STANDARDS RESEARCH

Towards the Harmonization of Canadian and American Masonry Structures Design Standards

February 2021

Author

Hélène Dutrisac, P.Eng., Canada Masonry Design Centre

Bennett Banting, Ph.D., P.Eng., Canada Masonry Design Centre

Project Advisory Panel

Ana-Maria Tomlinson, Ph.D., P.Eng., CSA Group

Jennifer Teague, Ph.D., CSA Group

Project Contributors

Canadian Contributors

Bennett Banting, Canada Masonry Design Centre

Hélène Dutrisac, Canada Masonry Design Centre

Bart Flisak, Crosier Kilgour & Partners Ltd.

Kevin Hughes, Tacoma Engineers

Carlos Cruz-Noguez, University of Alberta

Clayton Petitt, University of Alberta

David Stubbs, Canada Masonry Design Centre

American Contributors

Richard Bennett, University of Tennessee

Ece Erdogmus, University of Nebraska-Lincoln

Lane Jobe, Miller Consulting Engineers

Philippe Ledent, Masonry Institute of Michigan

Russ Peterson, Ensoltech

Heather Sustersic, Colby Co. Engineering

Jason Thompson, National Concrete Masonry Association

Acknowledgements

The authors gratefully acknowledge the valuable interactions and discussions with the participants of the CANUS Masonry Standard Harmonization Project as well as the funding support provided by the National Concrete Masonry Association (NCMA) foundation, Canadian Concrete Masonry Producers Association (CCMPA), Canada Masonry Design Centre (CMDC), and Canadian Standards Association (CSA Group).

Table of Contents

Executive Summary

Although the Canadian Standards Association (CSA) S304-14 (R2019) *Design of Masonry Structures* and The Masonry Society (TMS) 402-16 *Building Code Requirements for Masonry Structures* have been derived from the same pool of research, notable differences have been observed over the years and questions have been raised over the accuracy, degree of conservatism, and overall economy associated with each standard. To study disparities which may exist between these two standards, a Canadian-American research project was initiated to identify requirements within the standards which may in one case be insufficiently rigorous and in other cases be overly conservative. This report highlights the findings of this initiative and summarizes areas that have been identified for improvement in the context of the Canadian standard.

The primary goals of the research project were to conduct a comprehensive comparison of the masonry requirements contained within the CSA S304-14 and TMS 402-16 design standards and establish a collaborative Canadian-American front to strive for better long-term cross-border harmonization between the two standards. The collaborative Canadian-American initiative involved three key activities:

1. Comparison of the Canadian limit states and the US strength design provisions, including load (NBCC/ASCE 7) and resistance (CSA S304-14/TMS 402-16) provisions;
2. Parametric studies of reinforced masonry beams and in-plane and out-of-plane bending of reinforced masonry walls; and
3. Comparison of preliminary archetype building designs.

The comparison of the National Building Code of Canada (NBCC) and the American Society of Civil Engineers, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7) loading criteria revealed similarities with respect to dead loads and live loads due to use and occupancy. Although the approach for calculating snow, wind, and earthquake loads are similar in both codes, diverging results for the archetype designs were observed. Differences in the design loads were primarily attributed to variations in climatic and seismic hazard data.

The review of the CSA S304-14 and TMS 402-16 design standards revealed that, similar to the loading provisions, the methodologies used by both standards for computing reinforced masonry element resistances are generally similar in nature. Key nuances identified include:

- Lower CSA S304-14 masonry compressive strength, f'_m and flexural tensile strength, f_t which were noted to be approximately half of those specified by the TMS 402-16 standard;
- Use of a directionality factor, χ, in CSA S304-14 which impacts the resistance of masonry elements where compressive stresses are applied normal to the head face;
- A lower CSA effective compressive width of 4t when computing out-of-plane resistance of masonry walls which is triggered much sooner compared to 6t used in TMS 402-16; and
- Differences in resistance (strength) reduction factors with impacts noted to be more prominent in compression-controlled responses.

Parametric studies were carried out to investigate the nuances identified during the standard comparison.

The studies illustrated the following:

- Lower axial resistance in CSA S304-14 under combined axial load for out-of-plane bending response due to lower masonry compressive strength values and lower masonry material resistance factors;
- Lower CSA S304-14 squat wall resistances due to reduced moment arm;
- The CSA S304-14 reduced moment arm provision was shown to overestimate the actual moment for wall aspect ratios near 1.0;
- Reduction in CSA S304-14 resistance in tension-controlled regions of combined axial and out-of-plane bending response due to smaller effective compression width of 4t versus the TMS 402-16 provision of 6t;
- Reduced CSA S304-14 beam resistances, nuances attributed mainly to the directionality factor, χ and to greater compression stress block depth as a consequence of the smaller masonry resistance factor and lower masonry compressive strength; and
- Overall reduced seismic capacity of shear walls inhibiting their use in regions of high seismicity in Canada.

Preliminary designs of two building archetypes were carried out at two locations along the Canada-US border to identify nuances in location-specific design. In general, the two-storey mixed-use archetype design in the US was achievable with either smaller masonry block units or units of the same size but with significantly less reinforcement. Several differences were noted in the beam designs and highlighted how restrictive the Canadian design provisions are in comparison to the US. On the other hand, the multi-storey residential archetype exercise revealed that designs with a greater number of storeys can be achieved using the Canadian provisions. The number of storeys was restricted in the US due to the maximum reinforcement limit, a provision which is not included in the CSA S304-14 design standard.

Although a number of suggested changes and research needs have been identified, the following three areas have been identified as having the most significant impact on CSA S304-14 masonry behaviour response:

- The masonry compressive strength, f'_m;
- The effective compressive width, 4t; and
- The directional factor, χ.

"A collaborative Canadian-American research initiative was undertaken to identify requirements within CSA S304-14 (R2019) *Design of Masonry Structures* and TMS 402/602-16 B*uilding Code Requirements and Specification for Masonry Structures* to determine areas for improvement and increased harmonization."

1.0 Introduction

Many similarities exist between Canadian and American masonry construction materials, however several key differences have been identified in relation to each country's respective masonry design standard, namely CSA S304-14 (R2019) *Design of Masonry Structures* [1] and TMS 402/602-16 *Building Code Requirements and Specification for Masonry Structures* [2], henceforth referred to as CSA S304-14 and TMS 402-16, respectively. As both standards have been derived from the same pool of research, this has raised questions over the accuracy, degree of conservatism, and overall economy associated with each standard.

A collaborative Canadian-American research initiative was undertaken to identify requirements within the two standards to determine areas for improvement and increased harmonization. This report highlights the key findings of this initiative and summarizes opportunities for improvement of the requirements, with a focus on the Canadian standard.

This report is primarily written in the context of a Canadian audience. As such, limit states design terminology is used throughout the document with US terminology denoted in brackets. The strength design method used in the US aligns with the limit states methodology in Canada, but with slightly different terminology. In this method, masonry members are required to be proportioned so that the design strength equals or exceeds the required strength, where design strength is defined as the nominal strength multiplied by a strength-reduction factor. Hence, "design strength" can be synonymous to "factored resistance" and "required strength" to "factored loads".

1.1 Research Goals and Objectives

Canadian and US teams were formed (the project was given the name CANUS), with representation from various masonry key stakeholder groups, including academia, code users (designers), and industry representatives. The comparative research project entailed self-directed investigations, a week-long summit in Collingwood, Ontario, in October 2019 and a post-project summit in Orlando, Florida, in January 2020.

The goal of this project was to conduct a comprehensive comparison of the masonry requirements contained within CSA S304-14 and TMS 402-16 design standards. To achieve this goal, the following objectives were identified as described below.

- Identify differences between the design standards within the context of the respective national model code environments;
- Identify short-term changes that can be implemented in the next edition of CSA S304 to address gaps and better harmonize masonry design between the two standards; and
- Identify gaps and develop a prioritized needs list for research that may be used to identify future industry-funded projects and standard research initiatives.

2.0 Methodology

To achieve the desired objectives, a three-tier approach was used, which entailed the key activities listed below.

1. **Code and Design Standard Comparison:** a side-by-side comparison of the limit states design loading requirements and resistance design provisions of each country's respective code and design standard including load and resistance factors.
2. **Parametric Study:** parametric studies of primary masonry structural members selected for comparison, i.e., beams and walls considering in-plane and out-of-plane one-way bending effects.
3. **Archetype Building Designs:** the design of two building archetype elements at two locations along the Canada-US border intended to compare environmental loads and building code requirements for locations sharing similar geographic coordinates.

