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THE NEW CSA S304-14 DESIGN OF MASONRY STRUCTURES: PART 2
DISCUSSION OF SEISMIC CHANGES TO THE STANDARD

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ABSTRACT

The most significant change to CSA S304-14 is the creation of the new Clause *16 Special provisions for seismic design*. This clause contains all provisions related to seismic design of masonry. The *Limited ductility* shear wall seismic force resisting system (SFERS) category has been removed and a *Conventional construction* shear wall SFERS category has been defined and consolidated to include sites of high seismic hazard indices. *Moderately ductile* squat shear wall SFERS now provides designers with clear directions on the detailing requirements of flexurally controlled versus shear controlled mechanisms. Moderately ductile shear walls now permit partial-grouting of the plastic hinge region under certain circumstances. A new *ductile* shear wall category has been included with a $R_d = 3.0$ and $R_o = 1.5$ as well as explicit calculations for the inelastic rotational demand and capacity of the plastic hinge region for moderately ductile and ductile shear walls. Finally, design provisions have been provided for the use of masonry boundary elements as a means to increase the usable masonry compression strain, ϵ_{mu} .

KEYWORDS: *boundary elements, design standards, ductility, plastic hinge, seismic design, shear walls*

INTRODUCTION

Historically, masonry has been used in structures or monuments considered to have high importance in society because of masonry's durability and architectural aesthetic. The modern equivalent to buildings which society places a high importance on may also be described as buildings such as hospitals or fire stations that are critical to the function of civil society and are expected to operate during and after crisis events with a lifespan longer than normal structures.

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These *post-disaster* buildings are defined by the National Building Code of Canada (NBCC) as "...a building that is essential to the provision of services in the event of a disaster..." and includes hospitals, electrical substations, pumping stations, etc. [1]. These buildings have historically been examples where loadbearing concrete block masonry is used because of its fire resistance, durability and strength.

The above tradition has gradually diminished over time as other materials took masonry's place taking advantage of advances in research and changes to building codes regarding combustible construction. Of particular importance was the requirement that *post-disaster* buildings be designed with a SFRS that possessed a ductility-based force modification factor (R_d) equal to or greater than 2.0. Because of this recent change, conventionally designed and detailed masonry was excluded from any post-disaster buildings at sites subject to the seismic design provisions of the 2005 NBCC. Seismically governed *post-disaster* masonry buildings were required to conform to one of two SFRS categories: *Moderately ductile shear walls* or *Moderately ductile squat shear walls*. In the following years, it became evident that the requirements for these two categories of post-disaster SFRS were often very difficult and costly to meet. Fortunately, these series of events led the masonry industry to take a deep introspective look at current Canadian research resulting in a renewed focus on seismic design and behaviour of masonry.

A focused national research effort on earthquake resistant masonry was collaboratively engaged over the following decade between the masonry industry and Universities across Canada. This culminated in creation of a new Clause 16 within CSA S304-2014 [2] entitled "*Special provisions for seismic design*". All of the seismic design provisions were consolidated, modernized and expanded for ease of reference; the level of detail and the methodology present now more closely mirrors what they may be found in CSA A23.3-2014 *Design of concrete structures* [3]. It will be noted that the categories of masonry SFRS have been entirely redefined, updated and expanded to now also include a new *Ductile shear wall* category of design.

SFRS CATEGORIES

To take advantage of the ability of any structure to deform beyond its elastic limit in a ductile manner, a designer may select from 45 recognized SFRS categories in the 2015 NBCC for a variety of construction materials. The NBCC divides its force reduction factor, R , into two independent seismic force modification factors which are multiplied together: the first is related to the ductility of the SFRS, R_d , and the second is based on the ratio of the anticipated overstrength (*actual* versus *assumed* strength) of the SFRS, R_o . Higher levels of R_d typically involve more restrictions on detailing and more design limitations to safely permit large inelastic deformations. By contrast, R_o is normally consistent for a given type of material based on the anticipated conservatism of material strength assumptions used in design and actual installed behaviour. The categories of masonry SFRS recognized by the 2015 NBCC, R_d and R_o values and height limitations as a function of seismic hazard index are provided in Table 1.

