SEISMIC DESIGN GUIDE FOR MASONRY BUILDINGS

CHAPTER 2

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Canadian Concrete Masonry Producers Association

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The Canadian Concrete Masonry Producers Association (CCMPA) is a non-profit association whose mission is to support and advance the common interests of its members in the manufacture, marketing, research, and application of concrete masonry products and structures. It represents the interests of Region 6 of the National Concrete Masonry Association (NCMA).

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2 SEISMIC DESIGN OF MASONRY WALLS TO CSA S304.1-04

2.1 Introduction

Chapter 1 provides background on the seismic response of structures and seismic analysis methods, and explains key NBCC 2005 seismic provisions relevant to masonry design. This chapter provides an overview of seismic design requirements for reinforced masonry walls. Relevant CSA S304.1-04 design requirements are presented, along with related commentary, to provide detailed explanations of the NBCC provisions. Topics range from reinforced masonry shear walls subjected to in-plane and out-of-plane seismic loads, to a number of special topics such as masonry infill walls, stack pattern walls, veneers, and construction-related issues. Differences between CSA S304.1-04 seismic design requirements and those of the previous (1994) edition are identified and discussed, along with their design implications. For easy reference, relevant CSA S304.1 clauses are shown in a framed textbox where appropriate. Appendix B contains research findings and international code provisions related to seismic design of masonry structures. Appendix C contains relevant design background used in the design examples included in Chapter 4.

2.2 Masonry Walls – Basic Concepts

Structural walls are the key structural components in a masonry building and are used to resist some or all of the following load effects:

- axial compression due to vertical gravity loads,
- out-of-plane bending (flexure) and shear due to transverse wind, earthquake or blast loads and/or eccentric vertical loads, and
- in-plane bending and shear due to lateral wind and earthquake loads applied to building system in a direction parallel to the plane of the wall.

In a masonry building subjected to earthquake loads, horizontal seismic inertia forces develop in the walls, and the floor and roof slabs. The floor and roof slabs are called diaphragms where they transfer lateral loads to the lateral load resisting system. These inertia forces are proportional to the mass of these structural components and the acceleration at their level. An isometric view of a simple single-storey masonry building is shown in Figure 2-1a (note that roof diaphragm has been omitted for clarity). For earthquake ground motion acting in the direction shown in the figure, the roof diaphragm acts like a horizontal beam spanning between walls A and B. The end reactions of this beam are transferred to the walls A and B. These walls, subjected to lateral load along their longitudinal axis (also called *in-plane* loads), are called *shear walls*. Along with the floor and roof diaphragms, shear walls are the components of the building's lateral load path that transfers the lateral load to the foundations. A well-designed and well-built masonry building has a reliable load path, established by design, which transfers the forces over the full height of the building from the roof to the foundation.

Note also that the earthquake ground motion causes vibration of the transverse walls C and D. These walls are subjected to inertia forces proportional to their self-weight and are loaded *out-of-plane* (or transverse to their longitudinal axis). A vertical section through wall D that is loaded in the out-of-plane direction is shown in Figure 2-1b, while an elevation of shear wall A and its in-plane loading is shown in Figure 2-1c.

It is important to note that walls are subjected to shear forces in both the in-plane and out-of-plane directions during an earthquake event. However, the main difference between *shear walls* and other types of walls is that shear walls are key vertical components of a lateral load resisting system for a building, referred to as the Seismic Force Resisting System or SFRS by NBCC 2005. Usually not all walls in the building are shear walls; some walls (loadbearing and/or nonloadbearing) are not intended to resist in-plane loads, and are not designed and detailed as shear walls; in that case, they cannot be considered to form a part of the SFRS.

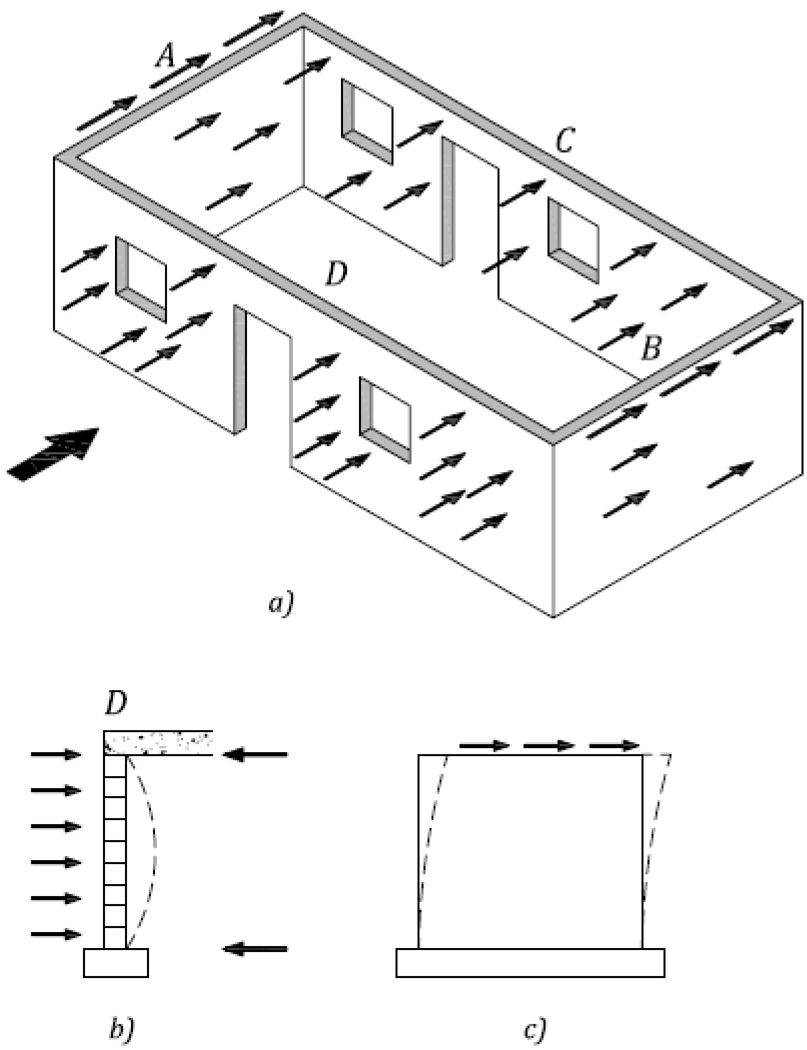


Figure 2-1 Simple masonry building: a) isometric view showing lateral loads; b) out-of-plane loads; c) in-plane loads (resisted by shear walls).

A typical reinforced concrete block masonry wall is shown in Figure 2-2. Vertical reinforcing bars are placed in the open cells of the masonry units (note that the term *cores* is also used in masonry construction practice), and are provided at generally uniform spacing along the wall length. The role of vertical reinforcement is to enhance the ability of the wall to resist forces due to vertical loads, and forces resulting from induced moments due to vertical eccentricities, as well as the out-of-plane loads. Horizontal wall reinforcement is usually provided in two forms: i) ladder- or truss-type wire reinforcement placed in mortared bed joints (see Figure 2-2b), and ii) steel bars (similar to vertical reinforcement) placed in grouted bond beams at specified locations along the wall height (see Figure 2-2c). Horizontal wire and bar reinforcement restrict in-plane movements due to temperature and moisture changes, resist in-plane shear forces and/or forces due to moments caused by out-of-plane loads. Grout, similar to concrete but with higher slump, is used to fill the cells of the masonry units that contain vertical and horizontal reinforcement bars. Grout increases the loadbearing capacity of the masonry by increasing its area, and serves to bond the reinforcement to the masonry unit so that the reinforcement and unit act compositely.

Grade 400 steel (yield strength 400 MPa) is nearly always used for horizontal and vertical reinforcing bars, whereas cold-drawn galvanized wire is used for joint reinforcement (also known as American Standard Wire Gauge – ASWG). Yield strength for joint reinforcement varies, but it usually exceeds 480 MPa for G30.3 steel wire. In design practice, 400 MPa yield strength is used both for the reinforcement bars and the joint wire reinforcement. The properties of concrete masonry units are summarized in Appendix D, while mechanical properties of masonry and steel materials are discussed by Drysdale and Hamid (2005) and Hatzinikolas and Korany (2005). The material resistance factors for masonry and steel prescribed by CSA S304.1-04 are as follows:

ϕ_m = 0.6 resistance factor for masonry (Cl.4.3.2.1)

ϕ_s = 0.85 resistance factor for steel reinforcement (Cl.4.3.2.2)

The following notation will be used to refer to wall dimensions (see Figure 2-2a):

l_w - wall length

h_w - total wall height

t - overall wall thickness

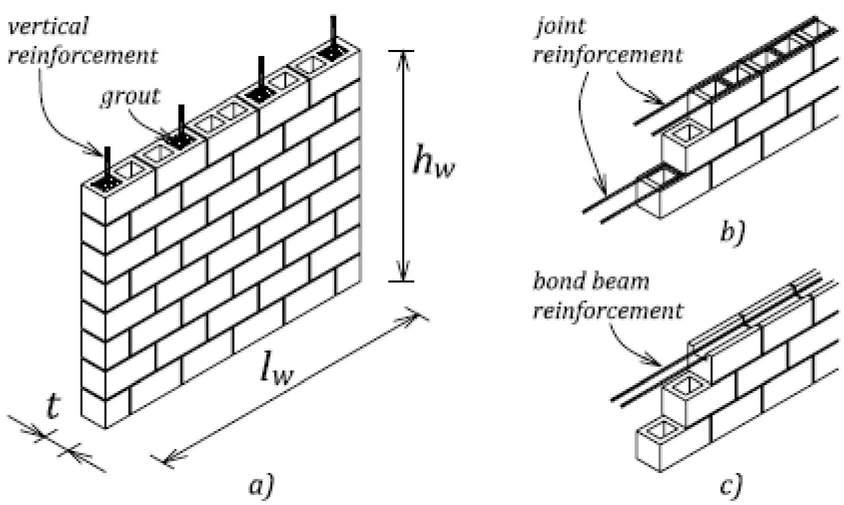


Figure 2-2. Typical reinforced concrete masonry block wall: a) vertical reinforcement; b) joint reinforcement; c) bond beam reinforcement.

Typical reinforced concrete masonry wall construction is shown in Figure 2-3. The lower section of the wall has been grouted to the height of a bond beam course. Vertical bars extend above the bond beam to serve as bar splices for the continuous vertical reinforcement placed in the next wall section.

Figure 2-3 Masonry wall under construction (Credit: Masonry Institute of BC).

Walls in which only the reinforced cells are grouted are called *partially grouted walls*, whereas the walls in which all the cells are grouted are called *fully grouted walls*. Irrespective of the extent of grouting (partial/full grouting), cross-sectional area of the entire wall section (considering the overall thickness t) is termed *gross cross-sectional area*, A_g. In partially grouted or hollow (ungrouted) walls, the term *effective cross-sectional area,* A_e, denotes that area which includes the mortar-bedded area and the area of grouted cells (S304.1 Cl.10.3). Both the gross and effective wall area are shown for a wall strip of unit length (usually equal to 1 metre). The difference between A_g and A_e is illustrated in Figure 2-4. In ungrouted masonry construction, the webs are generally not mortared, however in partially grouted reinforced masonry construction, the webs on each side of a grouted cell are sometimes mortared to ensure that grout does not flow into the adjacent cells not intended to be grouted. In any case, coarse grout will flow from the grouted core to fill the gap between the webs adjacent to the cell. In exterior walls the effective area is often significantly reduced by raked joints (this is not a concern with a standard concave tool joint). The designer should consider this effect in the calculation of the depth of the compression stress block.

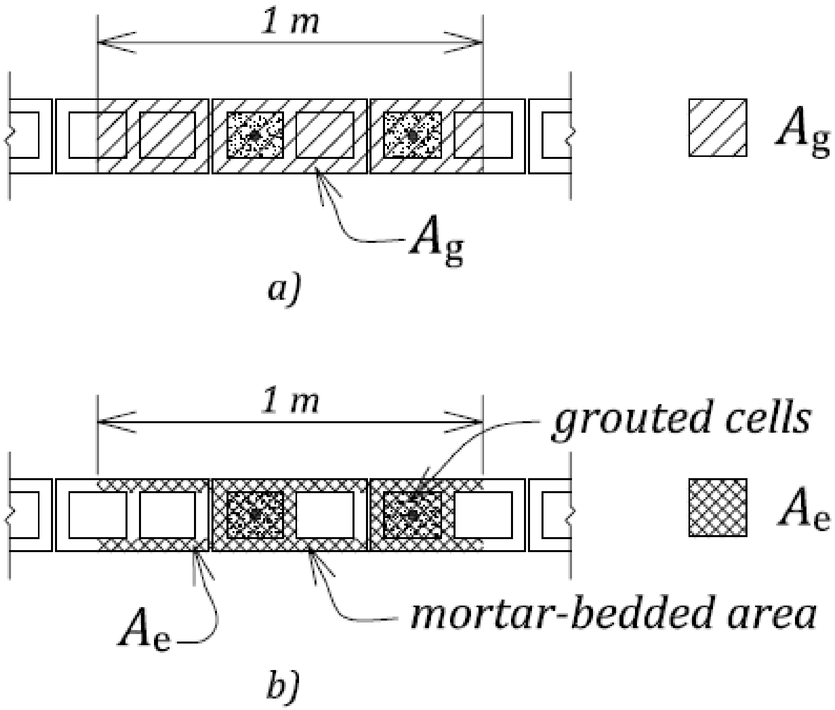


Figure 2-4. Wall cross-sectional area: a) gross area; b) effective area.

Shear walls without openings (doors and/or windows) are referred to as *solid* walls (Figure 2-5a), while the walls with door and/or window openings are referred to as *perforated* walls (Figure 2-5b). Regions between the openings in a perforated wall are called *piers* (see piers A, B, and C in Figure 2-5b). Perforated shear walls in medium-rise masonry buildings with uniform distribution of vertically aligned openings over the wall height are called *coupled walls.*

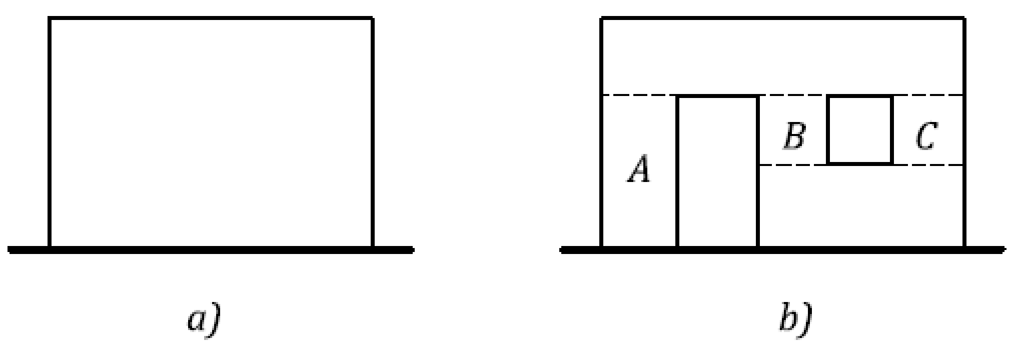


Figure 2-5. Masonry shear walls: a) solid, and b) perforated.

Depending on the wall geometry, in particular the height/length (h_w/l_w) aspect ratio, shear walls are classified into the following two categories:

- Flexural shear walls with a height/length aspect ratio of 1.0 or higher (Figure 2-6a), and
- Squat shear walls with a height/length aspect ratio less than 1.0 shown in Figure 2-6b (see S304.1 Cl.4.6.6 and 10.10.1.3).

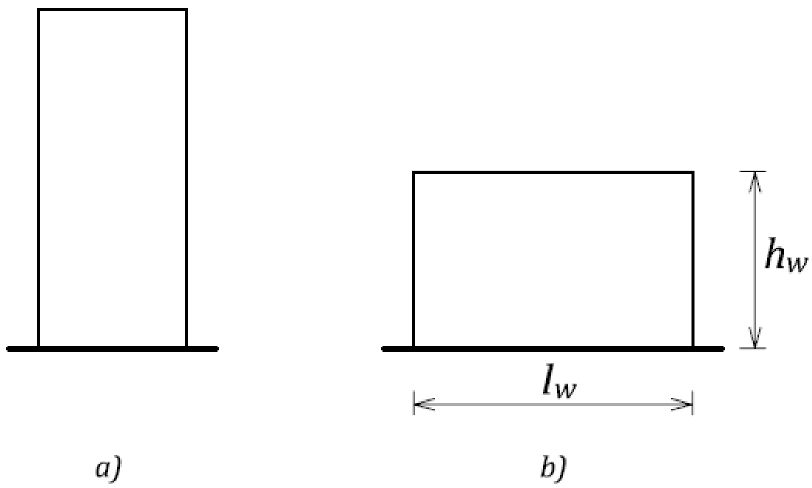
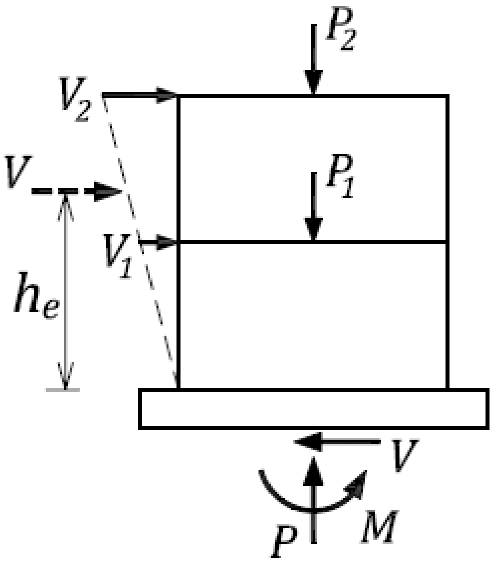


Figure 2-6. Shear wall classification based on the aspect ratio: a) flexural walls; b) squat walls.

Depending on whether the walls resist the effects of gravity loads in addition to other loads, masonry walls can be classified as loadbearing or nonloadbearing walls. *Loadbearing* walls resist the effects of superimposed gravity loads (in addition to their selfweight) plus the effects of lateral loads. *Nonloadbearing* walls resist only the effects of their selfweight and possibly out-of-plane wind and earthquake loads. Shear walls are loadbearing walls, irrespective of whether they carry gravity loads or not.

In masonry design, selection of the locations where movement joints (also known as control joints) need to be provided is one of the initial and very important detailing decisions. Some movement joints are provided to facilitate design and construction while others prevent cracking at undesirable locations. In any case, wall length is determined by the location of movement joints and so this detailing decision carries an implication for seismic design. For more details on movement joints refer to MIBC (2008).

In general, shear walls are subjected to lateral loads at the floor and roof levels, as shown in Figure 2-7. (Note the inverse triangular distribution of lateral loads simulating earthquake effects.) Distribution of forces in a shear wall is similar to that of a vertical cantilevered beam fixed at the base. Figure 2-7 also shows internal reactive forces acting at the base of the wall. Note that the wall section at the base is subjected to the shear force, V, equal to the sum of the horizontal forces acting on the wall, the bending moment, M, due to all horizontal forces acting at the effective height h_e, and the axial force, P, equal to the sum of the axial loads acting on the wall.



$$P = \sum P_i$$
$$V = \sum V_i$$
$$M = V \cdot h_e$$

Figure 2-7. Load distribution in shear walls.

2.3 Reinforced Masonry Shear Walls Under In-Plane Seismic Loading

2.3.1 Behaviour and Failure Mechanisms

The behaviour of a reinforced masonry shear wall subjected to the combined effect of horizontal shear force, axial load and bending moment depends on several factors. These include the level of axial compression stress, the amount of horizontal and vertical reinforcement, the wall aspect ratio, and the mechanical properties of the masonry and steel. The two main failure mechanisms for reinforced masonry shear walls are:

- Flexural failure (including ductile flexural failure, lap splice slip, and flexure/out-of-plane stability), and
- Shear failure (includes diagonal tension failure and sliding shear failure).

Each of these failure mechanisms will be briefly described in this section. The focus is on the behaviour of walls subjected to cyclic lateral load simulating earthquake effects. Failure mechanisms for reinforced masonry walls are discussed in detail in FEMA 306 (1999).

2.3.1.1 Flexural failure mechanisms

Ductile flexural failure is found in reinforced walls and piers characterized by a height/length aspect ratio (h_w/l_w) of 1.0 or higher and a moderate level of axial stress (less than $0.1f'_m$). This failure mode is characterized by tensile yielding of vertical reinforcement at one end of the wall and simultaneous cracking and possible spalling of masonry units and grout in the toe area (compression zone). In some cases, buckling of compression reinforcement accompanies the cracking and spalling of masonry units. Experimental studies have shown that the vertical reinforcement is effective in resisting tensile stresses and that it yields shortly after the cracking in masonry takes place (Tomazevic, 1999). Damage is likely to include both horizontal flexural cracks and diagonal shear cracks of small size concentrated in the plastic hinge zone, as shown in Figure 2-8a. (The plastic hinge zone is the region of the member where inelastic deformations occur and it will be discussed in Section 2.5.4.2.) In general, this is the preferred failure mode for reinforced masonry shear walls, since the failure mechanism is ductile and effective in dissipating earthquake-induced energy once the yielding of vertical reinforcement takes place.

Flexure/lap splice slip failure may take place when starter reinforcing bars projecting from the foundations have insufficient lap splice length, or when the rebar size is large relative to wall thickness (e.g. 25M bars used in 200 mm walls), resulting in bond degradation and eventual rocking of the wall at the foundation level. Initially, vertical cracks appear at the location of lap splices followed by the cracking and spalling at the toes of the wall (see Figure 2-8b). This mode of failure may be fairly ductile but it results in severe strength degradation and does not provide much energy dissipation.

Flexure/out-of-plane instability takes place at high ductility levels (see Figure 2-8c). Ductility is a measure of the capacity of a structure to undergo deformation beyond yield level while maintaining most of its load-carrying capacity (ductile seismic response will be discussed in Section 2.5.3). When large tensile strains develop in the tensile zone of the wall, that zone can become unstable when the load direction reverses in the next cycle and the compression takes place. This type of failure has been observed in laboratory tests of well detailed highly ductile flexural walls (Paulay and Priestley, 1993), but it has not been observed in any of the post-earthquake field surveys so far (FEMA 306, 1999). This failure mechanism can be prevented by ensuring stability of the wall compression zone through seismic design (see Section 2.5.4.4 for more details).

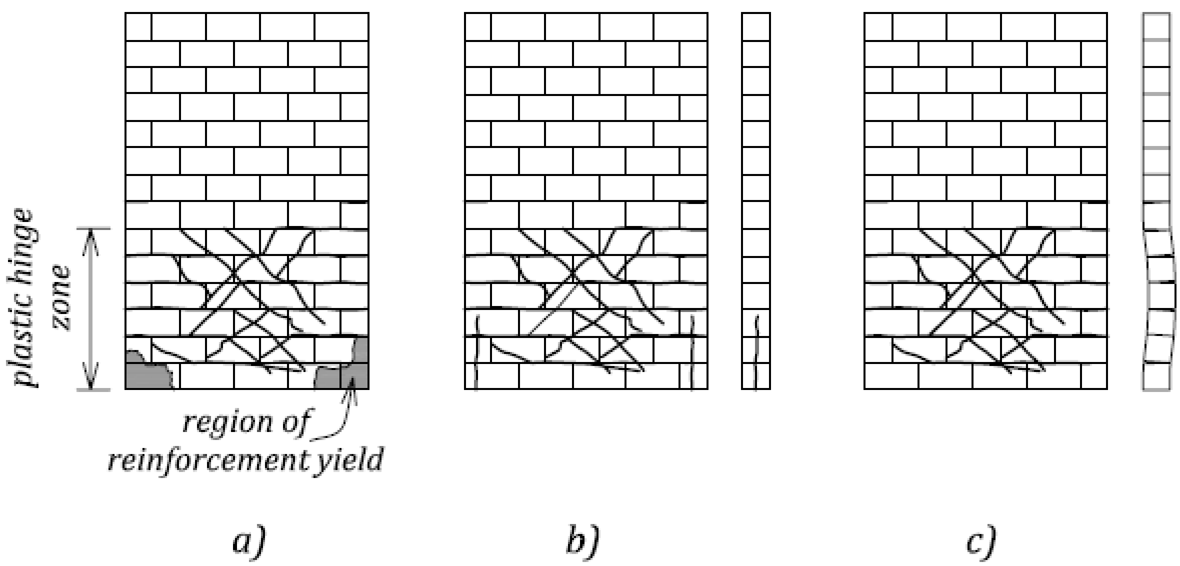


Figure 2-8. Flexural failure mechanisms: a) ductile flexural failure; b) lap splice slip, and c) out-of-plane instability (FEMA 306, 1999, reproduced by permission of the Federal Emergency Management Agency).

2.3.1.2 Shear failure mechanisms

Shear failure is common in masonry walls subjected to seismic loads and has been observed in many post-earthquake field surveys. Due to the dominant presence of diagonal cracks, this mode is also known as *diagonal tension* failure (see Figure 2-9a). It usually takes place in walls and piers characterized by low aspect ratio (h_w/l_w less than 0.8). These walls are usually lightly reinforced with horizontal shear reinforcement and so the shear failure takes place before the wall reaches its full flexural capacity.

This mode of failure is initiated when the principal tensile stresses due to combined horizontal seismic loads and vertical gravity loads exceed the masonry tensile stress. When the amount and anchorage of horizontal reinforcement are not adequate to transfer the tensile forces across the first set of diagonal cracks, the cracks continue to widen and result in a major X-shaped diagonal crack pair, thus leading to a relatively sudden and brittle failure. Note that these "diagonal cracks" may develop either trough the blocks, or along the mortar joints.

In modern masonry construction designed according to the code requirements, it is expected that adequate horizontal reinforcement is provided and that it is properly anchored within wall end zones. Horizontal reinforcement can be effective in resisting tensile forces in the cracked wall and in enhancing its load-carrying capacity. After the initial diagonal cracks have been formed, several uniformly distributed cracks develop and gradually spread in the wall. Failure occurs gradually as the strength of the masonry wall deteriorates under the cyclic loading. Voon (2007) refers to this mechanism as "ductile shear failure". It should be noted that ductile behaviour is usually associated with the flexural failure mechanism, while shear failure mechanisms are usually characterized as brittle, however in very squat shear walls a ductile shear mechanism may be the only ductile alternative.

<u>Sliding shear failure</u> may take place in masonry walls subjected to low gravity loads and rather high seismic shear forces. This condition can be found at the base level in low-rise buildings or at upper storeys in medium-rise buildings where accelerations induced by earthquake ground motion are high, but it can also take place at other locations. Sliding shear failure takes place when the shear force across a horizontal plane (usually base in reinforced masonry walls) exceeds the frictional resistance of masonry and a horizontal crack is formed at the base of the wall, as shown in Figure 2-9b. There may be very limited cracking or damage in the wall outside the sliding joint. The frictional mechanism at the sliding interface is activated after the clamping force develops in vertical reinforcement which yields in tension. Even though this mode of failure is often referred to as shear failure mode, it may also take place in the walls characterized by flexural behaviour. Pre-emptive sliding at the base limits the development of full flexural capacity in the wall.

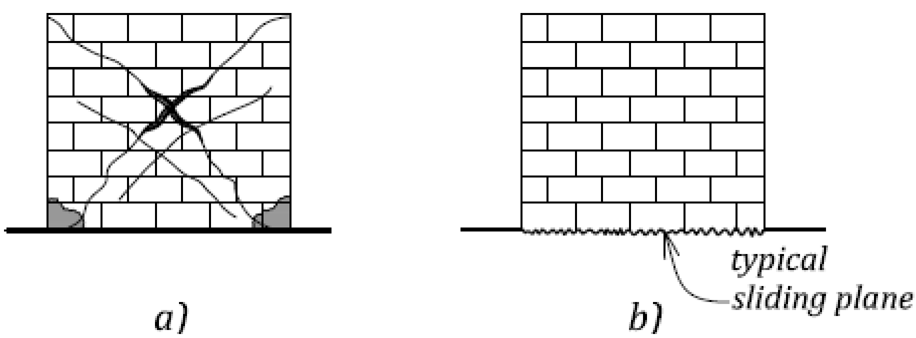


Figure 2-9. Shear failure mechanisms: a) diagonal tension[1], and b) sliding shear.

2.3.2 Shear/Diagonal Tension Resistance

Shear resistance of reinforced masonry shear walls depends on several parameters, including the masonry compressive strength, grouting pattern, amount and distribution of horizontal reinforcement, magnitude of axial stress, and height/length aspect ratio. Over the last two decades, significant experimental research studies have been conducted in several countries, including the US, Japan, and New Zealand. Although the findings of these studies have confirmed the influence of the above parameters on the shear resistance of masonry walls, it appears to be difficult to quantify the influence of each individual parameter. This is because of

[1] Source: FEMA 306, 1999, reproduced by permission of the Federal Emergency Management Agency

the complexity of shear resistance mechanisms and a lack of effective theoretical models. As a result, shear resistance equations included in the Canadian masonry design standard, S304.1-04, and those of other countries are based on statistical analyses of test data obtained from a variety of experimental studies. The diagonal tension shear resistance equation for reinforced masonry walls in CSA S304.1 (both 1994 and 2004 editions) is mainly based on the research by Anderson and Priestley (1992) and other research based on wall tests in the US and Japan. Refer to Section B.1 for a detailed research background on the subject.

