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A PROPOSED METHOD TO ADDRESS THE LIMITATION ON AXIAL LOAD FOR CONVENTIONAL CONSTRUCTION SHEAR WALLS IN THE 2014 CSA S304

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ABSTRACT

Design for earthquake forces in Canada underwent major changes at both the national model building code and masonry design standard levels from 2004 to 2014. The most recent edition of the CSA S304 Design of Masonry Structures published in 2014 introduced a new section: Clause 16 *Special provisions for seismic design*. One of the technical additions to the standard was a limit to the design level of axial load for *Conventional Construction* shear walls for buildings that possess with a *moderate* seismic hazard ($I_E F_a S_a(0.2) \geq 0.35$). A maximum axial compressive stress of not more than $0.1f'_m$ under seismic load cases is permitted. It is well understood that flexurally governed shear walls possess a greater level of inelastic energy dissipation when axial loads are low. In seismic design, it is generally preferable to have a small ratio of the depth of neutral axis, c, to the length of the wall, ℓ_w, to ensure inelastic yielding of flexural reinforcement. Although seismic response is likely enhanced by restricting axial loads to the required level, the proposed limit has shown itself to be difficult to meet within typical multi-storey loadbearing masonry raising concerns from the design community about its application. It can also be observed that the current limit is often more restrictive than what one could calculate using rational calculations or the limits for c/ℓ_w adopted by comparable walls systems in reinforced concrete and masonry design in the U.S. In lieu of the axial load limit of $0.1f'_m$, a designer is permitted by CSA S304 to carry-out *a more comprehensive analysis*, the basis of which is not defined by the standard. The following paper provides a rational means to meet the *more comprehensive analysis* requirements of the CSA S304 in order to design conventional construction shear walls with axial load levels that are over $0.1f'_m$.

KEYWORDS: *CSA S304, conventional construction, seismic design, shear walls*

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INTRODUCTION

A number of changes were made in the development of the 2014 edition of the CSA S304 design standard that have been summarized previously [1]. One such change was the consolidation and expansion of seismic design requirements with the creation of Clause 16 *Special provisions for seismic design*. New to the 2014 CSA S304 is the explicit definition of the design requirements for the seismic force resisting system (SFRS) *Conventional Construction Shear Walls* (R_d = 1.5, R_o = 1.5), which were previously inferred from the standard [2]. Notably, some technical changes were also introduced in 2014 including the introduction of an axial load limit on walls based on the seismic hazard index summarized in the text below (based on CSA S304 (16.5.3)[2]).

> ***CSA S304 16.5.3 Limitation on axial loads for $I_E F_a S_a(0.2) \geq 0.35$***
> *The axial compressive stress on a reinforced masonry wall due to factored load effects designed for seismic loadings corresponding to R_d = 1.5 shall be limited to not more than $0.1f'_m$ where the seismic hazard index, $I_E F_a S_a(0.2)$, is equal to or greater than 0.35.*

There are several issues raised with this requirement in light of contemporary design requirements. Firstly, National Building Code of Canada (NBCC) [3] already restricts the use of Seismic Force Resisting Systems (SFRS) based on the seismic hazard index. In the 2015 edition of NBCC *Conventional Construction* masonry shear walls are already limited to a maximum height of 30 m for a seismic hazard index between 0.35 and 0.75, and a height of 15 m when the hazard is greater than 0.75. Limitations to building height are in place to account for systems that possess relatively low levels of ductility, R_d = 1.5, and for which design requirements are left fairly simplistic and prescriptive in nature.

Next, the limit on axial load equal to $0.1f'_m$ represents a sudden, and dramatic, drop to wall capacity compared to *Conventional Construction* shear walls with a seismic hazard index equal to 0.34 or less. This has the appearance of being arbitrary and in the case of multi-storey construction is rather punitive. When the seismic hazard index is below 0.35, CSA S304 (10.4.1)[2] limits reinforced masonry walls, which do not contain compression reinforcement, to a maximum axial load equal to:

$$P_{r(\max)} = 0.80\left(0.85\phi_m f'_m A_e\right) \tag{1}$$

Considering a fully-grouted cross-section, the effective cross-section area, A_e, is equal the wall length, ℓ_w, multiplied by the wall thickness, t. If Eq. 1 is compared to an effective compression block equation for design of masonry under compression force, C_m, the following relationship can be solved for:

$$P_{r(\max)} = C_m$$
$$0.80\left(0.85\phi_m f'_m \ell_w t\right) = 0.85\phi_m f'_m \beta_1 ct \tag{2}$$
$$\frac{c}{\ell_w} = 1.0$$