The seismic hazard index $IEF_a S_a(0.2)$ is defined as the product of the building importance factor (I_E), the site coefficient (F_a, F_v) and the spectral acceleration for a given natural period ($S_a(T)$). It is an indicator of the severity of earthquake to be designed for a particular location and building use. Building height restrictions are limited to the height above grade except when no limited is specified (NL) or where a SFRS is not permitted (NP).

Table 1: Masonry SFRS Categories as Specified by [1]

| Type of Masonry SFRS | R_d | R_o | $IEF_a S_a(0.2)$ | | | | $IEF_v S_a(1.0)$ |
|---|-------|-------|--------------------------------|----------|-----------|-------|------------------|
| | | | <0.2 | 0.2-0.35 | 0.35-0.75 | >0.75 | >0.3 |
| | | | <i>Height Restrictions (m)</i> | | | | |
| Ductile Shear Walls | 3.0 | 1.5 | NL | NL | 60 | 40 | 40 |
| Moderately Ductile Shear Walls | 2.0 | 1.5 | NL | NL | 60 | 40 | 40 |
| Moderately Ductile Squat Shear Walls | | | | | | | |
| Conventional Construction Shear Walls | 1.5 | 1.5 | NL | 60 | 30 | 15 | 15 |
| Conventional Construction Moment Resisting Frames | | | NL | 30 | NP | NP | NP |
| Unreinforced Masonry | 1.0 | 1.0 | 30 | 15 | NP | NP | NP |
| Any Other SFRS not Listed | | | 15 | NP | NP | NP | NP |

Unreinforced Masonry ($R_d = 1.0, R_o = 1.0$)

Unreinforced masonry (URM) buildings can resist lateral earthquake loads through a number of different configurations which can include masonry frames, shear walls, squat walls and flanged walls. The available design freedom is due to the requirement for the preservation of elastic dynamic behaviour through design for seismic forces determined with $R_d R_o = 1.0$. URM is restricted to regions with a low seismic hazard index and URM should not be used in combination with reinforced masonry (RM) walls when sharing of seismic load depends on relative stiffness. Load sharing between URM and RM through tributary area and not wall stiffness is not restricted by CSA S304. Otherwise there are no specific restrictions to detailing of URM that differ from the design for other loads.

Conventional Construction: Shear Walls ($R_d = 1.5, R_o = 1.5$)

In this SFRS category, masonry must be reinforced to resist seismic loads. However, the selection of conventional construction generally means that either the seismic hazard is low or that there is ample resistance to lateral load so as not to require a more rigorous design. Conventional SFRS categories of masonry were not explicitly described in the 2004 edition of CSA S304; it was an implicit category based on minimum seismic reinforcement requirements. New in CSA S304-2014 are Equations 1 and 2 for effective moment of inertia (I_e) and effective shear area (A_e) that are provided for seismic modelling across all SFRS categories. A further significant change is that walls of low stiffness may be considered to be “minor walls” and can be excluded from the SFRS. By definition, a minor wall must resist no more than 2.5% of the total seismic force nor more than 50% of the average force resisted by walls in the SFRS. Combined, minor walls must resist less than 10% of the seismic force. Minor shear walls must be able to remain elastic under seismic loads and deflections and be able to support any applied axial loads. Minimum reinforcement and restrictions to the axial loads given in Table 2 need not apply to minor walls.

$$I_e = I_g (0.3 + P_s / (A_g f'_m)) \quad \text{where } I_{cr} \leq I_e \leq I_g \quad (1)$$

$$A_e = A_g (0.3 + P_s / (A_g f'_m)) \quad \text{where } A_{cr} \leq A_e \leq A_g \quad (2)$$

Where the factored dead plus live load at the base of the wall is given as P_s , the gross uncracked cross-sectional moment of inertia and area are given as I_g and A_g , respectively, and the cracked section moment of inertia and area are given as I_{cr} and A_{cr} , respectively.

