13TH CANADIAN MASONRY SYMPOSIUM
HALIFAX, CANADA
JUNE 4TH – JUNE 7TH 2017

TOWARDS THE EVALUATION OF EMPIRICAL AND RATIONAL DESIGN PROVISIONS FOR UNREINFORCED MASONRY WALLS

Rezaeivahdati, Amir[1] and Feldman, Lisa R.[2]

ABSTRACT
CSA S304-14, Design of Masonry Structures, continues to include empirical design provisions despite the introduction of rational design methods for masonry structures in Canada in 1965. There are cases in which masonry structures designed according to CSA S304-14 empirical provisions produce less conservative results compared to when they are designed in accordance with the rational provisions. This indicates either the empirical design provisions are not as safe as they should be or the rational design provisions are unnecessarily conservative. One of the cases in which masonry design based on empirical and rational provisions produces significantly different results relates to unreinforced, vertically-spanning, non-loadbearing exterior concrete block walls subjected to wind loads. For walls with certain slenderness ratios, walls designed in accordance with the empirical design provisions included in CSA S304-14 are capable of resisting a greater 1-in-50 year allowable wind pressure than those resulting when designed in accordance with the rational provisions. To ensure economic efficiency and an appropriate level of safety, the two design methods should be reconciled. To that end, different factors affecting the development of design provisions for such walls should be understood and quantified. This paper examines the status quo of the literature related to a number of these factors to highlight potential shortcomings in current knowledge. A research program underway at the University of Saskatchewan with the aim of resolving this discrepancy is introduced.

KEYWORDS: *unreinforced masonry, concrete block walls, empirical design, rational design, flexure*

INTRODUCTION
The Canadian standard used for the design of masonry structures, CSA S304-14 [1], includes both rational and empirical design provisions for unreinforced masonry walls. The two methods

[1] M.Sc. Student, University of Saskatchewan, Dept. of Civil, Geological, and Environmental Engineering, 57 Campus Drive, Saskatoon, SK, S7N 5A9, amr277@mail.usask.ca

[2] Associate Professor and Director of the Saskatchewan Centre for Masonry Design, University of Saskatchewan, Dept. of Civil, Geological, and Environmental Engineering, 57 Campus Drive, Saskatoon, SK, S7N 5A9, lisa.feldman@usask.ca

produce inconsistent results in certain cases. One of the cases relates to unreinforced, vertically-spanning, non-loadbearing exterior concrete block walls subjected to wind load, where after a certain slenderness ratio rational designs produce more conservative results. This difference in design outcome indicates either a lack of safety of the empirical design or economic inefficiency of the rational design.

This discrepancy has not been addressed directly in past studies. The safety of rational and empirical provisions has not been quantitatively investigated using reliability analysis and it has not been determined which one of the two design methods predicts the strength of walls more accurately. There have, however, been several studies that have quantified various aspects related to the strength of unreinforced masonry walls in pure flexure. This information could be used to resolve the cases of disagreement between the results of empirical and rational design provisions.

This paper includes a review of the current state of knowledge related to the behavior of unreinforced masonry walls in flexure as it applies to the design of unreinforced concrete block walls. An experimental program is currently underway at the University of Saskatchewan to resolve the issue.

EMPIRICAL AND RATIONAL PROVISIONS IN CSA S304-14

Empirical design provisions refer to rules of thumb established from past construction experience. Rational design provisions, in contrast, are based on engineering principles. The first design provisions in Canada for masonry structures were empirical. The first rational principles were developed in the 1960s [2]. Currently, CSA S304-14 [1] includes both rational and empirical provisions for the design of masonry structures. An overview of both provisions as they apply to the design of unreinforced masonry walls in flexure is provided here.

The empirical provisions of CSA S304-14 [1] allow walls satisfying a set of criteria to be constructed without engineering calculations. Some of the limits applying to unreinforced, non-loadbearing masonry walls in flexure are presented below:

- Hollow units should have a minimum width of 190 mm and a maximum slenderness ratio of 20.
- Walls cannot be constructed such that they are greater than 20 m above grade.
- The 1-in-50 year hourly wind pressure cannot exceed 0.55 kPa.
- The design method is not permitted for the design of foundation walls subjected to lateral loads, structures with a seismic hazard index that is greater or equal to 0.35, or for walls subject to lateral loads originating from sources other than wind or earthquake.