The same process can be applied to the axial load limit of CSA S304 (16.5.3)[2]. Ignoring the presence of tension reinforcement, the maximum axial stress of $P_f \leq 0.1f'_m$ can be substituted into the formula for the masonry under compression to yield the following:

$$P_f = C_m$$
$$0.10\left(f'_m \ell_w t\right) \leq \phi_m\left(0.85 f'_m\right)\beta_1 ct$$
$$\frac{c}{\ell_w} \leq \frac{0.10}{\phi_m\left(0.85\right)\beta_1} \tag{3}$$
$$\frac{c}{\ell_w} \leq 0.245$$

In this case, the actual neutral axis depth in a wall would be less than $c/\ell_w = 0.245$ because of the presence of tension reinforcement. To maintain equilibrium that tension force would appear on the applied axial load side of Eq. 3, consequently making $c/\ell_w < 0.245$, where the actual ratio would change with the masonry strength and reinforcement ratio in the wall. A summary of these limits can be visualized and compared through the lens of an interaction diagram, one of which is plotted for a *Conventional Construction* shear wall as depicted in Fig. 1. Axial load limits identified above are denoted along the vertical axis.

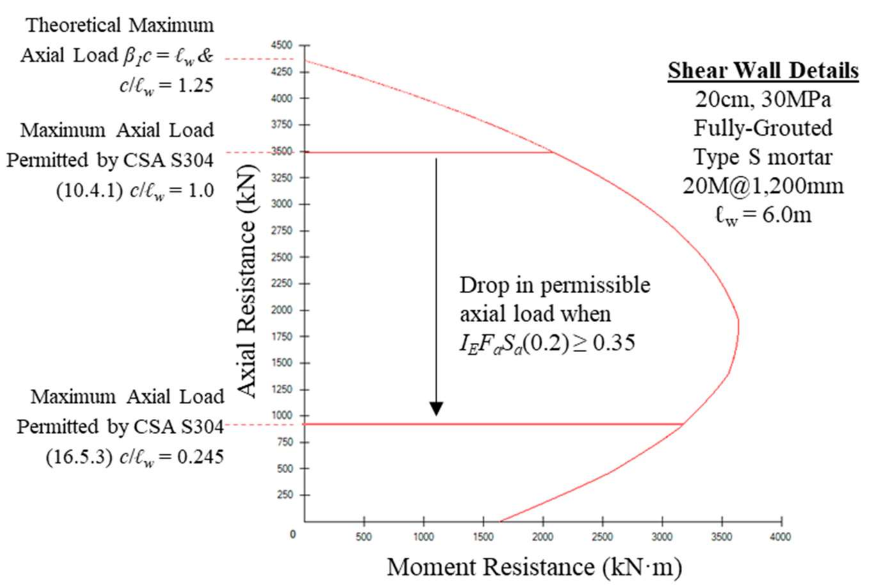

Figure 1: Interaction Diagram and Axial Load Limits of a Conventional Construction Shear Wall

Regardless of what the actual limit for a wall's detailing works out to be, the upper bound of 0.245 represents a sudden and significant drop in allowable axial load in a *Conventional Construction* shear wall when the seismic hazard changes from 0.34, where $c/\ell_w \leq 1.0$, to 0.35, where $c/\ell_w < 0.245$. Furthermore, since axial stress is already a function of building height, this limit appears to supersede provisions already provided by the NBCC regarding height limits for the SFRS.

The final issue raised by this limit is that appears to be arbitrary. There is currently no theoretical basis to assign a design level of axial load to be based on a singular percentage of the specified masonry strength. It has been well established that c/ℓ_w ratios provide insight into the theoretical ductility capacity of a shear wall undergoing plastic hinging. However, the level of ductility capacity is also related to the extent of inelastic rotations within the wall (plastic hinge height, h_p) and the height-to-length (aspect) ratio, A_r, of the wall, and the strains at ultimate for the masonry and steel reinforcement. Furthermore, the ductility capacity required for a shear wall must equal or exceed the ductility demand imposed on the wall for a given seismic hazard. The ductility demand itself is also a function of h_p, A_r and, per NBCC [3], the design levels of wall overstrength (R_o), and ductility (R_d) for the SFRS.