Table 2: Reinforcement Requirements for Conventional Construction Shear Walls

| Seismic Hazard Index | Vertical Reinforcement | | Horizontal Reinforcement | | Axial Load (MPa) |
|-------------------------------------|---|---|------------------------------|--|------------------|
| | Min. Area (mm ²) | Max. Spacing ($S_{(vert.)}$) (mm) | Min. Area (mm ²) | Max. Spacing ($S_{(hor.)}$) (mm) | |
| $I_E F_a S_a(0.2) < 0.35$ | 0.00125 A_g^i or 0.00125(4 $t \times t$) ⁱⁱ | 12($t+10$) or 2,400 mm | N/A | 1,200 mm or 0.5 ℓ_w^v ; | No limit |
| $0.35 \leq I_E F_a S_a(0.2) < 0.75$ | 0.00125 A_g^i or 0.00125(4 $t \times t$) ⁱⁱ and 0.00067 $A_g^{iii,iv}$ | 12($t+10$) or 2,400 mm 6($t+10$) or 1,200 mm | 0.00067 $A_g^{iii,iv}$ | 400 mm ^{vi} ; 2,400 mm or 0.5 ℓ_w^{vii} and 400 mm ^{viii} | 0.1 f'_m |

ⁱ If reinforcement is spaced less than 4 t apart, Cl. 10.15.1.1 applies if vertical reinforcement is required for axial compression and bending;

ⁱⁱ If reinforcement is spaced more than 4 t apart, Cl. 10.15.1.1 applies if vertical reinforcement is required for axial compression and bending;

ⁱⁱⁱ A_g measured perpendicular to the direction of the reinforcement considered;

^{iv} The sum of the areas of vertical and horizontal reinforcement must be at least 0.002;

^v When only bond beams are used;

^{vi} When only joint reinforcement is used;

^{vii} Bond beam spacing when used in conjunction with joint reinforcement;

^{viii} Joint reinforcement spacing when used in conjunction with bond beams;

Moderately Ductile Squat Shear Walls ($R_d = 2.0$, $R_o = 1.5$)

Researchers have noted that squat masonry walls with an aspect ratio (wall height-to-length) equal to or close to 1.0 may possess shear- or flexural-governed failure modes, often displaying characteristics of both [4][5]. Although shear is typically considered to be quite a brittle type of failure, squat shear walls which experience a mixed failure mode may typically demonstrate significant post-peak load capacity and ductility [6][7][8]. CSA S304 recognizes that squat shear walls, which have an aspect ratio less than 1.0, may achieve sufficient energy dissipation through either shear or flexural mechanisms. Predicting the nonlinear displacement and ductility of squat walls is very difficult without adopting complex nonlinear shear models [9], although a simplified ductility check for shear governed walls is provided by [10]. This is why there is no ductility verification check for squat shear walls nor is there any plastic hinge. Assurance of adequate ductility is established by providing sufficient reinforcement to maintain post-peak strength.

The minimum ratios of vertical and horizontal reinforcement are based on the force equilibrium of an idealized 45° crack in the wall as indicated in Figure 1. Equilibrium of shear stress in the wall results in a vertical and horizontal projection of shear force (V_f) which must be less than the horizontal projection of force (shear reinforcement) which, for an idealized crack with height and length equal to h_w , has a value of $F_{hor.} = \phi_s(\rho_v b_w h_w) f_y$. In the vertical direction the shear force must be less than the summation of the contribution of axial load over the cracked area ($P_s (h_w/\ell_w)$) and the vertical projection of force (vertical reinforcement) of $F_{vert.} = \phi_s(\rho_v b_w h_w) f_y$. Re-arranging these,

one can derive the required vertical (ρ_v) and horizontal (ρ_h) reinforcement ratios required for squat walls, indicated in Equations 3 and 4 as they appear in the CSA S304.

