stress design method.

3.2 Allowable stress design

Allowable stress design states that under working loads, the stresses developed in a member must be less than the permissible stresses. In case of unreinforced masonry, it is assumed that tensile stresses, not exceeding allowable limits, are resisted by the masonry. For reinforced masonry, tensile strength of masonry is neglected. This design approach has been followed in the ACI code for reinforced as well as unreinforced masonry while in the IS code, it is applied only to unreinforced masonry. This design method does not find place in Eurocode and the New Zealand Standard.

3.3 Strength Design or Limit State Design

Strength design requires that masonry members be proportioned such that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by a strength reduction factor (φ). Required strength shall be determined in accordance with the strength design load combinations of a legally adopted building code. This procedure has been adopted by the ACI code, IBC 2000 and the New Zealand code, and more emphasis has been laid on reinforced masonry rather than unreinforced masonry in all these three codes. Eurocode 6 specifies a limit state design for collapse and serviceability, wherein instead of strength reduction factors, partial safety factors for loads and materials are specified separately. Partial safety factor for loads depend on the load combination and partial safety factor for materials depend on the type of masonry unit and the failure mode. In these codes, on the basis of the following assumptions, the strength of reinforced masonry members are calculated.

(a) There is strain continuity between the reinforcement, grout and masonry.

(b) The maximum usable strain (ε_{mu}) at the extreme masonry compression fiber shall be assumed to be 0.0035 for clay masonry and 0.0025 for concrete masonry. The New Zealand code also specifies that the maximum usable strain will be 0.008 for confined concrete masonry.

(c) Reinforcement stress below specified yield strength (f_y) shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be taken equal to f_y.

(d) The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

(e) Masonry stress of $0.80f_m$ (ACI code) or $0.85f_m$ (IBC 2000, New Zealand Standards) shall be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at a distance of a = 0.80c or 0.85c respectively from the fiber of maximum compressive strain, as shown in Fig.1. The value of uniformly distributed masonry stress for confined masonry, as specified in the New Zealand Standards, is $0.9Kf_m$ up to a distance a = 0.96c (Fig. 2), where K is a factor greater than 1, for increase in masonry strength due to confinement provided by confining plates.

4. COMPARISON OF KEY CONCEPTS FOR UNREINFORCED MASONRY

In this section, provisions related to the design of unreinforced masonry specified in various codes have been compared and discussed to bring out the differences or similarities. Design provisions of both allowable stress design and strength (limit state) design have been discussed for design of masonry member subjected to axial compression, flexure and shear.

4.1 Allowable Stress Design

4.1.1 Axial Compression

Axial compression on masonry arises due to vertical loads, especially from dead load and live load. Compression tests of masonry prisms are used as the basis for determining specified compressive strength of masonry f_m, which is further modified for slenderness, eccentricity, shapes of cross-section, etc. to derive allowable compressive stress values.

In ACI code, calculated compressive stress (f_a) should be less than the allowable compressive stress F_a which is obtained by multiplying f_m with 0.25 and slenderness ratio, *R*. The factor 0.25 accounts for material uncertainty and reduces f_m to working stress level. R is the capacity reduction factor for slenderness as given by the following equations.

$$R=1-\left(\frac{h}{40t}\right)^2 \quad \text{for } h/t \le 29$$

$$R=\left(\frac{20t}{h}\right)^2 \quad \text{for } h/t > 29$$

Slenderness can affect capacity either as a result of inelastic buckling or because of additional bending moments due to the deflection. Applied axial load must be less than 25% of the Euler buckling load, as given below.

$$P_e = \frac{\pi^2 E_m I_n}{h^2}\left(1 - \frac{2e}{t}\right)^3$$

Therefore, according to ACI code, the permissible value is function of slenderness ratio whereas the limiting value of axial load depends on both slenderness ratio as well as eccentricity of the axial load.