CSA S304 (16.5)[2] does not provide any rationale or any explicit alternative approach to analysis that might otherwise overcome its strict reduction to axial load levels. The only means permitted to deviate from the axial load limit is through CSA S304 (16.5.1)[2], which describes that "a more comprehensive analysis" may be performed.

> ***CSA S304 16.5.1 General***
> *Shear walls of conventional construction designed for seismic loadings corresponding to R_d = 1.5 shall be designed in accordance with Clause 16.5.2 to 16.5.4, unless a more comprehensive analysis is performed.*

A MORE COMPREHENSIVE ANALYSIS METHOD

In the proposed design requirements meant to meet the criteria as a more comprehensive analysis, an important distinction should first be made that is currently not addressed in CSA S304. There is a difference in the behaviour, and means of inelastic energy dissipation, between squat and non-squat shear walls. Inelastic seismic energy dissipation can be facilitated through flexural plastic hinging in non-squat walls: a behaviour that is easy to quantify and evaluate. However, in squat walls inelastic energy dissipation is facilitated through shear dominated response in the wall, something that is not currently quantified or measured within the CSA S304, but rather can be prescriptively met through detailing, as done for *Moderately Ductile* squat shear wall design.

Design of Conventional Construction Squat Shear Walls

Squat walls are defined by CSA S304 as possessing an $A_r < 1$. It was previously inferred that limiting axial loads is done as a means to limit the c/ℓ_w ratio in the wall to permit sufficient inelastic rotational capacity within a plastic hinge. However, squat walls are not expected to undergo an inelastic flexural response to lateral loading. Rather, to ensure that a *Conventional Construction* squat wall can meet the inelastic demands required for R_d = 1.5, the process for designing R_d = 2.0 *Moderately Ductile* squat shear walls in CSA S304 (16.7)[2] (which recognizes a shear-governed inelastic energy dissipation mechanism) can be adapted to support a more comprehensive analysis.

Since ductility is not explicitly quantified for squat walls, only provisions related to *axial load effects* in the design of *Moderately Ductile* squat shear walls in CSA S304 (16.7)[2] will be considered and conservatively applied to *Conventional Construction* squat walls. Clauses that are deemed to relate to ensuring ductility and resisting cyclic loads effects will not be adopted based on the fact that a ductility force reduction factor of only R_d = 1.5 is being demanded, instead of R_d = 2.0, which these provision were originally intended for. An overview of the design requirements that form *a more comprehensive analysis* for *Conventional Construction* squat shear walls is summarized as the following additional design requirements in Table 1.

Table 1: Design Requirements for Conventional Construction Squat Shear Walls

Proposed Design Requirement	***Commentary***
Squat shear walls having a height-to-length ratio (h_w/ℓ_w) less than 1 shall be designed to the following:	*These requirements are in addition to all those already applicable to Conventional Construction shear walls. The following can be used in lieu of the axial load limit of CSA S304 (16.5.3) [2].*
a. The unsupported height of the wall shall be such that the height-to-thickness ratio $h_w/(t+10)$, of the wall in the compression zone is less then 20 (per CSA S304 (16.7.4) [2])	*Since there is no rational way to alter this limit for the reduced ductility demand of R_d = 1.5 versus 2.0, it is suggested that the Moderately Ductile squat shear wall limit is used directly.*
b. Horizontal and vertical reinforcement ratios shall not be less than that determined using the following equations (per CSA S304 (16.7.5) [2]): $\rho_h \geq \frac{V_f}{\left(\phi_s b_w h_w f_y\right)}$ $\rho_v \geq \rho_h - \frac{P_s}{\left(\phi_s b_w \ell_w f_y\right)}$	*Reinforcement requirements that relate to axial load are included here. This is done to ensure that compression struts in the cracked cross-section are properly accounted for within the equilibrium of forces in the reinforcement. These requirements are designed to ensure R_d = 2.0 behaviour and are conservatively also adopted here.*

Design of Conventional Construction Non-Squat Shear Walls

Non-squat walls permit direct evaluation of the c/ℓ_w ratio to ensure adequate inelastic flexural capacity to facilitate inelastic rotational demands placed on the wall. In reinforced concrete design, *Conventional Construction* concrete shear walls conforming to CSA A23.3 (R_d = 1.5, R_o = 1.3) are to be designed such that c/ℓ_w is less than 0.5 (CSA A23.3 (21.6.3.7.5)[4]) under seismic load cases. No specific limits are given based on the seismic hazard index. In the NBCC, *Conventional Construction* concrete shear walls are permitted to heights of 40 m when the seismic hazard index is between 0.35 and 0.75 and up to 30 m when the seismic hazard index is over 0.75, which are both heights greater than that permitted for *Conventional Construction* masonry shear walls [3].