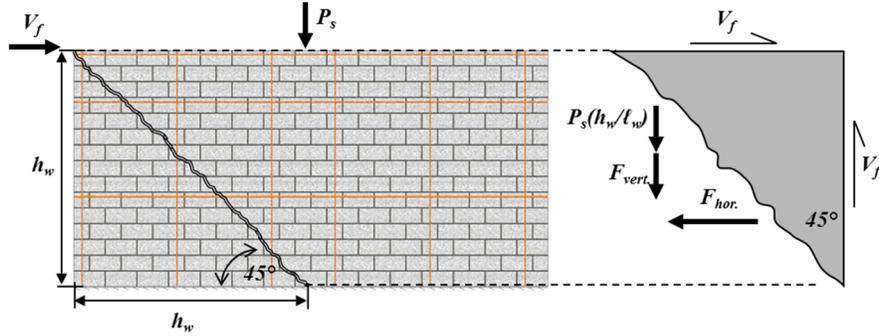


Figure 1: Shear Crack Equilibrium for Squat Shear Walls

$$\rho_h \geq V_f / (\phi_s b_w h_w f_y) \quad (3)$$

$$\rho_v \geq \rho_h - P_s / (\phi_s b_w l_w f_y) \quad (4)$$

Very squat walls, light axial load and partially-grouted masonry all tend to lead to shear-governed designs. For instance, a partially-grouted shear wall may have a moment capacity that is almost identical to a fully-grouted shear wall, but the shear strengths will differ more significantly because of the hollow cells and reduced area. The governing failure mechanism (shear versus flexure) determines the required level of over-design. A *flexurally governed* shear wall must satisfy the criteria: $V_r/V_f > M_r/M_f$, whereas a *shear governed* shear wall must satisfy $V_r/V_f < M_r/M_f$. A flexurally governed wall must have shear strength greater than the lesser of the shear force corresponding to the development of the factored moment resistance or the shear force corresponding to the seismic force generated using a reduced ductility of $R_d R_o = 1.3$, selected to maintain capacity design principles [3].

Moderately Ductile Shear Walls ($R_d = 2.0$, $R_o = 1.5$)

Recent experimental research in Canada [11][12] and the U.S. [13][14][15] has helped refine design requirements for plastic hinging in flexurally governed masonry walls. This includes partial-grouting in the plastic hinge region based on recent testing by numerous researchers [16][17][18][19]. The original clauses contained in the 2004 edition of CSA S304 for moderately ductile shear walls were viewed by many designers as being confusing and difficult to apply. Construction of Post-disaster masonry building in moderate seismicity regions suffered as a result. Based upon the more recent scientific publications, CSA S304-2014 provisions for moderately ductile shear walls were updated. Changes made from 2004 to 2014 are summarized in Table 3.

The previous simplified expression given to evaluate the ductility capacity of a moderately ductile shear wall was related to the aspect ratio of a wall and the neutral axis depth calculated for earthquake load cases. However, this ductility check made a number of assumptions that had limited design applications with no guidance on how to account for walls outside of the defined properties. CSA S304-2014 provides explicit means to determine the rotational demand, θ_{id} ,

(Equation 5) and capacity, θ_{ic} (Equation 6) for the plastic hinge to permit an empirical comparison of ductility capacity [20]. Furthermore, this more comprehensive ductility verification readily adapts to the use of alternative solutions, such as the use of masonry confinement, in cases where conventional masonry systems cannot satisfy ductility demands.

Table 3: Summary of Important Changes to Moderately Ductile Shear Walls 2004 to 2014