2.1 Code and Design Standard Comparison

A side-by-side comparison and discussion of design loads and member resistance requirements was conducted using the design loads prescribed in the NBCC 2015, National Building Code of Canada [3] and ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures [4] for Canada and the US, respectively. Canadian masonry design requirements are found in CSA S304-14 whereas US requirements are specified in TMS 402-16. The comparison was focused on covering design topics of interest and areas where departures between the standards are known to exist. Although US masonry design can be carried out using allowable stress design and empirical design can still be used in some situations in Canada, the comparison was done on the basis of limit states design and strength design from the Canadian and American standards, respectively.

Although several design loads are prescribed by both the NBCC 2015 and ASCE 7-16, the side-by-side comparison focused on the following design loads of interest:

Design loads (ASCE 7-16 and NBCC 2015)

1. Dead loads
2. Live loads
3. Snow loads
4. Wind loads
5. Earthquake loads and system restrictions
6. Importance and risk categories
7. Load combinations and load factors

Comparison between the standards was limited to design of reinforced masonry beams, out-of-plane walls, and shear walls. Although other aspects of design such as reinforcement provisions and material testing are also described in the standards, these are intended to be explored in future work. The side-by-side comparison focused on the following member resistance requirements of interest:

Masonry member resistance (TMS 402-16 and CSA S304-14)

1. Resistance reduction (ϕ) factors
2. Material properties
3. Design of masonry beams: flexure, shear, and deflections
4. Design of fully-grouted in-plane masonry walls: flexure, shear, and seismic detailing
5. Design of out-of-plane masonry walls: flexure, slenderness effects and shear

A systemic review of all applicable design standard clauses was completed. The results of the code and design standard comparison are not presented here but serve as the basis for the parametric study (see Section 3.1) and archetype design comparison (see Section 3.2). The complete study can be found as a separate report available for CSA S304 technical committee use.

2.1.1 Resistance Reduction Factors

With the limit states design approach, both standards require that the factored resistances (or design strength) be greater than the factored loads (or required strength). In each case, the factored loads are specified by other regulatory documents, namely the NBCC 2015 and ASCE 7-16 for Canadian and US design applications, respectively.

Although loads and load combinations are established in a similar fashion, factored resistances are computed using a different approach. In Canada, resistance factors are applied to material properties to compute the factored resistance. In the US, strength reduction factors are applied to the nominal resistance as a whole based on the type of expected failure.

2.1.2 Material Properties

Engineered design of reinforced concrete masonry uses material properties and section properties to determine the strength, stiffness, and deflection characteristics of masonry members. The following material properties were reviewed for the Canadian and US standards:

- Concrete block masonry unit dimensional properties
- Specified masonry assembly compressive strength, f'_m
- Reinforcing bar properties and steel yield stress, f_y
- Mortar and grout requirements
- Maximum usable masonry strain
- Modulus of elasticity
- Reinforcement size limitations

2.1.3 Design of Masonry Beams

The following masonry beam design requirements were reviewed for their differences:

- Beam construction
- Effective span length
- Lateral support
- Moment capacity and detailing requirements
- Shear resistance and detailing requirements
- Deep beams
- Serviceability deflections

2.1.4 Design of In-Plane (Shear) Walls

The differences between masonry shear wall design requirements were determined. The following components were examined:

- Axial load resistance
- Moment capacity
- Shear resistance
- Squat wall design
- Seismic force resisting systems

2.1.5 Design of Out-of-Plane Walls

An examination of the key CSA S304-14 and TMS 402-16 design provisions for reinforced masonry walls subjected to axial and out-of-plane (weak axis) flexural loads was conducted. To simplify the analysis, only reinforced walls subject to out-of-plane loads bending under one-way vertical flexure were considered. For most cases, only walls where moments are derived from face loads (e.g., wind, seismic) were considered and imposed eccentricities or other irregular loading situations were not considered. The following components were examined:

- Design assumptions
- Effective mortared area
- Effective compression width
- Minimum reinforcement requirements
- Maximum reinforcement limits
- Consideration of slenderness effects
- Design for second order effects
- Shear resistance

2.2 Parametric Study

To better investigate the impacts of design equations and parameters contained within CSA S304-14 and TMS 402-16, a series of parametric studies were conducted. These studies explored the specific effects that the standard provisions have in a quantifiable manner. Parametric studies included:

1. Design of masonry beams;
2. Design of masonry shear walls subjected to in-plane (strong axis, cantilevered) bending and axial loads; and

3. Design of masonry walls subjected to out-of-plane (weak axis, one-way) bending and axial loads.

The effects of material properties and ϕ-factors were explored within each of these different sections. Parametric studies were largely focused on the use of typical or conventional masonry construction, such as type S mortar, 20 cm (8 in) units and typical reinforcement details. The studies themselves were not exhaustive and, much like the design standard comparison, are limited in their scope. It was the intention of this report to highlight the most significant variation between the standards and to present a summary of those results and conclusions. The objective was to establish a baseline for efforts that are expected to continue in the future for each standard in their development. Nevertheless, the parametric studies offered quantifiable and normalized comparison between designs.

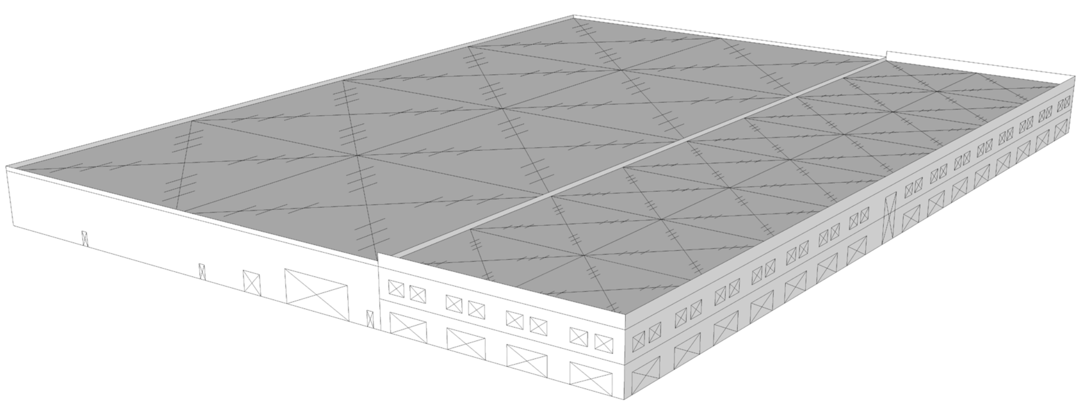
To allow for a proper side-by-side comparison of the CSA S304-14 and TMS 402-16 provisions without introducing block geometric or material property nuances, baseline studies were carried using Canadian block dimensions and sectional properties, and Canadian spacing and rebar sizes were used.

2.3 Archetype Building Designs

The third area of research focused on the design of selected structural elements of two archetype buildings at two different locations. Archetypes were selected to reflect typical North American masonry block construction.

The first archetype is a two-storey mixed-use building with a two-storey office space at the front and an attached single storey warehouse space at the back of the building for a total building area of approximately 16,050 m^2 (172,800 sf) (Figure 1). The composition of the roof consists of a lightweight roof system supported on open web steel joists and steel beams. The second floor consists of a concrete slab on steel deck supported on open web steel joists and beams. Both the roof and floor levels are supported at the perimeter by masonry walls and on the interior by steel columns. The warehouse space is separated from the office area with a full height reinforced masonry bearing wall. A 1.83 m (6 ft) high parapet is included at the building roof perimeter over the office space to simulate construction practices to conceal rooftop mechanical equipment. The remainder of the perimeter parapet is 0.71 m (2.33 ft). The office space storey height was set at 4.27 m (14 ft) with the overall storey height of the warehouse space fixed at 8.54 m (28 ft).

