14TH CANADIAN MASONRY SYMPOSIUM
MONTREAL, CANADA
MAY 16TH – MAY 20TH, 2021

INFLUENCE OF COMPRESSIVE STRESS ORIENTATION ON THE FLEXURAL STRENGTH OF CONCRETE BLOCK MASONRY

Vachon, Thomas C. J.[1] and Feldman, Lisa R.[2]

ABSTRACT

The χ-factors as included in Clauses 10.2.6 and 11.2.1.6 of CSA S304-14 – *Design of Masonry Structures* account for anisotropy and grout continuation in concrete block masonry construction by reducing the resulting capacity of the flexural compression strength for members where loading is oriented parallel to bed joints. These factors were incorporated based on the results of experimental research programs. However, more recent research suggests that these χ-factors are overly conservative and so may lead to uneconomical designs. A test database was therefore assembled based upon experimental results as reported in the literature, and, of the 211 total specimens as included, was reduced to include data from 62 reinforced concrete block masonry members representing cases in which block webs interrupted the depth of the compression stress block and loading was applied normal to the head face. A resulting analysis showed that the χ-factor of 0.5 as currently included in CSA S304-14 for such scenarios may be more reasonably increased to 0.7 while still ensuring that adequate structural safety is provided. Additionally, this review highlighted the fact that past experimental programs neglected to include direct comparisons of geometrically similar wall and beam specimens with equivalent arrangements of reinforcing steel and so provided the motivation for an updated experimental program as is currently underway at the University of Saskatchewan.

KEYWORDS: *reinforced concrete block masonry, flexural compressive strength, direction of loading, web interruption, χ-factor*

[1] M.Sc. Student, Department of Civil, Geological, & Environmental Engineering, University of Saskatchewan, 57 Campus Drive, Saskatoon, SK, Canada, thomas.vachon@usask.ca

[2] Professor and Director, Saskatchewan Center for Masonry Design, Department of Civil, Geological, & Environmental Engineering, University of Saskatchewan, 57 Campus Drive, Saskatoon, SK, Canada, lisa.feldman@usask.ca

INTRODUCTION AND MOTIVATION

Reinforced concrete block masonry members subject to flexure exhibit similar behaviour and failure modes as reinforced concrete beams [1]. However, in contrast to concrete, the compressive strength of masonry assemblies is anisotropic with masonry assemblies loaded parallel to the bed joint found to be weaker than those loaded normal to the bed joint [2,3,4].

Figure 1 shows the difference in assembly configuration for both orthogonal loading directions. Figure 1(b) shows the vertical mortar joint extending the entire height of the masonry assemblage for the case of loading applied parallel to the bed joint and so may cause a reduction in strength due to the resulting preferential shearing plane [2,5]. Grouting the masonry assemblies reduces anisotropy because the grout reinforces the mortar joints [5]; however, horizontal grout flow between adjacent blocks is restricted by the presence of the block webs and causes voids to form in the head space between blocks. The voids cause the compressive stress to concentrate in the face shells of the block and so reduce the capacity of the masonry assembly [2,6]. These voids reduce the effective area of masonry assemblies when load is applied parallel to the bed joint.

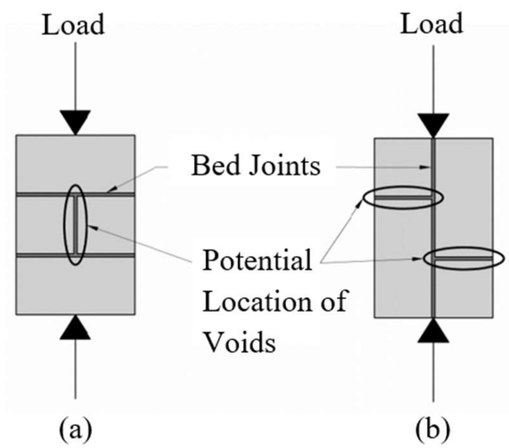

Figure 1: The effect of mortar joint orientation on masonry compressive strength for load applied: (a) perpendicular to the bed joint, and (b) parallel to the bed joint.