| Provision | CSA S304.1 2004 | CSA S304 2014 |
|---|--|--|
| Unsupported wall height-to-thickness ratio of wall section with plastic hinge. | Cl. 10.16.5.2.3 provided a hard limit of $h/(t + 10) < 14$. For a wall made with 20cm units this limited storey heights in the plastic hinge to 2.8m. | Cl. 16.8.3 gives different limits depending on the wall cross-section in the plastic hinge. $h/(t+10) < 30$ if: distance to neutral axis is less than $4b_w$ of $0.3\ell_w$, if neutral axis for a flanged wall lies within $3b_w$ and the flange is at least $h/5$ wide and 190 mm thick. $h/(t+10) < 30$ for the remainder of a wall if a thickened wall end is used such as a boundary element that extends at least half the distance to the neutral axis. $h/(t+10) < 20$ for all other cases unless additional analysis is shown for low axial load under $10\%f'_m$. |
| Partial-grouting of the plastic hinge region of a wall. | Cl. 10.16.4.1.3 does not permit partial-grouting of the plastic hinge. Partial grouting may be used outside the plastic hinge. | Cl. 16.8.5.2 permits partial-grouting of the plastic hinge region of a wall when $h_w/\ell_w \leq 2.0$ and either: $I_E F_a S_a(0.2) < 0.35$ or; $I_E F_a S_a(0.2) \geq 0.35$ and axial stress is under $10\%f'_m$. Partial-grouting outside the plastic hinge is permitted. |
| Horizontal reinforcement bending details. | Cl. 10.16.5.4.2 requires reinforcing bars with 180° hooks around vertical bars (throughout the wall including areas outside the plastic hinge) and spaced at not more than 1,200mm in the plastic hinge. | Cl. 16.8.5.4 requires reinforcing bars with 90° hooks spaced at the lesser of 1,200mm or $\ell_w/2$ in the plastic hinge. No special restrictions outside the plastic hinge. |
| Lap splices in horizontal and vertical reinforcement. | Cl. 10.16.4.3.3 restricts lapping of horizontal reinforcement to outside a region equal to the greater of 600 mm or neutral axis depth from the wall edge. Cl. 10.16.5.4.1 requires that no more than half the area of vertical steel in a wall may be lapped at any section in the plastic hinge. | Cl. 16.8.5.4 restricts lapping of horizontal reinforcement to outside a region equal to the greater of 600 mm or $\ell_w/5$ from the wall edge. No restriction on amount lapped at any section. Cl. 16.8.5.5 requires that lap splices in the plastic hinge shall be at least $1.5\ell_d$ long. |
| Extent of the plastic hinge region. | Cl. 10.16.5.2.1 defines the plastic hinge region as a height extending to the greater of: ℓ_w or; $h_w/6$ where ℓ_w is the length of the wall being designed. | Cl. Cl. 16.8.4 defines the plastic hinge region as a height extending to the greater of: $\ell_w/2$; $h_w/6$ but $< 1.5\ell_w$ where ℓ_w is the length of the longest wall in the SFRS. |
| Ductility verification of the wall and the inelastic rotation capacity the plastic hinge. | Cl. 10.16.5.2.3 the maximum compression strain in the masonry compression zone is restricted to 0.0025. The ductility capacity of a wall must be evaluated and verified to meet or exceed $R_d = 2.0$. This is deemed to be satisfied when: $c < 0.2\ell_w$ (Aspect ratio < 4) $c < 0.15\ell_w$ ($4 < \text{Aspect ratio} < 8$) | Cl. 16.8.6 the maximum compression the maximum compression strain in the masonry compression zone is restricted to 0.0025. Cl. 16.8.7, Cl. 16.8.8 contain explicit methods to evaluate ductility an inelastic rotational capacity as will be discussed in more detail in this chapter. |
| Seismic shear resistance in the plastic hinge. | Cl. 10.16.5.3.1 shear resistance in the plastic hinge region is determined by reducing the contribution of masonry and axial load from Cl. 10.10.1 by one-half. Cl. 10.16.5.3.2 sliding shear resistance in the plastic hinge is determined by reducing the compressive force C acting at the base of a wall by the yield force of vertical reinforcement in the compression zone in Cl. 10.10.4. | Cl. 16.8.9.1 shear resistance in the plastic hinge region is determined by reducing the contribution of masonry and axial load from Cl. 10.10.2 by 25%. Cl. 16.8.9.2 requires that shear resistance of the entire wall be greater than the smaller of either: the shear corresponding to the nominal moment resistance (M_n) determined with ϕ_m and $\phi_s = 1.0$, or; the shear determined using $R_o R_d = 1.3$. |

$$\theta_{id} = \frac{(\Delta_{fl} R_d R_o - \Delta_{fl} \gamma_w)}{h_w - \frac{\ell_w}{2}} \geq \theta_{min} \quad (5)$$

$$\theta_{ic} = \left(\frac{\varepsilon_{mu} \ell_w}{2c} - 0.002 \right) \leq 0.025 \quad (6)$$