Figure 1: Two Storey Mixed-use Warehouse/Office Archetype

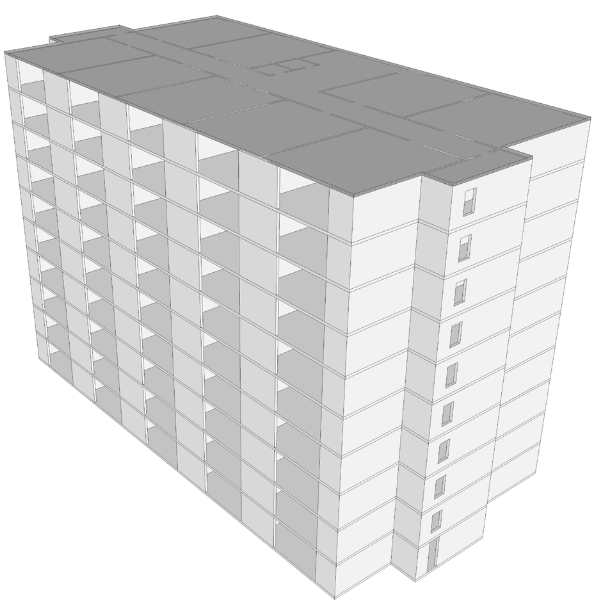


The second archetype is a multi-storey residential building consisting of a rectangular floor plan with a central corridor and stairwells at each end of the building (Figure 2). The overall floor plan area covers an approximate area of 990 m^2 (10,656 sf.) The roof and floor construction assemblies consist of 250 mm (10 in) hollow-core slab systems supported on reinforced masonry walls which also form the separation between residential units. The storey height was fixed at 3.048 m (10 ft). Although Figure 2 depicts a ten-storey structure, the intent of the study was to determine how high one can go with various block sizes.

To minimize the differences with respect to climatic data, seismic data, and prescribed design loads, it was decided to consider two locations along the Canada-US border as follows:

1. **Niagara Falls, New York, and Niagara Falls, Ontario –** these locations were selected to capture wind-driven Eastern Canada designs.
2. **Blaine, Washington, and White Rock, British Columbia –** these locations were intended to capture seismic-driven designs.

Figure 2: Multi-storey Residential Building Archetype



2.4 Limitations and Use

This study focused on reinforced masonry following engineering design methods. It is limited to a comparison between the design standards, although where appropriate, reference to material standards are also included. Several especially important aspects of design are not included within this initial report such as veneers or material testing methods. Some aspects of design that are presented here require additional and focused research to properly resolve, such as shear strength of partially-grouted shear walls. This report is intended to give the reader an appreciation for the intrinsic differences between how masonry is designed in both countries and to highlight where there may be gaps or conflicts within the respective methodologies.

3.0 Results and Discussion

3.1 Code and Design Standard Comparison

The review of the CSA S304-14 and TMS 402-16 design standards revealed that, similar to the loading provisions, the methodologies used by both standards for computing reinforced masonry element resistances are generally similar in nature. Key nuances identified include:

- Lower CSA S304-14 masonry compressive strength, f'_m and flexural tensile strength, f_t which were noted to be approximately half of those specified by the TMS 402-16 standard;
- Use of a directionality factor, χ, in CSA S304-14 which impacts the resistance of masonry elements where compressive stresses are applied normal to the head face;
- A lower CSA S304-14 effective compressive width of 4t when computing out-of-plane resistance of masonry walls which is triggered much sooner compared to 6t used in TMS 402-16; and
- Differences in resistance (strength) reduction factors with impacts noted to be more prominent in compression-controlled responses.

"The review of the CSA S304-14 and TMS 402-16 design standards revealed that, similar to the loading provisions, the methodologies used by both standards for computing reinforced masonry element resistances are generally similar in nature."

3.2 Parametric Study

The objective of the parametric study was to provide a detailed comparison of how the differing material properties and design equations affect the final design of masonry members. A summary of the results are presented only to highlight where major differences are observed. The complete study has been summarized within a separate technical report available for CSA S304 technical committee use.

3.2.1 Beams

The cumulative effect of the masonry material resistance factor, ϕ_m, and the compressive stress directionality factor, χ, result in a compressive stress block used by TMS 402-16 that is up to 3.1 times larger in magnitude. Combined with lower values of masonry strength, f'_m, this will create a compressive force in the masonry that is up to six times larger in TMS 402-16 designed beams. The result of this discrepancy is that TMS 402-16 designed beams can contain a larger maximum permissible area of reinforcing steel compared to the CSA S304-14 designed beams. A summary of the parametric study conclusions are as follows:

- The maximum reinforcement ratio, ρ, for CSA S304-14 designed beams is 0.0038 when χ = 0.7 and 0.0027 when χ = 0.5. The maximum reinforcement ratio, ρ, for TMS 402-16 designed beams is 0.0095.
    - A TMS 402-16 designed beam may have 2.5 to 3.5 times more tension reinforcement area than a CSA S304-14 designed beam, when using country-specific properties.
- The moment resistance for a TMS 402-16 designed beam using its maximum permitted reinforcement area was 138.4 kN·m. The moment resistance for a CSA S304-14 beam using its maximum permitted reinforcement area was 46.8 kN·m, when χ = 0.7, and 33.1 kN·m, when χ = 0.5. Moment resistance values were determined from a 3-course beam comprised of 20 cm (8 in) units.
    - The TMS 402-16 designed beam possessed 2.96 to 4.18 times more moment resistance capacity than a similar beam configuration designed with the CSA S304-14, when using country-specific properties.

The above comparison was made using normalized values to permit comparison of country-specific properties. Other notable conclusions from the parametric study of beams are as follows:

- Deep beam provisions for CSA S304-14 resulted in a greater reduction in the effective moment arm when compared to TMS 402-16 beams. This gap would lead to lower design moment resistances in CSA S304-14 designed beams even if country-specific properties are accounted for.
- Shear strength equations in the CSA S304-14 appear to provide a range of possible results that are, on average, greater than that of the TMS 402-16 when comparing normalized equations. This difference is explained by the more complex analysis method used by CSA S304-14, which can provide higher strengths with greater computational effort. However, when using country-specific properties, the TMS 402-16 produced strengths that were on average 1.25

times greater than the maximum theoretical value determined for the CSA S304-14. TMS 402-16 shear strengths were also on average 2.8 times greater than the minimum shear strength determined from the CSA S304-14.

- Service deflections were determined to be lower for TMS 402-16 designed beams. The magnitude of this difference was highly dependent on the ratio of service moment to cracking moment and the reinforcing ratio. In some cases, a maximum difference by a factor of five was observed between the standards. This would translate into a deflection of TMS 402-16 beams being as little as one-fifth that of a CSA S304-14 designed beam, for comparable loading conditions and designs, and when using country-specific properties.

3.2.2 Shear Walls

Shear wall behaviour was compared using interaction diagram (axial load, P, versus moment resistance, M) analysis. Comparisons were limited to fully-grouted shear walls. Design using country-specific properties for shear walls suggested that, in general, parts of the interaction diagram closer to the tension-controlled (lower) region near the state of pure bending responded in a similar manner for both countries. Shapes of the interaction diagrams were consistently similar although magnitudes of loads were often different. Observable differences in interaction diagram resistances between the two standards begin as the wall response transitions from that associated with pure bending. A summary of the parametric study conclusions are as follows:

- Squat wall provisions in the CSA S304-14, which are not present in the TMS 402-16, lead to reduced flexural strength of CSA S304-14 designed walls when using normalized values. Notably, wording of the CSA S304-14 requirements may also produce unintentional increases to wall strength where a reduction should otherwise be expected.
- Upper bound limits to maximum shear resistance somewhat mitigated observed differences between the standards in squat wall flexural strength, bringing the two standards into closer alignment. Nevertheless, TMS 402-16 limits for maximum shear strength still exceed CSA S304-14 by as much as a factor of two, when using country-specific properties.
- Reinforcement limits to the TMS 402-16 cap the compression-controlled response of the interaction diagram; however, in all cases it remained greater than the limits to the CSA S304-14, when comparing country-specific properties.
- Seismic design provisions for comparable seismic-force-resisting system (SFRS) categories demonstrated a similar approach taken to seismic design in each standard. Prescriptive reductions to wall resistance (both axial and shear) were applied to walls design by both standards. However, shear walls conforming to the CSA S304-14 still possessed lower moment and axial load strengths compared to TMS 402-16 designed walls, when using country-specific properties.

An interaction diagram analysis tool was developed to permit a nuanced comparison between standards for the parametric study. Although results are limited to the selected parameters, the trends reported were observed by the authors to also be true when other properties were varied (e.g., block size, aspect ratio) from those chosen.

3.2.3 Out-of-Plane Walls

Out-of-plane wall design and behaviour was compared through the use of interaction diagrams as was also done for shear walls. Using normalized properties resulted in different shapes to the interaction diagrams while using country-specific properties resulted in different magnitudes of resistance. To illustrate the different points and areas of interest on these diagrams, factored axial load and moment resistance curves were drawn for a 3 m high 20 cm - 15 MPa Canadian masonry block wall reinforced with 20M @ 1,400 mm using the CSA S304-14 and TMS 402-16 provisions, as shown in Figure 3 (a) and (b), respectively. A spacing of 1,400 mm was used to trigger the effective compression zone width requirement in both standards. Several curves were plotted to illustrate the following three aspects:

- The effects of CSA S304-14 effective compression zone width 4t (6t for TMS 402-16) versus taking a compression zone width equal to reinforcement spacing (b = s),
- The axial load cap limit, and
- The reinforcement limit in TMS 402-16.