This section discusses the in-plane shear resistance provisions of CSA S304.1-04 for non-seismic conditions; seismic requirements related to shear design are discussed in Section 2.5.4.5. The design of walls built using running bond is discussed in this section, and walls built using the stack pattern are discussed in Section 2.6.3.

2.3.2.1 Flexural shear walls

10.10.1.1

Flexural shear walls are characterized by height/length aspect ratio of 1.0 or higher (see Figure 2-6a). Consider a reinforced masonry shear wall built in running bond which is subjected to the effect of factored shear force, V_f, and the factored bending moment, M_f.

Factored in-plane shear resistance, V_r, is determined as the sum of contributions from masonry, V_m, and steel, V_s, that is,

$$V_r = V_m + V_s \qquad (1)$$

Masonry shear resistance, V_m, is equal to:

$$V_m = \phi_m (v_m b_w d_v + 0.25 P_d) \gamma_g \qquad (2)$$

Wall dimensions (b_w and d_v):
$b_w = t$ overall wall thickness (mm) (referred to as "web width" in CSA S304.1); note that b_w does not include flanges in the intersection walls
d_v = effective wall depth (mm)
$d_v \geq 0.8 l_w$ for walls with flexural reinforcement distributed along the length
Wall cross-sectional dimensions (b_w and d_v) used for shear design calculations are illustrated in Figure 2-10.

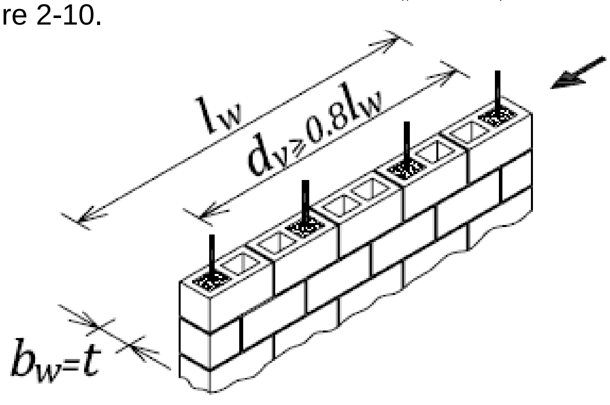


Figure 2-10. Wall cross-sectional dimensions used for in-plane shear design.

Effect of axial load (P_d):

P_d = axial compression load on the section under consideration, based on 0.9 times dead load, P_{DL}, plus any axial load, N, arising from bending in coupling beams or piers (see Figure 2-11)

$P_d = 0.9 P_{DL}$ for solid walls

$P_d = 0.9 P_{DL} \pm N$ for perforated/coupled walls

Note that the net effect of tension and compression forces $\pm N$ on the total shear in the wall is equal to 0.

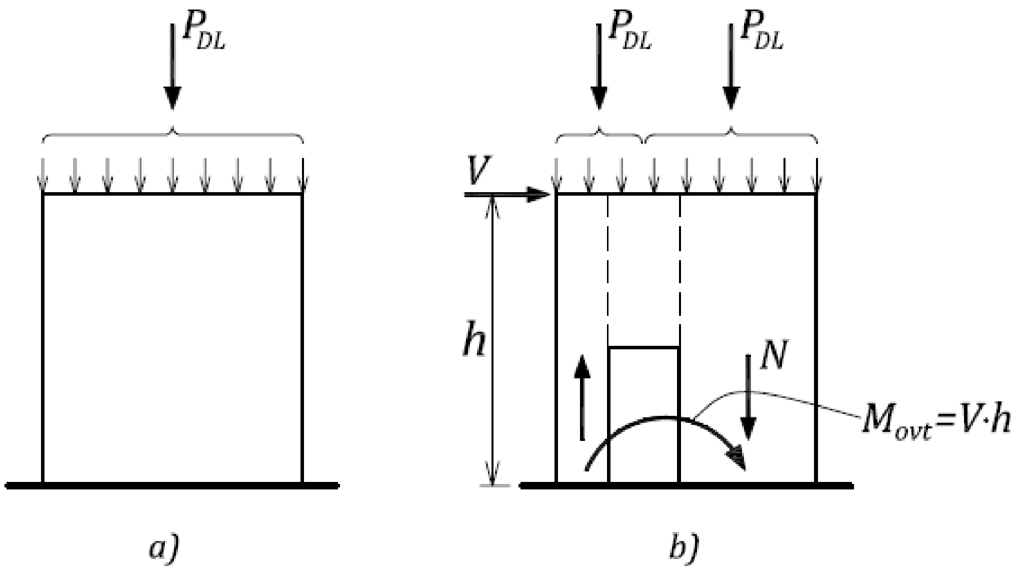


Figure 2-11. Axial load in masonry walls: a) solid; b) perforated.

Effect of grouting (γ_g):

γ_g = factor to account for partially grouted walls that are constructed of hollow or semi-solid units

$\gamma_g = 1.0$ for fully grouted masonry, solid concrete block masonry, or solid brick masonry

$\gamma_g = \frac{A_e}{A_g}$ for partially grouted walls, but $\gamma_g \le 0.5$

where (see Figure 2-4)

A_e = effective cross-sectional area of the wall (mm^2)

A_g = gross cross-sectional area of the wall (mm^2)

Masonry shear strength (v_m):

v_m represents shear strength attributed to the masonry in running bond, which is determined according to the following equation:

10.10.1.4

$$v_m = 0.16\left(2 - \frac{M_f}{V_f d_v}\right)\sqrt{f'_m} \qquad \textit{MPa units} \qquad (3)$$

Shear span ratio ($\frac{M_f}{V_f d_v}$):

The following limits apply to the shear span ratio:

$$0.25 \le \frac{M_f}{V_f d_v} \le 1.0$$

10.10.1.1

Reinforcement shear resistance, V_s, is equal to:

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} \qquad (4)$$

where
A_v = area of <u>horizontal</u> wall reinforcement (mm^2)
s = vertical spacing of horizontal reinforcement (mm)

As discussed in this section, the factored in-plane shear resistance, V_r, is determined as the sum of contributions from masonry, V_m, and reinforcement, V_s, that is,

$$V_r = V_m + V_s \qquad (5)$$

where

$$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g \qquad (6)$$

and

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} \qquad (7)$$

CSA S304.1 prescribes the following upper limit for the factored in-plane shear resistance V_r for flexural walls:

$$V_r \le \max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g \qquad (8)$$

Commentary

<u>Axial compression:</u>
The equation for the factored shear resistance of masonry, V_m, in accordance with CSA S304.1 [equation (2)], takes into account the positive influence of axial compression. The term $0.25P_d$ was established based on the statistical analyses of experimental test data carried out by Anderson and Priestley (1992). The 0.25 factor is consistent with that used for concrete in estimating the shear strength of columns.

Consider a masonry shear wall subjected to the combined effect of axial and shear forces shown in Figure 2-12a. A two-dimensional state of stress develops in the wall: axial load, P, causes the axial compression stress, σ, while the shear force, V, causes the shear stress, v. The presence of axial compression stress delays the onset of cracking in the wall since it reduces the principal tensile stress due to the combined shear and compression. Shear cracks develop in the wall once the principal tensile stress reaches the masonry tensile strength (which is rather low). It should be noted, however, that the masonry shear resistance decreases in a wall section subjected to high axial compression stresses (see the diagram shown in Figure 2-12b). This is based on experimental studies – for more details refer to Drysdale and Hamid

(2005). Note that shear walls in low-rise masonry buildings are subjected to low axial compression stresses, as shown in Figure 2-12b.

Grouting pattern:
CSA S304.1-04 takes into account the effect of grouting on the masonry shear resistance through the γ_g factor, which assumes the value of 1.0 for fully grouted walls and 0.5 or less for partially grouted walls. Research evidence indicates that fully grouted reinforced masonry walls demonstrate higher ductility and strength under cyclic lateral loads than otherwise similar partially grouted specimens, as discussed in Section B.5.

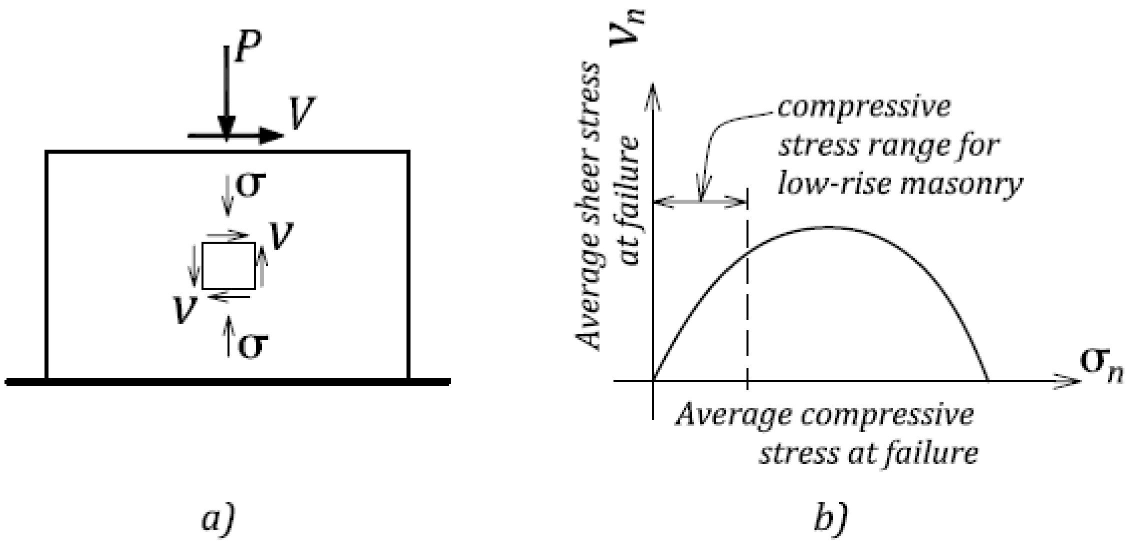


Figure 2-12. Effect of axial stress: a) a shear wall subjected to the combined shear and axial load; b) relationship between the shear stress at failure and the compression stress.

Masonry shear strength (v_m):
Masonry shear strength defined by equation (3) depends on masonry tensile strength represented by the $\sqrt{f'_m}$ term, as well as on the shear span ratio, $M_f/V_f d_v$. Walls with shear span ratios of less than 1.0 behave like squat walls and are characterized by the highest masonry shear resistance, as illustrated in Figure 2-13.

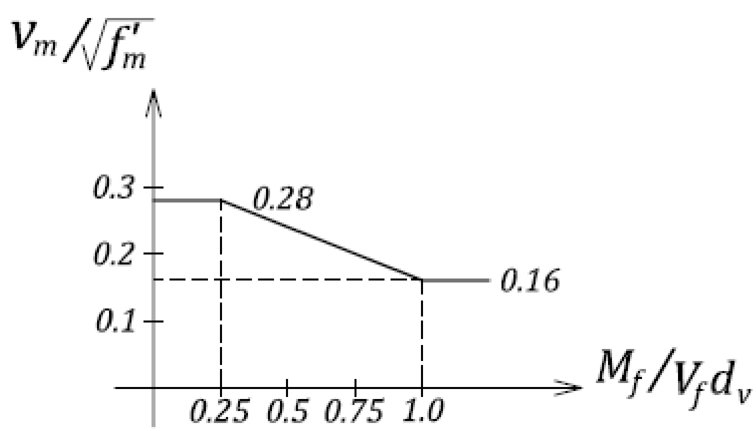


Figure 2-13. Effect of shear span ratio on the masonry shear strength.

For shear walls, the ratio M_f / V_f is equal to the effective height, h_e, at which the resultant shear force V_f acts, thereby causing the overturning moment $M_f = V_f \times h_e$ (see Figure 2-14). The term d_v denotes the effective wall depth, which is equal to a fraction of the wall length, l_w. Hence, $M_f / V_f d_v$ is equal to shear span ratio, h_e / d_v, which is related to the height-to-length aspect ratio.

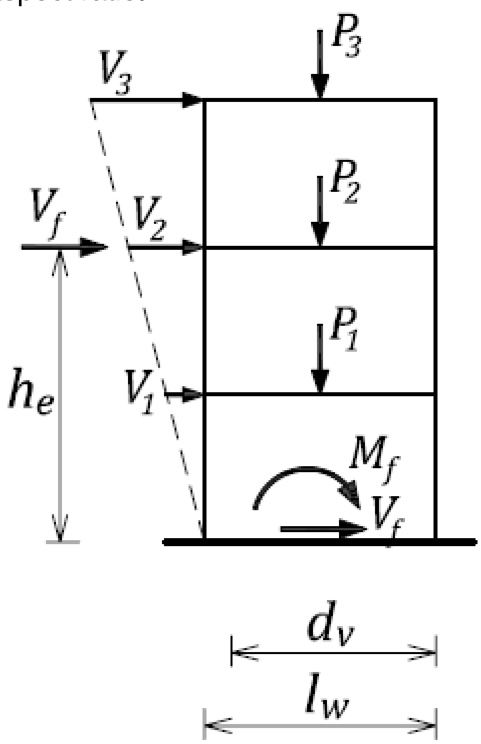


Figure 2-14. Shear span ratio $\frac{h_e}{d_v}$.

<u>Reinforcement shear resistance</u> (V_s):
Reinforcement shear resistance in reinforced masonry shear walls of running bond is mainly provided by horizontal steel bars and/or joint reinforcement. This model assumes that a hypothetical failure plane is at a 45° angle with regard to the horizontal axis, as shown in Figure 2-15a. When diagonal cracking occurs, tension develops in the reinforcing steel crossing the crack. (Before the cracking takes place, the entire shear resistance is provided by the masonry.)

The resistance provided by shear reinforcement is taken as the sum of tension forces developed in steel reinforcement (area A_v) which crosses the crack, as shown in Figure 2-15b. The number of reinforcing bars crossing the crack can be approximately taken equal to d_v / s.

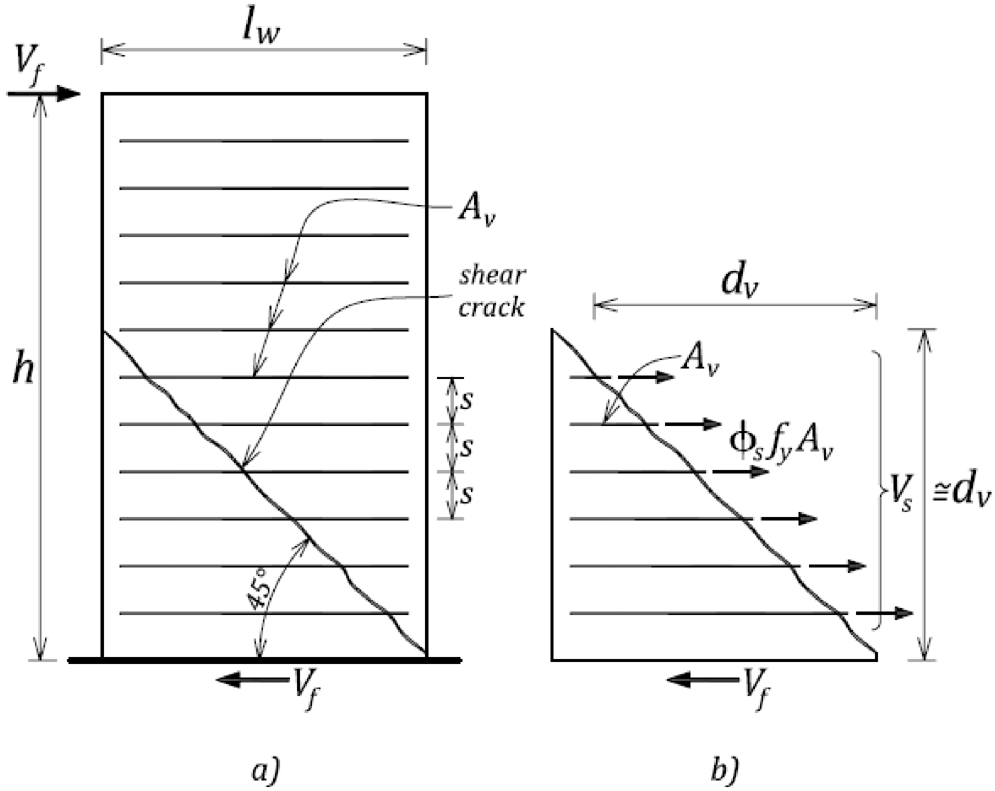


Figure 2-15. Steel shear resistance in flexural walls: a) wall elevation; b) free-body diagram showing reinforcement crossing a diagonal crack.

It appears that the steel reinforcement is less effective in resisting shear in masonry walls than in reinforced concrete walls. This may be due to a rather low masonry bond strength, so that not all bars crossing the assumed failure plane are fully stressed, plus the failure plane may be at an angle of less than 45° in this high moment region. Even in lightly reinforced masonry walls, horizontal reinforcement is less effective than in otherwise similar reinforced concrete walls. It is difficult to exactly estimate the contribution of steel reinforcement toward the shear resistance of masonry walls. Anderson and Priestley (1992) came to the conclusion that the contribution of steel shear reinforcement in a masonry wall is equal to 50% of the value expected in reinforced concrete walls. As a result, they proposed the following equation for the nominal steel shear resistance, V_s, (note that ϕ_s is equal to 1):

$$V_s = 0.5 A_v f_y \frac{d_v}{s}$$

CSA S304.1-04 uses the same V_s equation (4), except that the coefficient 0.6 is used instead of 0.5. Note also that, when 0.6 is multiplied by the ϕ_s value of 0.85, the resulting value is equal to $0.6 \times 0.85 = 0.51 \approx 0.5$.

The contribution of vertical reinforcement to shear resistance in masonry walls is not considered to be significant and it is not accounted for by the CSA S304.1-04 shear design equation. The analysis of experimental test data by Anderson and Priestley (1992) showed an absence of correlation between the wall shear resistance and the amount of vertical steel reinforcement.

2.3.2.2 Squat shear walls

10.10.1.3

Squat shear walls are characterized by a low height/length aspect ratio, h_w/l_w, less than unity. The factored shear resistance of squat shear walls, V_r, should be determined from the same equation as prescribed for flexural walls. To recognize the fact that shear resistance of masonry walls increases with a decrease in the height/length aspect ratio, CSA S304.1-04 prescribes an increased upper limit for the factored shear resistance as follows:

$$V_r \leq \max V_r = 0.4\phi_m\sqrt{f'_m}b_w d_v \gamma_g (2-\frac{h_w}{l_w}) \qquad \frac{h_w}{l_w} \leq 1.0 \qquad (9)$$

Cl.10.10.1.3 also prescribes that this maximum shear resistance can be used only when it is ensured that the shear input to the wall is distributed along the entire length and that a failure of a portion of the wall is prevented. Application is subsequently discussed under Commentary.

Commentary

The first term in equation (9) is equal to the maximum V_r limit for flexural shear walls (equation 8). Equations (8) and (9) have the same value for a wall with the aspect ratio $h_w/l_w = 1.0$. The term $(2-h_w/l_w)$ that accounts for the effect of wall aspect ratio has the minimum value of 1.0 for the aspect ratio of 1.0 and its value increases for squat walls – it is equal to 1.5 for the aspect ratio of 0.5.

Cl.10.10.1.3 prescribes that an increased maximum V_r limit for squat shear walls applies only when the designer can ensure that the shear input to the wall can be distributed along the entire wall length. Earthquake-induced lateral load in a masonry building is transferred from the floor or roof diaphragm into shear walls. Floor and roof diaphragms in masonry buildings range from flexible timber diaphragms to rigid reinforced concrete slab systems. The type of load transfer at the wall-to-diaphragm connection depends on the diaphragm rigidity (see Section 1.5.9.4 for more details).

CSA S304.1-04 Cl.10.15.1.3 requires that a bond beam be placed at the top of the wall, where the wall is connected to roof and floor assemblies. The bond beam therefore acts as a “transfer beam” that ensures a uniform shear transfer atop the wall, as shown in Figure 2-16a (this can be effectively achieved when the vertical reinforcement extends into the beam).

Shear forces are transferred from the top to the base of the wall by means of a compression strut. It should be noted that a majority of experimental studies used specimens with a rigid transfer beam cast on top of the wall, as discussed by Anderson and Priestley (1992). Provision of the top transfer beam (or an alternative means to apply shear force uniformly along the wall length) is required for seismic design of moderately ductile squat shear walls (Cl.10.16.6.2).

Where there is no transfer beam or bond beam at the top of the wall as shown in Figure 2-16b, a partial shear failure of the wall is anticipated. In such cases, the designer cannot take advantage of the increased maximum V_r limit for squat shear walls; the limit pertaining to flexural shear walls should be used instead.

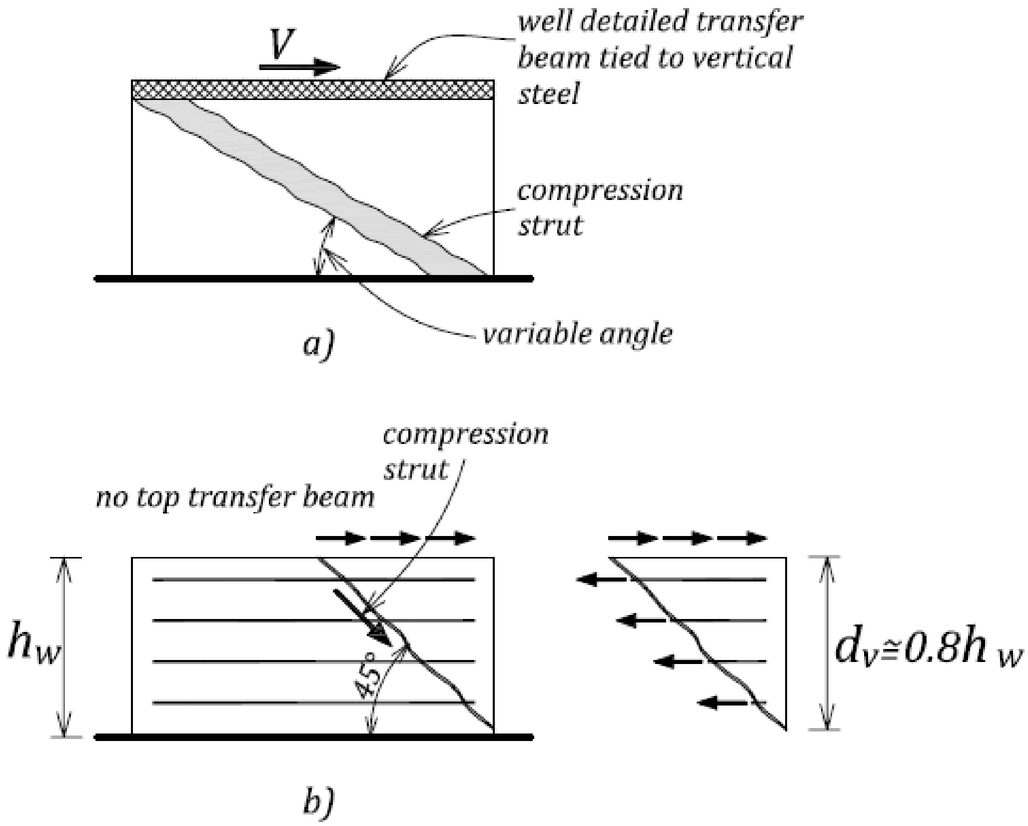


Figure 2-16. Shear failure mechanisms in squat shear walls: a) wall with the top transfer beam – a desirable failure mechanism; b) partial failure of a squat wall without the top beam.

2.3.3 Sliding Shear Resistance

Sliding shear failure may occur before walls fail in the flexural mode. Experimental studies (Shing et al., 1990) have shown that for squat walls sliding shear mechanism can control the failure and prevent the development of their full flexural capacity. This section discusses the sliding shear resistance provisions of CSA S304.1-04 for non-seismic conditions; seismic requirements related to sliding shear resistance will be discussed in Section 2.5.4.6.

10.10.4

Sliding shear failure can occur in both squat and flexural walls; however, it is much more common in squat walls having high shear resistance, V_r. Sliding shear resistance is usually checked at the foundation-to-wall interface (construction joint), but may need to be checked at other sections as well (especially upper portions of multi-storey flexural walls).

10.10.4.1

Sliding shear resistance is generally taken as a frictional coefficient times the maximum compressive force at the sliding plane. In accordance with CSA S304.1-04, the factored in-plane sliding shear resistance, V_r, shall be taken as:

$$V_r = \phi_m \mu P_2 \qquad (10)$$

where

μ is the coefficient of friction
= 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane
= 0.7 for a masonry-to-smooth concrete or bare steel sliding plane
= other (where flashings reduce friction that resists sliding shear, a reduced coefficient of friction accounting for the flashing material properties should be used)

P_2 is the compressive force in the masonry acting normal to the sliding plane, normally taken as $P_2 = P_d + T_y$

$T_y = \phi_s A_s f_y$ the factored tensile force at yield of the vertical reinforcement of area A_s (yield stress f_y)

P_d = axial compressive load on the section under consideration, based on 0.9 times dead load, P_{DL}, plus any axial load acting from bending in coupling beams

Note that A_s denotes the total area of vertical reinforcement crossing the sliding plane for seismic design of limited ductility shear walls and moderately ductile squat shear walls. However, A_s denotes the area of reinforcement in the tension zone only for moderately ductile shear walls (Cl.10.16.5.3.2). Note that, when sliding takes place at the base of the wall, the vertical reinforcement is in the form of dowels. For more details refer to Section 2.5.4.6.

Commentary

When sliding begins, the sand grains in the mortar tend to ride up and over neighbouring particles causing the mortar to expand in the vertical direction. This creates tension (and ultimately yielding) in the vertical reinforcing bars at the interface (note that adequate anchorage of reinforcement on both sides of the sliding plane is necessary to develop the yield stress). As a result, a clamping force is formed between the support and the wall, normally taken equal to $\phi_s A_s f_y$, as shown in Figure 2-17. The shear is then transferred through friction at the interface along the compression zone of the wall.

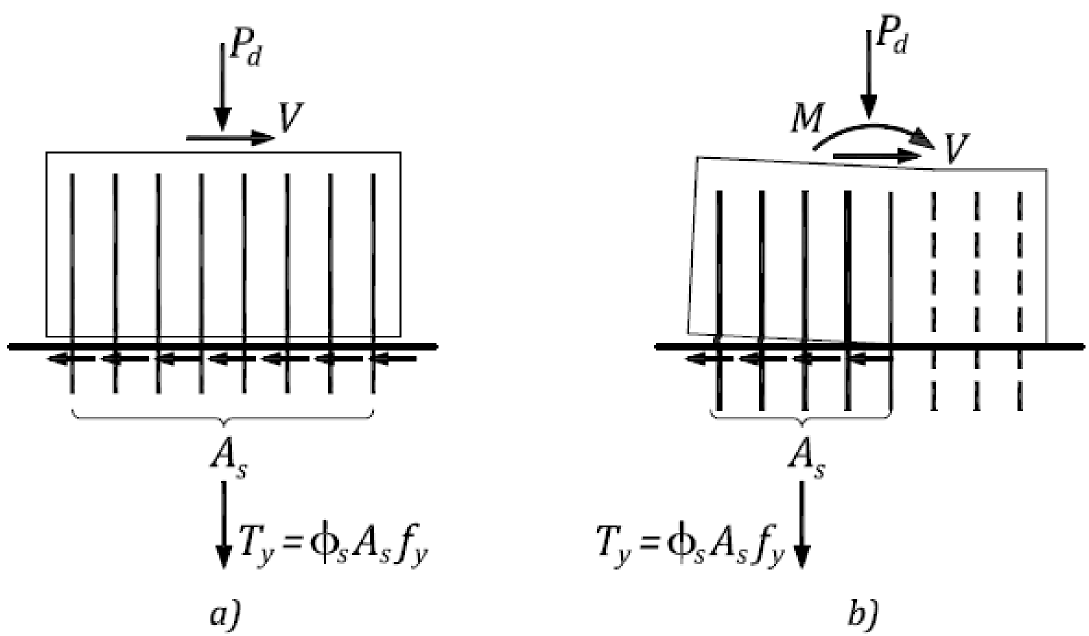


Figure 2-17. In-plane sliding shear resistance in masonry shear walls: a) conventional construction and limited ductility walls; b) moderately ductile walls.

In accordance with CSA S304.1-04, the maximum compression force, P_2 , is usually considered to be equal to the axial load plus the yield strength of the reinforcement/dowels crossing the sliding plane. Since the reinforcement yields in tension, shear resistance of the dowels cannot be included. This assumption is appropriate for walls that are not expected to demonstrate significant ductility.

However, if a wall is subjected to ultimate moment capacity which causes yielding of the compression reinforcement, there is a tendency for this reinforcement to remain in compression to maintain the moment resistance, especially after the wall has been cycled into the yield range once or twice. Thus, when the compression steel remains in compression, and the normal force resisting sliding will be limited to the resultant force in the tension steel, T_y , as shown in Figure 2-17b. This assumption is included in seismic design requirements for moderately ductile walls (to be discussed in Section 2.5.4.6).

Presence of flashing at the base of the wall usually reduces the sliding shear resistance when the frictional coefficient for the flashing-to-wall interface is low (Anderson and Priestley, 1992).