Whereby, Δ_{fl} is the lateral deflection at the top of the wall, γ_w is a wall over overstrength factor not less than 1.3, θ_{min} is the minimum inelastic rotational demand equal to 0.003 for moderately ductile

shear walls and 0.004 for ductile shear walls. Capacity design principles are applied to design of moderately ductile shear walls to ensure a ductile flexural failure with inelastic plastic hinge rotations. Yielding of vertical reinforcement in the plastic hinge during a design level earthquake leads to progressive cracking and weakening of the masonry in those areas. Furthermore, the cyclic nature of earthquake loads with directional changes will exacerbate the degradation of masonry within the plastic hinge region. To account for this factor, the contribution of masonry and axial load to shear resistance are reduced by 25% for moderately ductile shear walls. Furthermore, the minimum factored shear force is increased to the lesser of the shear corresponding to the nominal moment resistance (M_n) of the wall or the shear corresponding to an earthquake load calculated using $R_o R_d = 1.3$. M_n can be determined using material resistance factors of ϕ_s, ϕ_m equal to 1.0. CSA S304-2014 takes a conservatively large assumption of the plastic hinge with respect to the detailing requirements (above) and a conservatively large plastic hinge ($h_p = \ell_w$) is assumed for the inelastic rotational demand calculations. By contrast, the inelastic rotational capacity assumes a conservatively small $h_p = \ell_w/2$.

Ductile Shear Walls ($R_d = 3.0, R_o = 1.5$)

Ductile Shear Walls is a new category of masonry SFRS in 2015 NBCC and CSA S304-2014. The ductile and moderately ductile categories share the same height restrictions in the NBCC, however the increased R_d factor, representing a 33% decrease in seismic design force, leads to higher ductility demand within the plastic hinge region and correspondingly more stringent detailing requirements including the use of weldable grade reinforcement. Ductile shear walls cannot be squat and must have fully-grouted plastic hinge regions under all circumstances. Due to the higher ductility demand, the unsupported height-to-thickness ratio limits are more restrictive than moderately ductile shear walls. A summary of the height-to-thickness ratios for the plastic hinge region of ductile walls are provided in Table 4. The plastic hinge height (h_p) is calculated as $0.5\ell_w + 0.1h_w$, subjected to $0.8\ell_w < h_p < 1.5\ell_w$, where ℓ_w the longest wall in the SFRS.

Table 4: Unsupported Height-to-Thickness Ratios for Ductile Shear Walls

| Section of Shear Wall containing Plastic Hinge | | Moderately Ductile Shear Walls | Ductile Shear Walls |
|---|------------------|--|--|
| Default Value of Unsupported Height-to-Thickness Ratio | | 20 | 12 |
| Wall Containing Boundary Element | Boundary Element | 20 | 12 |
| | Rest of Wall | 30 | 16 |
| Rectangular Cross-Section | | If $c < (4b_w \text{ or } 0.3\ell_w) = 30$ | If $c < (4b_w \text{ or } 0.3\ell_w) = 16$ |
| Flanged Cross-Section (Flange Width $> h/5$, Flange thickness $> 190\text{mm}$) | | If $c < 3b_w = 30$ | If $c < 3b_w = 30$ |

Lap splices are permitted in the plastic hinge region with a minimum length of $1.5\ell_d$. No more than 50% of all vertical rebar may be lapped at any cross section. Ductile shear walls have a maximum spacing of vertical reinforcement of $\ell_w/4$, but need not be less than 400 mm unless required for strength calculations. Horizontal reinforcement must be provided by bars with a maximum spacing of 600 mm or $\ell_w/2$ and detailed with 180° standard hooks around vertical bars at the ends of the wall. Such a detail can be difficult to include within normal 20 cm concrete masonry units especially when lap splices are also present at a hook location.

Shear resistance provided by the masonry and axial load equation are reduced by 50% within the plastic hinge region (damage zone) of ductile shear walls. In addition, for sliding shear resistance, the compressive force in the masonry (C) is reduced by the compressive yield force of the vertical reinforcement within the compression zone. The explanation is that under load reversals reinforcement which yields under tension for loading in one direction will be under compression for the reversed lateral load. Finally, the factored shear resistance must be equal to the smaller of the shear calculated assumed $R_d R_o = 1.3$ or the shear corresponding to the development of the probable moment resistance (M_p) calculated using the unfactored material strength ($\phi_m, \phi_s = 1.0$) and an assumed strength of the reinforcement equal to $1.25 f_y$. If designers have difficulty meeting the required inelastic rotational demand of $\theta_{min} = 0.004$ using rectangular wall cross-sections and conventional reinforcement, adding a small flange will likely work.