The rational principles that apply to the design of unreinforced walls are introduced in Clause 7 of CSA S304-14 [1] and are based on a linear elastic analysis with a maximum allowed slenderness ratio equal to 30. Accordingly, the flexural tensile strength of walls should be larger than the sum of the stresses caused by combined flexure and self-weight.

Figure 1 shows the maximum allowable wind pressure resisted by unreinforced walls designed using either the rational or empirical design provisions included in CSA S304-14 [1] with slenderness ratios ranging from 10 to 20, the maximum permitted for walls using the empirical design method. Figure 1 shows that, independent of the slenderness ratio of the wall, walls designed in accordance with the empirical provisions are consistently designed to resist a 1-in-50 year wind pressure of 0.55kPa, whereas the wind pressure capable of being resisted by walls designed using the rational provisions decreases non-linearly from 1.3 kPa at a slenderness ratio of 10 to 0.38kPa when the wall has a slenderness ratio equal to 20. The figure shows that the maximum allowable wind pressure resisted by walls using either the rational or empirical methods is equal when the slenderness ratio is approximately 16. Below a slenderness ratio of 16, walls designed using the rational provisions can resist higher values of wind pressure than walls designed using the empirical method, whereas for greater values of the slenderness ratio, walls designed using the rational provisions resist lower values of wind pressure.

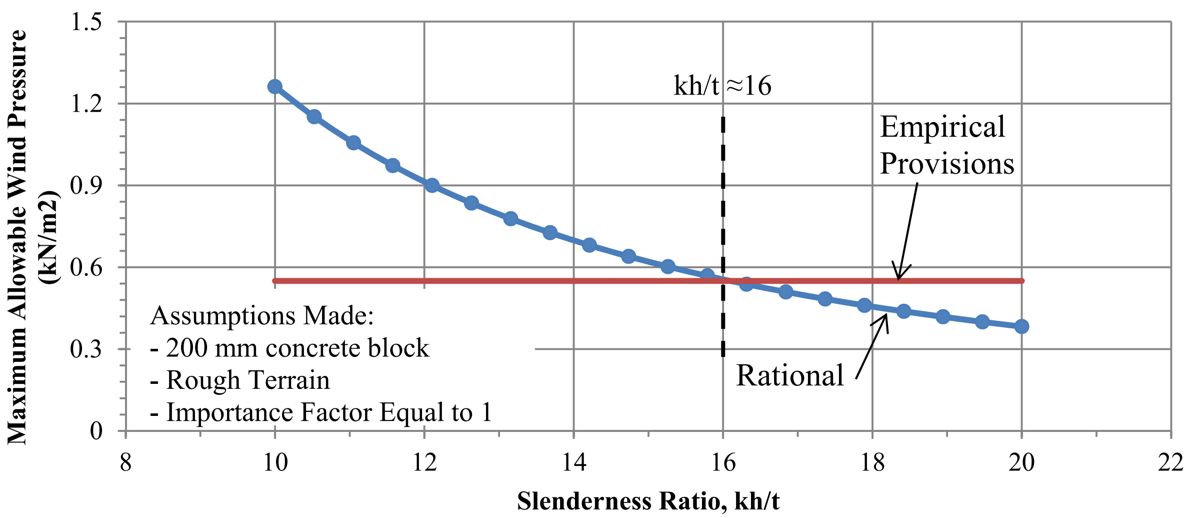

Figure 1: Effect of Slenderness Ratio on the Flexural Strength of walls based on Empirical and Rational Provisions

Although no failures have been recorded in the past when walls were designed based on the empirical provisions [2], the difference in the resulting designs using the two code methods indicates that the necessary safety and economic efficiency of walls should be determined. Past researchers have not specifically addressed this problem, but results of more fundamental research, as described in the following section, address the parameters that contribute to resolving the differences between these two design methods. This paper will review previous research programs and discusses some of these factors.

PAST WORKS AND TYPICAL TEST SPECIMENS USED

Several studies were conducted whose findings apply to the problem at hand. A review of past works related to the quantification of the flexural strength of unreinforced masonry walls will be made. Results of such works are crucial to understanding the behavior of walls in flexure and thus evaluating the rational and empirical provisions.