In the U.S., the 2016 edition of TMS 402 [5] contains a shear wall design category termed *Ordinary Reinforced Shear Walls* which are a reasonable approximation for *Conventional Construction* shear walls used in Canada. TMS 402 does not directly provide a c/ℓ_w ratio to be maintained for seismic design, but instead limits the maximum reinforcement ratio based on an imposed strain gradient for a prescribed load case [5]. In the case of *Ordinary Reinforced Shear Walls* a strain in the extreme tension reinforcement equal to at least 1.5 times the yield strain, ε_y, is required. Although not directly comparable (due to several major difference between design standards, including the fact this limit is applicable for a fictional load case) this limit approximates to a c/ℓ_w ratio of 0.45.

Nowhere in CSA S304 is a singular c/ℓ_w ratio limit adopted that could be adapted for use with *Conventional Construction* shear walls. Rather, CSA S304 (16.8.8.4)[2] design for *Moderately Ductile* shear walls provides an equation based on mechanics to directly estimate the limiting c/ℓ_w ratio based on the inelastic rotational demand and inelastic rotational capacity within the plastic hinge region of a shear wall. This equation also appears in reinforced concrete design [4]. Since R_d and R_o can be directly inputted as variables in the equation, its application is possible for the *Conventional Construction* category and serves as the basis for the proposed design requirements meant to meet the criteria as a more comprehensive analysis given in Table 2.

Table 2: Design Requirements for Conventional Construction Non-Squat Shear Walls

Proposed Design Requirement	***Commentary***
Shear walls having a height-to-length ratio (h_w/ℓ_w) greater than or equal to 1 shall be designed to the following:	*These requirements are in addition to all those already applicable to Conventional Construction shear walls. The following can be used in lieu of the axial load limit of CSA S304 (16.5.3) [2].)*
a. The unsupported height of the wall in the *plastic hinge* shall be such that the height-to-thickness ratio $h_w/(t+10)$, of the wall in the compression zone is less then 20. Exclusions are also permitted for walls with thicker sections at the end, neutral axis depths of a certain limit and walls with flanges per CSA S304 (16.8.3.2-16.8.3.4)[2] but are not repeated here for brevity.	*Since there is no rational way to alter this limit for the reduced ductility demand of R_d = 1.5 versus 2.0, it is suggested that the Moderately Ductile shear wall limits are used directly.*
b. The area of wall which contains the plastic hinge shall be fully-grouted.	*Moderately Ductile shear walls are permitted to have a plastic hinge region that is partially-grouted per CSA S304 (16.8.5.2)[2]. However, this is limited only to walls with a seismic hazard less than 0.35 and axial stress less than $0.1f'_m$. Therefore, it can be reasoned that this same conservatism should also be applied to walls of Conventional Construction with high axial loads and high seismic hazard. This is likely an overconservative approach.*