2.3.4 In-Plane Flexural Resistance Due to Combined Axial Load and Bending

Seismic shear forces acting at floor and roof levels cause overturning bending moments in a shear wall, which reach the maximum at the base level. The theory behind the design of masonry wall sections subjected to effects of flexure and axial load is well established and the design methodology is essentially the same as that related to reinforced concrete walls.
Note that CSA S304.1-04 Cl.10.2.8 prescribes the use of reduced effective depth, d , for flexural design of *squat shear walls*, that is:

$$d = 0.67 l_w \leq 0.7h$$

This provision was introduced for the first time in the 2004 edition of CSA S304.1, in order to account for the deep beam-like flexural response of squat shear walls. This provision can be rationalized for non-seismic design, but it should not be used in seismic conditions as all the tension steel is expected to yield, as shown in Figure 2-17b. The wall design according to this provision could give the flexural capacity of the wall larger than permitted according to the capacity design approach.

For a detailed flexural design procedure the reader is referred to Appendix C (Section C.1.1).

2.4 Reinforced Masonry Walls Under Out-of-Plane Seismic Loading

2.4.1 Background

Seismic shaking in a direction normal to the wall causes out-of-plane wall forces that result in bending and shear stresses and may, ultimately, cause out-of-plane collapse of the walls. Note that out-of-plane seismic response of masonry walls is more pronounced at higher floor levels (due to larger accelerations) than in the lower portions of the buildings, as shown in Figure 2-18. When walls are inadequately connected to top and bottom supports provided by floor and/or roof diaphragms, out-of-plane failure is very likely, and may also lead to a diaphragm failure. For more details on wall-to-diaphragm connections, the reader is referred to Section 2.6.7. The design of masonry walls for shear and flexure due to the effects of out-of plane seismic loads is discussed in this section.

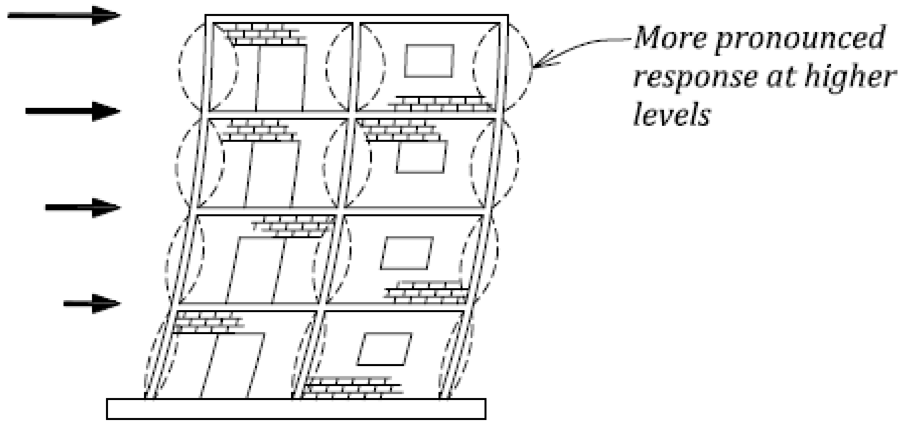


Figure 2-18. Out-of-plane vibration of walls (Tomazevic, 1999, reproduced by permission of the Imperial College Press).

2.4.2 Out-of-Plane Shear Resistance

10.10.2

The factored out-of-plane shear resistance, V_r, shall be taken as:

$$V_r = \phi_m (v_m \cdot b \cdot d + 0.25 P_d) \qquad (11)$$

where

$v_m = 0.16\sqrt{f'_m}$ *MPa units* (Cl.10.10.1.4)

with the following upper limit,

$$V_r \leq \max V_r = 0.4\phi_m \sqrt{f'_m}(b \cdot d) \qquad (12)$$

where
d is the distance from extreme compression fibre to the centroid of tension reinforcement, and
b is the cumulative width of cells and webs within a length not greater than four times the actual wall thickness $(4 \times t)$ around each vertical bar (for running bond), as shown in Figure 2-19a. Note that, for the purpose of this provision, the webs are the cross-walls connecting the face shells of a hollow or semi-solid concrete masonry unit or a hollow clay block (S304.1 Cl.10.10.2).

Commentary

Note that the equation for masonry shear resistance, V_m, is the same for shear walls subjected to in-plane and out-of-plane seismic loading. There is no V_s contribution because the horizontal reinforcement is provided only in the longitudinal direction and it does not contribute to the out-of-plane shear resistance.

In partially grouted walls, the out-of-plane shear design should be performed using a T-shaped wall section, where b denotes the web width (see Figure 2-19a).

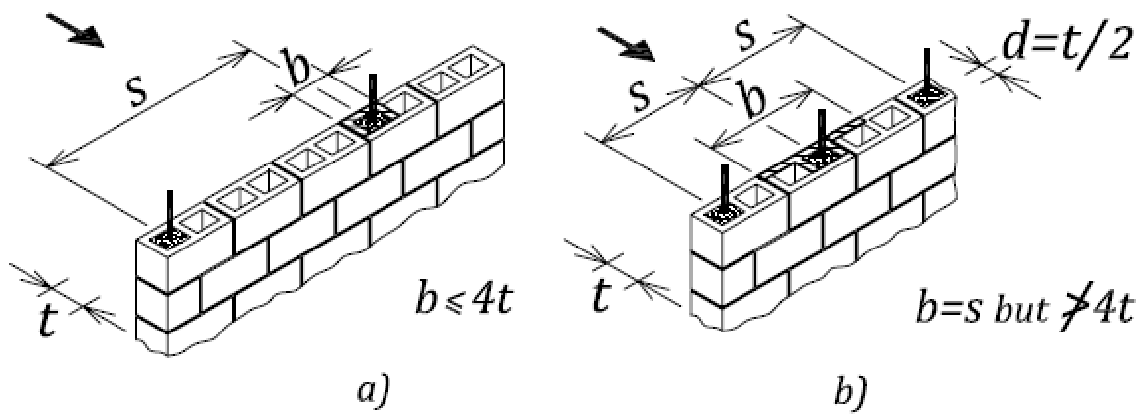


Figure 2-19. Effective width, b, for out-of-plane seismic effects: a) shear, and b) flexure.

2.4.3 Out-of-Plane Sliding Shear Resistance

10.10.4.2

The factored out-of-plane sliding shear resistance, V_r, is calculated from the following equation using the shear friction concept:

$V_r = \phi_m \mu C$ *(13)*

where

μ = the coefficient of friction (same as for the in-plane sliding shear resistance)

C = compressive force in the masonry acting normal to the sliding plane, taken as

$C = P_d + T_y$

T_y = the factored tensile force at yield of the vertical reinforcement detailed to develop yield strength on both sides of the sliding plane. In determining the out-of-plane sliding shear resistance, the entire vertical reinforcement should be taken into account in determining the factored tensile yield force, T_y, irrespective of the wall class and the associated ductility level.

For more details refer to the discussion on the sliding shear resistance of shear walls under in-plane seismic loading (Section 2.3.3).

2.4.4 Out-of-Plane Section Resistance Due to Combined Axial Load and Bending

Masonry walls subjected to out-of-plane seismic loading need to be designed for combined effects of bending and axial gravity loads. For flexural design purposes, wall strips of predefined width b (S304.1-04 Cl.10.6.1) are treated as beams spanning between the lateral supports. When the walls span in the vertical direction, floor and/or roof diaphragms provide lateral supports. Walls can also span horizontally, in which case lateral supports need be provided by cross walls or pilasters. For detailed design procedures, the reader is referred to Section C.1.2 in Appendix C. It should be noted that, for the purpose of out-of-plane seismic design, the permitted maximum compressive strain in the masonry is equal to 0.003 (note that this is an arbitrary value set for the purpose of the analysis). CSA S304.1 does not require ductility check, because the mechanism of failure is different for the in-plane and out-of-plane seismic resistance and the wall is not expected to undergo significant rotations at the locations of maximum bending moments. Very large curvatures would be required to cause compression failure of the masonry, corresponding to a high strain gradient over a very small length (equal to the wall thickness). Consequently, there is no need to use the reduced compressive strain limit of 0.0025 for this load condition.

10.6.1

For the case of out-of-plane bending, the effective compression zone width, b , used with each vertical bar in the design of walls with vertical reinforcement shall be taken as the lesser of (see Figure 2-19 b)

a) spacing between vertical bars s , or
b) four times the actual wall thickness $(4 \times t)$

Note that the discussion on out-of-plane stability issues is outside the scope of this document and it is covered elsewhere (see Drysdale and Hamid, 2006).

2.5 Seismic Design Considerations for Reinforced Masonry Shear Walls

2.5.1 Background

The focus of this section is mainly on the seismic design and detailing requirements for different classes of ductile reinforced masonry shear walls. General seismic design requirements for ductile shear walls are stated in Cl. 10.16. In the 1994 edition of this standard (CSA S304.1-94), seismic design requirements for ductile reinforced shear walls were included in Cl. 5.2.2, 6.3.3, and Appendix A. Changes in seismic design provisions between the two editions of CSA S304.1 will also be discussed in this section. Shear walls with conventional construction do not require seismic detailing since these walls are not designed for ductile performance.

It should be noted that NBCC 2005 also identifies moment-resisting frames with conventional construction as a possible masonry SFRS, however seismic design of moment-resisting masonry frames is beyond the scope of this document.

2.5.2 Capacity Design Approach

10.16.3.3

According to the design approach stated in Cl.10.16.3.3, a ductile reinforced masonry shear wall must be designed to resist a shear force not less than the shear that is present when the wall develops a plastic hinge mechanism.

Every structure or a structural component has several possible modes of failure, some of which are ductile while others are brittle. Satisfactory seismic response of structures requires that brittle failure modes be avoided. This is accomplished through the application of a *capacity design approach*, which has been used for seismic design of reinforced concrete structures since the 1970's (Park and Paulay, 1975). The objective of the capacity design approach is to force the structure to yield in a ductile manner without failing at the expected displacements (including other components of the structure, such as columns). At the same time, the rest of the structure needs to remain strong enough, say in shear, or flexible enough not to fail under gravity loads at these displacements.

This concept can be explained using an example of a chain shown in Figure 2-20, which is composed of brittle and ductile links. When subjected to force, F , and if the brittle link is weakest, the chain will fail suddenly without significant deformation (see Figure 2-20a). If a ductile link is the weakest, the chain will show significant deformation before failure and may not fail or break if the deformation is not too great (see Figure 2-20b). The structural designer is responsible for ensuring that the structure experiences a desirable ductile response when exposed to the design earthquake, that is, an earthquake of expected intensity for the specific building site location.

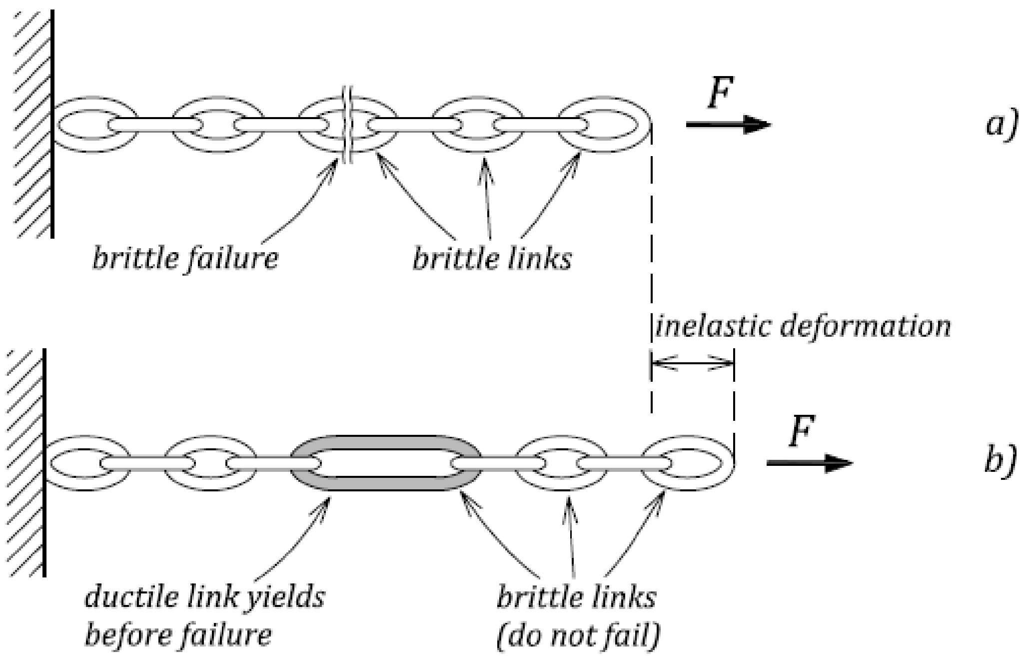


Figure 2-20. Chain analogy for capacity design: a) brittle failure; b) ductile failure.

The capacity design approach can be applied to seismic design of reinforced masonry shear walls. The key failure modes in reinforced masonry walls include flexural failure (which is ductile and therefore desirable in seismic conditions) and shear failure (which is brittle and should be avoided in most cases). For a detailed discussion of masonry failure modes refer to Section 2.3.1.

Note that the following three resistance "levels" are used in seismic design of masonry shear walls:

- *Factored resistances* M_r and V_r , determined using appropriate material resistance factors, that is, ϕ_m = 0.6 and ϕ_s = 0.85, and specified material strength;
- *Nominal resistances* M_n and V_n , determined without using material resistance factors, that is, ϕ_m = 1.0 and ϕ_s = 1.0, and specified material strength;
- *Probable resistances* M_p and V_p , determined without using material resistance factors; stress in the tension reinforcing is taken equal to $1.25 f_y$, and the masonry compressive strength is equal to f'_m .

In relation to the probable resistance parameters discussed above, it needs to be clarified that the flexural resistance of a masonry shear wall is usually governed by the yield strength of the reinforcement, f_y , while the masonry compressive strength, f'_m , has a much smaller influence.

Thus the probable resistances are determined by taking the masonry strength equal to f_m' and the real yield strength of the reinforcement equal to 1.25 the specified strength, that is, $1.25 f_y$.

Consider a masonry shear wall subjected to an increasing lateral seismic force, V, and the corresponding deflection shown in Figure 2-21a. The wall has been designed for a "design shear force" shown by a horizontal line. However, the actual wall capacity typically exceeds the design force, and the wall is expected to deform either in a flexural or shear mode at higher load levels. Conceptual force-deflection curves corresponding to shear and flexural failure mechanisms are also shown on the figure. These curves are significantly different: a shear failure mechanism is characterized by brittle failure at a small deflection, while a ductile flexural mechanism is characterized by significant deflections before failure takes place.

The earthquake will cause significant lateral deflections, which are more or less independent of the strength. If the governing failure mode corresponding to the lowest capacity occurs at a smaller deflection, the wall will fail in that mode. For example, the wall shown in Figure 2-21a is expected to experience shear failure since the maximum force corresponding to shear failure is lower than the force corresponding to flexural failure.

Consider a wall which is designed to fail in shear when the shear resistance, V_A, and corresponding displacement Δ_A have been reached, and to fail in flexure when the shear force, V_B, and corresponding displacement Δ_B have been reached (see Figure 2-21b). If the wall is weaker in flexure than in shear, that is, $V_B < V_A$, the shear failure will never take place. In this case, a ductile link corresponding to the flexural failure is the weakest and governs the failure mode. Such a wall will experience significant deflections before the failure (note that $\Delta_B \geq \Delta_A$); this is a desirable seismic performance.

However, suppose that the wall flexural resistance is higher (this is also known as "flexural overstrength") and now corresponds to moments associated with the shear force, V_C, as shown in Figure 2-21c. Now the wall will fail in shear at the force, V_A, and will never reach the force V_C. This is not a desirable wall design, since shear failure is brittle and sudden and should be avoided. Thus, it is important that the member shear strength be greater than its flexural overstrength, as we will discuss later in this section.

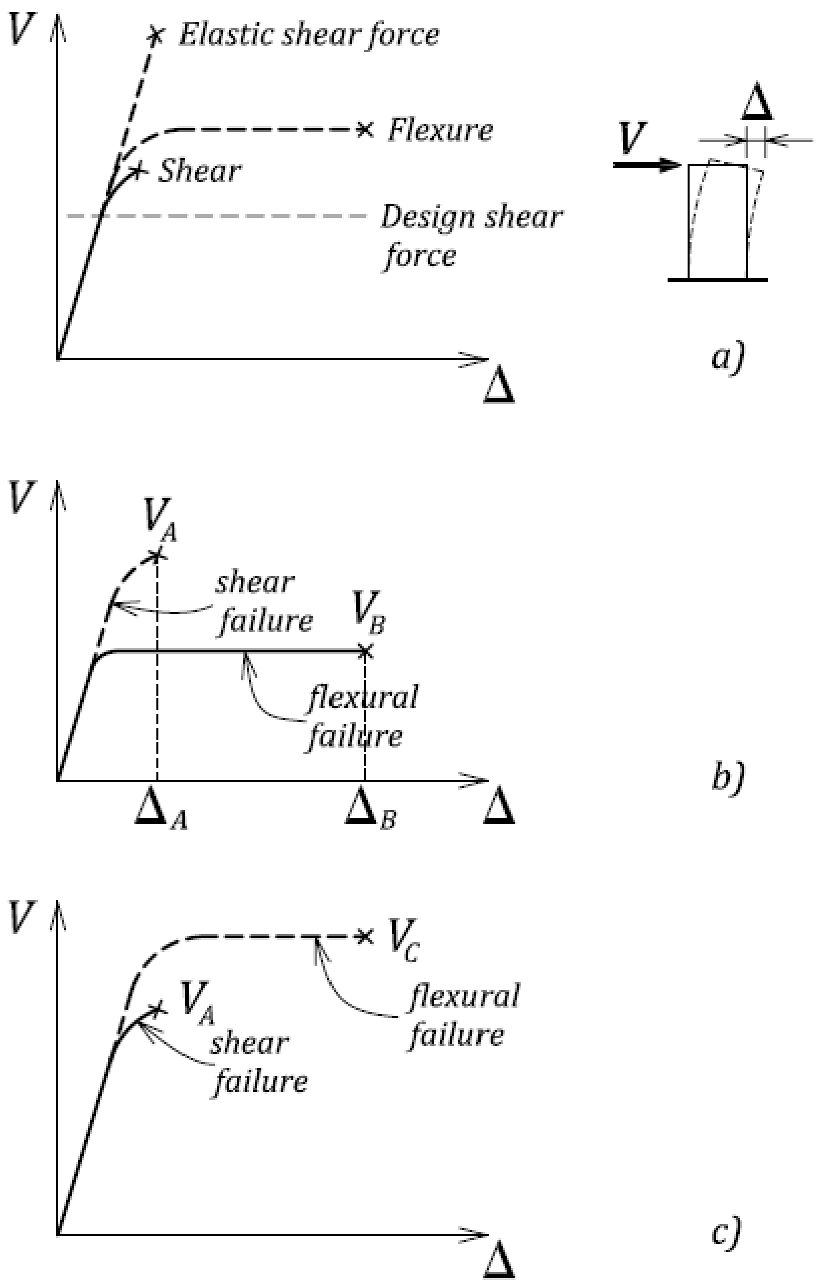


Figure 2-21. Shear force-deflection curves for flexural and shear failure mechanisms: a) a possible design scenario; b) flexural mechanism governs; c) shear mechanism governs (adapted from Nathan).

The last example demonstrates that making the wall "stronger" can have unintended adverse effects, and can change the failure mode from ductile flexural mode (good) to brittle shear mode (bad). Thus a designer should not indiscriminately increase member moment capacity without also increasing its shear capacity. According to the capacity design approach, ductile flexural failure will be assured when the shear force corresponding to the upper bound of moment resistance at the critical wall section is less than the shear force corresponding to the lower bound shear resistance of the shear failure mechanism. This will be explained with an example of the shear wall shown in Figure 2-22.

When the moment at the base is equal to the nominal moment resistance, M_n (this is considered to be an upper bound for the moment resistance value and it is explained below), the corresponding shear force acting at the effective height is equal to

$$V_{nb} = M_n / h_e$$

or

$$V_{nb} = M_n * (V_f / M_f)$$

as shown in Figure 2-22a. V_{nb} denotes the resultant shear force corresponding to the development of nominal moment resistance, M_n, at the base of the wall. To ensure the development of a ductile flexural failure mode, V_{nb} must be less than the corresponding factored shear resistance, V_r, as shown in Figure 2-22b.

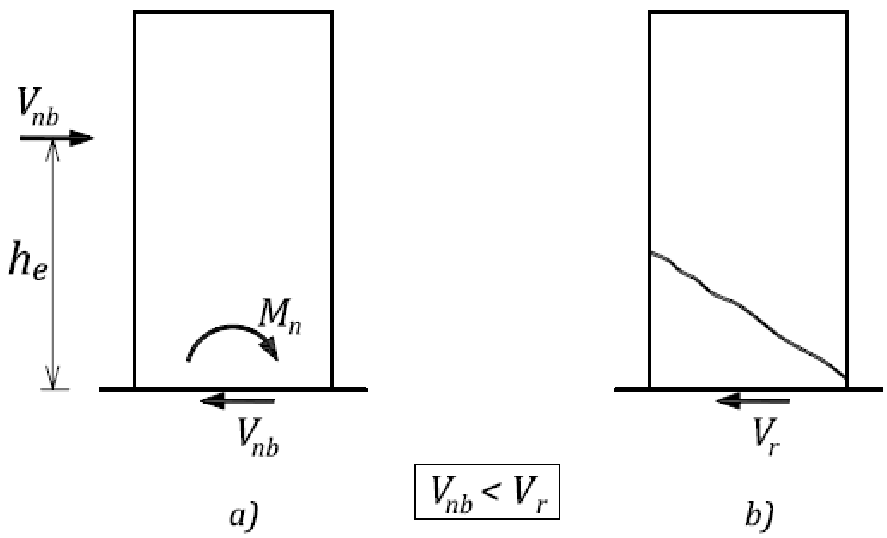


Figure 2-22. Comparison of shear forces at the base of the wall: a) shear force corresponding to the nominal flexural resistance, and b) shear force equal to the shear resistance.

The key capacity design concepts as applied to the design of masonry structures can be summarized as follows:

1. For the design of reinforced masonry walls, the factored shear resistance, V_r, should be greater than the shear due to effects of factored loads, but not less than the shear corresponding to the development of nominal moment capacity, M_n, of the wall system at its plastic hinge location (see Section 2.5.4.2 for detailed discussion on plastic hinges). The nominal moment capacity need not be taken larger than the load calculated with design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.0.

2. It is also important that other structural members which are not a part of the SFRS are able to undergo the same lateral displacements as the SFRS members without experiencing brittle failure.

Although CSA S304.1-04 Cl. 10.16.3.3 requires that the capacity design approach should be applied to ductile masonry walls, additionally, it is recommended that this approach be applied to all reinforced masonry shear walls. As a minimum, factored shear resistance, V_r, should be not less than the shear corresponding to the factored moment resistance, M_r, of the wall system at its plastic hinge location.

2.5.3 Ductile Seismic Response

A prime consideration in the seismic design is the need to have a structure capable of deforming in a ductile manner when subjected to several cycles of lateral loading well into the inelastic range. *Ductility* is a measure of the capacity of a structure or a member to undergo deformations beyond yield level while maintaining most of its load-carrying capacity. Ductile structural members are able to absorb and dissipate earthquake energy by inelastic (plastic) deformations, which are usually associated with permanent structural damage.

The concept of ductility and ductile response was introduced in Section 1.4.3. Key terms related to ductile seismic response of masonry shear walls, including ductility ratio, curvature, plastic hinge, etc. are discussed in detail in Section B.2. It is very important for a structural designer to have a good understanding of these concepts before proceeding with the seismic design and detailing of ductile masonry walls in accordance with CSA S304.1-04.

2.5.4 CSA S304.1-04 Seismic Design Requirements

2.5.4.1 Classes of reinforced masonry shear walls

Table 4.1.8.9 of NBCC 2005 identifies the following five classes of masonry walls based on the expected seismic performance quantified by means of the ductility-related force modification factor, R_d (see also Section 1.5.5):

1. Unreinforced masonry and other masonry structural systems not listed below ($R_d = 1.0$)
2. Shear walls with conventional construction ($R_d = 1.5$)
3. Limited ductility shear walls ($R_d = 1.5$)
4. Moderately ductile shear walls ($R_d = 2.0$)
5. Moderately ductile squat shear walls ($R_d = 2.0$) (note that this wall class was not identified in NBCC 2005 Table 4.1.8.9, however specific design and detailing provisions are stipulated by S304.1-04 Cl.10.16.6).

The last three classes are referred to as "ductile shear walls". Although shear walls with conventional construction and shear walls of limited ductility have the same $R_d = 1.5$ value, the difference between these walls is that the limited ductility walls have additional detailing requirements, and so can be used in taller structures. The same value of overstrength factor, R_o, of 1.5 is prescribed for all the above listed wall classes except for unreinforced masonry where R_o is equal to 1.0.

CSA S304.1-04 Clause 4.6 outlines the classes of reinforced masonry walls, while the seismic design provisions are prescribed in Clause 10.

New seismic design provisions for reinforced masonry walls in NBCC 2005 and CSA S304.1-04 are summarized below (note that new terms were introduced to provide consistency with the concrete design standard, CSA A23.3-04):

1. New term "shear walls with conventional construction" (previously "reinforced masonry")
2. New term "limited ductility shear walls" (S304.1-04, Cl.10.16.4)
3. New term "moderately ductile shear walls" (S304.1-04, Cl.10.16.5); note that S304.1-94 used term "masonry with nominal ductility"
4. New requirements for "moderately ductile squat shear walls" (S304.1-04, Cl. 10.16.6)
5. Height-to-thickness ratio restrictions introduced for ductile shear walls: limited ductility shear walls, moderately ductile shear walls, and moderately ductile squat shear walls.

Seismic design and detailing requirements for various masonry Seismic Force Resisting Systems (SFRSs) are summarized in Table 2-1. In accordance with NBCC 2005 Sent.4.1.8.1.1), seismic design must be performed when $S(0.2) > 0.12$. Also, it is permissible to use unreinforced masonry constructions at sites where $I_E F_a S_a(0.2) < 0.35$ (S304.1-04, Cl.4.5.1). Minimum CSA S304.1-04 seismic reinforcement requirements for masonry walls are summarized in Table 2-2.

Note that squat shear walls are most common in low-rise masonry construction, ranging from warehouses, school buildings, and fire halls. Some of these buildings, for example fire halls, are considered to be post-disaster facilities according to NBCC 2005. A new restriction has been introduced in NBCC 2005 (Sent. 4.1.8.10.2), by which post-disaster facilities require an SFRS with R_d of 2.0 or higher. An implication of this provision is that squat shear walls in post-disaster buildings be designed following the CSA S304.1-04 provisions for "moderately ductile squat shear walls".

Table 2-1. Summary of Seismic Design and Detailing Requirements for Masonry SFRSs in CSA S304.1-04

Type of SFRS	**Common applications**	R_d	R_o	**Expected seismic performance**	**Summary of CSA S304.1-04 design requirements**	**CSA S304.1 reinforcing and detailing requirements**
Unreinforced masonry	Low-rise buildings located in low seismicity regions	1.0	1.0	Potential to form brittle failure modes	▪ Can be used only at sites where $I_E F_a S_a(0.2) < 0.35$ ▪ Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads	Reinforcement not required
Shear walls with conventional construction	Used for most building applications	1.5	1.5	Design to avoid soft stories or brittle failure modes	Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads	Minimum seismic reinf. requirements (Cl.10.15.2.2) apply if $I_E F_a S_a(0.2) \geq 0.35$ otherwise follow minimum non-seismic reinf. requirements (Cl.10.15.1.1)
Limited ductility shear walls	Used only when required to comply with the NBCC 2005 height restrictions (Table 4.1.8.9)	1.5	1.5	Limited dissipation of earthquake energy by flexural yielding in specified locations; shear failure to be avoided	▪ Can be used for shear wall design when $h_w/l_w \geq 1.0$ ▪ Walls to be designed using factored moment resistance such that plastic hinges develop without shear failure and local buckling ▪ Sliding shear failure at joints to be avoided ▪ Expected ductility level to be verified ▪ Wall height-to-thickness ratio restrictions prescribed	Minimum seismic reinforcement requirements (Cl.10.15.2.2) must be satisfied, as well as seismic detailing requirements for limited ductility walls (Cl.10.16.4)

Type of SFRS	Common applications	R_d	R_o	Expected seismic Performance	Summary of CSA S304.1-04 design requirements	CSA S304.1 reinforcing and detailing requirements
Moderately ductile shear walls	Used for post-disaster or high risk buildings or where $R_d = 2.0$ is desired	2.0	1.5	Dissipation of earthquake energy by ductile flexural yielding in specified locations; shear failure to be avoided	▪ Walls to be designed using factored moment resistance such that plastic hinges develop without shear failure and local buckling 50% reduction in masonry resistance for V_r calculations ▪ Sliding shear failure at joints to be avoided (additional requirements compared to limited ductility walls) ▪ Expected ductility level to be verified ▪ Wall height-to-thickness ratio restrictions more stringent than limited ductility walls	Minimum seismic reinforcement requirements (Cl.10.15.2.2) must be satisfied, as well as seismic detailing requirements for moderately ductile walls (Cl.10.16.5)
Moderately ductile <u>squat</u> shear walls	Used for post-disaster buildings or where $R_d = 2.0$ is desired	2.0	1.5	Top transfer beam to ensure uniform shear transfer along the wall length; some flexural yielding expected	▪ Walls to be designed using factored moment resistance; shear failure and local buckling to be avoided ▪ Sliding shear failure at joints to be avoided ▪ Wall height-to-thickness ratio restrictions less stringent than limited ductility walls	Minimum seismic reinforcement requirements (Cl.10.15.2.2) must be satisfied, as well as special reinforcement requirements for moderately ductile squat shear walls

2.5.4.2 Plastic hinge region

10.16.4.1.1
10.16.5.2.1

The required extent of the plastic hinge region above the base of a shear wall in the vertical direction (also referred to as plastic hinge length, l_p) prescribed by CSA S304.1-04 is as follows (see Figure 2-23):

1. Limited ductility shear walls (Cl.10.16.4.1.1):
   l_p = greater of $l_w/2$ or $h_w/6$
2. Moderately ductile shear walls (Cl.10.16.5.2.1):
   l_p = greater of l_w or $h_w/6$

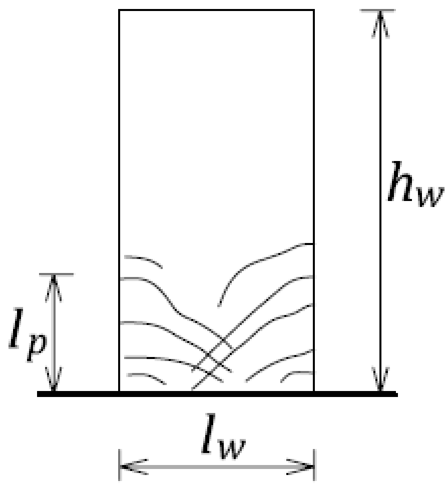


Figure 2-23. Plastic hinge length l_p.