Increasing Masonry Compression Strain with a Boundary Element.

Moderately Ductile and Ductile shear walls require verification of their inelastic rotational demand and capacity for seismic loads. Rotational capacity is a function of wall curvature which in turn is a function of the stress-strain relationships of masonry and reinforcing under compression and tension, respectively. Ductility verification Equations 5 and 6 use the peak compressive strain of masonry (ϵ_{mu}) directly in calculations. For seismic design, the default value for strain is $\epsilon_{mu} = 0.0025$. For determination of inelastic curvature capacity, a designer has practical control over only a few parameters. Wall length, height and aspect ratio are typically fixed values by the engineering stage of design and are not easily altered. This leaves the depth of compression zone, c , and the strain in the masonry. Either reducing c or also increasing ϵ_{mu} will improve the inelastic rotational capacity of the cross-section. CSA S304-2014 provisions for thickened sections at the wall ends (referred to as boundary elements) may be utilized to achieve both goals. A summary of the provisions for achieving a higher value of ϵ_{mu} are summarized in Table 5.

Table 5: Special Provisions for Shear Walls with Increased ϵ_{mu}

| Provision | Moderately Ductile Shear Walls | Ductile Shear Walls | Moderately Ductile or Ductile Shear Walls with $\epsilon_{mu} > 0.0025$ |
|---|--|---|---|
| Maximum ϵ_{mu} | 0.0025 | 0.0025 | 0.008 |
| Extent of Plastic Hinge | $h_p = \text{greater of: } \ell_w/2 \text{ or } h_w/6$ $h_p \leq 1.5\ell_w$ | $h_p = 0.5\ell_w + 0.1h_w$ $0.8\ell_w \leq h_p \leq 1.5\ell_w$ | $h_p = 0.5\ell_w + 0.1h_w$ $\ell_w \leq h_p \leq 2.0\ell_w$ |
| Shear Resistance of Masonry plus Axial Load (V_m) | $0.75 \times V_m$ | $0.5 \times V_m$ | $(0.0025 / 2\epsilon_{mu}) \times V_m$ |
| Sliding Shear Resistance | $\phi_m \mu C$ | $\phi_m \mu C - F_{s(\text{comp. zone})}$ | $\phi_m \mu C - F_{s(\text{comp. zone})}$ |
| Minimum Factored Shear | $M_n (\phi_m, \phi_s = 1.0)$ and $R_o R_d = 1.3$ | $M_p (\phi_m, \phi_s = 1.0, f_s = 1.25 f_y)$ and $R_o R_d = 1.3$ | $M_p (\phi_m, \phi_s = 1.0, f_s = 1.25 f_y)$ and $R_o R_d = 1.3$ |

Where, ϵ_{mu} can be increased through use of proprietary devices which must be experimentally verified for their efficacy and for which no prescriptive requirements are given in the CSA S304, or through special boundary elements constructed of conventional masonry units which do have prescriptive design requirements given based on experimental testing [21][22][23]. Boundary elements require at least four vertical reinforcing bars to be placed in at least two rows so that ties around these can confine the grout. A typical masonry boundary element is of modular length and width and may protrude out from the face of the wall on one or both sides depending on the

configuration of unit as indicated in Figure 2. Boundary elements are detailed to have buckling prevention ties in the form of hoops or stirrups as well as seismic cross-ties.