Table 1 summarizes the primary relevant parameters evaluated in past research programs. Table 2 provides the results of flexural tensile tests as reported in the available literature.

Table 1: Summary of relevant past research programs

Researchers	Curing Method		Ratio of flexural strength of full size walls to prisms	Retempering of Mortar	Load application	Wall span (m)
	Relative Humidity (%)	Temp. (C˚)				
Richart 1932 [3]	50-65	21-24	Not Reported	Not Reported	Single point load at mid-span	2.7
Hedstrom 1961 [4]	38-65	23	0.96–1.23	Permitted	Uniform loading	2.3
Fishburn 1961 [5]	≥50	27	1.16	Permitted	Two-point loading	2.3
Copeland and Saxer 1964 [6]	75	23	2 - 4	Permitted	Uniform loading	2.3
Drysdale and Essawy 1988 [7]	Not Reported	Not Reported	0.74[1]	Not permitted	Uniform loading	2.8
Matthys 1990 [8]	55	22	1.7-1.8 [2]	Permitted	Uniform loading	2.4
NCMA 1994 [9]	Sprayed and sealed in bags	24 ± 9	1.11	Permitted once	Uniform loading	2.4
Udey 2014 [10]	Not Reported	Not Reported	2.8	Permitted	Two-point loading	3.0

[1] Ratio of flexural strength of full-size wall to 5-course prism

[2] Ratios of flexural strength of full-size wall to 3-course prisms (This ratio was 0.94 -1.3 for prisms cut from walls)

Table 2: Reported Values of Flexural Tensile Strength of Unreinforced Masonry Walls

Researchers	Mortar Type(s) Used[1]	Flexural Tensile Strength (MPa)	Comments
Richart 1932 [3]	PCL	0.12 - 0.34	Two mortars mixes, different levels of lime
Hedstrom 1961 [4]	PCL	0.23 – 0.41	Two mortars mixes, different ratios of Portland Cement, lime and sand
Fishburn 1961 [5]	MC	0.07 – 0.23	Mixes of PC-based MC mortars used
Copeland and Saxer 1964 [6]	PCL & MC	0.21 – 0.71 [2]	Results based on walls only
Drysdale and Essawy 1984 [7]	PCL	0.31	Type S mortar
Matthys 1990 [8]	PCL & MC	0.16 for MC 0.29 for PCL	Type S mortar for both MC and PCL
NCMA 1994 [9]	PCL	1.3	Reported based on the average of types M, S and N PCL mortar
Udey 2014 [10]	Mortar Cement	0.09 – 0.14[3]	Type S mortar

[1] PC: Portland Cement, PCL: Portland Cement Lime, MC: Masonry Cement, MRC: Mortar Cement
[2] 0.034 – 1.2 MPa based on 2-course prisms
[3] 0.04 MPa based on bond wrench test

Support fixity is a parameter that was omitted in Table 1. All of the studies presented with the exception of Udey [10] allowed for rotation by providing supports similar to the recommendations of ASTM E72 [11]. The various studies reviewed provided different degrees of details related to the support conditions in test setups. Udey [10] tested walls using two different support conditions, one of which was based on common construction practice, referred to as realistic support condition as shown in Figure 2(a), and the other allowed for rotation, referred to as ideal support condition, similar to that shown in Figure 2(b).

DISCUSSION

The parameters introduced in Tables 1 and 2 will be discussed with regards to their effect on the flexural strength of unreinforced masonry walls and the conclusiveness of the available data. It will be shown how a reliability analysis could incorporate information from past works to determine the level of safety provided by the rational design provisions.

Mortar type and strength

The flexural tensile strength reported in Table 2 shows a wide range of variability. The lowest values belong to tests by Fishburn [5]. Flexural tensile strengths of 0.16 and 0.07 MPa were obtained for mortars including and not including Portland cement, respectively. The NCMA [9] reported the highest flexural tensile strengths: 1.71 MPa and 1.12 MPa for walls constructed with Type M Portland cement lime in conjunction with 300 mm concrete blocks, and Type S Portland cement mortar in conjunction with 200 mm blocks, respectively.