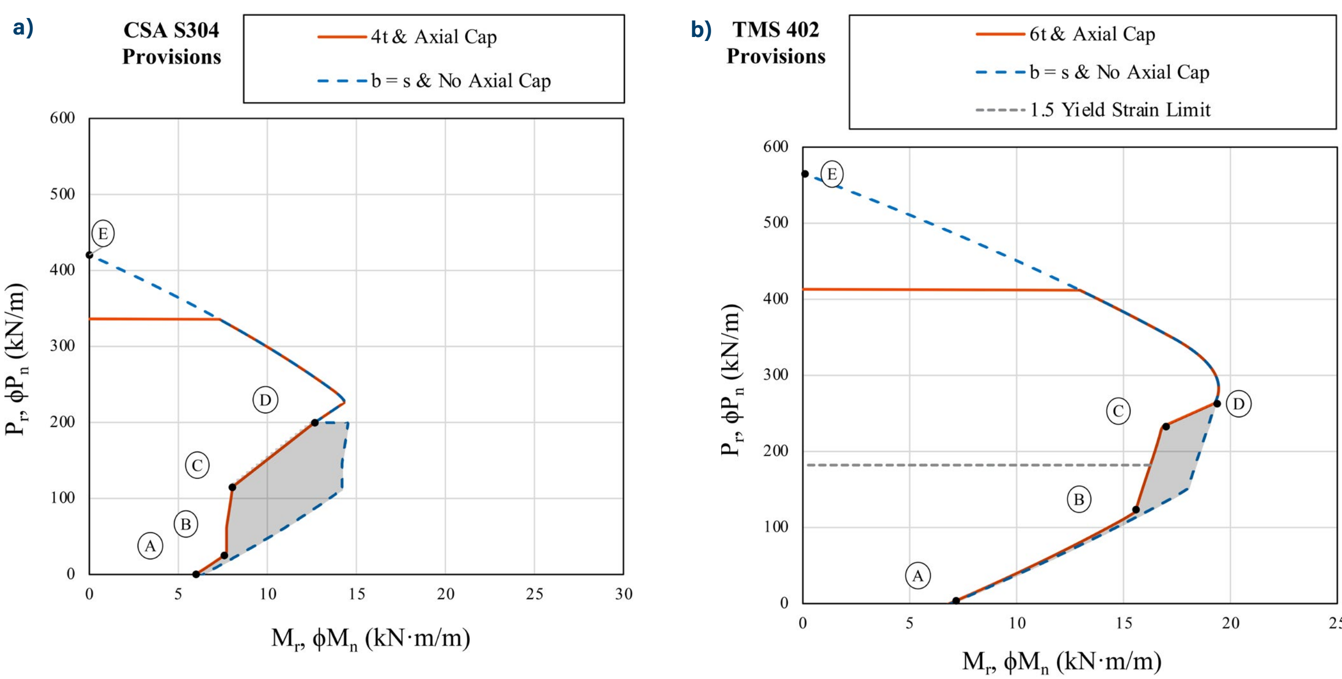
The shaded area in Figure 3 highlights the reduction in capacity due to the effective compression zone width (b) requirements. The following points of interest are also illustrated:

- Point A – pure bending (no axial load)
- Point B – balanced point (yield strain in reinforcement is reached)
- Line C to D – transition in effective compression zone width to full compression zone width
- Point E – theoretical point of pure compression (no moment)

The application of the provisions in the standards results in differing magnitudes of resistance, which are attributed to differences in masonry assembly compressive strength, f'_m, (specifically the use of different strengths for grouted, f'_{mg}, and ungrouted, f'_{mu}, masonry in the CSA S304-14) the effective compression zone width (4t vs 6t), and the resistance reduction ϕ-factors. A summary of the other parametric study conclusions are as follows:

- The resistance envelope for out-of-plane walls, much like shear walls, are smaller for CSA S304-14 designs compared to TMS 402-16 designs. The use of a smaller effective compression zone width as well as previously observed design standard differences contribute to this.
- Prescriptive limits to axial load resistance due to slenderness effects is more punitive for CSA S304-14 designed walls when the slenderness ratio exceeds 30.
- Moment amplification effects due to secondary moments and deflections were not explicitly compared; however, a normalized comparison of wall stiffness was conducted. The methods used by the TMS 402-16 resulted in an approximate increase to wall stiffness by 30%. Using country-specific properties would increase this.

Figure 3: Interaction Diagrams for Partially-Grouted Out-of-Plane Walls



3.3 Archetype Building Design Comparison

The objective of the two-storey archetype design was to determine the most economical reinforced masonry wall configuration for selected elements whereas the objective of the multi-storey archetype design was to see how many storeys can be accommodated with a standard 20 cm (8 in) unit (i.e., how tall can you build). The results for the two-storey mixed-use building and multi-storey residential archetypes include a comparison of design loads and masonry material proportioning.

3.3.1 Design Loads

3.3.1.1 Building Importance (Risk) Category

Both the NBCC 2015 and ASCE 7-16 require that buildings be assigned a building category, as a function of the building use and occupancy, for the purposes of determining applicable requirements. The categorization in the NBCC 2015 is given in terms of building importance categories and in the US is defined as Risk Categories. Both archetypes fall under the NBCC 2015 Normal Importance Category and the ASCE 7-16 Risk Category II. The building categorization defines, in both cases, applicable importance (load adjustment) factors for snow, wind, and earthquake design loads. The importance factors for the NBCC 2015 normal importance category and the ASCE 7-16 Risk Category II are 1.0 for snow, wind, and earthquake.

3.3.1.2 Dead and Live Loads

Superimposed dead load allowances account for roofing materials, mechanical and electrical systems and components, ceilings, flooring, and partitions where applicable. Partition allowances in the NBCC 2015 are considered as a dead load whereas this allowance is considered under live loads in ASCE 7-16. The NBCC 2015 also permits the partition (non-permanent) allowance to be reduced to 0.5 kPa for the purposes of computing the seismic weight of the structure. Live loads were based on the use and occupancy of the archetypes as prescribed by the respective codes. Dead and live loads are generally similar with the exception of the partition allowance. This allowance is set at 1.0 kPa (20.9 psf) in NBCC 2015 versus 0.72 kPa (15.0 psf) in ASCE 7-16.

3.3.1.3 Snow Loads

The snow loads for the archetypes study were determined in accordance with the respective code procedures and the prescribed climatic data. The one in 50-year snow parameters and the computed specified design uniform and drift (where applicable) snow loads at the two locations were determined for the two archetype buildings.

Whereas dead and live loads are nearly identical in the NBCC 2015 and ASCE 7-16, the snow loads differed considerably at the two locations under examination. Despite a lower ground snow load parameter, the basic uniform snow load for the two-storey mixed-use archetype in Niagara Falls, Ontario, was 20% greater than its counterpart location across the border. This difference in design snow load is attributed to the higher NBCC 2015 prescribed basic roof parameter, which is normally taken as 0.8. However, this parameter is increased for large roofs, for which the mixed-use building basic roof parameter increases to 0.898 compared to the 0.7 value used in the US equation. The NBCC 2015 snow equation also includes an associated rain load parameter that is not present in the US calculations. Differences decrease with the multi-storey residential archetype, which has a smaller roof area, but still results in a Canadian uniform design snow load that is 10% greater than the US. The differences in uniform design snow load are more pronounced at the western location. The Canadian uniform design snow loads are 160% and 133% greater in White Rock, British Columbia, than in Blaine, Washington, for the two-storey mixed-use and multi-storey residential archetypes, respectively.

The magnitude and extent of the snow drift loads at the 1.83 m (6 ft) high parapet at the front of the two-storey mixed-use building archetype were similar in White Rock, British Columbia / Blaine, Washington but greater for Niagara Falls, New York, compared to Niagara Falls, Ontario. Whereas the height of the 0.71 m (2.33 ft) parapet did not generate snow drifting loads according to the NBCC 2015 provisions, it did generate some according to the ASCE 7-16 provisions.