The flexural compressive strength of masonry beams is reduced when compared to walls because of the direction of internal compressive stress relative to the bed joint [7]. The internal flexural compressive stress of masonry walls acts perpendicular to the bed joint. In contrast, masonry beams spanning openings are typically constructed such that the internal compressive stress acts parallel to the bed joint; reducing both compressive strength and flexural capacity. As a result, Clauses 10.2.6 and 11.2.1.6 in CSA S304-14 [8] incorporate χ-factors to reduce the magnitude of the masonry compressive stress for the rectangular stress block to account for loading applied parallel to the bed joint.

Results of recent research programs [9-13] contradict earlier findings [2-4] regarding anisotropy in masonry. Some results indicate there is a potential increase in strength when load in masonry assemblies is applied parallel to the bed joint [9-12]. A statistical analysis showed; however, that the flexural capacity of concrete block masonry members is better approximated when the χ-factors are removed [13], and so suggest their incorporation in CSA S304-04 [14] resulted in overly conservative designs [9-13]. Despite these results, the χ-factors remained unchanged in the 2014 code edition and were reaffirmed in 2019, potentially due to a lack of research directly comparing the directional effect of compressive stress within flexural masonry specimens. A revised experimental research program is therefore underway at the University of Saskatchewan to re-evaluate the χ-factors for potential inclusion in a future edition of CSA S304.

INFLUENTIAL PARAMETERS ESTABLISHED FROM THE LITERATURE REVIEW

Early studies on anisotropy as related to masonry members [1-6] influenced the incorporation of the CSA S304-14 [8] χ-factors. The χ-factor is equal to 1 when the internal compressive strength acts normal to the bed joint. The χ-factor is equal to 0.7 when compressive stress acts perpendicular to the head joint and the compression stress is not interrupted by a web within the depth of the compression stress block (Figure 2(a)); otherwise, $\chi = 0.5$ to account for the potential development of voids in the headspace between blocks (Figure 2(b)) [8]. Selection of the appropriate χ-factor becomes problematic when beams are constructed using knockout blocks: the χ-factor can be taken as either 0.5 or 0.7 and depends on whether the resulting calculated depth of the compressive stress block is or is not interrupted by the remaining web height.

The χ-factor is therefore linked to the depth of the compressive stress block for calculating flexural resistance. The depth of the compression block, βc, is calculated as:

$$\beta c = \frac{A_s f_s}{0.85\, \chi\, f'_m\, b} \tag{1}$$

where A_s is the cross-sectional area of the reinforcing steel, f_s is the stress in the reinforcing steel, f'_m is the masonry assemblage strength as established normal to the bed joint, and b is the member width. An iterative procedure is often required because the χ-factor is both inversely proportional to and dependant on the calculated depth of the rectangular compressive stress block.

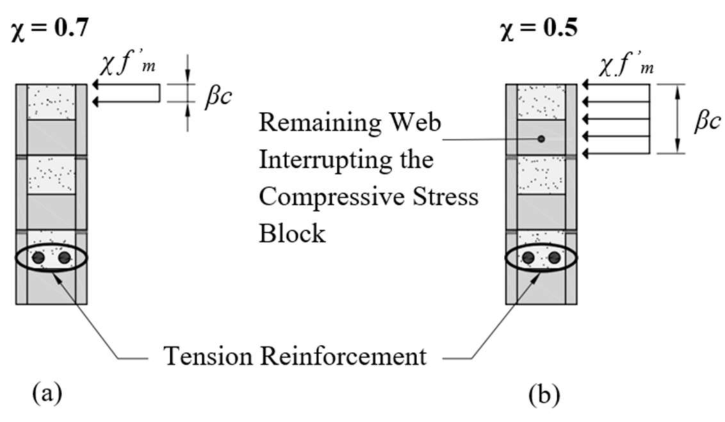

Figure 2: Effect of the χ-factor on the masonry compressive strength for cases when: (a) the depth of the rectangular compressive stress block is shallower than the knockout depth, and (b) the depth of the rectangular compressive stress block is deeper than the knockout depth.