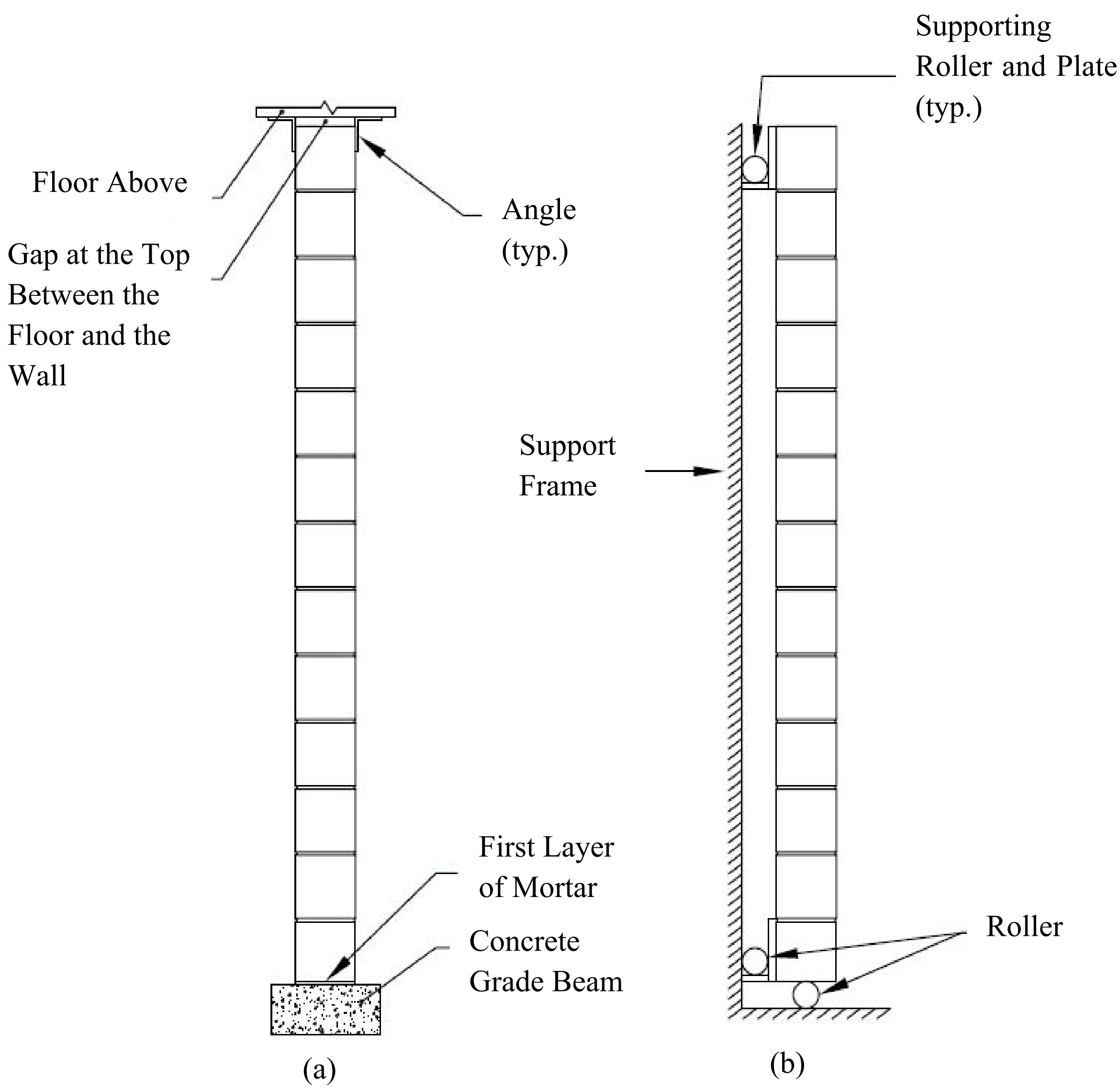

Figure 2: Typical Support Conditions Used in Experimental Investigations: (a) Realistic Support Condition, and (b) Ideal Support Condition Based on ASTM E72-15

The majority of previous tests reported in Table 2 included the use of Portland cement mortars. However, Portland cement mortar has been mostly replaced in Canada by masonry cement mortar [2]. Masonry cement mortar is known to have lower flexural strength than Portland cement mortar. Matthys [8] showed that the flexural tensile strength of walls built using masonry cement mortar is 56% of those built using Portland cement mortar. Mortar cement mortar is generally proportioned such that its strength is similar to Portland cement mortar; however it is mostly used in Western Canada [2]. Additional tests using masonry cement and mortar cement mortars are therefore required to establish reliable flexural tensile strength values.