3.3.1.4 Wind loads

The US procedure for computing wind design loads is slightly different compared to the Canadian procedures. The ASCE 7-16 prescribes wind velocity for various locations as a function of the risk category (importance category). The wind velocity was used in conjunction with several adjustment factors to compute a wind velocity pressure.

The pressures from ASCE 7-16 are higher than the Canadian values; however, the load combination in Canada requires the application of a load factor of 1.4 to the wind load whereas this factor is 1.0 in the US. In addition, gust and external pressure coefficients are combined in NBCC and are higher than those prescribed by ASCE 7-16.

3.3.1.5 Seismic Loads

The static force procedure was used to determine the seismic loading for the archetypes. Both codes make use of spectral acceleration values based on a 2% in 50 years probability of exceedance. These values were adjusted to account for site conditions and the importance of the structure. The 2% in 50-year values are commonly referred to in the US as the Maximum Considered Earthquake (MCE). However, the static force procedure in the US uses two-thirds the MCE hazard values whereas Canada uses the values as-is. Nonetheless, it is important to note that the NBCC 2015 caps the seismic design the design spectral acceleration in the computation of the base shear to the larger of two-thirds of the site-adjusted design spectral accelerations at 0.2s and the value at 0.5s, which generally governs the design of low-rise buildings. As such, it is reasonable to say that a similar approach is used for the determination of the seismic base shear force for short period structures (low-rise buildings). However, the seismic base shear for structures which do not benefit from the NBCC 2015 short hazard cap are, in essence, designed for 1.5 times the prescribed seismic design forces in comparison to the US.

There are some differences in seismic values between the NBCC 2015 and ASCE 7-16. The short period (0.2s) seismic hazard value in Niagara Falls, Canada is 45% higher than the corresponding value in the US while the 1.0s values are nearly identical. In White Rock, the values are 81% of those specified in Blaine, Washington. Except for short periods, the adjusted NBCC 2015 response spectrum (two-thirds) closely matches the US design response spectrum.

Both the NBCC 2015 and ASCE 7-16 recognize various masonry SFRS with various degrees of ductility. These systems are subject to restrictions and height limitations as prescribed within each code. In Niagara Falls, the use of a masonry SFRS with limited ductility is permitted in the US (Ordinary Shear Walls) without any height limitation and is also permitted in Canada (Conventional Construction) but limited to 30 m in height. In Blaine, Washington, the ASCE 7-16 requirements trigger the use of special reinforced masonry shear walls (ductile system). For White Rock, British Columbia, the NBCC 2015 permits the use of conventional masonry wall construction; however, the height is limited to 15 m. To the extent possible, the SFRS with the lowest ductility were selected for the study.

Another way of comparing seismic effects is to compare the base shear ratio as a function of the weight, as this value incorporates all required effects (hazard, site effects, importance of structure, force modification factors, higher mode effects, short-period cut-off, long-period cut-off). The base shear ratios as a function of the period demonstrate that the NBCC 2015 base shear ratio is consistently higher than the ASCE 7-16 ratio.

The NBCC 2015 design base shear ratios for low-rise buildings governed by the short-period cap are consistently higher than those prescribed by the ASCE 7-16 for comparable seismic force resisting systems. Notably, seismic design forces for low-rise buildings in Niagara Falls, Ontario, are 8 to 69% greater than in the US, depending on the type of SFRS; whereas values are 8 to 41% greater in White Rock, British Columbia, compared to Blaine, Washington. These base shear ratios were used to determine the seismic base shear of the two-storey mixed-use building archetype.

"In Canada, MASS models masonry walls along a given building line (elevation) and computes the resistance of masonry elements in accordance with the CSA S304-14 requirements."

3.3.2 Two-Storey Mixed-Use Building Archetype Design Results

Modelling of the archetype was carried out using the Direct Design software (Ensoltech Inc., Montana) and MASS software platforms (National Masonry Design Programs, Ontario) in the US and Canada, respectively. Direct Design is an all-inclusive software which computes and distributes the loads to structural elements as well as computes the resistance of the masonry elements in accordance with the TMS 402-16 material design standard. However, the software is limited to modelling uniform floor plans. As such, the two-storey mixed-used warehouse/office building was modelled as two separate buildings due to the software limitations.

In Canada, MASS models masonry walls along a given building line (elevation) and computes the resistance of masonry elements in accordance with the CSA S304-14 requirements. Although the software distributes the overall shear force inputted for the line in accordance with the rigidity of the elements, the user must manually compute and determine the distribution of the design forces to each wall line for input into the software.

3.3.2.1 Governing Design Elevation and Elements

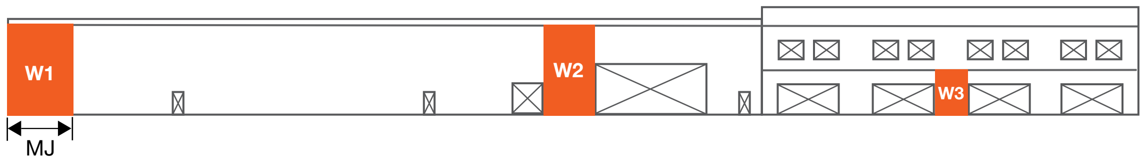
To simplify the design process, comparison was made only between governing elements within the structure between the two codes. As indicated in Figure 4 a), the critical wall cross-sections for in-plane and out-of-plane design were located in the East/West exterior elevations of the building. Pictured are the three selected wall design locations, defined as:

- **W1 –** A generic warehouse wall that would represent the majority of the design including other exterior elevations. The unit and reinforcement configuration selected here would dictate the economic feasibility of the project. The design is dominated by wind loads acting along the out-of-plane direction.
- **W2 –** The most critical wall in the structure due to the high tributary axial and out-of-plane loads imposed on the wall from adjacent beams. The design of this wall was considered to be unique and would require details that would differ from the rest of the building, at an added expense, but not as to dictate the economic feasibility of the structure as a whole necessarily.
- **W3 –** Generic office wall, which represents the construction in the two-storey section of the structure. Since these walls are relatively short in height, their design is governed by in-plane loads.

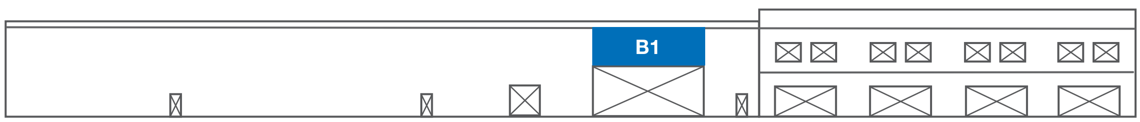
As indicated in Figure 4a), the warehouse design drawings did not provide movement joint locations (MJ). This was left as a design variable as it impacted in-plane design of the walls, specifically for the regions of high seismicity as will be described.

Figure 4: Governing Elevation for Two-Storey Mixed-Use Building

a) East/West Elevation with Beams Indicated



b) East/West Elevation with Walls Indicated



Design of the masonry beam critical in the East/West elevation is also pictured in Figure 4b), which is defined as the following:

- **B1 –** Is considered to be an exceptionally large masonry beam by Canadian practice and is located over a 11.0 m (36 ft.) wide opening. This is not a typical masonry beam but represents some of the unique designs that are currently being completed by US designers.

3.3.2.2 Two-Storey Mixed-Use Building US Archetype Design Results for Niagara Falls, New York

The points below were noted for the warehouse portion of the two-storey mixed-use building US archetype design for Niagara Falls, New York. Results for the office area were not reported as they did not govern the design. The design was completed using a SFRS with limited ductility: ordinary reinforced masonry walls (R = 2.0). Out-of-plane and axial loads dictated the design of the warehouse.

- Wall W1 and W3 could be designed with 8 in (20 cm) block wall with f'_m of 2,000 psi (13.8 MPa) and #7 (387 mm^2) vertical reinforcement at 48 in (1,220 mm) on-centre and horizontal heavy duty (HD) bed joint wire reinforcement (BJR) at 26 in (660 mm) on-centre. The selection of this size and strength of block with such a wide-spaced rebar suggest that from an American perspective loads are relatively minor. It is a fair conclusion that from the American perspective these structures are relatively easy to construct with masonry.
- Wall W2 required the use of 12 in (30 cm) block with f'_m of 2,000 psi (13.8 MPa) with #8 (509 mm^2) vertical at 120 in (3,048 mm) on-centre and horizontal HD BJR at 16 in (406 mm) on-centre. The larger unit size was needed strictly to address maximum reinforcement requirements in the wall. A larger unit size is a design strategy to help decrease the depth of the neutral axis permitting reinforcement to yield.
- Beam B1 could be designed with 10 courses of 8 in (20 cm), 2,000 psi (13.8 MPa) units and only 2 × #9 (645 mm^2) reinforcing bars as the main tension reinforcement. Notably, the beam would not require compression reinforcement or shear stirrups, making its construction relatively simple.