The proportion of web intercepting the rectangular compressive stress block, defined as the intact web height over the total block height, proportionally influences the magnitude of the uniform compressive stress [10,11]. Results from beam tests showed a 7% reduction in compression strength between the cases of no web and full webs intercepting the compression stress [10]. In contrast, results from prism tests showed a larger reduction in compression strength of 19%. The strain gradient in beams is linear as opposed to the uniform strain found in prism and so may explain the smaller reduction in compression strength for the beam specimens [11]. The theoretical reduction related to the proportion of web intercepting the compression stress is characterized as the percent difference between a χ-factor of 0.7 to 0.5 and so is calculated as the difference between these values divided by 0.7, and so roughly 29%. Results of experimental studies suggest that χ-factors for stress applied parallel to the bed joint appear to be conservative because measured reductions are less than the theoretical reduction of 29% [10,11].

These past studies did not consider the percentage of web area intercepting the rectangular compressive stress block as a function of block size. Additionally, provisions used for the determination of flexural resistance as included in CSA S304-14 [8] do not account for a change in the χ-factor with block size. Voids between the head joints may; however, increase for larger block sizes with wider webs and similar face shell thickness. The impact of potential void formation on the selection of an appropriate χ-factor may be established from the relationship between the percentage of web area intercepting the depth of the rectangular stress block and the effective magnitude of the uniform compressive stress.

The χ-factor reduces the magnitude of the compressive strength associated with the rectangular stress block and so potentially changes the calculated failure mode of a flexural masonry member. Clause 11.2.2 of CSA S304-14 [8] limits the maximum allowable area of reinforcement masonry beams to ensure an under-reinforced, and so ductile, failure. The maximum allowable area of steel reinforcement for masonry members reduces proportionally with χ-factor and so limits the resulting flexural resistance.

Samy et. al [13] compared the test results of 173 under-reinforced masonry specimens as available in the literature with theoretical predictions calculated in accordance with CSA S304-14 [8]. Both the theoretical failure mode and the moment resistance were calculated and compared to those reported during testing. Data was analyzed based upon two criteria: (1) whether theoretical predictions of failure mode matched those observed during testing, and (2) how closely the theoretically calculated flexural resistance matched that recorded during testing. The theoretically predicted failure mode for 72 specimens, and so 42% of the database, was incorrectly predicted by code when the χ -factors, as specified by CSA S304-14 [8], were incorporated into the calculations. The resulting mean ratio of the experimentally reported to theoretically calculated flexural capacity was 1.21 with a coefficient of variation of 0.16, and so suggested that the flexural resistance as calculated in accordance with CSA S304-14 [8] provisions is somewhat conservative. A revised analysis was then preformed setting the χ-factors equal to unity and resulted in the incorrect prediction of failure mode for 33 specimens, and so 19% of those included in the test database.

The resulting mean value of the ratio of the experimentally reported to theoretically calculated flexural resistance was 1.13 with a coefficient of variation of 0.14. Improvements to both criteria were therefore realized resulting in the recommendation that the χ-factors be removed from CSA S304 provisions in future code editions.

A critical review of the test database revealed that 114 of the total 135 beams included were constructed with either single course lintels or deep lintels, respectively. Grouting in these specimens would therefore be continuous with no bed joint acting parallel to the direction of compressive stress. A χ-factor of 1 may therefore more accurately approximate their flexural resistance. Samy et al. [13] further limited the test database to specimens that failed in an under-reinforced manner. It would therefore appear that the analysis is of limited value in assessing the appropriateness of χ-factors of 0.5 and 0.7 as included in CSA S304-14 [8] and suggests that an updated analysis, using a revised test database, is warranted.

REFINEMENT OF TEST DATABASE BASED ON SPECIMEN GEOMETRY

A critical review of the geometry of all specimens as reported in the literature [1, 6, 10, 12, 15 – 22] (Figure 3) was necessary to establish the test database for use in the updated analysis. Figures 3(a) and (b) [1, 17-20] shows those specimens as included in Samy's [13] analysis and so are best represented by $\chi = 1$. Figures 3(c) and (d) further show examples of columns, and walls with width that exceeds a single block course, respectively. These sections also represent situations for which $\chi = 1$. Specimens with these geometries were therefore excluded from the updated test database.