Curing

Curing by different combinations of spraying water and sealing the walls following construction significantly improves the flexural tensile strength of masonry. Copeland and Saxer [6] stated that covering specimens with plastic sheeting for seven days and wetting them once per day for the first four days could significantly increase the flexural tensile strength of specimens. The NCMA [9] reports that spraying the specimens a day after construction and sealing them in plastic bags, or curing specimens in the laboratory environment while spraying them at seven and fourteen days following construction increases the flexural tensile strength by a factor of three.

Table 1 shows that the variability of relative humidity of the environment in which walls were cured is quite large, while the curing temperature remained fairly consistent. This indicates that walls were exposed to different amounts of moisture during curing. Results from tests with different levels of relative humidity may therefore not be directly comparable.

Agreement between full-size walls and prisms

Table 1 shows that no reliable relationship between the flexural strength of full-scale walls and prisms has been established. CSA S304-14 [1] reflects on this by forbidding results from bond wrench tests to directly represent the flexural strength of full-scale walls. Therefore, until such relationship is established, the test database should only include results for full-scale walls.

Retempering of mortar

On-site construction includes retempering of mortar to improve workability. The majority of experimental programs reported include retempering with few exceptions as outlined in Table 1. The NCMA [9] permitted retempering once. It is unclear how much of an effect the difference in retempering practices between the reported experimental programs and typical construction practice has on the flexural strength of unreinforced masonry walls. Copeland and Saxer [6] reported results of the sole experimental program that included the effects of retempering. Results were inconclusive as retempering impacted the results of one mortar mix by reducing the flexural tensile strength of specimens by 42%, while the flexural strength of prisms made using the other two mortar mixes were insensitive to retempering. These results should therefore be used with caution as the limits on retempering as used in research are incompatible with typical construction practice.

Support conditions

Support conditions could have a significant impact on the flexural strength of unreinforced masonry walls. However, this impact has not been quantified. Realistic supports depicted in Figure 2(a) provide more fixity compared to idealized supports shown in Figure 2(b). The majority of previous experimental programs follow the setup depicted in Figure 2(b). Udey [10] examined the effect of support conditions on the strength of walls with a slenderness ratio equal to 16 and showed that realistic support conditions improve the flexural strength of unreinforced masonry walls by over 50%. This could at least partially account for the absence of failures in the past for walls designed according to the empirical provisions. Additional tests to confirm this relationship could contribute to a more accurate quantification of the flexural strength of unreinforced masonry walls. Until this additional research is completed, the current design assumption that walls built using typical construction practices behave as simply supported elements may potentially underestimate the flexural tensile strength of unreinforced masonry walls.

Load application

The loading configuration in past investigations included both uniform loading and point loads. The uniform load applied by the airbags has the advantage of better replicating the static component of wind load. Point loading is usually applied using two point loads one quarter of the height of the wall away from the supports and so produces a constant moment region in the mid-span wall section. The behavior of unreinforced masonry walls subjected to these two load application methods differ.

Kim and Bennett [12] report that Monk in 1954 experimentally determined a ratio of 1.97 for the flexural strength of walls subjected to uniform loading as compared to quarter point loading, while this ratio was found to be 1.99 for unpublished tests conducted by NCMA in 1967. These findings can be used to convert the results from past experimental programs which included quarter point loading to make them comparable with uniform loading.

Wall slenderness

A review of the literature did not reveal any past works that reported on investigations of the flexural strength of unreinforced masonry walls for slenderness ratios over which empirical and rational design outcomes differ. Unreinforced masonry walls in flexure constructed with 200 mm concrete blocks could be designed for heights up to 3.8 m using the empirical provisions included in CSA S304-14 [1]. Investigations reported in Table 1 included walls with spans that were 3 m or less exclusively, which corresponds to a maximum slenderness ratio of about 16 for concrete block units with 200 mm nominal width. An experimental investigation of the flexural strength of unreinforced masonry walls in pure flexure for slenderness ratios higher than those commonly studied in past studies is necessary for the comprehensive assessment of the rational and empirical design provisions.