In IS: 1905 code a stress reduction factor (k_s) is multiplied with the basic compressive stress for slenderness ratio of the element and also the eccentricity of loading. The basic compressive stress is either determined from prism test values or a standard table which is based on compressive strength of unit and mortar type. In this code there is a limit to the maximum slenderness ratio for a load bearing wall, depending on the number of storeys and the type of mortar. A comparison of R (based on ACI, NZS, and IBC) and k_s of IS:1905, which accounts for slenderness effects on the strength is shown in Figure 3. Strength reduction in IS: 1905 follows a near linear trend with slenderness ratio.

4.1.2 Axial Compression with Flexure

Masonry members are generally subjected to flexural stresses due to eccentricity of loading or application of horizontal loads such as wind and earthquake. According to the ACI code, if a member is subjected to bending only, calculated bending compressive stress f_b should be less than allowable bending stress F_b in masonry, taken as $0.33f_m$ which is 1.33 times the basic compressive stress allowed for direct loads ($0.25f_m$). This increase is due to the restraining effect of less highly strained compressive fibers on the fibers of maximum strain and is supported by experiment.

For combined action of axial and flexure loads, a masonry member is acceptable if the sum of quotients of the resulting compression stresses to the allowable stresses does not exceed 1 (Figure 4).

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

The above interaction equation (unity formula) is widely used and very conservative. IS: 1905 checks bending compression and tensile stresses independently against permissible values. The permissible values for bending compression are obtained first by increasing the basic compressive stress by 25% and then reducing it for eccentric loading causing flexure. The code provides permissible loads for three eccentricity values: (a) $e< t/24$, (b) $t/24<e<t/6$, (c) $t/6<e$. An applied moment can be converted into equivalent eccentricity.

4.1.3 Shear

Masonry load bearing walls also act as shear walls to resist in-plane lateral loads due to wind or seismic forces. The lateral load carrying capacity of shear wall structures mainly depends on the in-plane resistances of the shear walls because the in-plane stiffness of a shear wall is far greater than its out-of-plane stiffness. Three modes of shear failure in unreinforced masonry are possible:

1. Diagonal tension cracks form through the mortar and masonry units.
2. Sliding occurs along a straight crack at horizontal bed joints.
3. Stepped cracks form, alternating from head joint to bed joint.

The ACI code recognizes these modes of failure and addresses them while specifying permissible shear stresses. For prevention of diagonal cracks, in-plane shear stress should not exceed $0.125\sqrt{f_m}$. For sliding failure, the allowable shear stress is based on a Mohr-Coulomb type failure criterion ($\tau=c'+\sigma_d \tan\phi$) and to resist failure due to stepped cracks, different values of permissible shear stress are given for various bond pattern of masonry.

The IS: 1905 code only takes care of sliding failure by specifying that the permissible shear stress $F_v = 0.1 + \sigma_d/6$, which is a Mohr-Coulomb type failure criterion, where σ_d is average axial stress. However, this linear relationship is valid up to axial compression of 2.4 MPa, at which it reaches the maximum limiting value of 0.5 MPa.

4.2 Strength Design or Limit State Design

4.2.1 Axial Compression

In ACI code, the nominal axial strength is based on compressive strength of masonry modified for unavoidable minimum eccentricity and slenderness ratio, in addition to the strength reduction factor. The expression for effect of slenderness is the same as in allowable stress design. Eurocode 6 also considers the effect of slenderness and eccentricity by using capacity reduction factor. However, this capacity reduction factor is based on eccentricity not only at the ends of member but also at middle one-fifth; wherever the moment may be maximum.

4.2.2 Axial Compression with Flexure

In all the codes, the two failure modes of wall considered are parallel and perpendicular to bed joints. The codes require the section to be checked by calculating axial and flexural strength.

4.2.3 Shear

For ACI code considers the previously discussed three modes of failure in evaluating the nominal shear strength of masonry. Similarly IBC 2000 also considers those factors for determining nominal shear strength of masonry and differs only in magnitude from the ACI code. Eurocode 6 only considers a sliding mode of shear failure and prescribes an equation of Mohr-Coulomb type ($F_v = 0.1 + 0.4\sigma_d$).