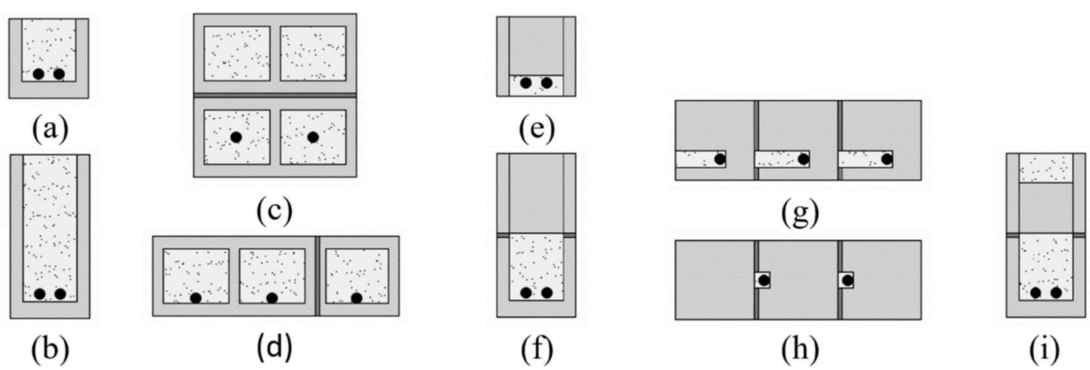

Figure 3: Cross-sections of the flexural specimens included in the database: (a) single course lintel beam, (b) deep lintel beam, (c) column specimen, (d) wall specimens, (e) single course with webs intact in compression zone, (f) multi-course beam with lintel in bottom course followed by standard blocks, (g) wide beams with narrow slits to accommodate reinforcing bars near the bottom of the specimen, (h) wide beams with narrow slits to accommodate reinforcement at mid-height of the specimen, and (i) multi-course beam constructed using knockout blocks.

Specimens shown in Figures 3(e) to (i) show cases for which χ-factors of either 0.5 or 0.7 would be assigned and so were included in the revised test database. Figure 3(e) [17,18] shows a single course standard block specimen with slits manually cut in the webs to accommodate the reinforcement. The webs were otherwise maintained and so intercepted the depth of the compression stress block resulting in assigning a $\chi = 0.5$. Figure 3(f) [1,6,10,20] shows a multiple course beam with webs intercepting the compression stress. Such a specimen would be assigned a χ-factor of 0.5. Figures 3(g) and (h) [16] show the cross-sections of three-course wide beams with narrow slits in the webs large enough to install the reinforcement. Sufficient web depth remains to intercept the depth of the compression stress block. These specimens were tested such that the compressive stress was applied parallel to the bed joint and are representative of a case for which $\chi = 0.5$. Figure 3(i) [10, 12, 21, 22] shows a multi-course beam constructed using knock-out blocks. The resulting χ-factor for this specimen is 0.5 because the depth of the compressive stress block extended beyond the region of continuous grout and was therefore intercepted by the remaining web. A total of 62 specimens in the test database had geometries matching those shown in Figures 3(e) to (i). These specimens were therefore included in the updated test database to reassess the appropriateness of $\chi = 0.5$ as currently included in CSA S304-14[8].

EVALUATION OF THE REVISED TEST DATABASE

The revised test database, including 62 of the total 211 concrete masonry specimens as identified in the literature [1, 6, 10, 12, 15 -22], was therefore assembled. This database was used to conduct an updated statistical analysis of experimentally reported and theoretically calculated failure modes and flexural resistances.