10.16.4.1.3

Masonry within the plastic hinge region must be fully grouted. This provision applies to limited ductility and moderately ductile shear walls in running bond. Note that this provision applies to moderately ductile shear walls by way of Cl.10.16.5.1. Refer to Section 2.6.3.5 for the discussion on plastic hinge requirements related to stack pattern walls.

Commentary

According to CSA S304.1-04 Cl. 10.16.2, the *plastic hinge* is the region of the member where inelastic flexural curvatures occur. In reinforced masonry shear walls which are continuous along the building height, this region is located at the base of the walls, as shown in Figure 2-23. Plastic hinge length can be defined as a fraction of the wall height. In taller flexural walls (three stories or higher), this region can be up to one storey high (usually located at the first storey level). In low-rise buildings, this length is smaller, but it does exist even in squat shear walls when they are subjected to combined effects of axial load and bending and show flexure-dominated response.

The ability of a plastic hinge to sustain these plastic deformations will determine whether a structural member is capable of performing at a certain ductility level. The extent of the plastic hinge region is usually termed the *plastic hinge length,* l_p , although in the case of vertical elements such as walls and columns, it would be more appropriate to refer to it as the plastic hinge height. The l_p value depends on the moment gradient, wall height, wall length, and the level of axial load.

Note that the CSA S304.1-prescribed plastic hinge length values are intended for detailing purposes, and that smaller l_p values should be used for curvature and deflection calculations. There are a few different equations for estimating the value of l_p to be used in curvature calculations, and the recommended on is that proposed by Corley (1966):

$$l_p = 0.5d + 0.032 h_w \big/ \sqrt{d}$$

where $d = 0.8 l_w$ for rectangular walls

CSA S304.1-04 provisions for plastic hinge length, to be used in detailing, are in line with the research findings and codes in other countries. For example, in the New Zealand Masonry Standard NZS 4230:2004 (SANZ, 2004), Cl. 7.4.3 prescribes the plastic hinge length to be the greater of l_w, $h_w/6$, or 600 mm. Findings of a research study by Shing et al.(1990a) indicated that the plastic hinge length is in the order of $h_w/6$.

CSA S304.1-04 plastic hinge length requirements for moderately ductile shear walls (Clause 10.16.5.2.1) are the same as the corresponding CSA S304.1-94 requirements for shear walls with nominal ductility (Clause A5.1).

Design and detailing of plastic hinge regions in ductile masonry shear walls is critical and will be discussed in the following sections. These regions are usually heavily reinforced, and it is critical to ensure proper anchorage of reinforcement. Open-end H-blocks may simplify construction in these regions.

Plastic hinge regions of ductile masonry walls must be fully grouted. Observations from past damaging earthquakes (e.g. 1994 Northridge, California earthquake, Magnitude 6.7) that caused damage to masonry buildings, have shown that the quality of grout placement, and the bond of the grout to the masonry units and reinforcement have a strong influence on the performance of reinforced masonry structures. Some reinforced block walls with large voids around reinforcing bars suffered severe damage (TMS, 1994). CSA S304.1-04 grouting requirements for ductile masonry walls are the same as S304.1-94 requirements related to shear walls with nominal ductility (Clause A5.3). Grout in accordance with CSA A179-04, “Mortar and Grout for Unit Masonry”, offers sufficient strength.

2.5.4.3 Ductility check

10.16.4.1.4 10.16.5.2.3

CSA S304.1-04 prescribes the following two ductility requirements for reinforced masonry shear walls:

1. the neutral axis depth/wall length ratio, c/l_w, should be within the following limits:

   a) For limited ductility shear walls (Cl.10.16.4.1.4):

   $c/l_w < 0.2$ when $h_w/l_w < 6$

b) For moderately ductile shear walls (Cl.10.16.5.2.3):

$c/l_w < 0.2$ when $h_w/l_w < 4$

$c/l_w < 0.15$ when $4 \leq h_w/l_w < 8$

2. if these requirements are not satisfied, the maximum compressive strain in the masonry in the plastic hinge region must be shown to not exceed 0.0025.

Commentary

For the purpose of the ductility check, the strain level in the masonry compression zone is limited to 0.0025. The intent is to limit deformations and the related damage in the highly stressed zone of a wall section.

Whether a structural member is capable of sustaining inelastic deformations consistent with an expected displacement ductility ratio, μ_Δ, will depend on the ability of its plastic hinge region to sustain corresponding plastic rotations. Plastic hinge rotations will depend on available curvature ductility, μ_φ, and the expected plastic hinge length. Refer to Section B.2 for a detailed explanation of curvature ductility and the relationship between curvature ductility and displacement ductility ratio.

It is important for a structural designer to have a sense for curvature ductility and its effect upon the ductile seismic performance. For example, a wall section shown Figure 2-24a is lightly reinforced and has a small axial compression (or tension) load. A small flexural compression zone will be required due to the light reinforcement, thus the neutral axis depth, c_1, will be small relative to the wall length (note the corresponding strain distribution - line 1 shown in Figure 2-24b). As a result, curvature, which is the slope of line 1, will be large and usually adequate to accommodate the plastic hinge rotations imposed on a structure during a major earthquake. However, when the wall is heavily reinforced and has significant axial compression load, a large flexural compression zone will be required, thus resulting in a relatively large neutral axis depth, c_2, as shown in Figure 2-24b (note the corresponding strain distribution - line 2 on the same diagram). For the same maximum strain in the concrete, the curvature (given by the slope of line 2) is much less than for lightly loaded wall. Thus the curvature ductility of the lightly loaded wall is much greater than the heavily loaded wall.

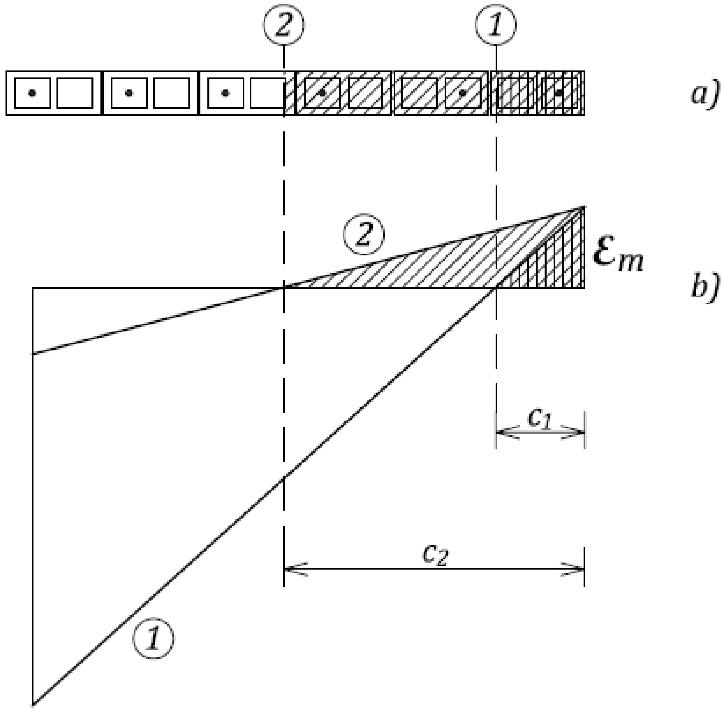


Figure 2-24. Strain distribution in a reinforced masonry wall at the ultimate: a) wall section; b) strain distribution.

Therefore, the ratio of neutral axis depth, c , relative to the wall length, l_w , that is, c/l_w ratio, is an indicator of the curvature ductility in a structural component. The c/l_w limits for ductile shear walls prescribed by CSA S304.1-04 cover most cases and save designers from performing time-consuming ductility calculations.

Where the c/l_w limit is not satisfied for a specific design, the designer may undertake detailed calculations to confirm that the ductility requirements have been met. Refer to Section B.2 for further guidance on detailed calculations related to the ductility requirements, and also Example 5a in Chapter 4 for a design application. In order to meet the S304.1-04 ductility requirements, the designer may want to consider an increase in the masonry strength or the wall thickness, or use flanged shear walls. Flanged wall sections will be discussed in Section 2.6.6.

<u>CSA S304.1-94 ductility check</u>

CSA S304.1-94 Clause A7 required a ductility check for shear walls with nominal ductility. The maximum masonry compression strain of 0.0025 was the same as the 2004 standard. The neutral axis depth requirement stated that

$$c/l_w < 0.2 \text{ when } h_w/l_w < 3$$

It can be concluded that no significant changes were made to the ductility check in CSA S304.1-04. The same c/l_w limit of 0.2 applies to both moderately ductile and limited ductility walls for h_w/l_w ratio values in the order of 4.0 or less, which is characteristic of low- to medium-rise masonry buildings.

As a reference, a discussion on the ductility check provisions of international standards is included in Section B.3.

2.5.4.4 Wall height-to-thickness ratio restrictions

10.16.4.1.2 10.16.5.2.2 10.16.6.3

CSA S304.1-04 prescribes the following height-to-thickness (h/t) limits for the compression zone in plastic hinge regions of ductile shear walls:

1. *Conventional construction*
   Slenderness limits and design procedures for masonry walls need to be followed (Cl.10.7.3.3) - it is possible to design walls with kh/t ratio greater than 30
2. *Limited ductility shear walls* (Cl.10.16.4.1.2):
   $h/(t+10) < 18$
3. *Moderately ductile shear walls* (Cl.10.16.5.2.2):
   $h/(t+10) < 14$
4. *Moderately ductile <u>squat</u> shear walls* (Cl.10.16.6.3):
   $h/(t+10) < 20$ (unless it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability).

Note that h denotes the unsupported wall height (between the adjacent horizontal supports), kh denotes the effective buckling length, and t denotes the actual wall thickness (e.g. 140 mm, 190 mm, 240 mm, etc.).

Commentary

The purpose of this provision is to prevent instability due to out-of-plane buckling of shear walls when subjected to combined effects of in-plane axial loads and bending moments, as shown in Figure 2-25. This phenomenon is associated not only with compression in the masonry, but also with the compression stresses in the flexural reinforcement that has previously experienced large inelastic tensile strains. According to Paulay (1986), this instability occurs when the neutral axis depth, c, is large, as illustrated in Figure 2-24 (see depth c_2), and the plastic hinge region at the base of the wall (length l_p) is large (one storey high or more); this is characteristic of taller reinforced masonry shear walls.

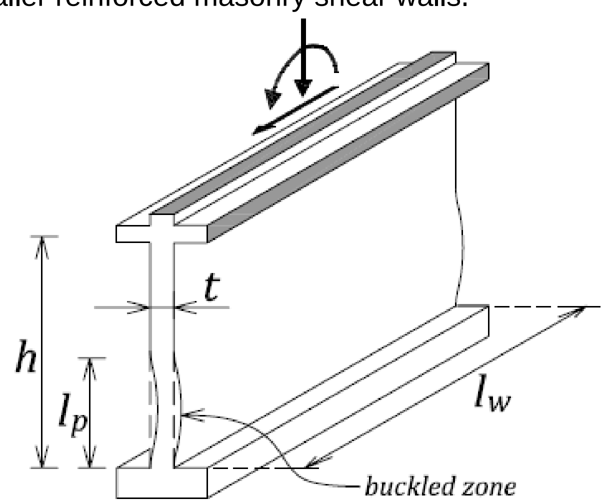


Figure 2-25. Out-of-plane instability in a shear wall subjected to in-plane loads (adapted from Paulay and Priestley, 1993, reproduced by permission of the American Concrete Institute).

A rational explanation for this phenomenon was first presented by Paulay (1986). When the wall experiences large curvature ductility, large tensile strains will be imposed on vertical bars placed at the extreme tension edge of the section. At this stage, uniformly spaced horizontal cracks of considerable width develop over the plastic hinge length (see Figure 2-26a). During the subsequent unloading, the tensile stresses in these bars reduce to zero. Change in the lateral load direction will eventually cause an increase in the compression stresses in these bars. Unless the cracks close, the entire internal compression within the section must be resisted by the vertical reinforcement, as shown in Figure 2-26b and d. At that stage, out-of-plane displacements may increase rapidly as the stiffness of the vertical steel to lateral deformation is small, thereby causing the out-of-plane instability. However, if the cracks close before the entire portion of the wall section previously subjected to tension becomes subjected to compression, masonry compressive stresses will develop in the section, the stiffness to lateral deformation is increased and the instability may be avoided (see Figure 2-26c and e). Refer to Section B.4 for a detailed discussion of the wall height-to-thickness ratio restrictions, and the analysis procedure developed by Paulay and Priestley (1992, 1993).

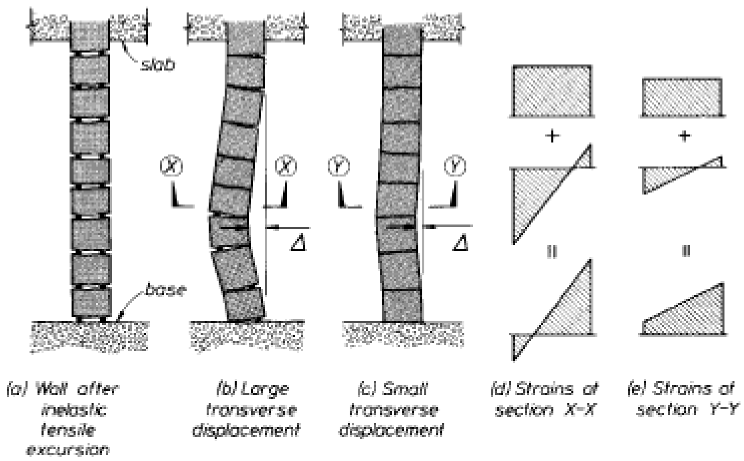


Figure 2-26. Deformations and strain patterns in a buckled zone of a wall section (Paulay,1986, reproduced by permission of the Earthquake Engineering Research Institute).

<u>Implications for seismic design</u>
This provision was first introduced in the 1994 edition of CSA S304.1 (Cl.A5.2 related to “nominally” ductile walls) and it is identical to the current CSA S304.1-04 provision for “moderately” ductile walls.

The height-to-thickness restrictions have significant effect on the required wall thickness in the plastic hinge region of a shear wall (usually located at the base of the wall). According to CSA S304.1-04, the clear (unsupported) height (h) limits for standard concrete block walls (190 mm nominal thickness) are as follows:

1. Limited ductility shear walls: maximum $h = 18(190 + 10) = 3600$ mm
2. Moderately ductile shear walls: maximum $h = 14(190 + 10) = 2800$ mm
3. Moderately ductile squat shear walls: maximum $h = 20(190 + 10) = 4000$ mm

The CSA S304.1-04 height-to-thickness restrictions for limited ductility and moderately ductile shear walls must be followed and cannot be relaxed according to the current version of CSA S304.1. However, Cl.10.16.6.3 states that the h/t ratio limit for moderately ductile <u>squat</u> shear walls can be relaxed, if it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability. A possible solution involves the provision of flanges at wall ends. However, the out-of-plane stability of the compression zone, which includes the flange and sometimes a portion of the web, must be adequate. This check is demonstrated in Example 4c (Chapter 4), where a moderately ductile squat shear wall with the h/t ratio of 33 and added flanges at its ends has been shown to satisfy the CSA S304.1 out-of-plane stability requirement.

The following analysis presents one method of checking if the flanged wall provides sufficient stiffness to prevent out-of-plane instability. For the purpose of this check, a wall can be considered as lightly loaded when the compressive stress f_c, due to the dead load (corresponding to the axial load, P_{DL}), is less than $0.1 f'_m$, that is,

$$f_c = \frac{P_{DL}}{l_w t} < 0.1 f'_m$$

Consider a wall section with flanges added at both ends to enhance the out-of-plane stability shown in Figure 2-27a. The wall is subjected to the factored axial load P_f, the bending moment M_f, and is reinforced with both concentrated reinforcement of area A_c, at each end, and distributed reinforcement along the wall length (total area A_d).

The effective flange width, b_f, can be initially estimated, and then revised if the out-of-plane stability is not satisfactory. A good initial minimum estimate would be

$$b_f \approx 2t$$

where t denotes the wall thickness (see Figure 2-27b). Note that this is an iterative procedure and the flange width may need to be increased to satisfy the stability requirements.

The buckling resistance of the compression zone should be checked according to the procedure described below.

First, the area of the compression zone A_L can be determined from the equilibrium of vertical forces shown in Figure 2-27a:

$$P_f + T_1 + T_2 - C_3 - C_m = 0$$

where

$$T_1 = C_3 = \phi_s f_y A_c$$

$$T_2 = \phi_s f_y A_d$$

$$C_m = (0.85 \phi_m f'_m) A_L$$

thus

$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85 \phi_m f'_m}$$

The area of the compression zone can be determined from the geometry shown in Figure 2-27b, that is,

$$A_L = a * t + (b_f - t) * t$$

Thus, the depth of the compression zone a can be found from the above equation as follows

$$a = \frac{A_L - b_f * t + t^2}{t}$$

The distance from the centroid of the masonry compression zone to the extreme compression fibre is equal to

$$x = \frac{t*(a^2/2) + (b_f - t)(t^2/2)}{A_L}$$

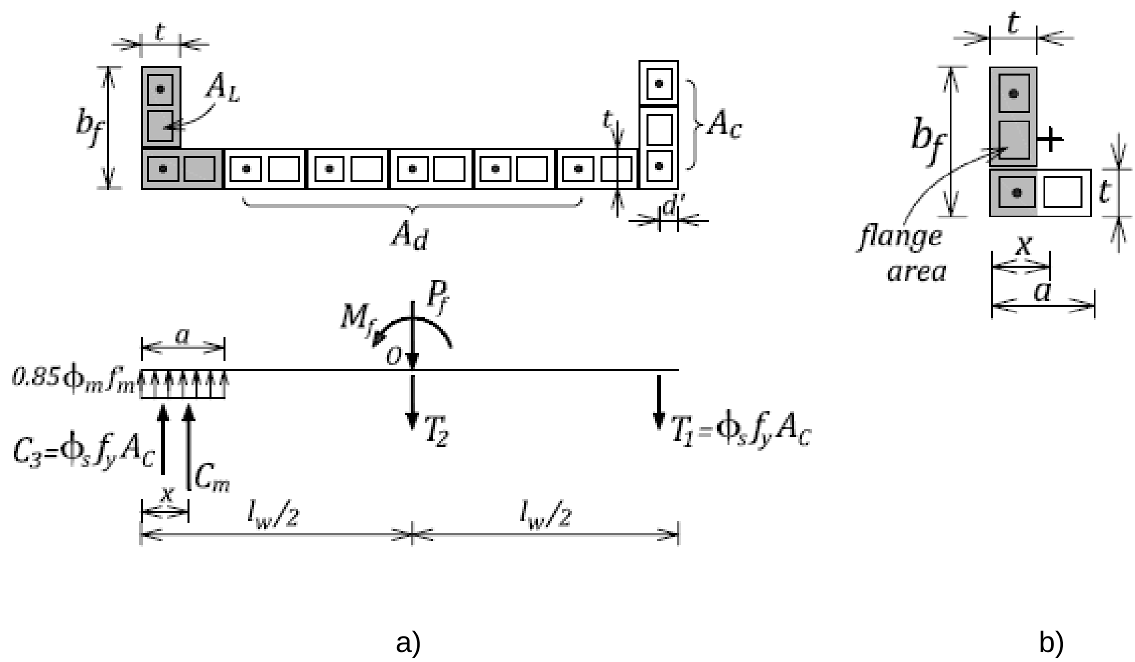


Figure 2-27. Flanged wall section: a) internal force distribution; b) flange geometry.

The compression zone of the wall may be either L-shaped or rectangular (non-flanged), however only the flange area will be considered for the buckling resistance check (the flange area is shown shaded in Figure 2-27b). This is a conservative approximation and it is considered to be appropriate for this purpose. The gross moment of inertia of the flange section around the axis parallel with the logitudinal wall axis can be determined from the following equation

$$I_{xg} = \frac{t*b_f^{\,3}}{12}$$

The use of gross moment of inertia, as opposed of partially or fully cracked one, is considered appropriate in this case, because the web portion of the compression zone and the effect of the reinforcement have been ignored.

The buckling strength for the compression zone will be determined according to S304.1 Cl. 10.7.4.3, as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I}{(1+0.5\beta_d)(kh)^2}$$

where

$\phi_{er} = 0.75$ resistance factor for member stiffness

$k = 1.0$ effective length factor for compression members (equal to 1.0 for pin-pin support conditions – a conservative assumption which can be used for this application)

$\beta_d = 0$ ratio of factored dead load moment to total factored moment (equal to 0 when 100% live load is assumed)

E_m - modulus of elasticity for masonry

The resultant compression force, including the concrete and steel component, can be determined as follows:

$$P_{fb} = C_m + \phi_s f_y A_c$$

The out-of-plane buckling resistance is considered to be adequate when

$$P_{fb} < P_{cr}$$

In some cases, it is not possible to use flanged wall sections due to architectural or other constraints. In such cases, structural designers may consider the following recommendations regarding the thickness restrictions for <u>moderately ductile squat shear walls</u>:

- When the CSA S304.1 h/t limits for ductile walls spanning in vertical direction (i.e. between adjacent floor slabs) have not been met, vertical supports in the form of pilasters can be provided to overcome this restriction. Clear span between adjacent pilasters should remain within the CSA S304.1 prescribed h/t limits. For more details related to the pilaster design refer to Drysdale and Hamid (2005) and Hatzinikolas and Korany (2005).
- The New Zealand Masonry Standard NZS4230:2004 (Cl.7.4.4.1) allows some relaxation for h/t ratio provided that c/t and c/l_w ratios are within certain limits. For shear walls of rectangular cross section shown in Figure 2-28a, the neutral axis depth needs to meet one of the following requirements (see Figure 2-28b):

  $$c \le 4t$$

  or

  $$c \le 0.3l_w$$

  For flanged shear walls the neutral axis depth needs to meet the following requirement (see Figure 2-28c):

  $$c \le 6t$$

  where $6t$ is the distance from the inside of a wall return of minimum length $0.2h$. Note that, in the case of a flanged wall section such as that shown in Figure 2-28c, the non-flanged wall end is more critical for out-of-plane instability (for more details refer to Section 2.6.6). This check gives conservative results, as shown in Example 5b in Chapter 4.

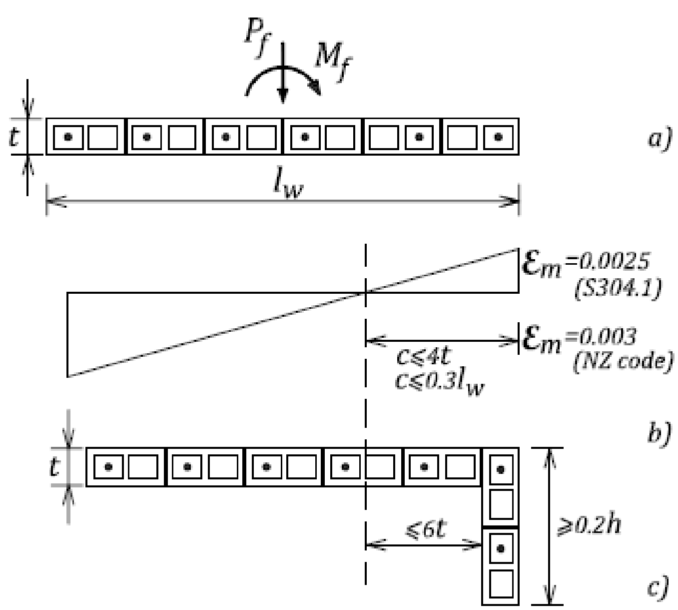


Figure 2-28. Neutral axis depth in ductile shear walls: a) rectangular (non-flanged) wall cross-section; b) corresponding neutral axis depth limits; c) flanged wall.

Note that CSA S304.1-04 restricts the maximum compressive strain in masonry ε_m in the plastic hinge zone to 0.0025, while NZS4230:2004 permits a larger strain value of 0.003.

2.5.4.5 Shear/diagonal tension resistance – seismic design requirements

10.16.4.2.1 10.16.5.3.1 10.16.6.4

CSA S304.1-04 general design provisions for shear (diagonal tension) resistance contained in Cl.10.10.1 were discussed in Section 2.3.2. Special seismic design provisions for the plastic hinge zone of the walls are as follows:

1. Limited ductility shear walls (Cl.10.16.4.2.1):
   $V_r = V_m + V_s$
   (the same equation used for the non-seismic design)

2. Moderately ductile shear walls (Cl.10.16.5.3.1):
   $V_r = 0.5V_m + V_s$
   (50% reduction in the masonry shear resistance)

3. Moderately ductile <u>squat</u> shear walls (Cl.10.16.6.4 and 10.10.1.3):
   $V_r = V_m + V_s$
   (the same equation used for the non-seismic design of squat shear walls)

For moderately ductile <u>squat</u> shear walls, Cl.10.16.6.2 requires that the shear force be applied along the entire wall length, and not concentrated near one end. The purpose of this provision is to ensure that a top transfer beam, or an alternative provision (bond beam provided at the top of the wall), will enable the development of the desirable shear failure mechanism shown in Figure 2-16a, and prevent the partial shear failure shown in Figure 2-16b. Shear failure mechanisms for squat shear walls were discussed in Section 2.3.2.2.

Commentary

Tests have shown that shear walls that fail in shear have a very poor cyclic response and demonstrate a sudden loss of strength. Also, walls that initially yield in flexure may fail in shear after several large inelastic cycles, with the resulting rapid strength degradation. Therefore, the shear steel (horizontal reinforcement) is usually designed to carry the entire shear load in the plastic hinge region of a wall (Anderson and Priestley, 1992). Seismic design provisions for ductile reinforced concrete shear walls (CSA A23.3 Cl.21.6.9) completely neglect the concrete contribution to the wall shear resistance in the plastic hinge zone.

CSA S304.1-04 provisions permit the use of the entire masonry shear resistance for all wall classes, except for moderately ductile shear walls, where only 50% of the masonry shear resistance, V_m, can be considered. CSA S304.1-94 Cl.A6.1. also contained the same reduction in the masonry shear resistance contribution for <u>nominally</u> ductile shear walls.

The overall shear strength is assumed to decrease in a linear fashion as the displacement ductility ratio increases, as discussed by Priestley, Verma, and Xiao (1994). This concept is illustrated in Figure 2-29 (note that displacement ductility ratio μ corresponds to the ductility-related force modification factor R_d). A ductile flexural response is ensured if the lateral force corresponding to the flexural strength is less than the residual shear strength, $V_{residual}$. A brittle shear failure takes place when the lateral force corresponding to flexural strength is greater than the initial shear strength, $V_{initial}$. When the lateral force corresponding to flexural strength is between the initial and residual shear strength, then shear failure occurs at a ductility corresponding to the intersection of the lateral force and shear force-displacement ductility plot. Anderson and Priestley (1992) recommended to allow 100% of the masonry shear strength up to ductility ratio of 2, and then to linearly decrease the masonry component of the shear strength to zero at the ductility ratio of 4. Note that CSA S304.1-04 allows 100 % of V_m up to $R_d = 1.5$, which corresponds roughly to a displacement ductility ratio of 1.5, but reduces the V_m contribution to 50 % at $R_d = 2.0$.

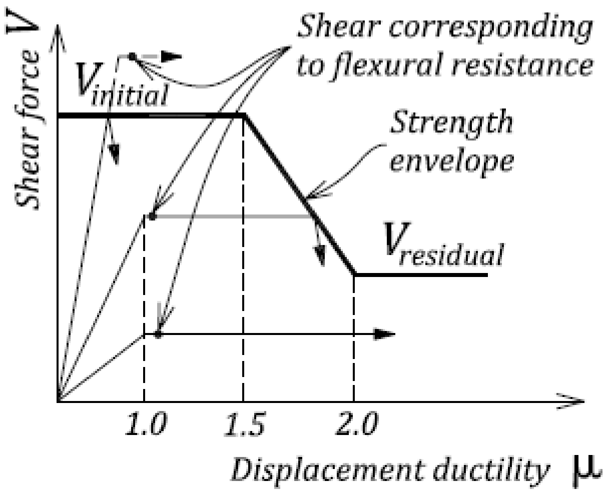


Figure 2-29. Interaction between the shear resistance and the displacement ductility ratio (adapted from Priestley, Verma, and Xiao, 1994, reproduced by permission of the ASCE[1]).