5. COMPARISON OF KEY CONCEPTS FOR REINFORCED MASONRY

Reinforced masonry is a construction system where steel reinforcement in the form of reinforcing bars or mesh is embedded in the mortar or placed in the holes and filled with concrete or grout. By reinforcing the masonry with steel reinforcement, the resistance to seismic loads and energy dissipation capacity can be improved significantly. In reinforced masonry, tension is developed in the masonry, but it is not considered to be effective in resisting design loads; reinforcement is assumed to resist all the tensile stresses.

5.1 Allowable Stress Design

Only the ACI code contains provisions on allowable stress design for reinforced masonry.

5.1.1 Axial Compression

In ACI code, the allowable axial compressive load (P_a) shall not exceed ($0.25f_mA_n+0.65A_{st}F_s$) R, which is obtained by adding the contribution of masonry and reinforcement. The contribution of longitudinal steel is given by the term $0.65A_{st}F_s$. The coefficient of 0.65 was determined from tests of reinforced masonry columns. The coefficient of 0.25 provides a factor of safety of about 4 against the crushing of masonry. Strength is further modified for slenderness effects by the factor R, which is the same as for unreinforced masonry.

5.1.2 Axial Compression with Flexure

For combined axial compression and flexure in reinforced masonry, the unity formula for interaction is not used in designing masonry members. The unity formula is suitable for unreinforced masonry but becomes very conservative for reinforced masonry. For reinforced masonry emphasis has been to compute nonlinear interaction diagram taking the effect of reinforcement and compression behavior of masonry into account. The axial load-bending moment interaction diagram is developed using equations and assumptions very similar to those used in analysis and design of reinforced concrete members. Interaction diagrams thus produced permit a rapid graphical solution.

5.1.3 Shear

When reinforcement is added to masonry, the shear resistance of masonry is increased. Shear reinforcement is effective in providing resistance only if it is designed to carry the full shear load. According to the ACI code, the minimum shear reinforcement is given by the following:

$$A_v = \frac{Vs}{F_s d}$$

This can be derived by assuming a 45^0 shear crack extended from the extreme compression fiber to the centroid of the tension steel, summing the forces in the direction of the shear reinforcement neglecting the doweling resistance of the longitudinal reinforcement. However the shear stress shall not exceed the permissible shear stress of masonry, which depends on the M/Vd_v ratio for shear walls. The term M/Vd_v is the product of h/d_v ratio and a factor depending on end restraints (Fig. 5). Figure 6 shows the variation of shear resistance with M/Vd_v ratio when shear reinforcement is provided or not, with the shear resistance being provided by reinforcement and masonry respectively.

5.2 Strength Design or Limit State Design

5.2.1 Axial Compression

The nominal strength of a member may be calculated using the assumptions of an equivalent rectangular stress block as outlined in the design assumptions. Slenderness effect on axial load carrying capacity is also taken into account except in IBC 2000. In the New Zealand Standards, nominal axial strength of a load bearing wall is given by $0.5f_mA_gR'$, where R' is always equal to $[1-(h/40t)^2]$.

5.2.2 Axial Compression with Flexure

For combined axial compression and flexure, nominal axial and flexural strength are computed similar to RC members with the design assumptions as discussed earlier. These design assumptions vary from one code to another.

According to the ACI code and IBC 2000, ε_{mu} shall be 0.0035 for clay masonry and 0.002 for concrete masonry. In wall design for out-of-plane loads according to both the codes, the required moment due to lateral loads, eccentricity of axial load and lateral deformation are assumed maximum at mid-height of the wall. In certain design conditions, such as large eccentricities acting simultaneously with small lateral loads, the design maximum moment may occur elsewhere. When this occurs, the designer should use the maximum moment at the critical section rather than the moment determined from the above equation.

In Eurocode 6, it is mentioned that the maximum tensile strain in reinforcement should be limited to 0.01. According to this code, no redistribution of moment is allowed with normal ductility steel. In this case, the ratio of depth of neutral axis to the effective depth should not be greater than

0.4. Redistribution of moments in a continuous beam should be limited to 15% when high ductility steel is to be used.

The New Zealand Standards, which deals with only concrete masonry, specifies that ε_{mu} shall be 0.0025 for unconfined masonry and 0.008 for confined masonry. Confinement is provided to the masonry walls to impart ductility to them.