Cross-section details for the above-described walls and beams are shown in Figure 5a) in comparison to the Canadian design details.

3.3.2.3 Two-Storey Mixed-Use Building Canadian Archetype Design Results for Niagara Falls, Ontario

As with the US results, out-of-plane loads dictated the design of the masonry in the warehouse. The design was completed using a SFRS with limited ductility: conventional construction masonry shear walls (R_dR_o = 2.25). A summary of the findings is provided below for the Canadian design, which are also illustrated in Figure 5b):

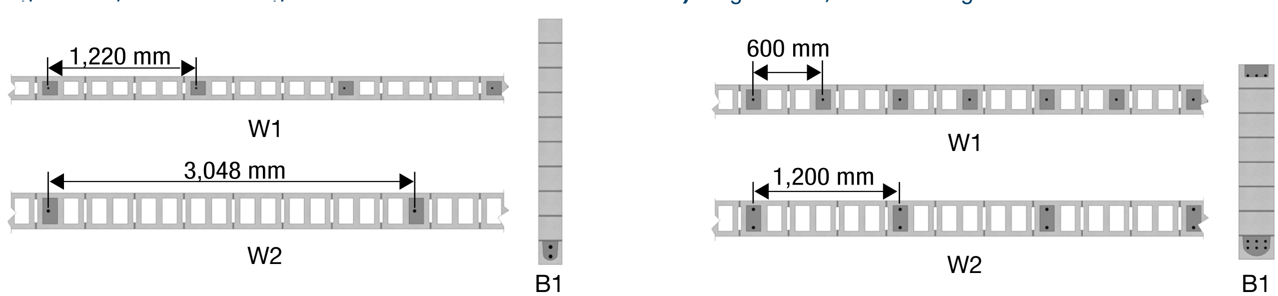
- Wall W1 could not be designed with a 20 cm unit. This was the most significant difference between the countries, as the starting point for Canadian design was to effectively look at 25 cm units. The only possible way to design with a 20 cm unit was to construct the wall with fully-grouted, 30 MPa units with 20M vertical bars spaced at 400 mm on-centre. This would be an incredibly costly and difficult wall to construct for an entire warehouse. Using a 25 cm unit can be achieved, but a high strength unit is still needed (20 MPa). Although a 25 cm unit needs to only be partially-grouted, reinforcement requirements are still high, requiring a 25M @ 1,000 mm or 20M @ 600 mm (the design option pictured in Figure 5b)) would both pass. Finally, using a 30 cm unit still requires a 20 MPa strength but permits a 25M bar @1,400 mm or 20M @ 800 mm.
- Wall W2, similar to the US design, requires a 30 cm unit for conventional construction techniques. It is possible to use a 25 cm unit, 30 MPa strength, that is fully-grouted with 2×20M bars at 400 mm on-centre so that eccentric reinforcement is produced. If a 30 cm unit is used, then a fully-grouted 30 MPa block is needed with a 25M @ 400 mm on-centre. Partially-grouted masonry is possible with a 30 cm unit, 25 MPa, and 2×25M bars placed at 1,200 mm (this is the option indicated in Figure 5b)).
- Wall W3 could be constructed with 20 cm units, but still require a high strength (20 MPa). Partial-grouting with 20M bars @ 800 mm is possible.
- Beam B1 could be designed as an 8 or 9 course beam, depending on the wall unit size selected. For 30 cm or 25 cm units, a 30 MPa unit strength is needed and compression reinforcement is also needed. For an 8-course 30 cm unit a tension layer of reinforcement comprised of 2 layers of 3×20M bars are required with 3×20M bars also needed as compression reinforcement (this is the option picture in Figure 5b) without shear stirrup details shown). For a 9-course 25 cm unit beam, tensile reinforcement of 2 layers of 2×20M bars are required with compression reinforcement of 2×20M. Both cases require shear stirrups and compression ties, making the design of this beam much more costly and complex than the US design which requires neither.

A summary of the wall and beam details for the two-storey mixed-use building in Niagara Falls (New York/ Ontario) is given in Figure 5. In general, the Canadian design would require a stronger and larger block size compared to the US design and requires a greater amount of reinforcement.

Figure 5: Design Cross-Section Details for 2-Storey Mixed-Use in Niagara Falls

a) Niagara Falls, New York Design

b) Niagara Falls, Ontario Design



3.3.2.4 Two-Storey Mixed-Use Building US Archetype Design Results for Blaine, Washington

The points below were noted for the warehouse portion of the two-storey mixed-use building US archetype design for Blaine, Washington. Results for the office area were not reported. The design was completed using a ductile SFRS: special reinforced masonry walls (R = 5.0). In this case, seismic in-plane loads dictated the design of the walls.

- Walls W1 and W3 could be designed with an 8 in (20 cm) unit with a unit strength of 2,000 psi (13.8 MPa). Vertical reinforcement required was a #7 (387 mm^2) bar at 32 in (813 mm) on-centre and #6 (284 mm^2) horizontal bond beams at 48 in (1,220 mm).
- Wall W2 required a 12 in (30 cm) unit with 3,000 psi (20.7 MPa) strength. Vertical reinforcement needed included #7 (387 mm^2) bars at 32 in (813 mm) on-centre and horizontal bond beams of #6 (284 mm^2) bars at 48 in (1,220 mm).
- Beam B1 was not affected by the change in location.

Notably, the US design for a warehouse did not fundamentally differ from the two locations in the masonry unit selection needed for the majority of the structure. Design details are not presented because a Canadian design was not possible.

3.3.2.5 Two-Storey Mixed-Use Building Canadian Archetype Design Results for White Rock, British Columbia

The design was initially investigated for considering a SFRS with conventional reinforced shear walls. This design was not possible. Block size was increased, reinforcement was tightened, and movement joint locations were altered; however, as block size increases and spacing of reinforcing decreases, the seismic weight increases. As such, selecting a ductile shear wall SFRS (R_d = 3.0) was not capable of accommodating the shear force demand. It was observed in each design iteration that at least one wall segment in the structure would reach their maximum shear resistance limit. The only exception to this was the case of a theoretical wall elevation with no movement joints. This was due to piers being created within the wall that were defined as squat and had a higher maximum limit to their shear resistance. In that particular case, 20M bars would still be required along the base of the wall in every other cell to resist shear sliding. Overall, a passing design was only possible by going beyond what would be considered typical construction practice and by providing extreme wall details that would push the limits of constructible masonry.

3.3.3 Multi-Storey Residential Building Archetype Design Results

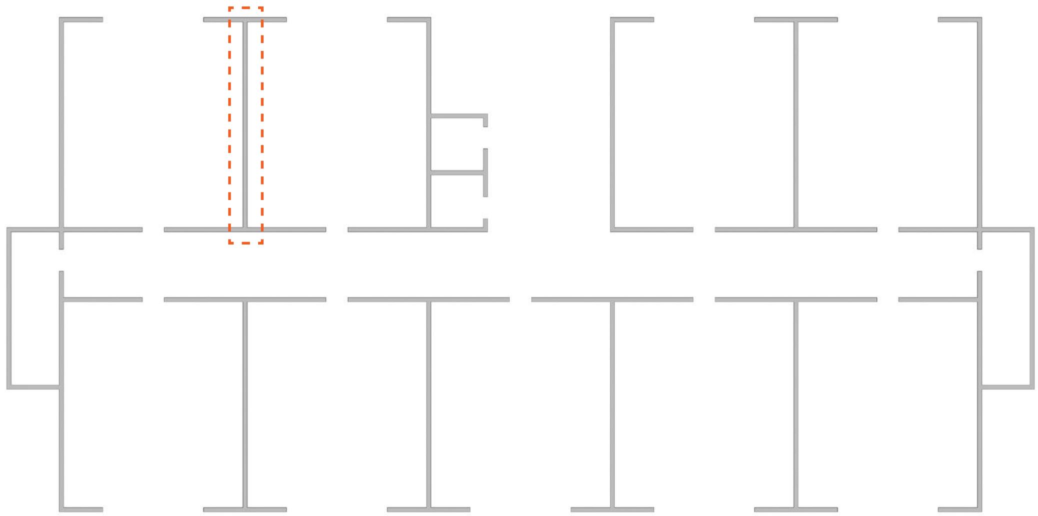
The objective of the multi-storey residential building archetype was to determine how many storeys could be built based on a storey height of 3.0 m. Modelling of the archetype was carried out in a similar fashion as noted for the two-storey mixed-use archetype. Although the layout of the floor plan indicated that flanged walls would be a design option, for simplicity and based on typical design practice, this option was not considered. The building plan of the multi-storey residential structure is pictured in Figure 6 with a typical shear wall indicated by the dashed red line. Results are reported in the sections below.