2.5.4.6 Sliding shear resistance – seismic design requirements

10.16.4.2.2
10.16.5.3.2
10.16.6.5

CSA S304.1-04 general design provisions for sliding shear resistance in Cl.10.10.4 were discussed in Section 2.3.3. The special seismic design provisions for sliding shear resistance are as follows:

1. *Limited ductility shear walls (Cl.10.16.4.2.2) and moderately ductile squat shear walls (Cl.10.16.6.5):*

   $V_r = \phi_m \mu P_2$

   The same equation is used for non-seismic design.
2. *Moderately ductile shear walls (Cl.10.16.5.3.2):*
   In this case, only the reinforcement in the tension zone should be used for the P_2 calculation. (The compressive reinforcement is assumed to have yielded in tension in a

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previous loading cycle and is now exerting a compressive force across the shear plane as it yields in compression.)

Commentary

The mechanism of sliding shear resistance was discussed in detail in the Commentary portion of Section 2.3.3. The sliding shear resistance mechanism for moderately ductile walls is illustrated in Figure 2-17b, and is unchanged from CSA S304.1-94 Clause 6.2.

It should be noted that sliding shear often governs the shear strength of reinforced masonry walls, particularly for squat shear walls in low-rise masonry buildings. To satisfy the sliding shear requirement, an increase in the vertical reinforcement area is often needed. However, this increases the moment capacity and the corresponding shear force required to yield the ductile flexural system, so the sliding shear requirement is not satisfied. Dowels at the wall-foundation interface can improve sliding shear capacity but they may also increase the bending capacity if they are too long. Note that, for squat shear walls it is impossible to prevent sliding shear if the shear reinforcement is designed to meet the capacity design requirements. In that case, shear keys could be used to increase the sliding shear resistance.

To minimize the chances of sliding shear failure, TCCMAR's findings recommended roughening the concrete foundation surface at the base of the wall with the roughness ranging from 1.6 mm (1/16 in) to 3.2 mm (1/8 in). A more effective solution is to provide the shear keys at the base of the wall that are as wide as the hollow cores and 38 mm (1.5 in) deep, with sides tapered 20 degrees. Tests have shown that these shear keys eliminate the wall slippage under severe loading (Wallace, Klingner, and Schuller, 1998).

2.5.4.7 Seismic reinforcement requirements

CSA S304.1-04 includes several requirements pertaining to the amount and distribution of horizontal and vertical wall reinforcement. It should be noted that shear walls with conventional construction do not require the special seismic detailing required for limited ductility and moderately ductile walls. These walls only need to be designed to resist the effect of factored loads, and to satisfy minimum seismic reinforcement requirements. Note that, according to NBCC 2005 Cl.4.1.8.1.1), seismic design needs to be considered when $S(0.2) > 0.12$. Also, it is possible to use unreinforced masonry constructions at sites where $I_E F_a S_a(0.2) < 0.35$ (S304.1-04 Cl.4.5.1). Shear walls reinforcement requirements are summarized in Table 2-2, with a reference being made to pertinent CSA S304.1 clauses.

Table 2-2. CSA S304.1-04 Wall Reinforcement Requirements: Loadbearing Walls and Shear Walls

	Non-seismic design requirements	**Additional seismic requirements for** $I_E F_a S_a(0.2) \geq 0.35$
	Clause 10.15.1.1	***Clause 10.15.2.2***
Minimum area: vertical & horizontal reinforcement	<u>Minimum vertical reinforcement</u> for loadbearing walls subjected to *axial load plus bending* shall be $A_{v\min} = 0.0013 A_g$ for $s \leq 4t$ $A_{v\min} = 0.0013(4t^2)$ for $s > 4t$ S304.1 does not contain provisions regarding the minimum horizontal reinforcement area.	Loadbearing walls (including shear walls) shall be reinforced horizontally and vertically with steel having a minimum total area of $A_{stotal} = 0.002 A_g$ distributed with a minimum area in one direction of at least $A_{v\min} = 0.00067 A_g$ (approximately one-third of the total area) Reinforcement equivalent to at least one 15M bar shall be provided around each masonry panel and around each opening exceeding 1000 mm in width or height. Such reinforcement shall be detailed to develop the yield strength of the bars at corners and splices.
	Clause 10.15.3	
Maximum area: vertical & horizontal reinforcement	<u>Maximum horizontal or vertical reinforcement area</u> $A_{s\max} = 0.02 A_g$ for $s \leq 4t$ $A_{s\max} = 0.02(4t^2)$ for $s > 4t$ Maximum vertical reinforcement for flexural walls under low axial load (Cl.10.7.4.6.5) $\frac{c}{d} \leq \frac{600}{600 + f_y}$ or $\rho \leq \rho_b$	

<table>
<tr><th></th><th>Non-seismic design requirements</th><th>Additional seismic requirements for $I_E F_a S_a(0.2) \geq 0.35$</th></tr>
<tr><td rowspan="2">Spacing: vertical reinforcement</td><td>Clause 10.15.1.2</td><td>Clause 10.16.4.3.2</td></tr>
<tr><td>Where vertical reinforcement is required to resist flexural tensile stresses, it shall be
a) continuous between lateral supports;
b) spaced at not more than 2400 mm along the wall;
c) provided at each side of openings over 1200 mm long;
d) provided at each side of movement joints, and
e) provided at corners, intersections and ends of walls.</td><td>Vertical seismic reinforcement shall be uniformly distributed over the length of the wall. Its spacing shall not exceed the <u>lesser of</u>
a) $6(t+10)$ mm
b) 1200 mm
c) $l_w/4$ (for limited ductility or moderately ductile walls only)
but it need not be less than 600 mm</td></tr>
<tr><td rowspan="2">Spacing: horizontal reinforcement</td><td>Clause 10.15.1.3</td><td rowspan="2"><u>Outside plastic hinge regions (Cl.10.15.2.6):</u>
Horizontal seismic reinforcement shall be continuous between lateral supports. Its spacing shall not exceed
a) 400 mm where only joint reinforcement is used;
b) 1200 mm where only bond beams are used; or
c) 2400 mm for bond beams and 400 mm for joint reinforcement where both are used.

<u>Plastic hinge regions (Cl. 10.16.4.3.3):</u>
Reinforcing bars are to be used in the plastic hinge region, at a spacing not more than
a) 1200 mm or
b) $l_w/2$</td></tr>
<tr><td>Where horizontal reinforcement is required to resist effects of shear forces, it shall be:
a) continuous between lateral supports;
b) spaced not more than 2400 mm o/c for bond beam reinforcement;
c) spaced at not more than 600 mm for joint reinforcement for 50% running bond and 400 mm for other patterns;
d) provided above and below each opening over 1200 mm high; and
e) provided at the top of the wall and where the wall is connected to roof and floor assemblies.</td></tr>
</table>

Notes:
$A_g = 1000 \cdot t$ denotes gross cross-sectional area corresponding to 1 m wall length (for vertical reinforcement), or 1 m height (for horizontal reinforcement)
s = bar spacing
t = actual wall thickness
l_w = wall length

CSA S304.1-04 provisions related to detailing of reinforcement in ductile shear walls are summarized in Table 2-3.

Table 2-3. CSA S304.1 Reinforcement Detailing Requirements for Ductile Shear Walls

	Limited Ductility Shear Walls	**Moderately Ductile Shear Walls**
Vertical reinforcement	No special detailing requirements	***Clause 10.16.5.4.1.*** At any section within the *plastic hinge region*, no more than half of the area of vertical reinforcement may be lapped.
Horizontal reinforcement	***Clause 10.16.4.3.3*** Horizontal reinforcement shall not be lapped within a) 600 mm or b) c (the neutral axis depth) whichever is greater, from the end of the wall.	***Clause 10.16.5.4.2*** Horizontal reinforcement shall be: a) provided by reinforcing bars only (no joint reinforcement!); b) continuous over the length of the wall (can be lapped in the centre), and c) have 180° hooks around the vertical reinforcing bars at the ends of the wall.

CSA S304.1-04 minimum seismic reinforcement requirements for all classes of reinforced masonry shear walls are illustrated in Figure 2-30. To ensure desirable seismic performance of ductile shear walls, CSA S304.1-04 prescribes additional reinforcement requirements which are illustrated in Figure 2-31.

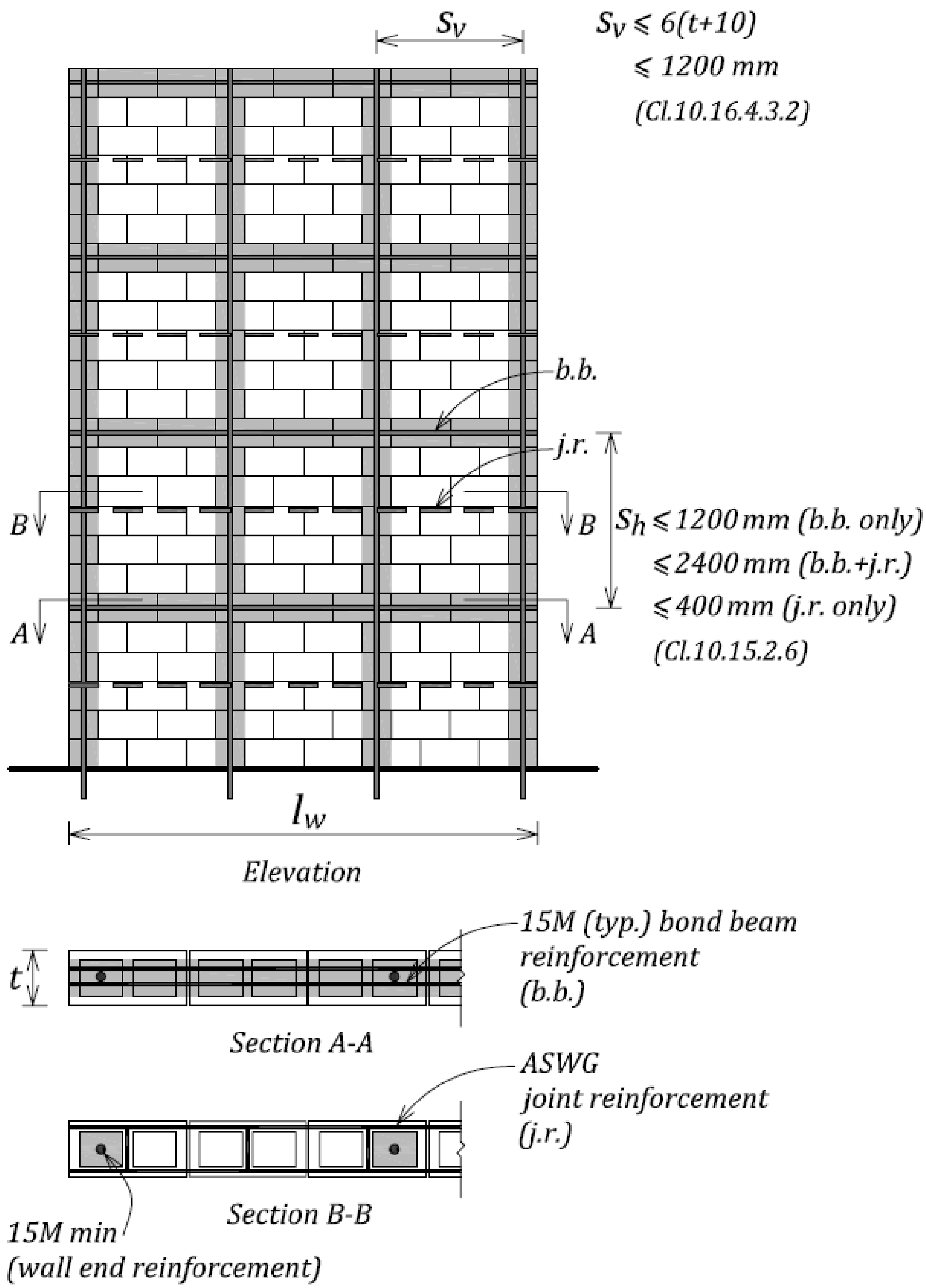


Figure 2-30. <u>Reinforced masonry shear walls:</u> CSA S304.1 minimum seismic reinforcement requirements.

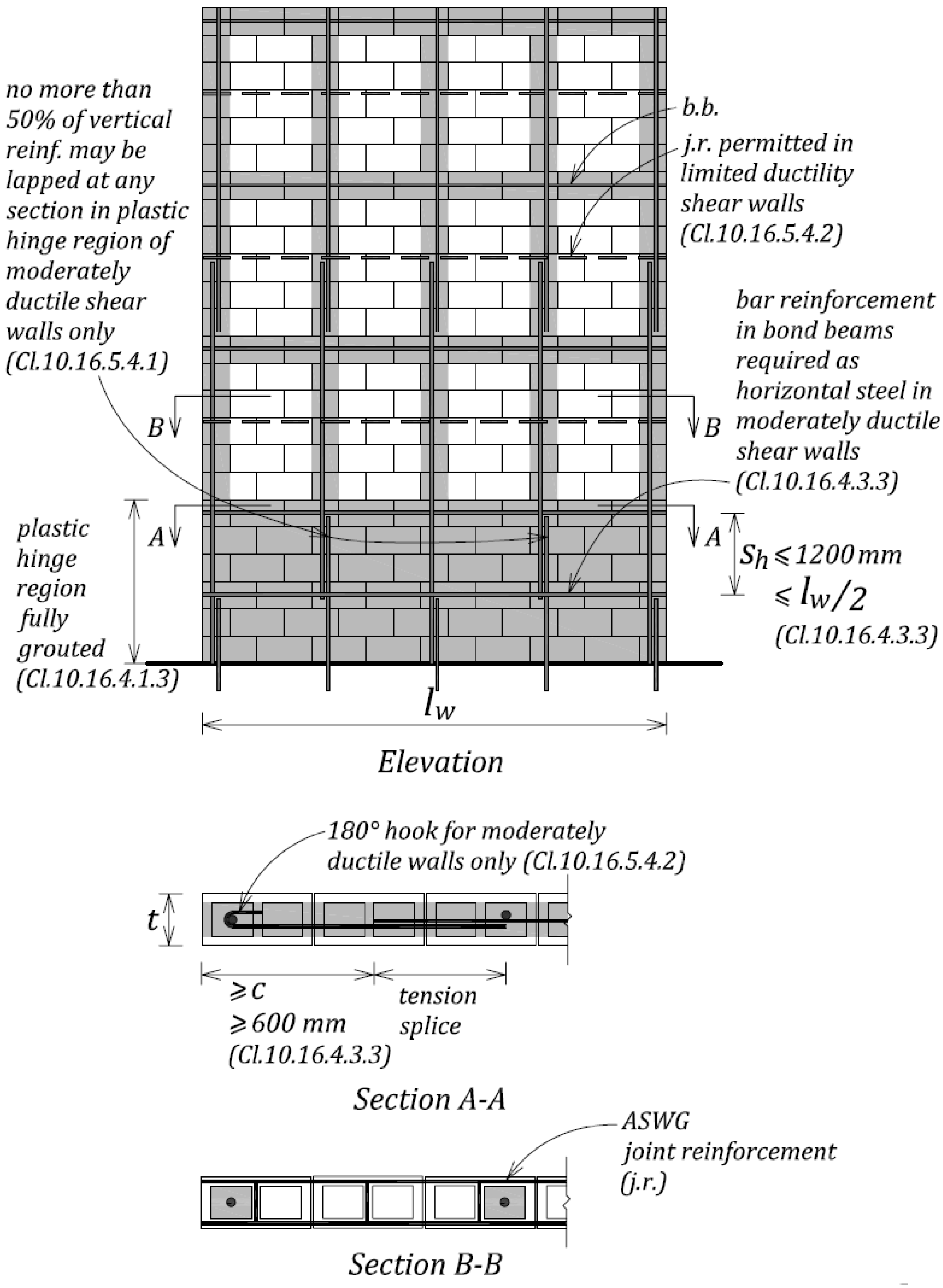


Figure 2-31. <u>Ductile reinforced masonry shear walls:</u> additional CSA S304.1 minimum seismic reinforcement requirements.

Commentary

According to Cl.10.16.4.3.2, vertical seismic reinforcement shall be uniformly distributed over the length of the wall. Shear walls with distributed reinforcement have almost the same moment resistance as shear walls with reinforcement concentrated at the edges, but distributed reinforcement has a beneficial effect on controlling cracking and maintaining shear strength in these walls.

Clause 10.16.4.3.3 requires that horizontal reinforcement laps not be within the greater of

- 600 mm or
- the neutral axis depth c

from the end of the wall, as shown in Figure 2-31. This requirement guards against lap splice failure in the end sections that may have either large masonry strains in the vertical direction, or masonry damage from previous cycles.

According to Cl.10.16.5.4.1, at any section within the *plastic hinge region* of moderately ductile walls, no more than half of the area of vertical reinforcement may be lapped, that is, laps should be staggered. This provision guards against failure of an entire lap splice, helps increase the hinge length, and thereby decrease the masonry strain.

Cl.10.16.5.4.2 prescribes the requirements for anchorage of horizontal shear reinforcement in moderately ductile shear walls. Adequate anchorage needs to be provided at each end of a potential diagonal crack. CSA S304.1-04 requires 180° hooks around the vertical reinforcing bars at the ends of the wall (see Figure 2-32a). Although this type of anchorage is most efficient, it may cause congestion at the end zone for narrow blocks. The New Zealand masonry design standard (NZS 4230:2004) C 10.3.2.9 recommends the use of 90° hooks bent downwards into the core as an alternative solution (see Figure 2-32b).

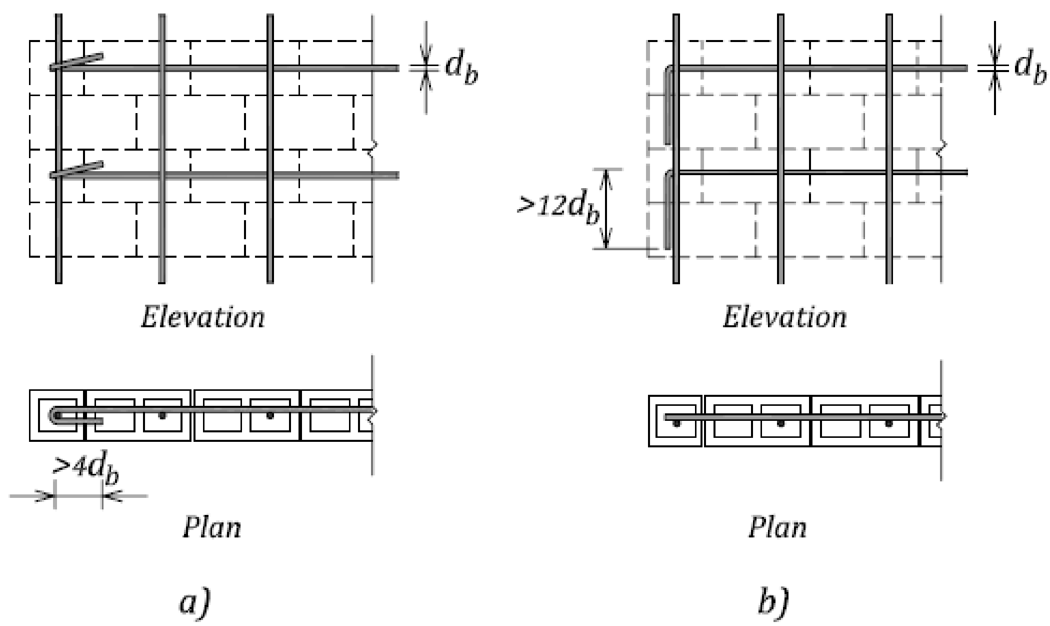


Figure 2-32. Anchorage of horizontal reinforcement: a) 180° hooks; b) 90° hooks (reproduced from NZS 4230:2004 with the permission of Standards New Zealand under Licence 000725).

CSA S304.1-94 and S304.1-04 requirements – a comparison

Most of the CSA S304.1-04 seismic requirements for shear wall reinforcement existed in the 1994 edition of the standard. A comparison is summarized below:

1. S304.1-94 Cl.5.2.2.2.2 contained the minimum seismic requirements related to reinforcement area in loadbearing walls and shear walls. These requirements remain unchanged, except that the 1994 requirements applied to the masonry structures located in seismic zone 2 or higher (this was compatible with NBCC 1995). The 2004 requirements apply to masonry structures located in areas where the seismic hazard index, $I_E F_a S_a(0.2) \geq 0.35$. This may result in changes for some locations.

   Reinforcement spacing requirements have been somewhat expanded in S304.1-04. Spacing requirements for reinforcement are clarified in Cl. 10.15.2.5 and 10.15.2.6. Cl.10.16.4.3.2 requires that the vertical reinforcement spacing for the limited ductility shear walls has an additional limit of $l_w/4$, but that it need not be less than 600 mm.

   S304.1-94 Clause A.8 included seismic reinforcement requirements for nominally ductile shear walls. The same requirements are now included in S304.1-04, Cls.10.16.4 and 10.16.5, for limited ductility and moderately ductile shear walls. A few new requirements introduced in S304.1-04 are discussed below.
2. S304.1-04 Cl.10.16.4.3.3 requires horizontal reinforcement in the form of reinforcing bars (no joint reinforcement) in the plastic hinge region of both limited ductility and moderately ductile walls (this is a new requirement).
3. S304.1-04 Cl. 10.16.5.4.2 requires 180° end hooks for horizontal reinforcement bars in the plastic hinge region of moderately ductile walls. It also requires that lines of horizontal reinforcing be continuous (this is a new requirement).

2.5.4.8 Reinforcement requirements for moderately ductile squat shear walls

10.16.6.6

CSA S304.1-04 introduced the following new requirements for the amount of reinforcement in moderately ductile squat shear walls:

- Vertical reinforcement ratio ρ_v (Cl.10.16.6.6.1):

$$\phi_s \rho_v \geq (V_f - P_f)/b_w l_w f_y$$

- Horizontal reinforcement ratio ρ_h (Cl.10.16.6.2) must be greater of

$$\phi_s \rho_h \geq \phi_s \rho_v + P_f / b_w l_w f_y$$

and that the value determined in accordance with Cl.10.10 be based on the shear resistance requirements (see Section 2.3.2).

Commentary

The seismic design requirements for moderately ductile squat shear walls were introduced in the 2004 edition of S304.1. In general, the squat wall requirements are more relaxed than those pertaining to moderately ductile flexural shear walls, because shear failure in squat shear walls is not as critical as in taller flexural walls, and can provide some ductility. Thus the design and detailing requirements related to the flexural failure mechanism (e.g. ductility check) are not required for squat walls.

The reinforcement requirements in Cl.10.16.6.6 have been derived from the mechanism of a squat shear wall failing in the shear-critical mode shown in Figure 2-33a. Consider a squat shear wall subjected to the combined effect of factored shear force, V_f, and the factored axial force, P_f. The effect of these forces can be presented in the form of distributed shear stress, v_f, and distributed axial stress, p_f, as follows

$$v_f = \frac{V_f}{b_w \cdot l_w} \qquad (14)$$

and

$$p_f = \frac{P_f}{b_w \cdot l_w} \qquad (15)$$

The wall is reinforced with horizontal and vertical reinforcement, where the reinforcement ratios ρ_h for horizontal reinforcement, and ρ_v for vertical reinforcement, are given by

$$\rho_v = \frac{A_v}{b_w \cdot l_w} \quad \text{and} \quad \rho_h = \frac{A_h}{b_w \cdot h_w}$$

where

$b_w = t$ overall wall thickness (referred to as “web width” in CSA S304.1)

l_w = wall length

h_w = wall height

If the yield stress of the reinforcement is given by f_y, the factored unit capacity of the reinforcement in the two directions is $\phi_s \rho_h f_y$ and $\phi_s \rho_v f_y$.

Once the shear force in the wall reaches a certain level, inclined shear cracks develop in the wall at a 45° angle to the horizontal axis, as shown in Figure 2-33b (note that this is an idealized model and that the angle may be different from 45°). The areas of masonry between these inclined cracks act as compression struts. Consider a typical unit length strut shown in Figure 2-33c. This strut remains in equilibrium only if there is enough force in the vertical reinforcement to satisfy moment equilibrium about the base. Note that the force in both the vertical and horizontal bars that pass through the strut do not create any net force on the strut.

Equilibrium of the strut requires that

$$p_f + \phi_s \rho_v f_y = v_f \qquad (16)$$

When v_f and p_f equations (14) and (15) are substituted into equation (16), the vertical reinforcement ratio is

$$\phi_s \rho_v = \frac{V_f - P_f}{b_w \cdot l_w \cdot f_y}$$

Note that the above equation is presented in Cl.10.16.6.6.1.

Equilibrium in the horizontal direction requires that the tensile capacity of the horizontal reinforcement, $\phi_s \rho_h f_y$, be (see Figure 2-33d)

$$\phi_s \rho_h f_y = v_f \qquad (17)$$

This equation can be presented in an alternative form useful for design purposes:

$$v_f = \frac{v_f}{\phi_s f_y} = \frac{V_f}{b_w \cdot l_w \cdot \phi_s \cdot f_y}$$

When the v_f expression is substituted from equation (17) into equation (16), it follows that

$$\phi_s \rho_h f_y = p_f + \phi_s \rho_v f_y$$

This gives the following relationship between the horizontal and vertical reinforcement, which is also presented in Cl.10.16.6.6.2:

$$\phi_s \rho_h = \phi_s \rho_v + \frac{P_f}{b_w l_w f_y}$$

It is worth noting that the required ratios of horizontal and vertical reinforcement are equal for walls with low axial load, that is, $P_f \cong 0$. This scenario applies to low-rise masonry buildings with a light roof weight.

Note that the vertical and horizontal reinforcement design should be based on flexure and shear requirements, but the designer should confirm that the minimum reinforcement requirements discussed in this section are also satisfied.

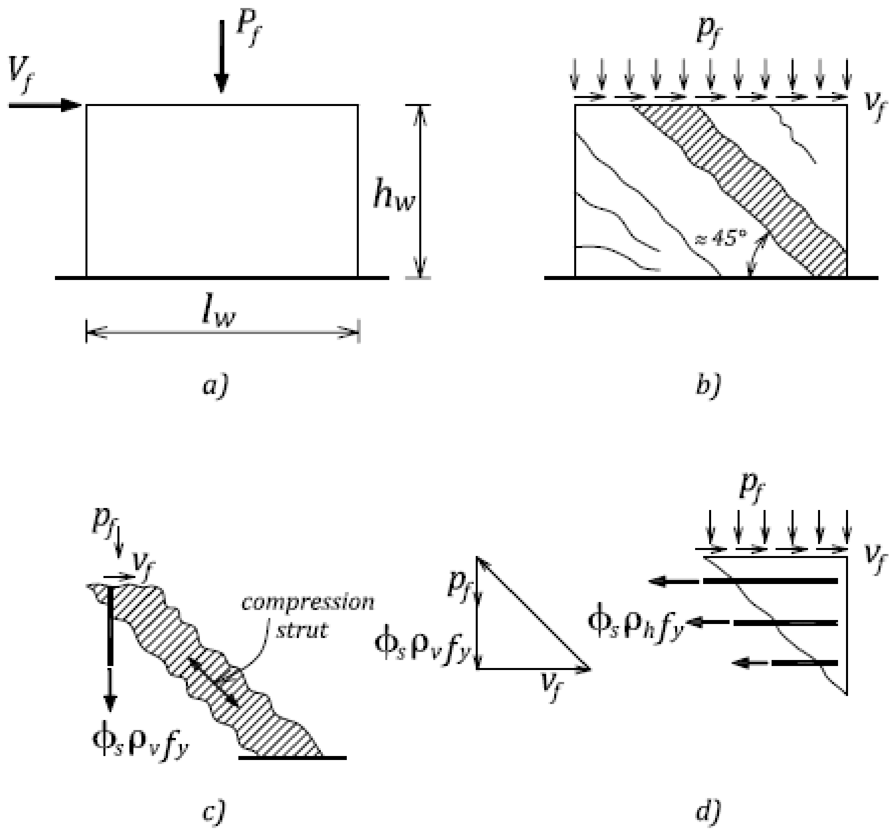


Figure 2-33. Shear failure mechanism for a squat shear wall: a) wall subjected to shear and axial load; b) crack pattern; c) compression strut; d) free-body diagram.