5.2.3 Shear

Shear force is assumed to be resisted by both, masonry and reinforcement. The formulas given in ACI code and IBC 2000 to derive the nominal shear strength of masonry and reinforcement are empirically derived from research. The concept of minimum shear reinforcement is to help restrain growth of inclined cracking, and provide some ductility for members (by confining the masonry) subjected to unexpected force or catastrophic loading. The provisions of shear design are similar to as discussed in the previous section for allowable stress design.

In Eurocode 6, there is a maximum limit to the shear strength provided by masonry and shear reinforcement together, which is given by $0.3f_mbd/\gamma_m$. It is mentioned in the New Zealand Standards that for masonry members subjected to shear and flexure together with axial load, the shear stress provided by the masonry shall be multiplied by the factor $(1 + 12P_u/A_gf_m)$, where P is negative for tension. It is evident that V_m will decrease because of a reduction of aggregate interlock resulting from axial tension. The code considers instances where shear transfer is required by shear friction along a known or likely crack path. Resistance to sliding along a potential shear failure plane is provided by frictional forces between the sliding surfaces. The frictional forces are proportioned to the coefficient of friction and the total normal force acting across the joint, which may be provided by axial force, P_u, and distributed reinforcement, $A_{vf}f_y$. The effective clamping force across the crack will be $A_{vf}f_y + P_u$. Thus the dependable shear force, V_u, which can be transmitted across the crack by shear friction, is $\varphi\mu_f\ (A_{vf}f_y + P_u)$. Thus the required area of shear friction reinforcement shall be computed from:

$$A_{vf} = \frac{1}{f_y}\left(\frac{V_u}{\mu_f \varphi} - P_u\right)$$

During the placing of grout, if the interface has been intentionally roughened, μ_f equals 1.0; else μ_f is taken to be 0.7.

6. DISCUSSION

In this article, only strength provisions related to axial compression, flexure and shear for designs of masonry members were considered. Presently, most design codes prefer Limit State Design approach because of better reliability and economy, which is a major departure from the conventional empirical design method. Moreover, codes such as ACI 530-02, IBC 2000, New Zealand Standard and Eurocode 6, include design provisions for reinforced masonry. For reinforced masonry, only the ACI code contains provisions based on allowable stress values, whereas all other codes follow only limit state design approach. The International Building Code 2000 specifies some minor changes to the ACI code in the form of design assumptions and strength reduction factors.

For allowable strength of masonry shear walls, ACI emphasizes on the aspect ratio and boundary conditions by a parameter M/Vd_v. Also the strength of masonry is based on prism tests instead of placing reliance on standard tables, which relate it to the strength of unit and type of mortar. The advantage of prism test is that the prisms are built of similar materials under the same conditions with the same bonding arrangement as in the structure.

The design approach in IS: 1905-1987 is semi-empirical; which combines allowable stress design with rules of thumb for unreinforced masonry only. Neither limit state methodology has been adopted in this code nor there are any provisions related to reinforced masonry for any design philosophies. Enhancements and modifications of IS: 1905-1987 is urgently required to address these issues.

Among the codes studied in this document, only the New Zealand Standards contains provisions on ductility of masonry structures and confined masonry. Regarding shear it contains provisions on shear friction reinforcement and also considers the case when masonry members are subjected to shear and flexure together with axial tension. These salient features are not covered in other documents.

7. CONCLUDING REMARKS

IS: 1905-1987 provides a semi-empirical approach to the design of unreinforced masonry, especially for stresses arising from vertical and moderate lateral loads, such as wind. The permissible stress values are not directly linked to prism test values and do not address the strength and ductility of masonry members under large lateral loads due to earthquakes. Further the use of reinforcement is necessary to improve its flexural resistance and ductility required for seismic loads. The masonry codes of other countries provide detailed provision for the design of reinforced masonry members. IS: 1905 should be expanded to incorporate such provisions.