Table 2: Continued

c. The extent of the plastic hinge region, h_p, above the base of the wall shall be taken as ℓ_w.	*To achieve R_d = 1.5 through flexural yielding of reinforcement the theoretical height over which this yielding occurs must be assumed. This limit only applies to detailing requirements a. and b. The same extent of plastic hinge is assumed for Conventional Construction reinforced concrete shear walls [4].* *Other detailing requirements for the plastic hinge (i.e. lap splices, bar spacing, ε_{mu} etc.) are intended to facilitate a plastic hinge that can develop R_d = 2.0. Existing detailing requirements for Conventional Construction walls R_d = 1.5 should still apply since there is no change in ductility demand/capacity. i.e. "The plastic hinge region is an already existing feature in flexurally-governed Conventional Construction shear walls that can achieve R_d = 1.5 using the provisions of CSA S304 (16.4 and 16.5)[2]."*
d. The inelastic rotational demands and maximum axial load for a Conventional Construction non-squat shear wall are deemed to be satisfied by the following limit per CSA S304 (16.8.8.4)[2]: $\frac{c}{\ell_w} \le \frac{\varepsilon_{mu}}{\frac{4A_r}{(2A_r-1)}\left(\frac{\Delta_{f1}R_dR_o}{h_w}\right)\left(1-\frac{\gamma_w}{R_dR_o}\right)+2\varepsilon_y}$ Where: c = the neutral axis depth for the factored earthquake load case under consideration ℓ_w = the length of the wall being designed ε_{mu} = 0.003 A_r = the aspect ratio of the *wall being designed* (h_w/ℓ_w) Δ_{fl} = the lateral elastic deflection at the top of the shear wall under factored loads γ_w = the wall overstrength factor equal to the ratio of load corresponding to the nominal moment resistance (determined using $\phi_m = \phi_s$ = 1.0) to the factored load on the wall. It need not be taken as less than 1.3 and shall not be taken greater than R_oR_d = 2.25. ε_y = 0.002	*This equation can be derived by assuming that the plastic hinge region at the base of the wall is conservatively taken as equal to ℓ_w for determining inelastic rotational demands per CSA S304 (16.8.8.2)[2] and as $\ell_w/2$ when determining inelastic rotational capacity per CSA S304 (16.8.8.3)[2].* *In order to apply the same principles to Conventional Construction shear walls the following is assumed:* *1. A plastic hinge region of a Conventional Construction shear wall is not subject to detailing restrictions of Moderately Ductile shear walls as explained previously. The "hinge" is simply a recognition of where flexural yielding of reinforcement can take place. Walls are still on expected to meet R_d = 1.5 requirements.* *2. Having a R_d = 1.5 designation recognizes very limited ductility. There are already height limits in place in the NBCC [3] for regions of moderate to high seismic risk. It is widely recognized that the required inelastic capacity in R_d = 1.5 shear walls can be facilitated through a combination of flexural yielding of reinforcement, cracking of masonry, shear deformations, and system overstrength.* *3. The use of CSA S304 (16.8.8.4)[2] explicitly accounts for scenarios when seismic demands are high by requiring a direct calculation of Δ_{fl} without the need for arbitrary limits based on hazard.*

IMPACT ANALYSIS OF THE PROPOSED DESIGN REQUIREMENTS

Design of Conventional Construction Squat Shear Walls

For the case of squat shear walls the limit of $0.1f'_m$ for *Conventional Construction* shear walls was actually more severe that the limits already in place for *Moderately Ductile* squat shear walls. Although not typical in single-storey buildings, it is plausible that a squat wall may extend over several stories and may be used in situations where axial stresses are quite high. Simply adopting the provisions for *Moderately Ductile* squat shear walls ensures that walls of *Conventional Construction* have sufficient inelastic capacity when axial stresses exceed $0.1f'_m$. The proposed limits to height-to-thickness ratio of the wall as well as the new minimum reinforcement ratio provisions are unlikely to cause much disruption for the cases where axial loads are already high in squat walls.

Design of Conventional Construction Non-Squat Shear Walls

The proposed methodology permits a maximum overstrength value, γ_w, equal to R_oR_d. This is probably the most significant assumption of the new proposed methodology compared to *Moderately Ductile* shear walls. As noted throughout Table 2, specific detailing requirements intended to ensure R_d = 2.0 behaviour (e.g. limiting compression strain in the masonry to ε_{mu} = 0.0025) are not required to ensure R_d = 1.5 behaviour. Rather, the proposed method is simply using a well-established mechanics equation to directly determine inelastic rotational demand and capacity for the shear wall. Other design requirements provided in Table 2 are based on established design parameters for *Conventional Construction* shear walls and justified by the fact that anticipated demands and behaviour of the system remains consistent with provisions contained in CSA S304 (16.4 and 16.5)[2].

Moderately Ductile shear walls do not limit γ_w, however, a minimum inelastic rotational demand of 0.003 is required to be met per CSA S304 (16.8.8.2)[2]. This minimum demand is empirically derived from those used in reinforced concrete design [4]. Minimum demands are only employed for *Moderately Ductile* and *Ductile* shear wall systems and there is no rational means to provide a similar limit to walls of *Conventional Construction*. To account for this difference and ensure only rational solutions are provided, an upper bound to the maximum overstrength must be provided, in this such that $\gamma_w \leq R_dR_o$. The impact of this assumption will be explored further as it related to c/ℓ_w.