3.3.3.1 Multi-Storey Residential US Archetype Design Results

Unlike the two-storey structure, in this case, design axial loads became a dominant design parameter. This resulted in significant problems with US design in both locations. The following observations were made by the US design team for both Niagara Falls, New York and Blaine, Washington:

- Using an 8 in unit (20 cm) with a strength of 2,000 psi (13.8 MPa) permitted three storeys of loadbearing masonry (9.0 m) in the structure. This required vertical reinforcement of #5 (200 mm^2) bars at 88 in (2,240 mm) on-centre and horizontal HD BJR at 16 in (406 mm).
- Building to 4 storeys, an 8 in (20 cm) unit could not be accommodated.
- Maximum reinforcement provisions, which limit the area of reinforcement in walls, began to govern design. As a result, the US design team could not accommodate the axial loads over 3 storeys in height.

Figure 6: Floor Plan of Multi-Storey Residential Archetype



3.3.3.2 Multi-Storey Residential Canadian Archetype Design Results for Niagara Falls, Ontario

The Canadian design team first explored a 6-storey structure and then a 10-storey structure. Only a 20 cm unit was considered for the design to compare with the US design team. A conventional construction shear wall SFRS was selected and seismic in-plane loads were judged to be the governing factor to the design. The points below were noted for the multi-storey residential Canadian archetype design for Niagara Falls, Ontario.

- Building to a height of 6 storeys (18 m) required 20 cm units with a unit strength of 20 MPa, fully-grouted to resist in-plane shear. Vertical reinforcement consisted of 15M bars @ 1,200 mm on-centre and horizontal reinforcement consisted of HD BJR @ 200 mm.
- At a height of 10 storeys (30 m), a 20 cm unit with strength of 30 MPa was required, fully-grouted. Vertical reinforcement still consisted of a 15M bar @ 1,200 mm and horizontal reinforcement consisted of HD BJR @ 200 mm.
- It was clear that it was still possible to build higher with a 20 cm unit; however, the NBCC 2015 would require that, for this location, the SFRS would have to comply with a moderately ductile shear wall system. It is likely then that when using the higher ductility category, design issues similar to the US team would be encountered due to the requirement for reinforcement to yield in tension.

3.3.3.3 Multi-Storey Residential Canadian Archetype Design Results for White Rock, British Columbia.

The higher seismic hazard in this location limited the height for conventional construction shear walls to just 15 m (5 storeys). The following observations were made by the Canadian design team:

- At 5 storeys (15 m) in height, 20 cm and 25 cm units did not have enough in-plane shear capacity. A 30 cm unit with 25 MPa strength could not be accommodated due to in-plane shear failure.
- A design at 4 storeys (12 m) was attempted and succeeded with a 30 cm unit with 25 MPa strength, fully-grouted. Vertical reinforcement consisted of 20M bars @ 1,000 mm on-centre and horizontal reinforcement was HD BJR @ 200 mm with 10M bond beams @ 1,200 mm.
- As noted for the 2-storey mixed-use design, the shear limit, V_{max}, governed design of the masonry for regions of high seismicity.

3.4 Preliminary Archetype Design Key Findings

The key findings below were noted from the preliminary archetype designs.

- The lower masonry compressive strength, f'_m, values in Canada often limit the design. This was most notable for the design of out-of-plane governed walls and beams in the two-storey mixed-use building archetype. Low strength and stiffness due to f'_m (compounded by the masonry material reduction factor ϕ_m) caused differences to the magnitude of the slenderness amplification effects. It was observed that, for the warehouse generic wall W1, American design loads resulted in a primary moment that was about 35% less than that use in Canadian design. In addition to this, the moment amplification factor determined using Canadian stiffness properties was equal to 1.6. By comparison, American passing designs had a moment amplification factor of less than 1.1 for a smaller unit size.
- The maximum reinforcement limit in the TMS 402-16 often controls the design in the US in all cases. This is observed where high axial loads are present such as Wall W2 in the two-storey mixed-use building archetype, and in the multi-storey residential structure. It was observed that yielding criteria for CSA S304-14 walls will come into play for out-of-plane walls when the slenderness ratio is greater than 30. Otherwise, in the multi-storey residential structure yielding of flexural reinforcement would only be required if a moderately ductile SFRS is designed.
- Large beams are technically feasible by Canadian design; however, the steel detailing requirements often make construction impractical and expensive. Beam B1 in the two-storey mixed-use building archetype demonstrated that restrictions to Canadian design may be overcome to resist the loads, but the necessity of tied compression steel, shear stirrups and multiple layers of tension reinforcement would make this impractical to physically construct in an economical manner. Other beams in the structure were also designed, but not reported here as the conclusions remain the same.
- Higher seismic design categories in ASCE 7-16 forces the use of a higher ductility seismic force-resisting system for all materials. In Canada, the short and long period triggers, 0.2s and 1.0s, do not necessarily force a higher ductility, but rather limit the height of systems with lower ductilities. In higher Canadian seismic zones, masonry construction of low-rise buildings with conventional construction SFRS are unable to accommodate the high seismic design forces; thus, requiring the use of a SFRS with higher ductility to accommodate the associated seismic demand. In such cases, the use of masonry SFRSs with higher ductilities renders this type of construction cost-prohibitive in comparison to conventional construction SFRSs of other materials, such as concrete, which can accommodate higher seismic demands.
- Limits to the maximum factored shear resistance in seismically designed walls combined with an amplification to the factored shear force can make it impractical to design loadbearing masonry structures in the seismically controlled designs in British Colombia.

4.0 Conclusions and Recommendations

4.1 Summary of Major Proposed Changes to the CSA S304-14

4.1.1 Masonry Compressive Strength, f'_m

A common theme observed during this analysis was the effect that the masonry compressive strength has on the results from both standards. Masonry prisms are the standard assembly test to verify masonry compressive strength that is to be used for design. Masonry prisms constructed to the requirements of the CSA S304-14 are normalized to a height-to-thickness ratio of 5, must consist of a full unit length and constructed in face shell bedded mortar. By contrast, masonry prisms constructed to the requirements of design with the TMS 402-16 are normalized to a height-to-length ratio of 2, may consist of a half unit length and may be constructed in face shell and web bedded mortar.

"The masonry prism specified for Canadian design purposes is meant to replicate the workmanship, failure mechanisms, and boundary conditions of walls."

The masonry prism specified for American design purposes is meant to provide a simple means to validate the assembled strength of the masonry materials (concrete block, mortar, and grout). Then, in the same manner as done for reinforced concrete design and concrete cylinder testing, design equations and force modification factors (ϕ-factors) are used to correlate the prism strength to that of the final structural element. This is done because masonry prisms cannot account for all possible variables for in-situ conditions (workmanship, tolerances, size, loading conditions, confining effects, etc.) and the failure mode of a masonry prism would not reflect the variety of possible failure modes within walls or beams as constructed.

The masonry prism specified for Canadian design purposes is meant to replicate the workmanship, failure mechanisms, and boundary conditions of walls. Presumably, the resulting design equations and ϕ-factors would correlate the strength of these prisms to that of the structural elements being designed. However, this project has revealed that the net result from the combined material properties, ϕ-factors, and design equations between the two standards does not result in the same design. This observation is maintained when masonry designs are evaluated within the lens of the prevailing building code loads, as done with the archetype designs. The net result is that Canadian masonry structures require larger and stronger units with more reinforcing. A possible reason for this is due to the development of the Canadian masonry prism test at a time before limit states design was adopted elsewhere in the standard. It is speculated that the current Canadian prism test was derived to account for variables that would be otherwise accounted for with the adoption of the ϕ-factors. Notably, the masonry strength design values in the CSA S304-14 are largely unchanged from the original imperial values for strength given in the 1977 edition of the CSA S304 [5] and the soft metric conversions used in the 1984 and 1994 editions [6]–[7].