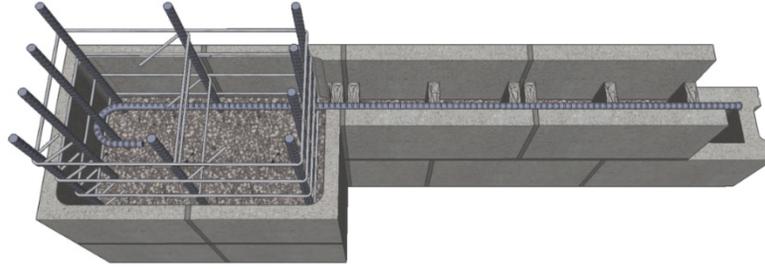


Figure 2: One Possible Boundary Element Configuration as defined by the CSA S304-14

Buckling prevention ties must be spaced at a distance not lesser than: six times the vertical bar diameter, twenty four times the tie diameter or one-half the least dimension of the member. The effective confinement and new design value of ϵ_{mu} can be determined directly from Equation 6 from the minimum buckling prevention tie area calculation of [3].

$$A_{sh} = 0.2k_n k_{pl} \frac{A_g}{A_{ch}} \frac{f'_m}{f_{yh}} s h_c \quad (7)$$

Similar to the approach given in [3], the minimum tie area, A_{sh} , is a function of spacing, s , gross area of the boundary element, A_g , and the outside-to-outside area of the confined core of the boundary element, A_{ch} . The factor k_n accounts for the number of vertical bars that are laterally supported by the corners of ties or by seismic cross ties, n_t , as depicted in Figure 3 and determined as $k_n = n_t / (n_t - 2)$. The factor k_{pl} , related to the target strain in the boundary element masonry, is determined as $0.1 + 30\epsilon_{mu}$. The outside-to-outside dimension of the core measured perpendicular to the direction of the buckling prevention ties, h_c , as depicted in Figure 4.

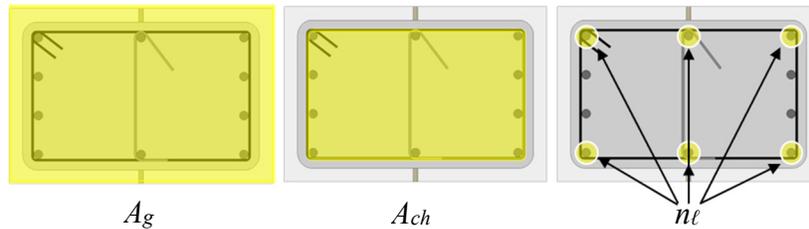


Figure 3: Definitions of A_g , A_{ch} and n_t

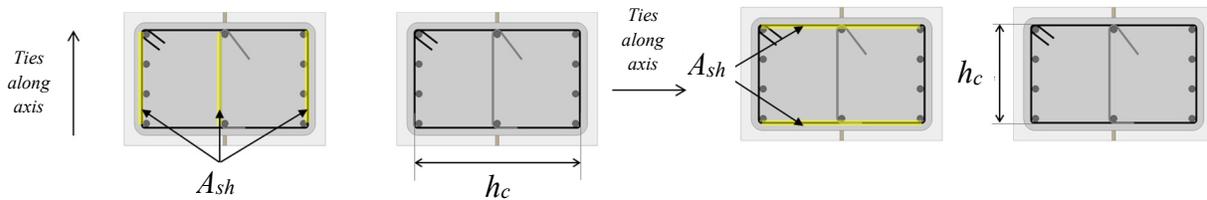


Figure 4: Definitions of Axis-Specific Parameters A_{sh} and h_c

CSA S304-2014 specifies minimum vertical reinforcement in a boundary element based on the overall wall area within the plastic hinge region of a shear wall as $0.00075b_wl_w$. Outside of the plastic hinge region, a boundary element must have a vertical reinforcement area of at least $0.0005b_wl_w$. It can be advantageous to reduce the size of boundary elements in multi-storey construction when inelastic rotational and moment demands reduce in size. However, compatibility for structural regularity according to the NBCC must be maintained and that such changes to strength and stiffness of the structure must be properly reflected in the overall seismic design. Finally, the interface shear at the connection between boundary element and the web of the shear wall should be checked with Equation 8.

$$V_{fr} = \phi_m \mu F_s \quad (8)$$

Where V_{fr} is the interface shear strength at the connection facilitated by the frictional force created by the tensile force of horizontal reinforcing crossing the connection, F_s , and the coefficient of friction of masonry-to-masonry, μ , taken as 1.0. Horizontal reinforcement must be detailed to achieve its yield strength on both sides of the interface.

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