Structural Reliability

Structural reliability is used to determine the probability of the violation of a limit state. For practical reasons, the reliability of a structure is often estimated by a combination of the review of historical records of the performance of structures and subjective estimations [13]. The reliability index is a tool to relate and compare levels of safety of various structural members. Target reliability indices are established by code committees to ensure acceptable levels of safety for different situations. To determine the reliability index of a current design, all variables involved in relating the load effects to resistance are expressed using their probability density functions [13]. The violation of a limit state depends on the probability of the occurrence of specific values for each variable.

Kim and Bennett [12] analyzed test results from several investigations of unreinforced masonry walls and reported reliability indices in the range of 2.34 to 3.85 for TMS 402-02, the Building Code for Masonry Structures [14] for unreinforced hollow masonry walls in flexure using types M or S Portland cement lime or masonry cement mortars. They also reported that Ellingwood et al. [15] found the reliability index for members subjected to wind loading to be approximately 2.5. The higher calculated reliability indices for masonry in pure flexure compared to members constructed of other materials justified increases in allowable flexural tension stresses in TMS 402 [2,12].

A reliability analysis has not been conducted for unreinforced masonry walls in flexure in Canada [2]. A reliability index of around 2.5 is justified when wind is the principal load [12,15]. Ellingwood et al. [15] states this rather low value of reliability index may be attributed to the mitigating effects of the redistribution of forces among members which are difficult to quantify. However, CSA S408-11, Guidelines for the Development of Limit States Design Standards, [16] requires a reliability index of 4 for structures in the normal importance category which fail in a brittle manner.

Performing a reliability analysis provides information about the safety level of rational design provisions for unreinforced concrete block masonry in flexure in Canada and could be done using the test database reported in Table 2. Determination of the safety level of the rational design method could contribute to the reconciliation of disagreements in design outcomes from the rational and empirical design methods.

The discussion above sheds light on the shortcomings and strengths of the current state of knowledge as applied to the behavior of unreinforced masonry walls under flexure and the resolving of the problem at hand. Accordingly, a research program is underway at the University of Saskatchewan to address some of the shortcomings discussed above. In particular, the following factors are being investigated:

- The effects of slenderness and support conditions on the flexural strength of unreinforced masonry walls including both realistic and ideal supports for walls and slenderness ratios ranging from 16 to 20.
- Determination of the safety level of unreinforced masonry walls in flexure for CSA S304-14 [1] rational provisions incorporating results from past experiments.

CONCLUSION

Unreinforced, vertically-spanning, non-loadbearing exterior concrete block walls subjected to wind load may be designed by either the empirical or rational provisions included in CSA S304-

14. Results obtained from the two design methods disagree in some cases. Understanding the parameters affecting the flexural tensile strength of unreinforced masonry walls could help eliminate such disagreements. Some of such parameters include: mortar type, the size of the specimens tested, method of load application, retempering of mortar, method of curing, support conditions, and slenderness ratio. The following conclusions were drawn:

- The majority of previous tests were performed using Portland cement mortar while the use of this mortar is currently limited in Canada. The database of flexural tests on unreinforced masonry walls using mortar cement mortar and masonry cement mortar should be expanded.
- A reliable relationship has not been established for the flexural tensile strength as reported using full-scale masonry walls versus prisms. Until such a relationship is determined, tensile strength results obtained from prism tests are recommended to be excluded from the test database.
- Some of the previous experiments had limits on the retempering of mortar, even though retempering is allowed on-site. Results from these tests should be used with caution since the effect of retempering of mortar on its flexural strength has not been determined.
- Curing has been shown to have significant impact on the flexural tensile strength of masonry. Previous experiments used different curing methods which makes direct comparability difficult. The comparison of curing conditions in a laboratory environment as compared to those achieved on-site must be considered.
- The database of flexural tensile strengths based on past tests should be made consistent considering the effects of load application method. It has been established that the flexural resistance obtained from tests conducted using quarter point loading should be multiplied by a factor of 1.97 to match those conducted using uniform loading.
- Ideal support conditions were used in most of the past experiments. The difference in the degree of fixity provided by the different types of supports and so the resulting flexural tensile strength determined at mid-height of the wall requires further study.
- The majority of previous experiments investigated walls with spans less than three meters. Rational design provisions produce results that are more conservative compared to empirical provisions for walls with spans larger than those studied in past experiments. Therefore, the flexural tensile strength of unreinforced masonry for walls with larger spans should be investigated.
- A reliability analysis has not been done for masonry in flexure in Canada. It is important for a reliability analysis to be conducted to determine the safety levels of the rational design provisions of CSA S304-14.
- A research program at the University of Saskatchewan is underway to investigate the effects of support conditions and slenderness for unreinforced masonry walls in flexure. Additionally, a reliability analysis will be performed as part of this program to determine the level of safety of such walls.