Future editions of CSA S304 may consider the adoption of US masonry compressive design strengths. Design equations that otherwise achieve the same behaviour have largely been adopted on both sides of the border. Design equations have changed substantially since the early editions of the CSA S304 (e.g., the growing prevalence of reinforced masonry and move away from working stress) and the adoption ϕ-factors since that time suggests that the definition of masonry strength may need to be updated. The application of the ϕ_m resistance factor for masonry and the newly proposed compression-controlled ϕ strength-reduction factor of 0.6 for the TMS 402 would result in a greater harmonization between the standards if the same f'_m was used. Proposed specified compressive strength normal to the bed joint masonry design strengths are provided in Table 1 for consideration based on a linear interpolation of TMS 402/602-16.

Table 1: Proposed Masonry Design Strengths

Specified compressive strength of unit (MPa)	Specified masonry compressive strength normal to the bed joint, f'_m (MPa)	
	Type S	Type N
30	20.3	17.6
25	18.2	16.0
20	16.3	14.4
15	14.3	12.5

Use of these new masonry strength values would further require better statistical alignment to current specified strength calculations, integration within a robust reliability analysis to evaluate ϕ_m, as well as a re-evaluation of the impacts of different grout strengths and their design implications.

4.1.2 Use of the Effective Face Shell Thickness

Adopting US-based masonry strength values in CSA S304-14 would also require the use of the minimum face shell for resistance calculations. The use of the effective face shell was never adopted as originally intended as block producers do not provide such information and designers have historically not taken up the calculation of such a number. The fact that a designer may not know the block producer or type of unit being used during the design stage makes it difficult for values to be used in a meaningful way. As a result, industry-wide accepted values [8] have seen widespread use without consideration of different block configurations. Instead, it is suggested that the minimum face shell thickness be the standard's default value in strength calculations. However, provisions should be given to enable a designer or manufacturer to provide their own testing to warrant the use of a larger face shell thickness for strength calculations.

4.1.3 The Directionality Factor, χ

For cases of fully-grouted beams where normal construction practices are followed and reduced web units are used, it is proposed that $\chi = 1$. No directional strength factor is used in the TMS 402-16, and this is a reasonable compromise. Leaving the factor in for cases of hollow masonry bending in flexural wall panels would satisfy some historic precedent. Additional research may be needed to fully support a complete dissolution of the χ factor for all forms of construction, but in the case of fully-grouted beams with reduced webs, it should be removed.

4.1.4 Stack Pattern Masonry

CSA S304-14 is more stringent in its design requirements and prescriptive limits to stack pattern masonry than TMS 402-16. It is suggested that a provision be added in CSA S304 that permits use of stack pattern as a direct substitute for running bond for all applications (beams, shear walls, out-of-plane walls). The use of stack pattern in this manner should only be permitted when a minimum area of horizontal reinforcement in the form of a bond beam is used, when vertical shear interface strength is sufficient to prevent excess cracking, and when fully-grouted units are present. In the cases of partially-grouted masonry walls, and shear walls containing a plastic hinge additional investigation is required.

4.1.5 Change to β_1

As the masonry compressive strengths, f'_m in CSA S304-14 are less than 20 MPa, it was suggested to consider replacing the β_1 factor, that is the ratio of depth of rectangular compression block to depth to the neutral axis, with 0.8 for all cases.

4.1.6 Squat Wall Moment Arm Reduction Provisions

The shear wall parametric study demonstrated that the reduced moment arm provision as it is currently worded may be inadequate for squat walls with aspect ratios near 1.0 with low levels of axial load governed by flexure. It was suggested to further review this condition as it was noted that shear resistance is less likely to be a governing mechanism for walls with aspect ratios approaching 1.0.

4.1.7 Effective Compression Zone Width, 4t

The CSA S304-14 effective compression zone width provision of 4t may be conservative and is lower than the US provision of 6t. Furthermore, the nominal thickness of the wall is used in determining the application of the 6t provision in TMS 402-16. The 4t provision should therefore be reviewed as it greatly impacts the resistance of masonry walls in out-of-plane loading conditions, more specifically for tension-controlled responses. In addition to this, there are some locations where the CSA S304-14 opts for prescriptive rules based on nominal wall thickness and some where it is based on actual wall thickness. A singular approach should be used and applied throughout.

4.1.8 Slenderness Phi-Factor, ϕ_{er}

Although typical for reinforced concrete design in both countries, the notable absence of such a factor in US design suggests that the use of an added ϕ-factor for slenderness calculations may be unnecessary. A re-evaluation of the reliability analysis of such a design should be conducted to establish what the rationale is for such a factor other than simply replicating reinforced concrete design.

4.1.9 Stiffness of Masonry for Slenderness Calculations

Accounting for axial load and non-linear stress in the masonry when determining section properties of cracked masonry should be explored. The explicit limit in CSA S304-14 to restrict against including axial load seems contrary to US design practice. The current stiffness formula in the CSA S304-14 appears to have limitations outside a narrow scope of wall types. Additional research in this area is required for both standards.

4.2 Research Needs

In order to address some of the differences between the standards, additional study and research will be required. This may include physical testing, modelling, or reviews of existing literature.

4.2.1 Derivation and Application of ϕ-Factors

The fundamental philosophy behind the current prism strength procedures and the derivation and application of the masonry ϕ factor should be re-examined. A detailed reliability analysis with both design standards should be conducted within the context of their prevailing building codes to ensure that increased masonry strengths will still meet limit states design requirements. Some additional testing may be required; however, much of this can be done through analytical techniques.

4.2.2 Out-of-Plane Wall Stiffness

Modulus of Rupture (MOR) values in TMS 402-16 were derived from wall/wallette tests and may not include the effects of anisotropy, grout, beam arrangement, etc. More comprehensive calculations could be investigated through experimental research considering the arrangement of the setup, direction of loading, and grout. Conversely, the flexural tensile stress values in CSA S304-14 are based on the bond wrench test, which also has limitations in its ability to replicate in-situ masonry behaviour.

In CSA S304-14, the slenderness effects and effective stiffness affect the deflected shape and moment force magnification in the moment magnifier method. Both standards can benefit from simpler, more user-friendly equations, supported by research/experimental results. This research should be conducted as a collaboration between US and Canadian researchers to leverage resources, especially since they tie to the fundamentals of engineering mechanics that are the same in both countries.

The effective compression zone for masonry bending out-of-plane should also be evaluated. As limits should be based on rational analysis and experimental evidence, possible design conditions could be established where different effective compression zone widths would be applicable for different design scenarios. The current approach is using a singular value for all loading cases, and wall configurations may not be accurate for all cases and may be a reason as to why different values have been adopted by the two standards.

4.2.3 New Materials

Neither standard addresses high strength masonry nor high strength steel, even though the manufacturers may be making high strength block routinely due to their manufacturing processes. Design equations could be revisited based on material strength ranges. Further, neither standard addresses fibre-reinforced polymer (FRP) bar reinforcement or light-weight grouts. Additionally, use of new additives or changes in block composition may have different effects on masonry behaviour not currently captured in design standards. Given the current trends towards energy efficiency, both standards could benefit from including these newer materials.

References

[1] *Design of Masonry Structures,* CSA S304-14, Canadian Standards Association, Toronto, ON, CAN, 2014.

[2] *Building Code Requirements and Specification for Masonry Structures,* TMS 402/602-16, The Masonry Society, Longmont, CO, USA, 2016.

[3] *National Building Code of Canada,* Canadian Commission on Building and Fire Codes and National Research Council of Canada, Ottawa, ON, CAN, 2015.

[4] *Minimum Design Loads for Buildings and Other Structures,* ASCE 7, American Society of Civil Engineers, Reston, VA, USA, 2016.

[5] *Masonry Design and Construction for Buildings,* CSA S304, Canadian Standards Association, Toronto, ON, CAN, 1977.

[6] *Masonry Design for Buildings,* CSA CAN-S304, Canadian Standards Association, Toronto, ON, CAN, 1984.

[7] *Masonry Design for Buildings (Limit States Design),* CSA S304.1, Canadian Standards Association, Toronto, ON, CAN, 1994.

[8] R. Drysdale and A. Hamid, *Masonry Structures Behaviour and Design, Canadian Edition,* Mississauga, ON: Canada Masonry Design Centre, 2005.

CSA Group Research

In order to encourage the use of consensus-based standards solutions to promote safety and encourage innovation, CSA Group supports and conducts research in areas that address new or emerging industries, as well as topics and issues that impact a broad base of current and potential stakeholders. The output of our research programs will support the development of future standards solutions, provide interim guidance to industries on the development and adoption of new technologies, and help to demonstrate our on-going commitment to building a better, safer, more sustainable world.

CSA GROUP™