The masonry assemblage strength, f'_m, as required to theoretically calculate flexural resistance in accordance with CSA S304-14 [8], were not experimentally measured using tests of companion masonry prisms for a number of specimens in the test database. Instead, f'_m was theoretically estimated [15-20] as the average value of the measured block strength, f'_{bl}, as measured from compression tests of individual masonry units, and grout strength, f'_{gr}, as measured from either absorptive grout cubes [15-19] or non-absorptive grout cylinders [20]. This method is known to produce unconservative values [7]. Sarhat and Sherwood's 2013 [23] multiple linear regression model was instead used to calculate f'_m for the purposes of this analysis:

$$f'_m = 0.287f'_{bl} + 0.114f'_{mr} + 0.252f'_{gr} + 0.62 \qquad (3)$$

where f'_{mr} is the measured mortar strength as reported from mortar cubes tests.

The failure mode of each specimen was theoretically calculated according to CSA S304-14 [8] for all three values of χ included in Clause 10.2.6: 0.5, 0.7, and 1. Consideration of potential values of χ that are not currently included in CSA S304-14 [8] will be conducted once the test data from the experimental program underway at the University of Saskatchewan has been collected and analysed. Table 1 shows the theoretically calculated failure modes for each specimen, corresponding to each of the three χ-factors as evaluated, were then compared to the experimentally observed failure mode. Samy et al. [13] only analysed under-reinforced specimens and so the accuracy of the χ-factors were evaluated by comparing the proportion of specimens where the failure mode was conservatively estimated such that the theoretically calculated failure mode was over-reinforced (OR) when under-reinforced (UR) was observed during testing. However, it is unsafe to theoretically predict an under-reinforced failure when in fact an over-reinforced failure mode was identified during testing. Appropriate values of χ are therefore those that, when incorporated into provisions for the calculation of flexural resistance, allow for a correct or conservative prediction of failure mode in at least 90% of cases using a lower, one-sided, 95% confidence bound. The lower confidence bound was calculated using the simple asymptotic formula with continuity correction for proportions.

Table 1: Comparison of calculated and experimentally observed failure modes.

	$\chi = 1$	$\chi = 0.7$	$\chi = 0.5$
Successfully Estimated Failure Mode	91.9%	90.3%	71.0%
Estimated OR When UR Observed	3.2%	8.1%	29.0%
Estimated UR When OR Observed	4.8%	1.6%	0.0%
95% Lower Confidence Limit of Correctly or Conservatively Estimating Failure Mode	89.9%	94.9%	99.2%

Table 1 shows that failure modes were either correctly or conservatively predicted in 89.9, 94.9, and 99.2% of cases when values of χ = 1, 0.7, or 0.5 were incorporated in calculations, respectively. Ideally, the most appropriate χ-factor would be one that allows for the correct prediction of failure mode in at least 90% of cases. Table 1 shows that χ = 1 does not meet this requirement and so suggests that the conclusion resulting from the original analysis as conducted by Samy et al. [13] is not accurate, and likely results from the test database that formed the basis of this investigation. Results for both χ = 0.5 and 0.7 meet the criteria for prediction of failure mode; however, at 8.1%, the number of specimens predicted to fail in a more ductile manner than reported during testing (i.e. in an under- rather than over-reinforced manner) when a χ = 0.7 is included in calculations for flexural resistance is substantially less than the 23% when χ = 0.5 is included in calculations. A value of 0.7 for χ appears to provide the best fit.

The ratio of experimentally observed to theoretically predicted flexural resistance for all specimens calculated using all three values of the χ-factor as included in CSA S304-14 [8] were then evaluated statistically. The distribution of the data was not normal (p-value < 0.001 for each χ-factor) because: (1) while moment capacity increases with area of reinforcement, the rate of this increase decays with increasing steel area due to the incorporation of the χ-factor in flexural resistance calculations, and (2) the data consists of two populations given that the reinforcement in some specimens yielded prior to specimen failure whereas in other specimens it did not. A distribution test was therefore conducted and showed that the data is best approximated by transforming them to a normal distribution using the unbounded Johnson transformation for cases of χ = 0.7 and 1, and the bounded Johnson transformation when χ = 0.5. The resulting p-values for the Johnson transformations where 0.46, 0.41, and 0.37 for χ = 0.5, 0.7 and 1, respectively. The transformed data is normal and so allows the 95% lower single sided confidence bound to be calculated.