An absolute upper bound to the c/ℓ_w ratio permitted by the proposed design requirements would occur when wall overstrength, γ_w, is equal to R_oR_d = 2.25. Substituting this upper limit into the c/ℓ_w equation from CSA S304 (16.8.8.4)[2] will cancel out the terms related to aspect ratio and top displacement, and will yield the following:

$$
\begin{aligned}
\frac{c}{\ell_w} &\leq \frac{\varepsilon_{mu}}{\dfrac{4A_r}{(2A_r-1)}\left(\dfrac{\Delta_{f1}R_dR_o}{h_w}\right)\left(1-\dfrac{2.25}{R_dR_o}\right)+2\varepsilon_y} \\
&\leq \frac{\varepsilon_{mu}}{2\varepsilon_y} \\
&\leq \frac{0.003}{0.004} \\
&\leq 0.75
\end{aligned} \qquad (4)
$$

This value of 0.75 now represents the theoretical maximum c/ℓ_w ratio that could now be achieved in design using this proposed methodology. Although it is greater than that for *Conventional Construction* concrete shear walls and *Ordinary Reinforced Shear Walls*, it is judged to be reasonable because of the more comprehensive analysis used here. Recall that to establish the wall overstrength factor, γ_w, the nominal (unfactored) moment resistance properties of the wall are used. Therefore, the c/ℓ_w ratio for the nominal wall (which is closer to the actual wall behaviour) is a better indication of the true c/ℓ_w limit in terms of inelastic rotational capacity in the plastic hinge.

Nominal Moment Resistance of Axial Load Dominated Walls

A shear wall which is determined to have a nominal moment resistance, M_n, so large such that $M_n/M_f = \gamma_w = R_dR_o = 2.25$ would be limited by a factored moment resistance determined with an upper bound of $c/\ell_w = 0.75$ per Eq. 4. This c/ℓ_w ratio corresponds to an implied elastic response using factored wall properties. Where, by strain compatibility, the strain in the extreme reinforcement for the factored cross-section would be $\varepsilon_s = 0.001$. This is based on using $\phi_m = 0.6$ and $\phi_s = 0.85$ as a means to reduce specified strengths of masonry and reinforcement, respectively. However, when *nominal wall properties* are considered, the *nominal neutral axis depth* is reduced from the factored wall *for the same axial load*, as indicated in the interaction diagrams plotted in Fig. 2.

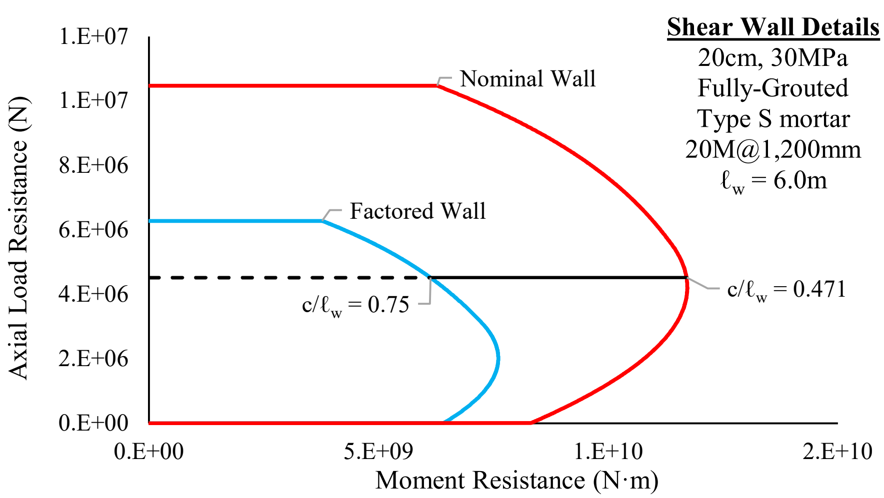

Figure 2: Factored and Nominal Interaction Diagrams

Where, the reduction to the c/ℓ_w ratio in the nominal wall is roughly proportionate to the *factored neutral axis depth* by ϕ_m. Meaning that, if the neutral axis depth, c, using factored material properties is limited to a maximum of $0.75\ell_w$, then the maximum *nominal neutral axis depth* would approximately equal to $c/\ell_w \approx 0.6 \times 0.75 = 0.45$, which results in inelastic strains in the reinforcement (balanced conditions exist at approximately $c/\ell_w = 0.6$). For the wall details given in Fig. 2 a c/ℓ_w limit of 0.75 in the factored wall corresponds to a c/ℓ_w ratio using nominal wall properties of 0.471. As the reinforcement ratio and masonry strength changes, so will to the maximum c/ℓ_w ratio in the nominal shear wall. To determine the range of variation between nominal and factored walls, a number of interaction diagrams were generated, similar to that in Fig. 2. A plot of the corresponding c/ℓ_w ratios for walls with varying vertical reinforcement ratios and masonry strengths (based on unit strength) are then plotted in Fig. 3.