2.5.5 Summary of Seismic Design Requirements for Reinforced Masonry Walls

Table 2-4. Summary of the CSA S304.1-04 Seismic Design Requirements for Reinforced Masonry Walls

Provision *(guide reference section shown in the brackets)*	**Shear walls with conventional construction**	**Limited ductility shear walls**	**Moderately ductile shear walls**	**Moderately ductile <u>squat</u> shear walls ($h_w/l_w<1$)**
Ductility factor	R_d =1.5	R_d =1.5	R_d =2.0	R_d =2.0
Plastic hinge region (2.5.4.2)	Not applicable	***Cl.10.16.4.1.1*** l_p = greater of $l_w/2$ or $h_w/6$ ***Cl.10.16.4.1.3*** Masonry within the plastic hinge region shall be fully grouted.	***Cl.10.16.5.2.1*** l_p = greater of l_w or $h_w/6$ Same as limited ductility walls	
Ductility check (2.5.4.3)	Not applicable	***Cl.10.16.4.1.4*** 1. $\varepsilon_m = 0.0025$ 2. $c/l_w < 0.2$ when $h_w/l_w < 6$	***Cl.10.16.5.2.3*** 1. Maximum compression strain: $\varepsilon_m = 0.0025$ 2. Ductility limits: $\boxed{c/l_w < 0.2}$ when $h_w/l_w < 4$ and $\boxed{c/l_w < 0.15}$ when $4 < h_w/l_w < 8$	No special provisions
Wall height-to-thickness ratio restrictions (2.5.4.4)	***Cl.10.7.3.3*** Must meet non-seismic slenderness requirements and design procedures	***Cl.10.16.4.1.2*** $h/(t+10) < 18$	***Cl.10.16.5.2.2*** $h/(t+10)<14$	***Cl.10.16.6.3*** $h/(t+10) < 20$ Unless it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability

<table>
<tr><th>Provision
(guide reference section shown in the brackets)</th><th>Shear walls with conventional construction</th><th>Limited ductility shear walls</th><th>Moderately ductile shear walls</th><th>Moderately ductile <u>squat</u> shear walls
($h_w/l_w < 1$)</th></tr>
<tr><td rowspan="2">Shear/diagonal tension resistance (2.5.4.5)</td><td rowspan="2">Cl.10.10.1
$V_r = V_m + V_s$
Same as non-seismic design</td><td rowspan="2">Cl.10.16.4.2.1
$V_r = V_m + V_s$
Same as non-seismic design</td><td rowspan="2">Cl.10.16.5.3.1
$V_r = 0.5V_m + V_s$
50% reduction in the masonry shear resistance</td><td>Cl.10.16.6.4
Same as limited ductility walls</td></tr>
<tr><td>Cl.10.16.6.2
Shear force applied uniformly along the wall length</td></tr>
<tr><td>Sliding shear resistance (2.5.4.6)</td><td>Cl.10.10.4
$V_r = \phi_m \mu P_2$
Same as non-seismic design</td><td>Cl.10.16.4.2.2
$V_r = \phi_m \mu P_2$
Same as non-seismic design</td><td>Cl.10.16.5.3.2
$V_r = \phi_m \mu P_2$
Only reinforcement in the tension zone to be taken into account for P_2 calculation.</td><td>Cl.10.16.6.5
Same as limited ductility walls</td></tr>
<tr><td rowspan="2">Minimum seismic reinforcement area (2.5.4.7)</td><td rowspan="2">Minimum seismic reinf. requirements (Cl.10.15.2.2) apply when $I_E F_a S_a(0.2) \geq 0.35$ otherwise apply minimum non-seismic reinf. requirements (Cl.10.15.1.1)</td><td colspan="3">Cl.10.15.2.2
Minimum seismic reinforcement area requirements apply for all classes of ductile masonry walls (see Table 2-2)</td></tr>
<tr><td colspan="2"></td><td>Cl.10.16.6.6
Additional reinforcement requirements</td></tr>
</table>

2.6 Special Topics

2.6.1 Unreinforced Masonry Shear Walls

S304.1-04 allows the use of unreinforced masonry construction for sites where the seismic hazard index, $I_E F_a S_a(0.2) < 0.35$ (Cl.4.5.1). Seismic design provisions for unreinforced masonry shear walls are presented in this section.

2.6.1.1 Shear/diagonal tension resistance (in-plane and out-of-plane)

7.10.1 7.10.2

Design provisions for factored in-plane and out-of-plane diagonal tension shear resistance, V_r, for unreinforced masonry shear walls are the same as those for reinforced masonry walls, except that there is no steel contribution term ($V_s = 0$). The background for these provisions is discussed in detail in Sections 2.3.2 and 2.4.2.

Commentary

Diagonal tension is a brittle failure mode, characterized by the development of a major diagonal crack that forms when the masonry tensile resistance has been reached (see Section 2.3.1.2). This is an undesirable failure mechanism and should be avoided, preferably by providing horizontal reinforcement in masonry walls loaded in-plane and located in regions where $I_E F_a S_a(0.2) > 0.35$.

2.6.1.2 Sliding shear resistance (in-plane and out-of-plane)

7.10.4.1 7.10.4.2

Design provisions for in-plane and out-of-plane sliding shear resistance for unreinforced masonry walls are somewhat different from those for reinforced masonry, in that bed-joint sliding masonry resistance (in addition to the frictional resistance) is assigned to the wall. Note that in reinforced masonry walls only frictional resistance is considered, as discussed in Section 2.3.3.

The in-plane sliding shear resistance, V_r, along bed joints between courses of masonry, also known as *bed-joint sliding resistance*, is given in Cl.7.10.4.1 as

$$V_r = 0.16\phi_m \sqrt{f'_m} A_{uc} + \phi_m \mu P_1$$

where

μ = the coefficient of friction
= 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane
= 0.7 for a masonry-to-smooth concrete or bare steel sliding plane
= other (when flashings reduce friction that resists sliding shear, a reduced coefficient of friction accounting for the flashing material properties should be used)

P_1 = the compressive force in masonry acting normal to the sliding plane, normally taken as P_d (equal to 0.9 times the dead load). For infill shear walls, an additional component, equal to

90% of the factored vertical component of the compressive force resulting from the diagonal strut action, should be added (see Figure 2-34c).

A_{uc} = uncracked portion of the effective cross-sectional area of the wall that provides shear bond capacity (note that both out-of-plane loads and in-plane loads can cause cracking of the masonry wall)

For the <u>in-plane</u> sliding shear resistance, A_{uc} should be determined as follows

$$A_{uc} = t_e \cdot d_v$$

where

t_e = effective wall thickness; t_e is equal to the sum of two face shell thicknesses for hollow walls, and to the actual wall thickness t for fully grouted walls

d_v = effective wall depth, equal to $0.8 l_w$

l_w = wall length

For the <u>out-of-plane</u> sliding shear resistance, A_{uc} should be determined as follows

$$A_{uc} = t_e \cdot l_w$$

The sliding shear resistance at the base of the wall (along the bed joint between the support and the first course of masonry) is equal to (see Figure 2-34b)

$$V_r = \phi_m \mu C$$

where C is compressive force in the masonry acting normal to the sliding plane, normally taken as P_d (equal to 0.9 times the dead load), since T_y =0, that is,

$$C = P_d + T_y$$

Design equations for the out-of-plane sliding resistance stated in Cl.7.10.4.2 are the same as the equations for the in-plane sliding shear resistance presented above.

Commentary

The two forms of the sliding shear failure mechanism (bed-joint sliding and base sliding), are presented in Figure 2-34 a) and b). Sliding shear failure is likely to govern the design of masonry shear walls in low-rise buildings, due to the low axial load acting on these walls (see Commentary in Section 2.5.4.6). In unreinforced masonry walls, dowels can provide the required sliding shear resistance at the base, but it should be noted that a sliding shear failure can still take place at the section at the top of the dowels, which is undesirable. However, it should be noted that the sliding shear failure mechanism is a ductile one, and has been characterized by significant lateral deformations along the failure plane in major earthquakes.

Note that in the equation for bed-joint sliding resistance, the first term represents the shear bond resistance of masonry mortar, while the second term represents the sliding shear resistance based on the Coulomb friction model. In determining the sliding shear resistance for the bed-joint sliding mechanism for seismic design of unreinforced masonry walls, the first term in the equation should be ignored if the wall cracks due to either in-plane or out-of-plane bending. If the wall remains uncracked, the second term (shear friction resistance) should not be included. The smaller of the two values should be used in the design.

For the sliding resistance at the base of the wall, sliding shear resistance is provided by the weight of the wall above and yielding of steel dowels. Note that the dowel contribution is possible only after a small shear slip at the base takes place and a horizontal crack forms at the wall-to-foundation interface.

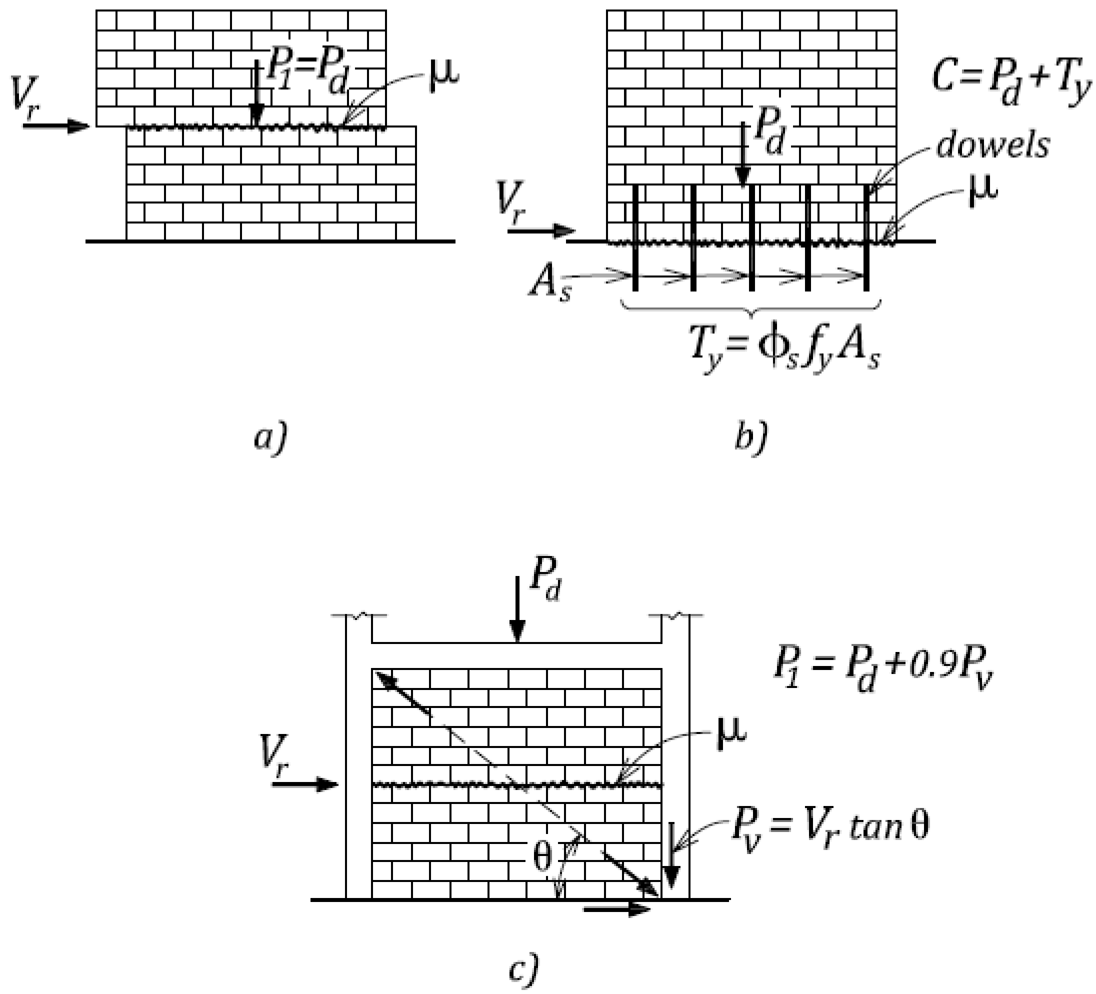


Figure 2-34. Sliding shear failure mechanism: a) bed-joint sliding; b) sliding at the base of the wall; c) sliding shear in infilled masonry walls.

The bed-joint sliding failure mechanism is also characteristic of infilled masonry walls, as shown in Figure 2-34c). Seismic design considerations for masonry infill walls are discussed in Section 2.6.2.

2.6.1.3 Flexural resistance due to combined axial load and bending

7.2

A masonry wall of length, l_w, and thickness, t, subjected to factored axial load, P_f, and factored bending moment, M_f, has an eccentricity, e, equal to

$$e=\frac{M_f}{P_f}$$

According to Cl.7.2.1, unreinforced masonry walls should be designed to remain uncracked when

$e \geq 0.33 l_w$ for in-plane bending, or

$e \geq 0.33 t$ for out-of-plane bending,

but the maximum stresses must not exceed $\phi_m f_t$ for tension and $\phi_m f'_m$ for compression (Cl.7.2.2), where f_t is the flexural tensile strength of masonry (see Table 5 of CSA S304.1-04).

The maximum stresses at the wall ends can be calculated as follows:

$$\max f_c = \frac{P_f}{A_e} + \frac{M_f}{S_e} \leq \phi_m f'_m$$

and

$$\max f_t = \frac{P_f}{A_e} - \frac{M_f}{S_e} \geq -\phi_m f_t$$

where

P_f and M_f are the factored axial load and the factored bending moment acting on the wall section

$A_e = t_e \cdot l_w$ effective cross-sectional area of masonry

t_e = effective wall thickness equal to the sum of two face shell thicknesses for hollow walls, and to the actual wall thickness t for fully grouted walls

$S_e = \dfrac{t_e \cdot l_w{}^2}{6}$ section modulus of effective wall cross-sectional area

When

$e < 0.33 l_w$ for in-plane bending, or

$e < 0.33 t$ for out-of-plane bending,

an unreinforced masonry wall can be designed assuming cracked wall sections (Cl.7.2.3) using an equivalent rectangular stress block, as per Cl.10.2.6.

The centroid of the compression zone must coincide with the load eccentricity, e, as shown in Figure 2-35b, and the compression capacity, P_r, can then be determined from the following equation:

$$P_r = (0.85 \chi \phi_m f'_m) \cdot t_e \cdot \left(\frac{l_w}{2} - e \right) \cdot 2$$

note that P_r must be greater than P_f.

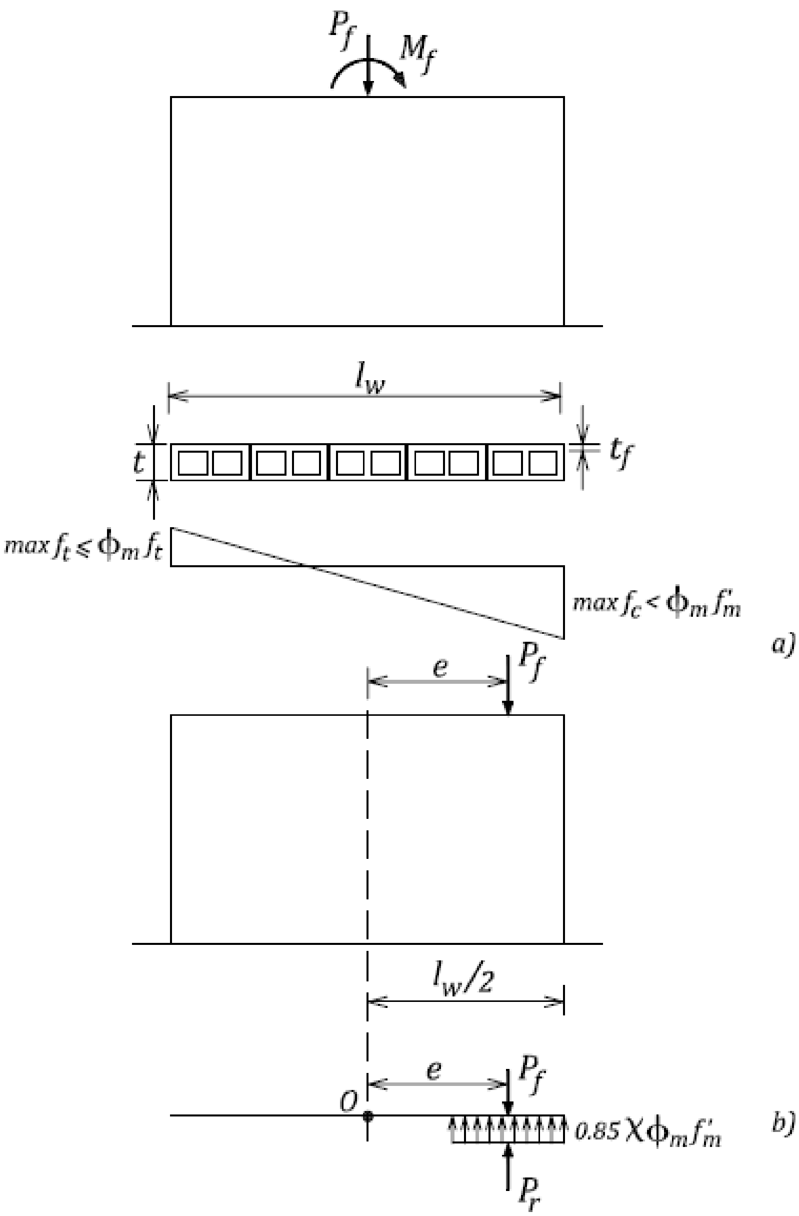


Figure 2-35. Stresses due to combined axial load and bending in an unreinforced masonry wall: a) uncracked wall; b) cracked wall.

Commentary

It is realistic to assume that unreinforced masonry wall sections will experience cracking under seismic conditions. Reports from the past earthquakes have shown that unreinforced masonry suffers extensive damage in earthquakes, e.g. 1994 Northridge, California earthquake (magnitude 6.7); for more details refer to TMS (1994). Despite the extensive damage, it should be noted that the building stock of unreinforced masonry block walls in California is very limited, since the provision for reinforcement in masonry structures started after the 1933 Long Beach earthquake. This cannot be said for most seismic zones in Canada.

2.6.2 Masonry Infill Walls

7.13 10.12

Infill walls are masonry wall panels enclosed by reinforced concrete or steel frame members on all four sides. Infill walls are not listed as a wall class in NBCC 2005, and therefore fall under the classification of "other masonry SFRS(s)",. They are only allowed in low seismic regions where $I_E F_a S_a(0.2) < 0.20$, and have $R_d = R_o = 1.0$ and a height limitation of 15 m.

CSA S304.1 design provisions for masonry infill walls, introduced for the first time in the 2004 edition of the code, are summarized below.

General design requirements

1. Masonry infill walls are treated as shear walls and should be designed to resist all in-plane and out-of-plane loads (Cl.7.13.1).
2. Masonry infill walls should be designed to resist any vertical loads transferred to them by the frame (Cl.7.13.2.4).
3. The increased stiffness of lateral load-resisting elements that consist of masonry infill shear walls working with the surrounding frame, should be taken into account when distributing the applied loads to these elements (Cl.7.13.2.5).
4. When a diagonal strut is used to model the infill shear wall according to Cl.7.13.3, an infill frame can be designed using a truss model (see the note to Cl.7.13.2.5).

Design approaches for masonry infill walls

CSA S304.1 offers three possible design and construction approaches for infill walls:

1. *Participating infill (diagonal strut approach)* – when there are no openings or gaps between the masonry infill and the surrounding frame, but the infill is not tied or bonded to the frame, the infill should be modelled as a diagonal strut according to Cl.7.13.3. Where openings or gaps exist, the designer must show through experimental testing or special investigations that the diagonal strut action can be formed and all other structural requirements for the infill shear walls can be developed (Cl.7.13.2.3).
2. *Frame and infill composite action* – when the infill shear wall is tied and bonded to the frame to create a composite shear wall, where the infill forms the web and the columns of the frame form the flanges of the shear wall (Cl.7.13.2.2).
3. *Isolated infill* - it is also possible to design an isolated infill panel (a note to Cl.7.13.1 and Cl.7.13.2.3), which is separated from the frame structure by a gap created by vertical movement joints along the ends and a horizontal movement joint under the floor above or beam. In that case, masonry infill is a nonloadbearing wall and cannot be treated as a shear wall. Restraints must be provided at the top of the wall to ensure stability for out-of-plane seismic forces.

Diagonal strut model

For structural design purposes, infill walls should be modelled as diagonal struts, as shown in Figure 2-36 (Cl.7.13.2.1). The key properties of the diagonal strut model are summarized below.

Diagonal strut width w should be determined as follows (Cl.7.13.3.2):

$$w = \sqrt{\alpha_h^2 + \alpha_L^2}$$

where

$$\alpha_h = \frac{\pi}{2}\left(\frac{4E_f I_c h}{E_m t_e \sin 2\theta}\right)^{1/4}$$

and

$$\alpha_L = \pi\left(\frac{4E_f I_b l}{E_m t_e \sin 2\theta}\right)^{1/4}$$

α_h = vertical contact length between the frame and the diagonal strut

α_L = horizontal contact length between the frame and the diagonal strut

E_m, E_f = moduli of elasticity of the masonry wall and frame material, respectively

h, l = height and length of the infill wall, respectively

$l_s = \sqrt{h^2 + l^2}$ length of the diagonal strut

t_e = sum of the thickness of the two face shells for hollow or semi-solid block units and the thickness of the wall for solid or fully grouted hollow or semi-solid block units

I_c, I_b = moments of inertia of the column and the beam of the frame respectively

θ = angle of diagonal strut measured from the horizontal, where

$$\tan\theta = \frac{h}{l}$$

Effective diagonal strut width, w_e, to be used for the calculation of the compressive strength of the strut should be taken as (Cl.7.13.3.3)

$$w_e = w/2$$

or

$$w_e \le l_s/4$$

whichever is the least.

The *design length* of the diagonal strut l_d should be equal to (Cl.7.13.3.5)

$$l_d = l_s - w/2$$

Depending on the strut end conditions (fixed or pinned), an effective length can be calculated by multiplying the design length by the effective length factor for compression members, k (see Annex B to CSA S304.1).

The design length for the diagonal strut in reinforced infill walls should be determined as the smallest of the following (Cl.10.12.3):

- design length l_d as defined above, or
- infill wall height h or length l, when minimum reinforcement and lateral anchorage are provided for the span in that direction.

<u>In-plane resistance of masonry infill walls</u>

According to CSA S304.1, masonry infills should be designed considering the following failure mechanisms:

- Compression or buckling failure in diagonal strut, and
- In-plane shear failure of the masonry infill.

Diagonal strut – compression resistance (Cl.7.13.3.4)

The compression strength of the diagonal strut, P_r, is equal to the compression strength of the masonry times the effective cross-sectional area, that is,

$$P_r = (0.85 \chi \phi_m f'_m) \cdot A_e$$

where

$$A_e = w_e * t_e$$

Note that the masonry compressive strength should be reduced by $\chi = 0.5$ (corresponding to the masonry strength for compression normal to the head joints). The concept of effective cross-sectional area is addressed by S304.1-04 Cl.7.3 (unreinforced masonry walls) and Cl.10.3 (reinforced masonry walls).

Diagonal strut – buckling resistance
In determining the compression resistance, P_r, slenderness effects should be included in accordance with Cl.7.7.

The designer should ensure that the horizontal component of the diagonal strut compression resistance, P_h, is larger than the factored shear load, V_f, acting on the infill (see Figure 2-36c).

Shear resistance of infill walls (Cl.7.13.3.1 on unreinforced infills and Cl.10.12.4 on reinforced infills)
In-plane sliding shear resistance (bed-joint sliding resistance) is the key shear resistance mechanism characteristic both of unreinforced and reinforced infill walls (Cl.7.10.4). See Section 2.6.1.2 for a discussion on the bed-joint sliding mechanism.

Infill shear walls should be designed so that a bed-joint sliding shear failure is prevented (Cl.7.13.3.1). This failure mechanism can lead to a knee-braced condition that could cause a premature failure of the column in the surrounding frame, as shown in Figure 2-39 a).

The vertical component of the diagonal strut compression resistance, P_v, must be considered in determining the sliding shear resistance, as shown in Figure 2-34 c) (see Note 2 to Cl.7.13.3.1).

CSA S304.1 Cl.10.12.4 states that the reinforced masonry infills need to be designed to resist all applied shear loads in accordance with Cl.10.10.1, as they relate to the diagonal tension shear resistance discussed in Section 2.3.2 of this guide. However, it should be noted that the shear resistance corresponding to the diagonal tension cracking does not represent the limited or ultimate load condition for infill walls (see the discussion in the commentary part of this section).

<u>Reinforcement</u>
The reinforcement is required to resist tensile and shear stresses in infills (Cl.10.12.2). The minimum reinforcement requirements stated in Cl.10.15 should be followed (see Section 2.5.4.7).

<u>Effect of masonry infill on frame members (Cl.7.13.3.1)</u>
Adjacent frame members and their connections should be designed to resist additional shear forces resulting from the diagonal strut action (see Note 3 to Cl.7.13.3.1).

Commentary

The infilling of frames is associated with the construction of medium- and high-rise steel and reinforced concrete (RC) buildings, where the frames carry gravity and lateral loads, and the infills provide the building envelope and internal partitions. Historically, the frames have been engineered according to the state of the knowledge of the time, with the infill panels considered to be “nonstructural” elements (FEMA 306, 1999). However, recent damaging earthquakes in several countries (e.g. the 1999 Kocaeli earthquake in Turkey, the 2001 Bhuj earthquake in India, the 2001 Chi Chi earthquake in Taiwan, the 2003 Boumerdes earthquake in Algeria, etc.) revealed significant deficiencies and poor seismic performance of RC frame buildings with masonry infills, thereby causing significant human and economic losses (Murty, Brzev, et al. 2006).

The introduction of infills into frames changes the lateral-load transfer mechanism of the structure from a predominantly frame action to a predominantly truss action, as shown on Figure 2-37 (Kaushik, Rai, and Jain, 2006). Masonry infills in RC or steel frame buildings are usually modelled as diagonal compression struts, so an infilled frame can be modelled as a braced frame with pin connections at beam-column joints.

It should be recognized that the seismic response of infilled frames is very complex. At low level of seismic loads, the infill panels are uncracked and often cause a significant increase in the stiffness of the entire structure. In some cases, the stiffness of a RC frame with infills may be in the order of 20 times larger than that of the bare frame. At that stage, infills usually attract most of the lateral forces, but as the load increases, the infills crack and their stiffness drops. As a result, the stiffness of an infilled frame progressively decreases in each subsequent loading cycle, and more of the load is transferred to the frame. For that reason, the frames must have sufficient strength to avoid the collapse of the structure (Kaushik, Rai, and Jain, 2006). CSA S304.1 requires that the masonry infills should be able to resist the lateral seismic loads without any assistance from the frames (Cl.7.13.3.1).

To safeguard frames from being designed for very low seismic forces, some building codes require that the frame alone be designed to independently resist at least 25% of the design seismic forces, in addition to the forces caused by gravity loads. CSA S304.1 Cl.7.13.3.1 (Note 3) states that the frame members and their connections should be designed to resist additional shear forces introduced by the diagonal strut action. For example, the columns will have to resist a shear force equal to the horizontal component of the diagonal strut compression resistance, P_h (see Figure 2-36c).

The following two analytical models can be considered in the design of infilled frames (see Figure 2-37):

i) uncracked braced frame with diagonal struts; this model results in a high stiffness (corresponding to a short period) and small lateral deflections, and
ii) bare frame with cracked frame members (assuming failed infills); this model results in a low stiffness (corresponding to a long period) and large deflections.

It should be noted that the first model will give the maximum design forces, while the second one will give the maximum lateral deflections. The designer needs to consider both models in the analysis and use the most critical values for the design.

Problems associated with seismic performance of infilled frame structures arise from discontinuities of infills along the building height, and the resulting vertical stiffness discontinuity (see the discussion on irregularities in Section 1.5.10). In such infilled frames, there is a high level of forces to be resisted by the frame components. In some cases, discontinuity of infills at the ground floor level results in a soft storey mechanism, which has caused the collapse of several buildings in past earthquakes (see Figure 2-38). In developing countries, construction quality combined with inadequate detailing of RC frame components may occur, which leads to a non-ductile seismic response of these structures.