The value of the transformed 95% confidence bound is arbitrary and therefore shown in Table 2 along with the resulting mean in terms of the untransformed values. The untransformed values are the ratios of experimentally recorded moment capacity divided by the capacity calculated in accordance with CSA S304-14 [8] provisions. The resulting ratio should not exceed unity at the 95% lower one sided confidence bound to ensure that the prediction of flexural resistance is not over-estimated. The resulting 95% lower confidence bound for χ-factors equal to 1, 0.7, or 0.5 are 0.81, 0.95, and 0.97, respectively. None of these limits exceed unity, but those for $\chi = 0.7$ and 0.5 are reasonably close to one. These findings suggest that either $\chi = 0.5$ and 0.7 may be reasonable for use in CSA S304-14 [8] provisions.

Table 2: Mean and 95% lower confidence interval of experimental moment divided by the calculated moment.

	$\chi = 1$	$\chi = 0.7$	$\chi = 0.5$
Mean	1.17	1.31	1.62
95% Lower Confidence Limit	0.81	0.95	0.97

Results of this statistical analysis show that $\chi = 0.7$ most reasonably predicts both the failure mode and flexural resistance of the specimens within the test database. This further indicates that consideration be given to adjust the χ-factors in a future edition of CSA S304-14 [8] to account for anisotropy of masonry assembly compressive stress.

EXPERIMENTAL PROGRAM AT THE UNIVERSITY OF SASKATCHEWAN

The χ-factors have yet to be quantified by comparing tests of beams and walls constructed with similar geometries and levels of reinforcement. The experimental program currently underway at the University of Saskatchewan was therefore established with this purpose.

The experimental program includes a total of 24 specimens, and consists of 12 pairs of beams and walls. Parameters evaluated include the orientation of flexural compressive stress (parallel and perpendicular to the bed joint for beams and walls, respectively), three block sizes (200mm, 250mm, and 300mm), and four reinforcing ratios selected based upon theoretically calculated predictions of compressive stress block depths of 165mm, 210mm, 240mm, and 260mm. The amount and arrangement of the longitudinal reinforcement in each beam and wall pair were selected to compare: (1) the potential effect of orthogonal compressive stress on observed failure mode and, (2) the nature of the relationship between the rectangular stress block and the proportion of the web intercepting the compression stress block. The four reinforcing ratios for each otherwise similar specimen geometry were selected to compare the orthogonal compressive stress to the observed failure mode. Reinforcement ratios of particular interest for study were those that cause an over-reinforced failure to be theoretically predicted when $\chi = 0.5$, yet instead suggest an under-reinforced failure if calculated by the same methods but instead assuming $\chi = 1.0$.

Walls and beams were constructed with their longitudinal axes perpendicular and parallel to the lab floor, respectively, though all specimens were tested with their longitudinal axes parallel to the lab floor. This required the walls specimens to be lowered and rotated into position on the testing bed using straps and a lift. Figure 4 shows that all specimens were tested under four-point loading. Steel and concrete strain gauges (not shown in Figure 4) were affixed to the reinforcement and the side faces of the concrete blocks, respectively, with the ultimate goal of estimating the neutral axis location.