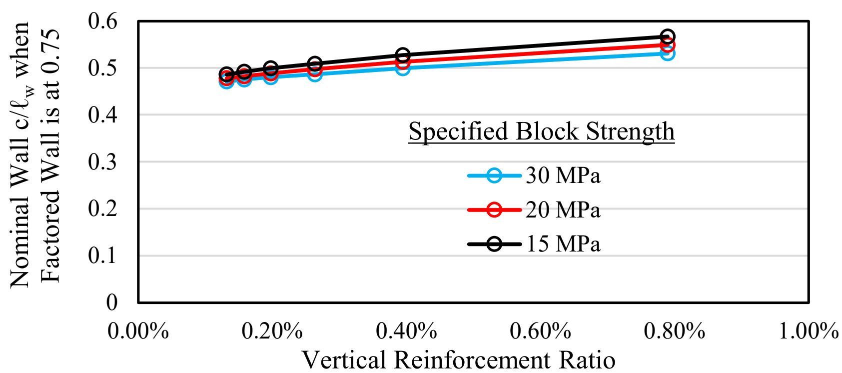

Figure 3: Nominal Wall Neutral Axis Depth Ratio (c/ℓ_w) for Vertical Reinforcement Ratios and Masonry Strengths Corresponding to typical Conventional Construction Walls

It can be observed that over the range of shear wall designs selected, the upper bound for the neutral axis ratio is around 0.5 (similar to that permitted reinforced concrete and masonry design in the U.S.). At a c/ℓ_w ratio of 0.5 based on nominal wall properties, the strain in the reinforcement would be equal to $\varepsilon_s = 0.003$ or the equivalent of 1.5 times the yield strain. Therefore, an upper bound to $\gamma_w = R_o R_d = 2.25$ will ensure that *flexural ductility of the nominal system is preserved*, even if the design level of neutral axis depth for the factored wall properties suggests otherwise.

Range of Expected Solutions

The range of c/ℓ_w limits which would apply using the proposed design requirements are plotted as a function of the wall aspect ratio, A_r, and the top of wall deflection, Δ_{f1}, which can be normalized against the wall height, h_w. This is shown for the following four cases of wall overstrength, γ_w: where γ_w is taken as just below the upper limit as 2.2 (Fig. 4a), as an intermediate value of 2.0 (Fig. 4b), as the current overstrength value $R_o = 1.5$ (Fig. 4c), and as the current minimum to for *Moderately Ductile* shear walls of 1.3 (Fig. 4d). A range of top drift levels are plotted as Δ_{f1}/h_w starting with 0.2%, which would represent a relatively small seismic demand, at increments of 0.1% up to 1.2% which would represent a rather large seismic demand (based on drift limits in the NBCC [3] a maximum permissible elastic drift of 1.16% can be derived).