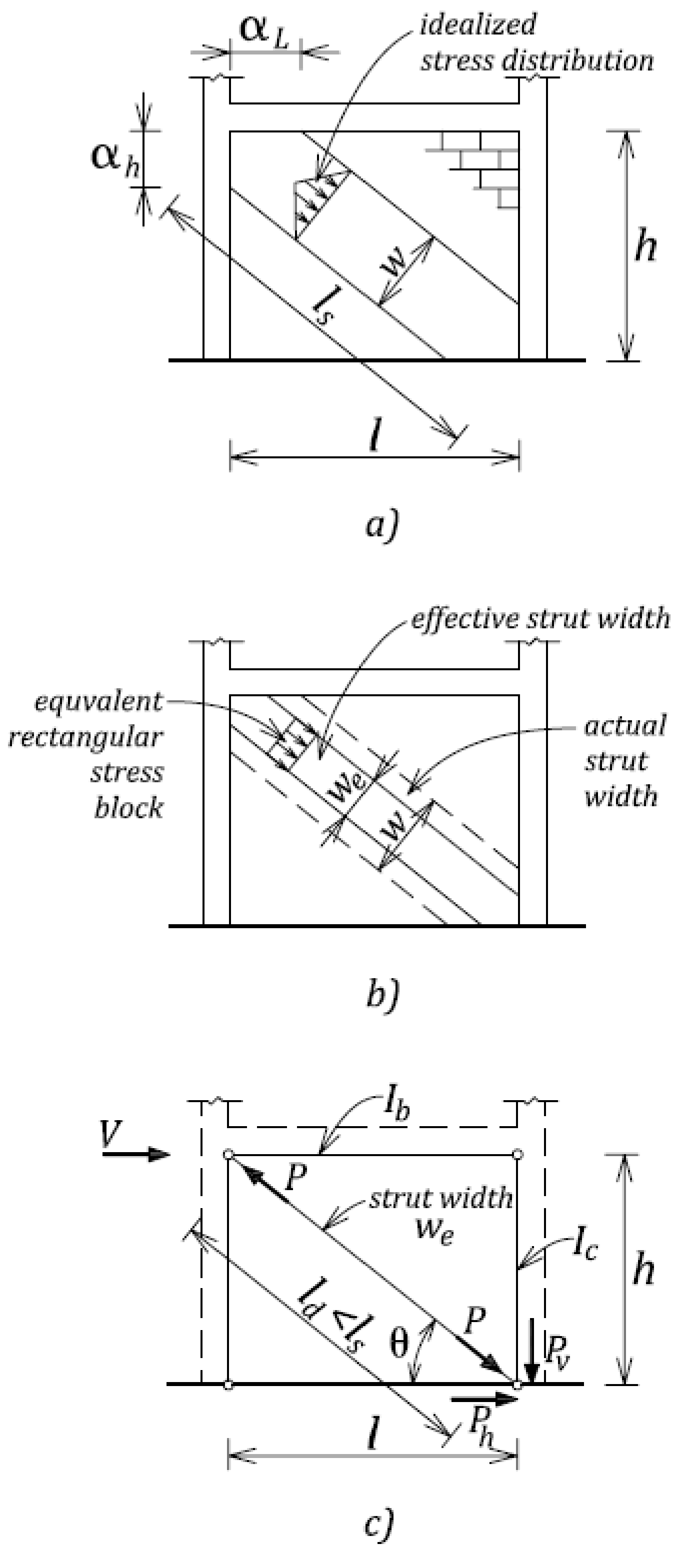
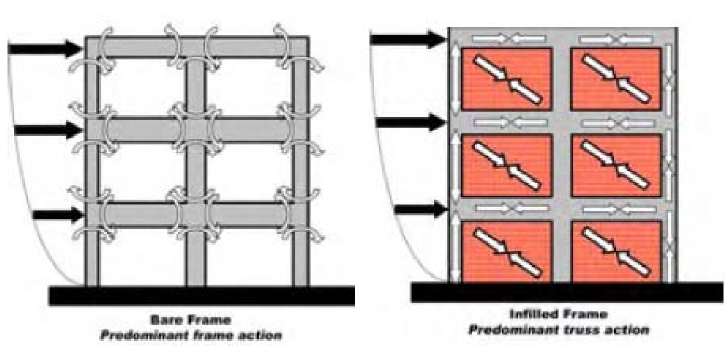


Figure 2-36. Diagonal strut model: a) actual strut width; b) effective strut width; c) analytical model.



a *b*

Figure 2-37. Masonry infills alter the seismic response of a frame structure: a) bare frame; b) diagonal strut mechanism (Source: Murty, Brzev, et al. 2006[1]).

a) *b)*

Figure 2-38. Soft storey mechanism: a) vertical discontinuity in masonry infills[2] ; b) building damage in the 2003 Boumerdes, Algeria earthquake[3].

Infill walls may fail due to the effects of *in-plane* or *out-of-plane* seismic forces. *The in-plane seismic response* of masonry infills is generally governed by shear failure mechanisms. The response depends on several factors, including the relative stiffness of the infill and frame, the material properties, and the contact between the infill and frame. The following behaviour modes are characteristic of masonry infills subjected to in-plane seismic loads (Tomazevic 1999; FEMA 306, 1999):

1. *Bed-joint sliding failure:* this mechanism takes place along horizontal mortar joints and results in the separation of infill into two or more parts (see Figure 2-39 a and b). The separated parts of the masonry infill cause free column deformations, ultimately resulting in plastic hinging in the columns. This is a ductile, displacement-controlled mechanism, since the earthquake energy is dissipated through the friction along the bed joints. This

[1] Reproduced by permission of the Earthquake Engineering Research Institute (EERI)
[2] Source: Murty, Brzev, et al., 2006, reproduced by permission of the EERI
[3] Source: S. Brzev

mechanism is likely to occur when the frame is strong and flexible. If the plane of weakness forms near the column mid-height, there is a chance for a short-column effect in the frame that can lead to a shear failure (see Figure 2-39 a). Note that when an infill panel experiences the bed-joint sliding failure, an equivalent diagonal strut may not form, so that sliding becomes the governing failure mechanism.

2. *Diagonal strut mechanism with corner compression failure*: this mechanism takes place due to the high concentration of compression stresses in the diagonal strut. The formation of a diagonal strut is preceded by diagonal tension cracking in the infill shown in Figure 2-39c. These cracks start in the centre of the infill and run parallel to the compression strut. As the load increases, the cracks propagate until they extend to the corners of the panel. When the capacity of the diagonal strut has been reached, the crushing takes place over a relatively small region (see Figure 2-39 d). The onset of diagonal shear cracking should not be considered as the limiting or ultimate load condition for infill walls, because the ultimate load is governed by either the capacity of the diagonal strut or the bed-joint sliding shear resistance.

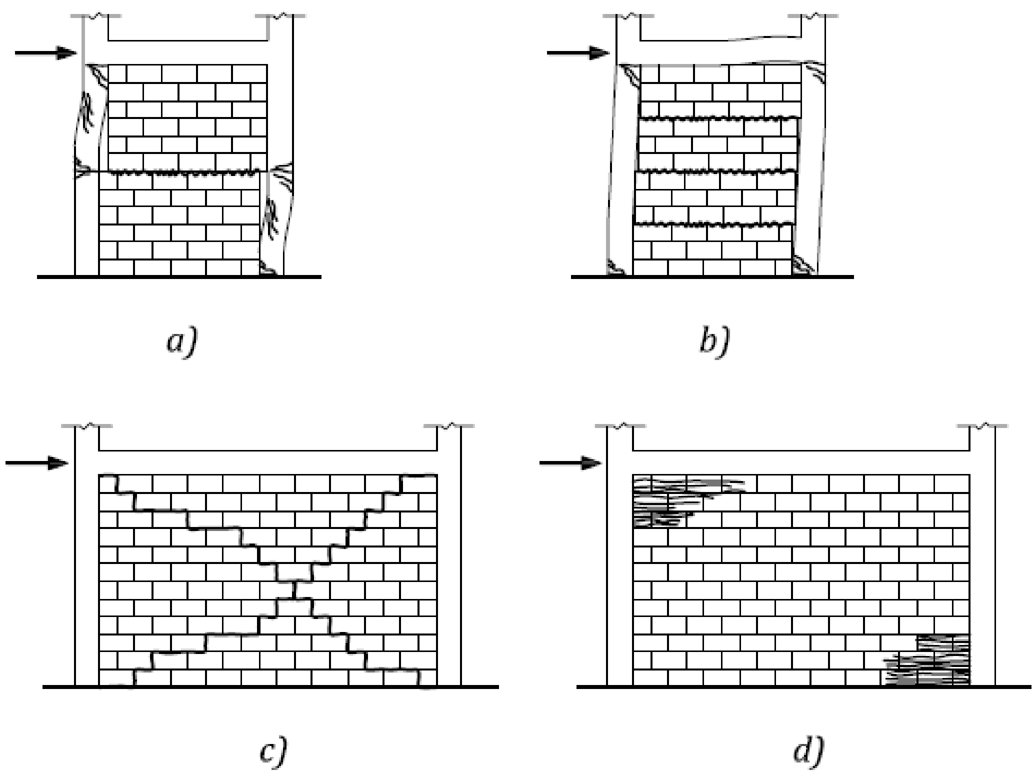


Figure 2-39. Masonry infill behaviour modes: a) and b) bed-joint sliding[1]; c) diagonal tension[2]; d) corner compression[2].

The diagonal strut mechanism can account for the additional stiffness provided by infill panels. It has been adopted by some design codes and guidelines for over 30 years, based on the pioneering research done in the1960s. It is the basis for the diagonal strut model proposed in CSA S304.1-04 (Stafford-Smith,1966), and its background has been further described in a more

[1] Tomazevic, 1999, reproduced by permission of the Imperial College Press
[2] FEMA 306, 1999, reproduced by permission of the Federal Emergency Management Agency

recent publication (Stafford-Smith and Coull, 1991). In this model, the effective strut width, w_e, is a function of the relative flexural stiffness of the column/beam and the infill, the height/length aspect ratio of the infill panel, the stress-strain relationship of the infill material, and the magnitude of diagonal load acting on the infill. Diagonal strut properties prescribed by international codes vary significantly (Kaushik, Rai, and Jain, 2006). For example, New Zealand Masonry Code NZS 4230:2004 prescribes that w_e should be taken as 25% of the length of the diagonal. Eurocode 8 (1988) prescribes that w_e should be taken as 15% of the diagonal length of the infill. The ACI 530-11 diagonal strut provisions are currently under development, as discussed by Henderson, Bennett, and Tucker (2007).

A key design parameter related to the diagonal strut model is the length of bearing (or contact) between the adjacent column and the infill (this parameter is denoted as α_h and α_L in CSA S304.1 Cl.7.13.3.2, for the column-infill or beam-infill contact length respectively). Experimental studies have shown that the bearing length is governed by the flexural stiffness of the column relative to the in-plane bearing stiffness of the infill. The stiffer the column, the longer the length of bearing, and the lower the compressive stresses at the interface (Stafford-Smith and Coull, 1991). This phenomenon is reflected in the CSA S304.1 equations used to determine α_h and α_L values. Note that the CSA S304.1 provisions are unique in that they prescribe two contact lengths – other codes and design recommendations use only the column contact length (corresponding to α_h in CSA S304.1).

Out-of-plane failure takes place due to ground shaking transverse to the plane of the wall. This mode of failure is more likely to occur at upper stories of a building, due to amplified accelerations, but it can also happen at lower stories due to concurrent in-plane loading that may damage the masonry. Arching is the prevalent mechanism in resisting out-of-plane seismic loads, because considerable out-of-plane strength can be developed even in cracked infills. This has been confirmed by several experimental studies (Dawe and Seah, 1989, and Abrams, Angel, and Uzarski, 1996). Note that the arching action is possible only for infills in direct contact with the frame (i.e. without a gap at the top). Out-of-plane strength estimates based on the flexural model of the infill acting as a vertical beam subjected to uniform load due to out-of-plane seismic load are rather conservative. Note that CSA S304.1-04 does not contain provisions related to out-of-plane resistance of masonry infills. Proposed ACI 530-11 design provisions for infill walls (currently under development), include an empirical design equation for the out-of-plane resistance of masonry infills based on the arching action, as proposed by Dawe and Seah (1989).

Isolated infill: when an infill panel is isolated from the frame, the gap (often called *seismic gap*), must be filled with a very flexible soundproof and fireproof material, e.g. boards of soft rubber or special caulking. The gap size (usually in the order of 20 to 40 mm) depends on the stiffness of the structure, the deformation sensitivity of the partition walls, and the desired seismic performance (Bachmann 2003). In addition to the gap on the sides and top of the panel, a restraint for out-of-plane resistance is required. This is typically provided in the form of clip angles or dowels at the top and/or sides that restrain out-of-plane motion only. These anchors should coincide with vertical and horizontal wall reinforcing (see CSA A370-04 for restraint information).

The above discussion mainly pertains to solid infills. Perforations within infill panels are the most significant parameter affecting seismic behaviour of infilled systems. Openings located in the centre portion of the wall can lead to weak infill behaviour. On the other hand, partial height infills can be relatively strong. The frames are often relatively weak in column shear, and partial height infills could potentially lead to a short-column mechanism (FEMA 306, 1999).

2.6.3 Stack Pattern Walls

Stack pattern is the arrangement of masonry units in which the head joints are vertically aligned (CSA S304.1 Cl.2.2.1). Stack pattern is not recommended for walls resisting seismic loads because, unlike a running bond pattern, the wall integrity provided by overlapping units is not available. The term stack pattern is now used, rather than stack bond, to highlight the lack of bond provided by this configuration of units. Stack pattern walls can be found in existing masonry buildings throughout Canada (see Figure 2-40a), and some older walls of this type are being demolished, as shown in Figure 2-40b. These walls act as a series of individual vertical columns, and the provision of horizontal reinforcement is essential to tie them together.

a)

b)

Figure 2-40. Stack pattern walls: a) stack pattern wall in an existing masonry building[1]; b) demolished stack pattern wall[2].

CSA S304.1-04 provisions regarding stack pattern walls of relevance for the seismic design are summarized in this section. CSA S304.1-94 did not contain any specific design provisions related to stack pattern walls.

[1] Credit: Svetlana Brzev
[2] Credit: Bill McEwen

2.6.3.1 Reinforcement requirements

CSA A371-04 Cl.8.1.3

Joint reinforcement or other horizontal reinforcement is required when structural or veneer masonry is laid in stack pattern, defined as less than a 50 mm overlap of masonry units.

10.10.3

Horizontal reinforcement for in-plane shear resistance in stack pattern walls shall be spaced at
- a) maximum 800 mm for bond beam reinforcing, and
- b) maximum 400 mm for wire joint reinforcing.

10.15.1
10.15.2

Reinforced stack pattern walls need to meet the minimum horizontal and vertical reinforcement requirements for non-seismic condition contained in Cl. 10.15.1, and the additional minimum seismic requirements of Cl.10.15.2 (see Section 2.5.4.7 and Table 2-2).

Commentary

Provision of horizontal reinforcement is critical for enhancing continuity in stack pattern walls. CSA S304.1-04 permits the use of joint reinforcement spaced at 400 mm or less, in addition to the bond beam reinforcement provided at a maximum spacing of 2400 mm (Cl.10.15.1.3). Codes in other countries, e.g. the U.S. masonry code ACI 530-08 (2008) Cl.1.11 states that the horizontal reifnforcement shall be placed at a maximum spacing of 48 in. (1219 mm) on center in horizontal mortar joints or in bond beams. Commentary to Cl. 1.11 states that “the use of horizontal reinforcement to enhance continuity in stack pattern walls is generally practical only by the use of bond beams”.

Note that gross cross-sectional area A_g for minimum area of vertical reinforcement according to Cl.10.15.1.1, should be calculated based on the effective compression zone width b discussed in Section 2.6.3.3.

2.6.3.2 In-plane shear resistance

10.10.3

The maximum factored vertical in-plane shear resistance in stack pattern walls shall not exceed that corresponding to the shear friction resistance of the continuous horizontal reinforcing used to tie the wall together at the continuous head joints (see Section 2.6.3.1 for horizontal reinforcement requirements).

Shear friction resistance shall be taken as

$$V_r = \phi_m \mu C_h$$

where
μ = 0.7 is the shear friction coefficient

C_h = compressive force in the masonry acting normal to the head joint. It is normally taken as the factored tensile force at yield of the horizontal reinforcement crossing the joint. This reinforcement must be detailed to develop its yield strength on both sides of the vertical joint.

Commentary

In-plane shear resistance of stack pattern walls is less than that of walls built in running bond. There is no masonry contribution to the shear resistance, so the resistance depends exclusively on the reinforcement crossing the vertical head joint. This is similar to the treatment of shear resistance at wall intersections prescribed in Cl.7.11.4 (see Section C.2).

Shear friction resistance, V_r, is proportional to the coefficient of friction, μ, and the clamping force, C_h, acting perpendicular to the wall height, h (see Figure 2-41). C_h is equal to the sum of tensile yield forces developed in reinforcement bars of area A_b, spaced at the distance s, that is:

$$C_h = \phi_s f_y A_b \, h/s$$

Reinforcing bars providing the shear friction resistance should be distributed uniformly across the vertical joint. The bars should be long enough so that their yield strength can be developed on both sides of the joint. Note that, in theory, a sliding shear plane can form along any vertical joint in a stack pattern wall.

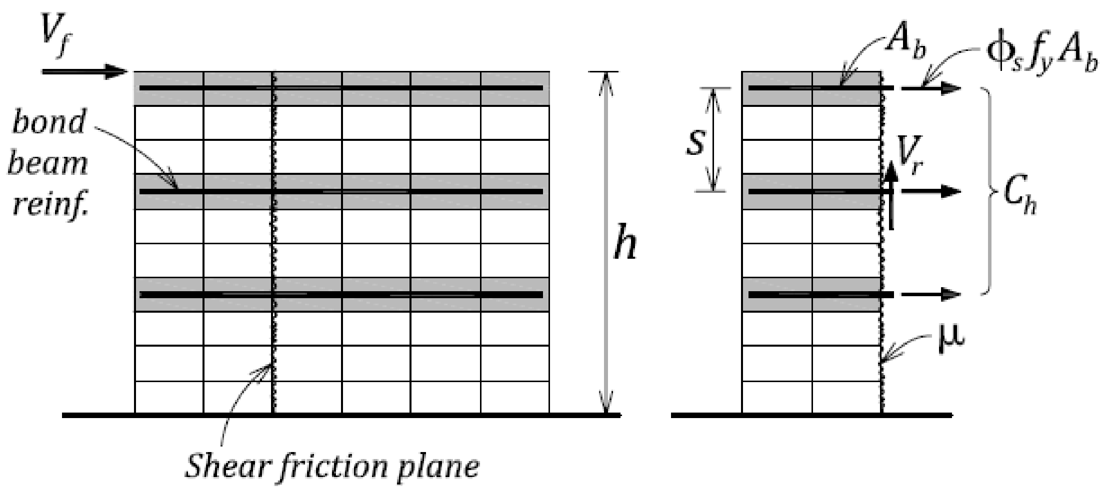


Figure 2-41. In-plane shear resistance of stack pattern walls.

2.6.3.3 Out-of-plane shear resistance

10.10.2

Out-of-plane shear resistance of stack pattern walls is determined according to the same provisions for walls built in running bond (see Section 2.4.3). Note that, for the purpose of shear resistance calculations, b includes the width of the cell and webs at a vertical bar within the length of the reinforced unit.

Commentary

Unless horizontal reinforcement is provided in sufficient amount (size and spacing), out-of-plane shear resistance of stack pattern walls is similar to that of a series of isolated vertical columns. In Figure 2-42 some stacks are not reinforced with vertical bars and so it is important to have adequate horizontal reinforcement to tie the stacks together.

2.6.3.4 Design for the combined axial load and flexure

The design approach for reinforced stack pattern walls for combined axial load and flexure is similar to that presented in Sections 2.3.4 and 2.4.4 for running bond. In determining the out-of-plane flexural resistance, the flexural tensile strength f_t should be taken equal to 0 for tensile resistance parallel to bed joints (S304.1 Cl.5.2.1). Also, the effective compression zone width b should be taken according to Cl.10.6.1.

10.6.1

For the case of out-of-plane loading (or "minor axis bending" as referred to in S304.1), the effective compression zone width, b, used with each vertical bar in the design of <u>stack pattern walls</u> with vertical reinforcement shall be taken as the <u>lesser of</u>

a) spacing between vertical bars, s, or
b) the length of the reinforced unit.

Figure 2-42 shows a portion of a reinforced stack pattern wall. In this example the length of the reinforced units is less than the spacing between bars and so the compression zone width, b, to be used with such bar is equal to the block length.

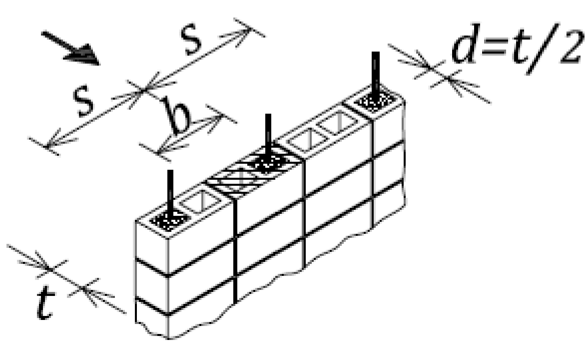


Figure 2-42. Effective compression zone width b for out-of-plane seismic effects in stack pattern walls.

2.6.3.5 Seismic requirements – plastic hinge region

10.16.4.1.3

CSA S304.1-04 permits the use of stack pattern in plastic hinge regions of ductile shear walls, however these regions must be solidly grouted and constructed of open-ended H-blocks.

Commentary

In addition to the proper amount and detailing of horizontal and vertical reinforcement in plastic hinge regions, the extent and continuity of grout is critical for the satisfactory seismic performance of reinforced masonry walls (see Section 2.5.4.2 for a detailed discussion on plastic hinge regions). Some codes, such as the New Zealand masonry code (NZS 4230:2004), do not permit the use of stack pattern walls in plastic hinge regions of the masonry walls, while the U.S. masonry code ACI 530-08 (2008) does (see Cl.1.17.3.2.6.e)..

2.6.3.6 Unreinforced stack pattern walls

CSA S304.1 does not contain any provisions related to unreinforced stack pattern walls. Cl.7.10.3 for unreinforced walls is identical to Cl.10.10.3 for the in-plane seismic resistance of reinforced stack pattern walls.

Commentary

The seismic performance of stack pattern walls without closely spaced horizontal reinforcement has been much less satisfactory than for walls constructed in running bond. The presence of horizontal reinforcement is critical for tying together vertical columns formed by stacked blocks (NZS 4230:2004).

Unreinforced stack pattern walls located in regions with moderate to high seismic risk are considered to be vulnerable to seismic effects and should be either retrofitted or demolished. It is suggested that unreinforced stack pattern walls not be used in seismic regions.

2.6.4 Nonloadbearing Walls

Nonloadbearing walls resist the effects of their own dead load and any out-of-plane wind and earthquake loads. This includes partitions and exterior walls that do not support floors and roofs (S304.1 Cl.2.2). However, walls that do not support floors and roofs, but resist the in-plane forces from wind and earthquake loads are considered loadbearing shear walls (see Section 2.5.4.7 for a detailed discussion on seismic reinforcement requirements for shear walls).

10.15.2.3 10.15.2.4

Minimum seismic reinforcement requirements for nonloadbearing walls are summarized below:

1. If $I_E F_a S_a(0.2) \le 0.35$

   Minimum seismic reinforcement is not required per CSA S304.1-04.
2. If $0.35 \le I_E F_a S_a(0.2) \le 0.75$ (Cl.10.15.2.4)

   Nonloadbearing walls shall be reinforced in one or more directions with reinforcing steel having a minimum total area of

$A_{stotal} = 0.0005A_g$

The reinforcement may be placed in one direction, provided that it is located to reinforce the wall adequately against lateral loads and that it spans between lateral supports.

3. If $I_E F_a S_a(0.2) \geq 0.75$ (Cl.10.15.2.3)

   Nonloadbearing walls shall be reinforced horizontally and vertically with steel having a minimum total area of

   $A_{stotal} = 0.001A_g$ distributed with a minimum area in one direction of at least

   $A_{v\min} = 0.00033A_g$ (approximately one-third of the total area).

   A_g denotes gross cross-sectional area corresponding to unit wall length (for vertical reinforcement), or unit height (for horizontal reinforcement). Note that this minimum total area is one half of that required for loadbearing walls.

10.15.2.6

Horizontal seismic reinforcement must be continuous between lateral supports in both loadbearing and nonloadbearing walls. Its spacing cannot exceed

(a) 400 mm where only joint reinforcement is used;
(b) 1200 mm where only bond beams are used; or
(c) 2400 mm for bond beams and 400 mm for joint reinforcement where both are used.

In terms of seismic design, the effect of out-of-plane seismic loads is likely going to govern the design of nonloadbearing walls. The approach for out-of-plane flexural design is similar to that presented in Section 2.4.4 for reinforced masonry walls. For unreinforced nonloadbearing walls, the design procedure presented in Section 2.6.1.3 should be followed.

Reinforced nonloadbearing walls include masonry enclosing elevator shafts and stairways, walls used as exterior cladding (not veneers), and masonry partitions which exceed 200 kg/m^2 in mass or are over 3 m in height, and are located at the sites where $I_E F_a S_a(0.2) > 0.75$ (Cl.4.6.1). Seismic requirements for nonloadbearing walls in CSA S304.1-94 were similar to the current code requirements. Cl.6.3.3.1 stated that minimum seismic reinforcement is required for nonloadbearing walls located in velocity or acceleration-related seismic zones 2 or higher. Minimum seismic reinforcement requirements stated in Cl.5.2.2.3 and 5.2.2.4 of CSA S304.1-94 are the same as the current requirements (S304.1-04 Cl.10.15.2.3 and 10.15.2.4) in terms of reinforcement area. S304.1-94 reinforcement spacing requirements were similar to those stated in Cl.10.15.2.6 of S304.1-04. Note that the item c) in Cl.10.15.2.6 of S304.1-04 covering the case of a combination of bond beams and joint reinforcement did not exist in the previous standard.

2.6.5 Masonry Veneers and their Connections

2.6.5.1 Background

In some applications and exposure conditions, the need for better control over rain penetration led to the incorporation of an air space or cavity in traditional masonry walls to provide a capillary break between two wythes. This type of two-stage wall can be referred to as a *rainscreen wall,* when the air space behind the outermost element is drained and ventilated to the exterior and an effective air barrier is included in the backup assembly. Masonry *veneer*, an important component of a modern rainscreen wall, is a nonloadbearing masonry facing attached to, and supported laterally by a structural backing. The structural backing may be structural masonry, concrete, metal stud or wood stud. A section of a typical rainscreen wall is shown in Figure 2-43.

While masonry veneers of brick, block or stone are nonloadbearing components, there are structural issues to be addressed if they are to perform satisfactorily. Veneers must be connected adequately to a structural backing by means of metal *ties* to ensure effective transfer of lateral loads due to wind and earthquakes. Steel angles are usually used to support veneers across openings (lintels), and to provide horizontal movement joints (shelf angles). For more information related to masonry veneers refer to the Masonry Technical Manual by MIBC (2008).

Veneer design is addressed by CSA S304.1-04 Cl.9 and CSA A370-04 Connectors for Masonry.

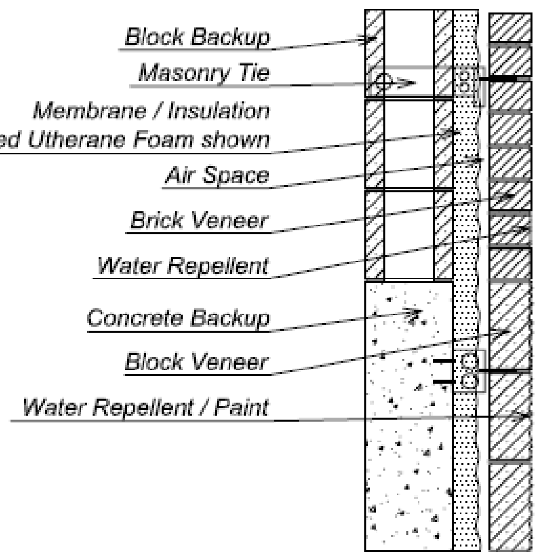


Figure 2-43. Key components of a masonry veneer (MIBC, 2008, reproduced by permission of the Masonry Institute of BC).

2.6.5.2 Ties

Ties are the key components that connect a veneer to a structural backing to ensure effective lateral load transfer. Tie requirements are outlined in CSA A370-04 Connectors for Masonry. The older kinds of ties, such as strip ties and Z-ties (now referred to as "Prescriptive Ties"), are seldom used in modern commercial construction, and cannot be used where the seismic hazard index, $I_E F_a S_a(0.2) > 0.35$. The newer, 2-piece, adjustable, engineered ties that are now in common use are now simply referred to as "Ties". CSA A370-04 contains strict design requirements for the corrosion resistance, strength, deflection and free play of ties.

CSA A370-04 requires stainless steel ties for masonry over 13 m high (formerly "buildings" over 11 m in CSA A370-94) for areas subject to high wind-driven rain. Hot dipped galvanized coatings are the acceptable minimum corrosion protection for walls 13 m or lower in these areas, and for all walls in drier areas. The standard provides wind-driven rain data for locations across Canada in Annex E, in terms of their Annual Driving Rain Index (aDRI).

The maximum tie spacing is prescribed by S304.1-04 Cl.9.1.3 and A370-04 Cl.7.1 as follows

- 600 mm vertically, and
- 820 mm horizontally

Note that S304.1-04 and A370-04 prescribe different value for horizontal tie spacing – the value of 820 mm prescribed by S304.1-04 is stated here because it better reflects construction dimensions.

While these tie spacings may be feasible for stiff backups like block and concrete, in most cases they cannot be achieved under the calculation method specified for flexible stud backups. The wind load lateral deflection limit for flexible stud backups supporting masonry veneer is span/360.

The factored resistance of a tie (P_r) is addressed by A370-04 Cl.9.4.2.1.2, and can be determined from the following equation

$P_r = \phi * P_{ult}$

where ϕ is the the resistance factor, which can assume the following values

ϕ = 0.9 for tie material strength

ϕ = 0.6 for embedment failure, failure of fasteners, or buckling failure of the connection.

P_{ult} denotes the ultimate tie strength. A370-04 requires that the ultimate strength of a masonry tie be not less than 1000 N.

2.6.5.3 Seismic load provisions for ties

Seismic load provisions for ties apply in areas in which the seismic hazard index $I_E F_a S_a(0.2) > 0.35$, and for all post-disaster buildings (NBCC 2005 Cl.4.1.8.17.2).