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APPENDIX: List of symbols

A_g Gross cross-sectional area of masonry

A_n Net cross-sectional area of masonry

A_s Total area of longitudinal reinforcing steel in a reinforced masonry wall, column or pilaster

A_v Cross section area of shear reinforcement

A_{vf} Area of shear friction reinforcement

a Depth of equivalent compression zone at nominal strength

b Width of section

b_w Effective web width

c Distance from extreme compression fiber to neutral axis

d Distance from extreme compression fiber to centroid of tension reinforcement

d_v Actual depth of masonry in direction of shear considered

E_m Modulus of elasticity of masonry in compression

e Eccentricity

e_a Accidental eccentricity

e_{hi} Eccentricity resulting from horizontal loads at top or bottom of wall

e_{hm} Eccentricity at mid-height resulting from horizontal loads

e_{mk} Eccentricity in the middle one-fifth of the wall

e_k Eccentricity due to creep

e_u Eccentricity of P_{uf}

F_a Allowable compressive stresses due to axial load only

F_b Allowable compressive stresses due to bending only

F_s Allowable tensile or compressive stress in reinforcement

F_v Allowable shear stress in masonry

f_a Calculated compressive stresses due to axial load only

f_b Calculated compressive stresses due to flexure only

f_m Specified compressive strength of masonry

f_r Mean compressive strength of mortar

f_{xk} Characteristic flexural strength of masonry

f_u Compressive strength of masonry unit

f_u Compressive strength of masonry unit

f_{xk1} Characteristic flexural strength in plane of failure parallel to bed joints

f_{xk2} Characteristic flexural strength in plane of failure perpendicular to bed joints

f_y Specified yield strength of steel for reinforcement

f_{yk} Characteristic strength of steel

h Effective height of columns, walls or pilasters

h'' Dimension of confined masonry core measured perpendicular to the direction of confining plate being considered

I_n Moment of inertia of net cross-sectional area of a member

L Length of a panel between supports

l_c Length of compressed part of wall, ignoring any part of wall that is in tension

M Maximum moment at the section under consideration

M_{cr} Nominal cracking moment strength

M_d Design moment

M_i Design moment at top or bottom of wall resulting from eccentricity of floor load at support

M_m Design moment within the middle one-fifth of the wall

M_n Nominal moment strength

M_u Factored moment

N_v Compressive force acting normal to shear surface

P Axial load

P_a Allowable compressive force in reinforced masonry due to axial load

P_e Euler buckling load

P_d Design axial strength

P_i Design vertical load at top or bottom of the wall

P_m Design vertical load at middle one-fifth of the wall

P_n Nominal axial strength

P_u Factored axial load

P_{uf} Factored load from tributary floor or roof areas

P_{uw} Factored weight of wall area tributary to wall section under consideration

P_{vf} Factored axial load normal to cross-section occurring with V_u, taken positive for compression and negative for tension

R Slenderness reduction factor

s Spacing of shear reinforcement

t Thickness of wall

V Shear force

V_d Design value of applied shear load

V_m Shear strength provided by masonry

V_n Nominal shear strength

V_s Shear strength provided by reinforcement

V_u Factored shear force at section

v_i Total shear stress corresponding to V_i

v_m Allowable shear stress of masonry

Z Section modulus of wall

α Bending moment coefficient depending on μ, degree of fixity at edge of panels and aspect ratio of panels

$\Phi_{i,m}$ Capacity reduction factors

φ Strength reduction factors

δ_u Deflection due to factored loads

ε_{mu} Maximum compressive strain in masonry

μ Orthogonal ratio of characteristic flexural strengths of masonry, f_{xk1}/f_{xk2}

μ_f Coefficient of friction

γ_f Partial safety factor for loads

γ_m Partial safety factor for materials

γ_s Partial safety factor for steel

θ Angle of shear reinforcement to the axis of the member

σ_d Permanent vertical stress on wall

ω_u Factored out-of-plane uniformly distributed load

ϕ_∞ Final creep coefficient