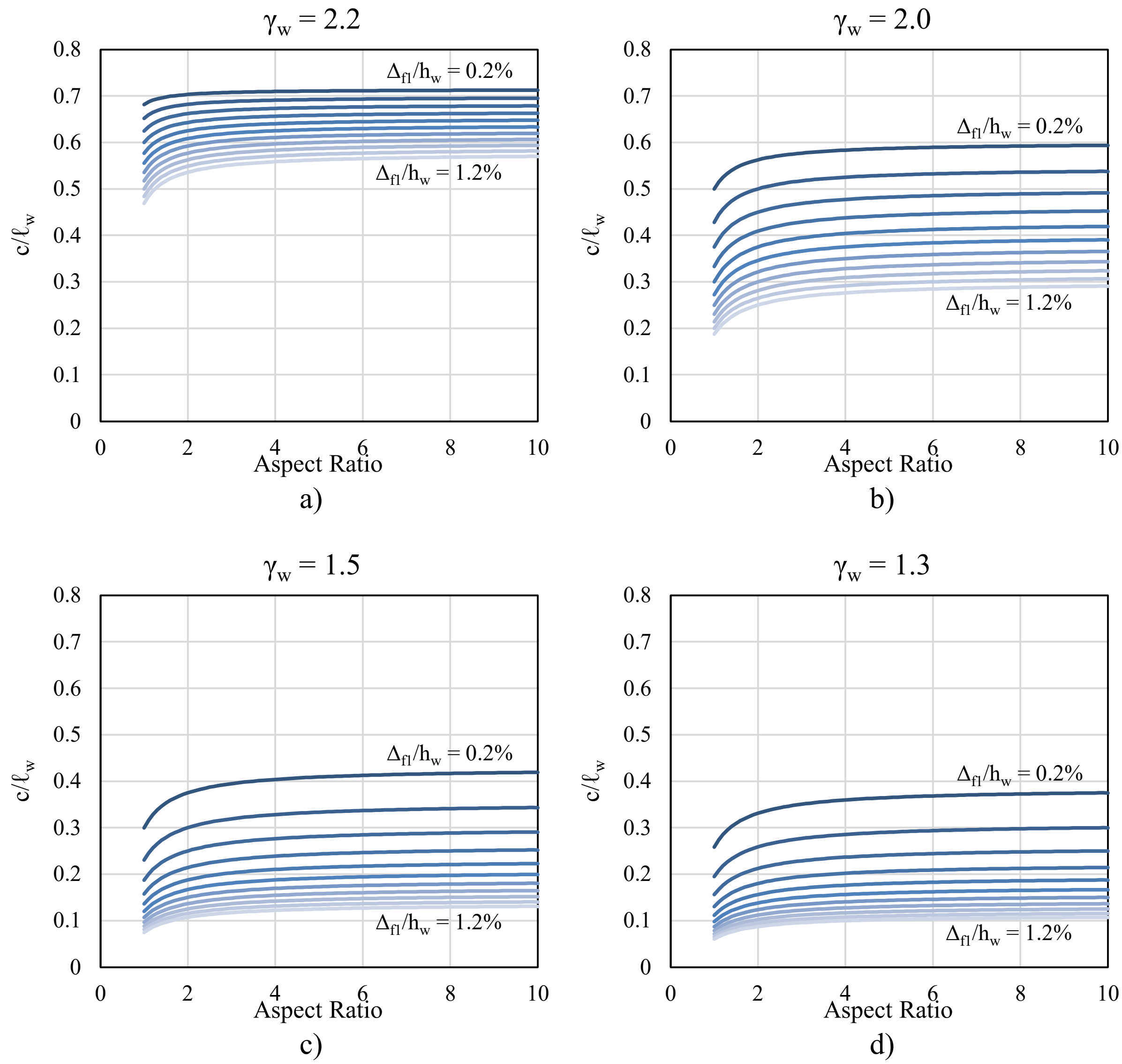

Figure 4: Neutral Axis Depth Ratios for Different Values of γ_w and Normalized Elastic Wall Displacement Δ_{f1}/h_w

For the extreme case of $\gamma_w = 2.25$ as the upper limit, all solutions converge to $c/\ell_w = 0.75$, as was discussed previously. Comparing the following plots to the limits in the current CSA S304, derived to be $c/\ell_w = 0.245$ and those in CSA A23.3, $c/\ell_w = 0.5$ and TMS 402, $c/\ell_w = 0.45$ it is clear that the appropriate limit based on rational calculations will actually vary significantly. It is also clear that using a single solution for c/ℓ_w will not provide a conservative answer for all combinations of seismic demand, overstrength and aspect ratio.

While the range of solutions in Fig. 4 coincides reasonably well with the hard limits adopted elsewhere, it is clear that in some cases employing a hard c/ℓ_w limit may not be appropriate and should be re-evaluated by other design standards that do so. It should be noted that TMS 402 [5] and ASCE 7 [6] will limit the shear wall category used (and consequently the c/ℓ_w limit) by seismic hazard, whereas the NBCC [3] and consequentially the CSA A23.3 [4] only restricts the height of building permitted based on hazard level.

CONCLUSION

The current phrasing of the CSA S304 (16.5.3.)[2] with respect to *Conventional Construction* shear walls in regions with a seismic hazard index over 0.35 forces a sudden drop in permissible axial load. It is proposed that adopting the methods described in CSA S304 (16.8.8.4)[2] to evaluate inelastic rotational demand/capacity directly for *Moderately Ductile* shear walls constitutes a sufficiently *more comprehensive method* permitted by CSA S304 (16.5.1)[2] as a rational alternative to the arbitrary limit to axial load of $0.1f'_m$. A maximum value for wall overstrength based on the factored loads and nominal moment resistance of the wall shall not be taken as a value greater than $R_oR_d = 2.25$. This was demonstrated to preserve inelastic ductility of the nominal wall system, for which is an implicit requirement for shear walls with $R_d = 1.5$ in order to preserve inelastic flexural strains needed to dissipate seismic energy.

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