Ties are designed to resist the lateral wind and seismic loads acting perpendicular to the veneer surface, based on the tributary tie area. Seismic lateral loads on ties are determined from the provisions for elements and components of buildings and their connections (NBCC 2005 Cl. 4.1.8.17). The seismic tie load V_p is determined from the following equation:

$$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p$$

where

- $S_a(0.2)$= 5 % damped spectral response acceleration for a 0.2 sec period (depends on the site location; values for various locations in Canada from NBCC 2005 Appendix C)
- F_a = foundation factor, which is a function of site class (soil type) and $S_a(0.2)$ (see Section 1.5.2)
- I_E = building importance factor equal to1.0, except 1.3 for schools and community centres, and 1.5 for post-disaster buildings
- $S_p = C_p A_r A_x / R_p$ (where $0.7 < S_p < 4.0$)
- S_p = horizontal force factor for part or portion of a building and its anchorage (see NBCC 2005, Table 4.1.8.17, Case 8)
- C_p = seismic coefficient for a particular nonstructural component (equal to 1.0 for ties)
- A_r = response amplification factor to account for the type of attachment (equal to 1.0 for ties)
- $A_x = 1 + 2h_x / h_n$ amplification factor to account for variation of response with the height of the building (maximum 3.0 for the worst case at top of wall for ties). Note that $A_x = 3$ is the worst case for a tall building that may have higher mode contribution to accelerations in the top part of the building; thus $A_x = 3$ would be used for the entire top floor. For a single-storey building this doesn't make much sense. However, the accelerations will be higher at the top of a wall where the capacity is reduced because of low vertical load on the bricks, so $A_x = 3$ may be reasonable for the top row of ties. This could be reduced in the lower part of the wall, but for construction simplicity it would be better to maintain one spacing on most projects.
- R_p = element or component response modification factor (equal to 1.5 for ties).

So, the S_p value for tie design is

$S_p = 1.0 \cdot 1.0 \cdot 3.0/1.5 = 2.0$

W_p = tributary weight for a specific tie, equal to the unit weight of the veneer masonry (typically taken as 1.8 kN/m^2 for brick and cored block) times the tributary area (equal to the product of tie spacing for each direction).

The tie design load depends on the type of veneer backup (rigid/flexible), as per S304.1 Cl.9.1.3.3:

- For rigid backups (e.g. concrete block walls), the tie force is equal to the seismic load V_p corresponding to the tributary area weight W_p.
- For flexible backups (e.g. steel or wood stud walls), a tie must resist 40% of the tributary lateral load on a vertical line of ties. However, a tie must also be able to resist the load from double the tributary area on the tie.

The new formula from the NBCC 2005 may result in lower lateral seismic loads than the NBCC 1995 and may result in wind loads governing in more cases.

Factored tie capacities V_r are normally provided by test data from the manufacturers. The tie capacity is considered to be adequate provided that

$$V_p \leq V_r$$

If this is not a case, the tributary area and resulting tie spacing can be reduced until the above requirement is satisfied, or a stronger tie can be considered. In many cases, the design will begin with a given tie strength, with the resulting spacing calculated and assessed (see design Example 7 in Chapter 4).

2.6.6 Boundary Elements and Flanged Shear Walls

CSA S304.1-04 does not contain any specific seismic provisions regarding boundary elements in reinforced masonry shear walls. Boundary elements are thickened and specially reinforced sections provided at the ends of shear walls (see Figure 2-44a). The practice of using boundary elements is common for reinforced concrete ductile shear walls, with the related seismic design provisions included in CSA A23.3-04. In tall shear walls subjected to significant bending moments at their base, boundary elements provide an additional space to accommodate confinement and additional vertical flexural reinforcement. Boundary elements also provide stability against lateral out-of-plane buckling in thin wall sections (this was discussed in Section 2.5.4.4). To sustain high flexural and normal stresses, vertical reinforcement in the boundary elements must be well confined using properly anchored transverse reinforcement. This applies particularly to the plastic hinge regions of shear walls. Since seismic bending moments in reinforced masonry shear walls in low- and medium-rise buildings are not as high as those expected in RC shear walls in high-rise buildings, the provision of boundary elements may not be required.

It is of interest to note that U.S. masonry design standard ACI 530-08 (Clauses 3.3.6.5.1 to 3.3.6.5.5) contains provisions for boundary elements in reinforced masonry shear walls. However, Cl.3.3.6.5.1 states that it is expected that boundary elements will not be required in lightly loaded walls (e.g. $P_f \leq 0.1 A_g f'_m$ for symmetrical wall sections), in walls that are either short (squat) or moderate in height (aspect ratio $M_f / V_f l_w < 1.0$), or in walls subjected to moderate shear stresses. It is expected that most masonry shear walls in low- to medium-rise buildings would not develop high enough compressive strains to warrant special confinement.

Boundary elements may be required in shear walls to satisfy height-to-thickness requirements, or in walls in which flexural failure governs with the c/l_w ratio exceeding certain limit (similar to the ductility check procedure discussed in Section 2.5.4.3). For more details refer to ACI 530-08 Cl.3.3.6.5.3 and the commentary.

Flanged shear walls are discussed in Section C.2. A typical L-shaped flanged wall section is shown in Figure 2-44b. CSA S304.1-04 does not contain any specific seismic provisions related to flanged shear walls. Flanged shear walls are required to resist earthquake forces in both principal directions.

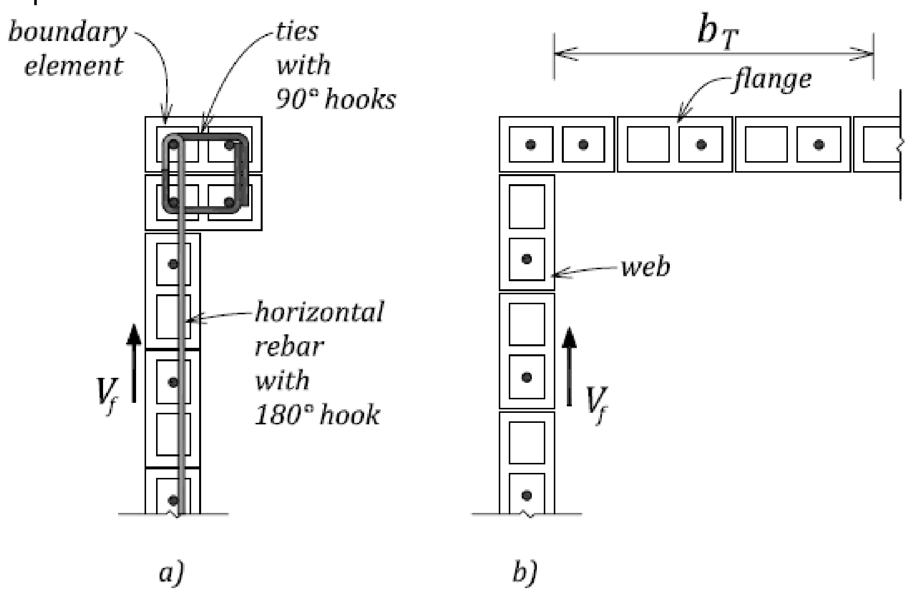


Figure 2-44. Boundary elements and flanges in reinforced masonry shear walls: a) boundary elements; b) flanged shear walls.

Paulay and Priestley (1992) proposed effective overhanging flange widths for reinforced concrete and reinforced masonry shear walls. For tension flanges, it is assumed that vertical forces due to shear stresses introduced by the web of the wall into the flange spread out at a slope of 1:2. For reinforced concrete flanged shear walls, the flexural strength of wall section with the flange in compression is insensitive to the effective flange width as the neutral axis is probably in the flange. After significant tension yield excursion in the flange, the compression contact area becomes rather small after load reversal, with outer bars toward the tips of the flange still in tensile strain.

As a result, the suggested the following overhanging flange width b_T to be used in seismic design for the flanges under tension and compression are as follows:

- Tension flange: $0.5h_w$
- Compression flange: $0.15h_w$

where h_w denotes the wall height. Note that these b_T values are different than the overhanging flange widths prescribed by CSA S304.1-04 for non-seismic design (see Table C-1 and Figure C-10 in Appendix C).

Shear walls with unsymmetrical flanges will have different flexural resistances, depending on whether flange acts in tension or in compression. Research studies on T-section walls have shown that such walls can exhibit larger ductility when the flanges are in compression. However, T- and L-section walls may have limited ductility when flanges are in tension (Paulay and Priestley, 1992; Priestley and Limin, 1995). Their experiments have shown that wall failure was sudden and brittle, and was initiated by a compression failure of the non-flange end of the wall, as shown in Figure 2-45 b). This was principally due to the large compression force needed to balance the large tension capacity of the reinforcement in the flange section.

In walls with unsymmetrical flanges, such as the T-section wall shown in Figure 2-45, the designer should be careful when applying the capacity design approach to determine flexural and shear capacity. The flexural capacity of the wall section is reached when the flange is in compression and the axial load is at minimum, $P_{f\,\min}$,as shown in Figure 2-45a. However, the maximum shear occurs when the flange is in tension and the axial load is at maximum, $P_{f\,\max}$, as shown in Figure 2-45b (this will result in a significantly higher flexural strength). A similar approach should be taken when the capacity design approach is applied to shear walls with pilasters.

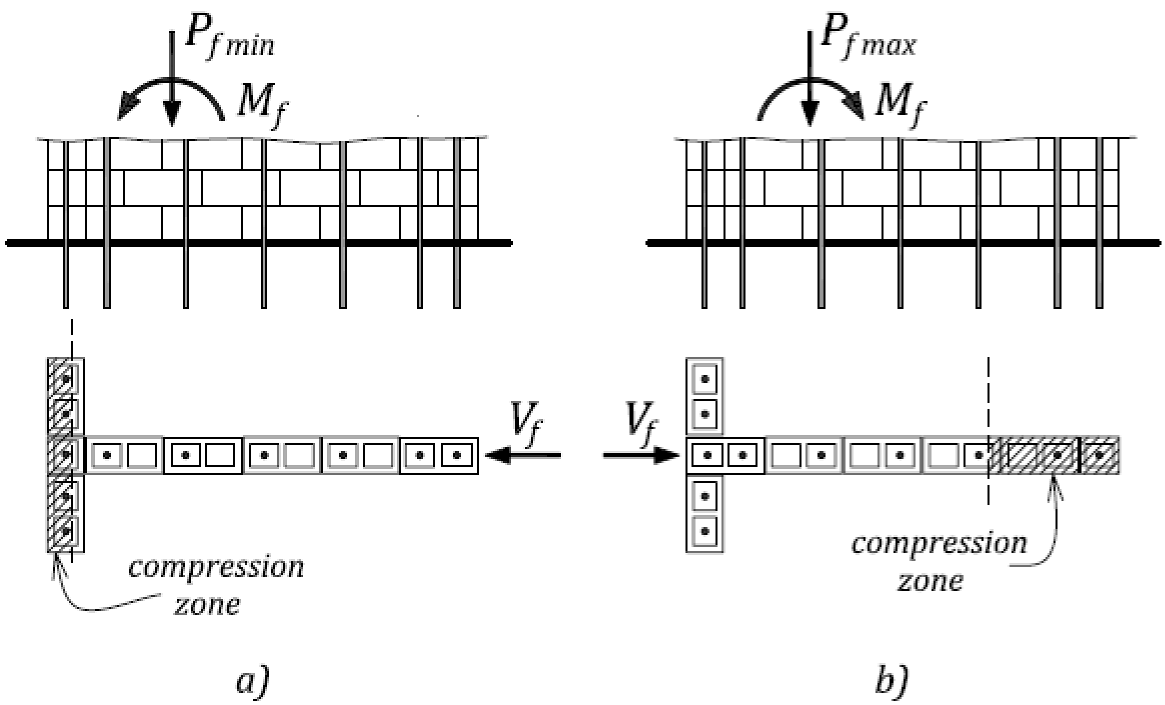


Figure 2-45. T-section flanged shear wall: a) flexural design scenario: web in tension; b) shear design scenario: web in compression.

The importance of web-to-flange connection for effective shear transfer in flanged shear walls is discussed in Section C.2. CSA S304.1-04 (clauses 7.11.1 to 7.11.3) prescribes three alternative approaches to achieve the effective shear transfer. Seismic studies in the U.S. under the TCCMAR research program resulted in recommendations related to horizontal reinforcement at the web-to-flange intersections (Wallace, Klingner, and Schuller, 1998). To ensure the effective shear transfer, horizontal reinforcement in bond beams needs to be continued from one wall into other, for a distance of 600 mm (2 feet) or 40 bar diameters, whichever is greater. The grout must be continued across the intersection by removing the face shells of the masonry units in one of the walls, as illustrated in Figure 2-46. Note that ACI 530-08 (Cl.1.17.3.2.6) requires that bond beams in ductile walls be provided at a vertical spacing of 1200 mm (4 feet).

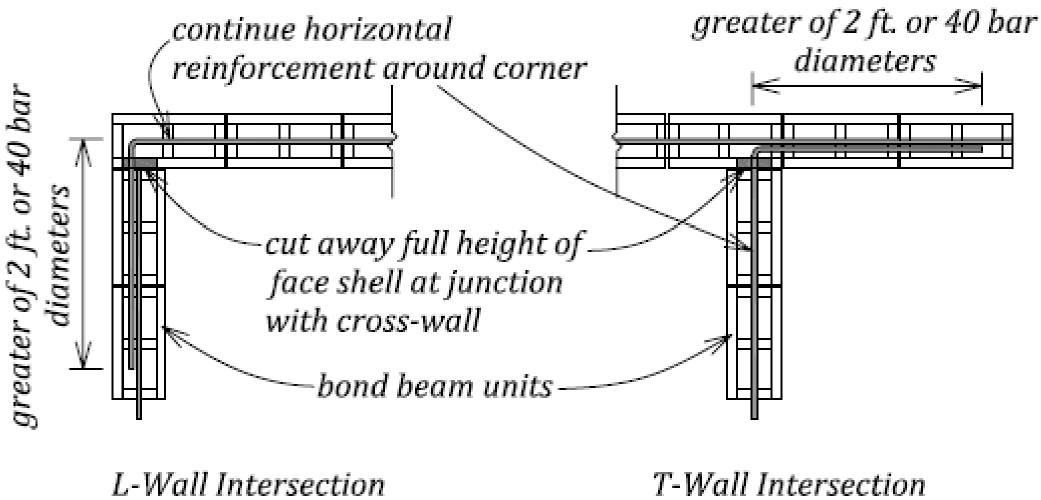


Figure 2-46. Horizontal reinforcement at the web-to-flange intersection: TCCMAR recommendations.

As an alternative to boundary elements, the New Zealand masonry standard NZS 4230:2004 Cl.7.4.6.5 prescribes the use of horizontal confining plates in ductile reinforced masonry walls. These thin perforated metal plates (made either of stainless steel or galvanized steel) are placed in mortar bed joints in the compression zone of rectangular walls. The confining plates are effective in increasing the maximum masonry compressive strain in plastic hinge regions to 0.008 (this value is significantly higher than the 0.0025 value prescribed by CSA S304.1-04 for unconfined walls). Provision of confining plates in the New Zealand masonry standard is based on the research done by Priestley (1982).

2.6.7 Wall-to-Diaphragm Anchorage

CSA A370-04

Masonry shear walls should be adequately anchored to floor and roof diaphragms.

The maximum anchor spacing between walls and horizontal lateral supports must not exceed ten times the nominal wall thickness (t+10 mm) (Cl.7.2.2). Anchors must be fully embedded in reinforced bond beams or reinforced vertical cells.

When the unfactored load applied normal to a wall is greater than 0.24 kPa, the ultimate strength of a wall anchor must not be less than 1,600 N (Cl.8.2.1).

Commentary

Anchorage is one of the most important, and in many cases the most vulnerable, components of existing masonry buildings exposed to earthquake effects. Many failures of masonry buildings result from wall-diaphragm failure that allows an out-of-plane wall failure, followed by a diaphragm failure.

Wall anchors must be effective in resisting the horizontal design forces from in-plane and out-of plane seismic loads. According to the capacity design approach, anchors should be designed to remain elastic in a seismic event (no yielding). This can be achieved by designing the anchor capacity based on the wall capacity, or on the elastic wall forces (corresponding to $R_d R_o$ of 1.0).

The anchors need to resist tension and shear forces, as shown in Figure 2-47.

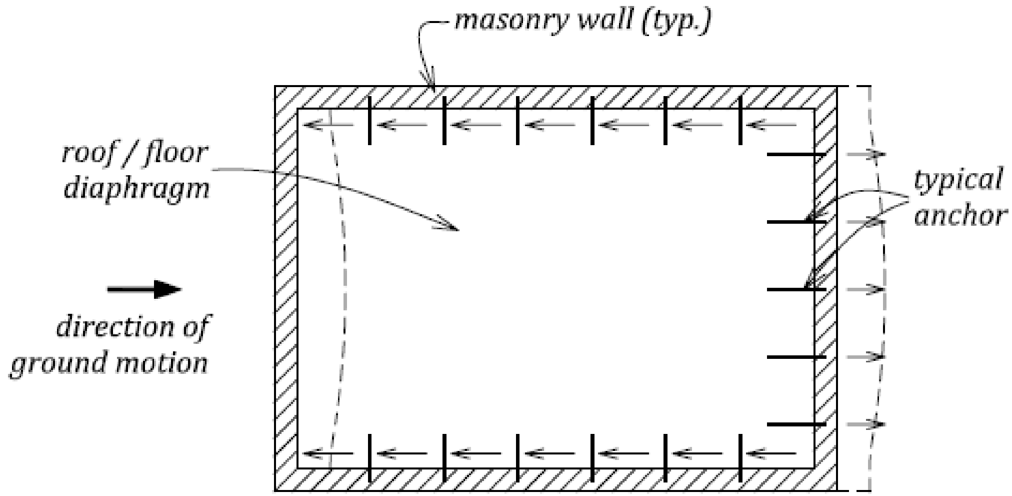


Figure 2-47. Tension and shear anchors at the wall-to-diaphragm connection.

Seismic load provisions for nonstructural components and their connections (including anchors) are provided in NBCC 2005 Cl.4.1.8.17.

2.6.8 Constructability Issues

Most of the information provided in this section has been adapted from the Masonry Technical Manual prepared by the Masonry Institute of BC (2008).

2.6.8.1 Reinforcement

Reinforced masonry is basically another form of reinforced concrete. However, reinforcing and grouting details should consider the core configuration of the masonry units. Care should be taken to disperse the rebar throughout the wall, and to avoid congestion in individual vertical cells. The cell size of the masonry units will dictate the size and number of bars that can be effectively grouted. A reinforcement arrangement, such as the one shown in Figure 2-48, is unsuitable and should be avoided. Typical reinforced masonry makes use of 15M or 20M bars. Units of 125, 150 and 200 mm nominal width should not contain more than one vertical bar per cell (2 bars at splices). 25M bars are occasionally used, but are more difficult to handle and require long laps. Vertical bars are typically placed in one layer in the centre of the wall. Site coordination is required to ensure that rebar dowels are installed to coincide with reinforced masonry cell locations.

Horizontal rebar is placed in bond beam courses using special bond beam blocks that have depressed or knock-out webs. Bond beams are typically spaced at 2400 mm vertically, but may also be positioned to coincide with lintel courses over openings. Joint reinforcement is often used in addition to bond beam bars. It is a ladder of 9 gauge (3.7 mm) galvanized wire installed in the mortar bed joint, which positions a wire in the centre of each block face shell. It is spaced at a maximum of 400 mm when used as seismic reinforcement. Joint reinforcement resists wall cracking and can contribute to the horizontal steel area in the wall. If joint reinforcement is not used, the maximum spacing of bond beams is 1200 mm for seismic detailing.

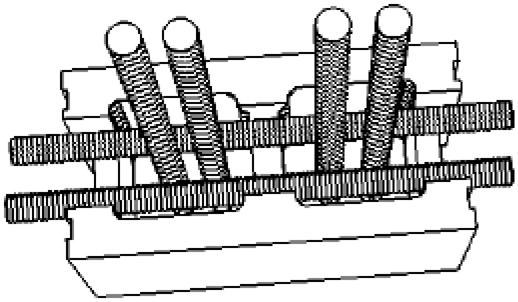


Figure 2-48. An example of inappropriate reinforcement arrangement: 2 bars vertically and 2 bars horizontally in a 20 cm wall are almost impossible to grout, particularly at splices where the steel is doubled (MIBC, 2008, reproduced by permission of the Masonry Institute of BC).

In addition to flexural, shear and minimum seismic steel, reinforcing is also required around openings over 1000 mm in loadbearing walls, at each side of control joints, and at the corners, ends, intersections and tops of walls. CSA S304.1-04 (Clause 4.6.1) allows unreinforced masonry partitions if they are less than 200 kg/m^2 in mass and 3 m in height, but only for seismic hazard indices $I_E F_a S_a(0.2) < 0.75$.

Nonloadbearing masonry partitions must have adequate top anchorage to avoid out-of-plane collapse. Dowels or angle clips must align with cells containing vertical bars (see CSA A370-04 for anchorage details). Bond beams at the tops of walls constructed under slabs or beams should be located in the second course below the slab to allow effective grouting of that course. Cells in the top course containing vertical bars can be dry packed with grout as they are laid with open-end units.

2.6.8.2 Masonry grout

Masonry grout, or “blockfill”, must flow for long distances through relatively small cells to anchor wall reinforcement. It is therefore placed at a much higher slump than regular concrete – in the range of 200 to 250 mm. While this water content would be problematic in cast-in-place concrete, in masonry the extra water necessary for placement is absorbed into the masonry units, thereby reducing the in-place water/cement ratio and providing adequate strength in the wall. Standard compressive strength tests using non-absorbent cylinders provide misleading data, as the extra water is trapped in the cylinder. Testing has shown the actual grout strength to be at least 50% higher than cylinder results. This situation is recognized in CSA S304.1 by basing masonry strength requirements on grout strengths of only 12.5 MPa by cylinder test. In some cases, a higher cement content grout (20 MPa) may be preferred for pumping reasons.

The most commonly used type of grout is Course Grout, which has a maximum aggregate size of 12 mm. Fine Grout uses coarse sand for aggregate and is usually used in small core units such as reinforced brick. Grout is supplied either by ready-mix truck or mixed on site, with quality control data available from the supplier or field test cylinders.

While grouting, care must be taken to completely fill the reinforced cores and to ensure that all bars, bolts and anchors are fully embedded. Vibration is usually not practical, but bars can be shaken to “puddle” the grout. Grout is often pumped in 2.4 m pours from bond beam to bond beam. The maximum pour height for “high-lift grouting” in CSA A371-04 Masonry Construction for Buildings is 4.5 m, but this should only be considered for H-block or 250 and 300 mm units. For total grout pours of 3 m or more, the grout must be placed in lifts of 2 m or less.

Sample base specification:

- Grout to meet CSA A179-04 requirements
- Minimum compressive strength 12.5 MPa at 28 days by cylinder test under the property specification
- Maximum aggregate size 12 mm diameter
- Grout slump 200 to 250 mm

2.6.8.3 Masonry mortar

Unlike reinforcing and grout, there are few issues in the specification, preparation and installation of mortar for structural masonry. CSA A179-04 covers mortar types and mixing, with Type S mortar almost always used for structural masonry. It provides the balance of mortar strength and bond that is required for good seismic performance. Unlike most cement-based products, compressive strength is not the dominant material criteria. Good bond results from mortar properties such as workability, adhesion, cohesion and water retention. Adequate bond binds the units together to provide structural integrity, tensile and shear capacity, and moisture resistance. In a mortar mix, Portland cement provides compressive strength and durability, while mortar cement, masonry cement or lime provides the properties that lead to good bond.

Most mortar is mixed on-site, and can be checked against the proportions specified in CSA A179-04. There are also pre-manufactured dry and wet mortars. The compressive cube strength required in CSA A179-04 for these products can be confirmed by plant or site test data. Site inspection of mortar mixing is generally not a significant concern for designers, because the bricklayer and the specifier are both looking for workable, well-proportioned mixes that provide installation efficiency for the mason, and long term performance for the designer. Mortar joints should be well filled and properly tooled for good performance. Concave tooled joints are the best shape for both structural and weather resistance.

Mortar joints compensate for minor dimensional variations in the masonry units, and provide coursing adjustment that may be necessary to meet required dimensions. Mortar joints also contribute to the architectural quality of the masonry assembly through colour and modularity.

2.6.8.4 Unit sizes and layout

Concrete masonry units are made in various sizes and shapes to fit different construction needs. Each size and shape is also available in various profiles and surface treatments. Concrete unit sizes are usually referred to by their nominal dimensions. Thus a unit known as 20 cm or 200x200x400 mm, will actually measure 190x190x390 mm to allow for 10 mm joints (see Figure 2-49). Standard nominal widths are 100, 150, 200, 250 and 300 mm, with 200 mm being the most common size for structural walls.

Working to a 200 mm module will minimize cutting, and maintain the alignment of vertical cells for rebar, as illustrated in Figure 2-50. Where possible, piers, walls and openings should be dimensioned in multiples of 200 mm. Foundation dowels must also be laid out to match the module of vertically reinforced cells.

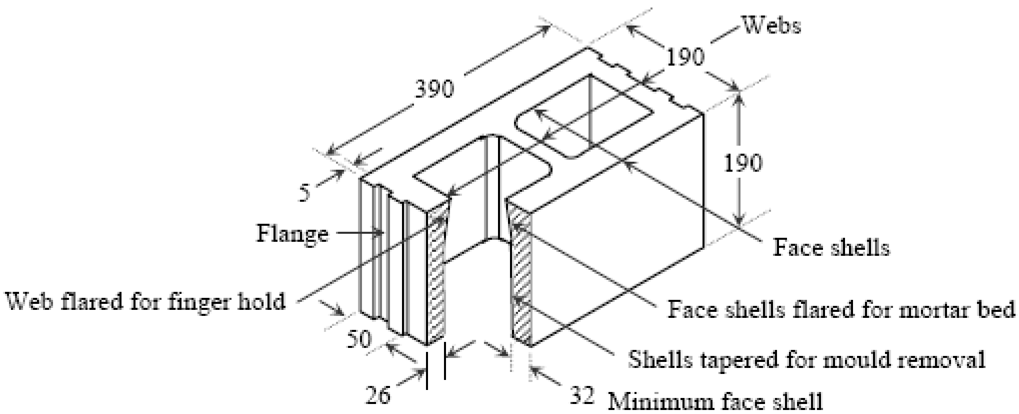


Figure 2-49. A typical 200 mm block unit (Hatzinikolas and Korany, 2005, reproduced by the authors' permission).

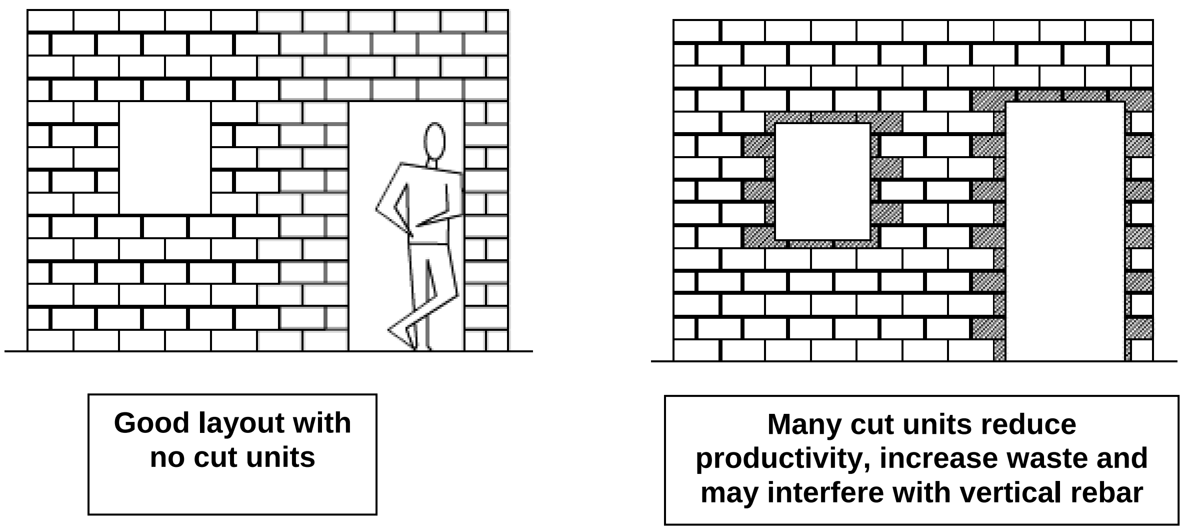


Figure 2-50. Examples of good and poor masonry layout (MIBC, 2008, reproduced by permission of the Masonry Institute of BC).

2.6.8.5 Other construction issues

In "high-lift grouting" (over 1.5 m), clean-out/inspection holes at the base of the reinforced cells may facilitate the removal of excessive mortar droppings and, more importantly, can confirm that grout has reached the bottom of the core. Clause 8.2.3.2.2 of CSA A371-04 allows the common practice of waiving the requirement for clean-out/inspection holes by the designer when the masonry contractor has demonstrated acceptable performance, or where the walls are not structurally critical. In some cases, the designer may require the initial walls to have clean-outs, pending demonstrated performance, and then waive them for the remaining walls.

Vertical movement joints in reinforced masonry walls are required to accommodate thermal and moisture movements, and possible foundation settlement. They are typically specified at a maximum spacing of 15 m.

Masonry walls should be installed to meet the requirements and tolerances of CSA A371-04 standard.