ACKNOWLEDGEMENTS

The authors of this paper are thankful for the technical and financial support provided by the Canada Masonry Design Centre, the Saskatchewan Masonry Institute, and the University of Saskatchewan. The first author is grateful for scholarships provided by the National Science and Engineering Research Council of Canada, the American Concrete Institute, and the University of Saskatchewan.

REFERENCES

[1] CSA. (2014). "S304-14-Design of Masonry Structures." Mississauga, ON, Canada: Canadian Standards Association.

[2] Laird, D.A. (2013). "The empirical dilemma." *Proc., 12th Canadian Masonry Symposium*, Vancouver, BC, Canada.

[3] Richart, F. E. (1932). "The structural performance of concrete masonry walls." *J. American Concrete Institute*, 28(2), 363-385.

[4] Hedstrom, R. O. (1961) "Load test of patterned concrete masonry walls." *Journal of the American Concrete Institute,* 57(4), 1265-1286 Detroit, MI.

[5] Fishburn, C. C. (1961). "Effect of mortar properties on strength of masonry." Washington D.C: U.S Dept. of Commerce, National Bureau of Standards.

[6] Copeland, R. E., and Saxer, E. L. (1964). "Tests of structural bond of masonry mortars to concrete block." *J. American Concrete Institute,* 61(11), 1411-1452.

[7] Drysdale, R. G., and Essawy, A. S. (1988). "Out-of-plane bending of concrete block walls." J. Struct. Eng., 114(1), 121-133.

[8] Matthys, J. (1990). "Concrete masonry flexural bond strength prisms versus wall tests." *Proc., 5th North American Masonry Conference*, Vol. 2. Urbana-Champaign: University of Illinois, 677-685.

[9] NCMA (1994). "Research evaluation of the flexural tensile strength of concrete masonry." Proj. No. 93-172, Order No. MR 10. Herndon, Virginia, USA. Retrieved on Nov. 20, 2016 from http://ncma-br.org/pdfs/masterlibrary/MR10.pdf.

[10] Udey, A. (2014). "Realistic wind loads on unreinforced masonry walls." (Master's Thesis.) University of Saskatchewan, Saskatoon, SK, Canada.

[11] ASTM. (2015) "Standard Test Methods of Conducting Strength Tests of Panels for Building Construction" (E72-15). West Conshohocken, PA: ASTM International.

[12] Kim, Y. S., and Bennett, M. (2002). "Flexural Tension in Unreinforced Masonry: Evaluation of Current Specifications." *J. The Masonry Society*, 20(1), 23-30.

[13] Melchers, R. E. (1987). *Structural Reliability Analysis and Prediction,* Ellis Horwood Limited, Southampton, Great Britain.

[14] TMS. (2002) "Building Code Requirements for Masonry Structures" (ACI 530-02/ASCE 5-02/TMS 402-02; ACI 530.1-02/ ASCE 6-02/TMS 602-02). Masonry Standards Joint Committee, Longmont, CO, USA.

[15] Ellingwood, B., Galambos, T. V., MacGregor, J. G., Cornell, C. A. (1980). "Development of a probability based load criterion for American National Standards A58: Building code requirements for minimum design loads in buildings and other structures." Vol. 13. US Department of Commerce, National Bureau of Standards. Washington D.C., USA

[16] CSA. (2011). "Guidelines for the development of limit states design standards (S408-11)." Mississauga, ON. Canada: Canadian Standards Association.