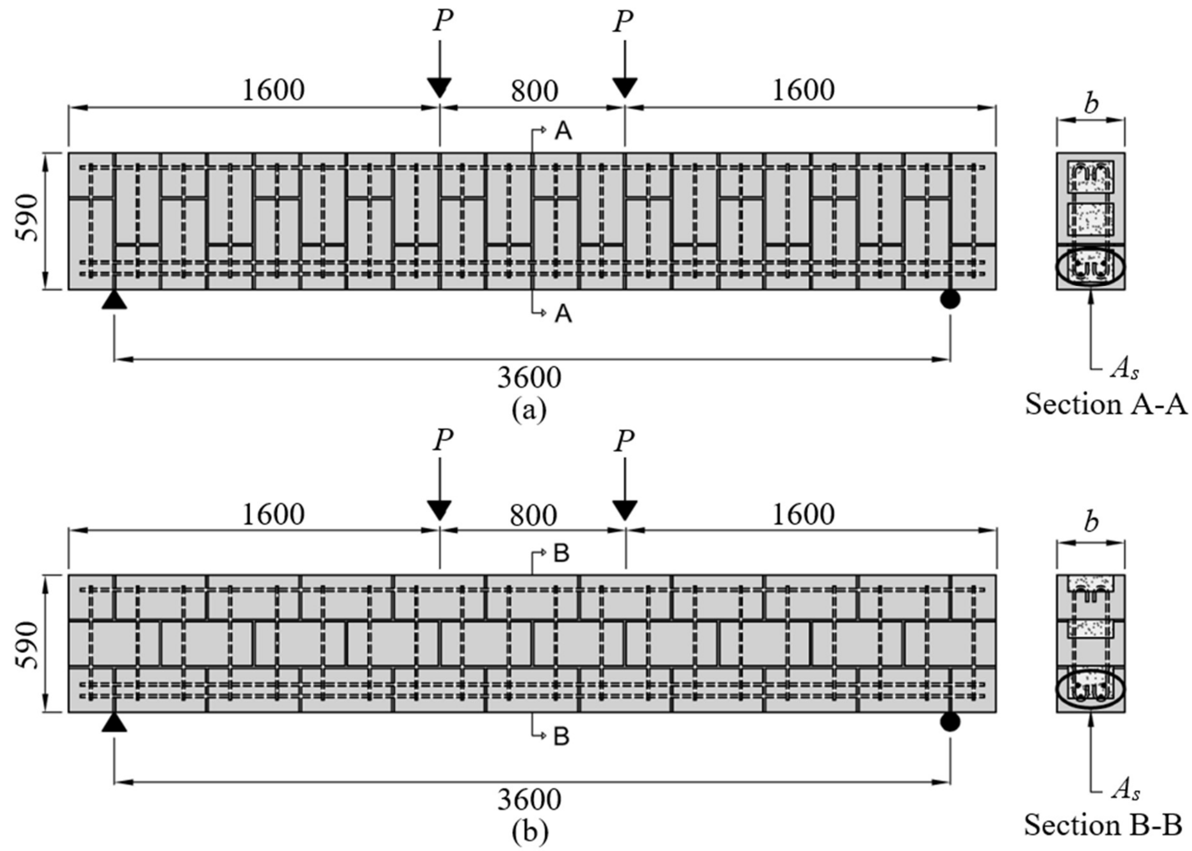

Figure 4: Proposed test setup for: (a) wall specimen, and (b) beam specimen.

The cross-sections as shown include similar arrangements of the reinforcement in each pair of specimens and is achieved by including knock-out blocks in the beams to accommodate the same cross-sectional area and effective depth to the reinforcing steel as in the corresponding wall. Stirrups were installed in each block cell of the beam specimens; however, the webs of the blocks in the walls obstruct the placement of the stirrups and so walls were also constructed using similar knockout blocks. The knockout blocks were created by manually cutting slits in the webs of standard blocks sourced from a single material batch. The flexural capacity and failure mode for each pair of specimens will be used to evaluate appropriate χ-factors for potential use in a future edition CSA S304.

SUMMARY AND CONCLUSIONS

The χ-factors as included in Clauses 10.2.6 and 11.2.1.6 of *CSA S304-14 – Design of Masonry Structures* account for anisotropy and grout continuation in concrete block masonry construction by reducing the resulting magnitude of the flexural compression stress for members when loading is oriented parallel to bed joints, and were incorporated based on the results of experimental research. However, more recent work suggests that these χ-factors are overly conservative and so may lead to uneconomical designs. This paper therefore provides a re-assessment of data available in the literature and describes an experimental program that is underway at the University of Saskatchewan with aim of resolving this issue.

The following conclusions were noted:

1. A normal distribution does not effectively capture test data for specimens in which the compressive stress block was intercepted by block webs. This results because the percentage reduction in moment capacity resulting from a χ-factor increases with increasing reinforcement ratio and because the data is, in fact, from two populations: (1) when specimen failure was initiated by yielding of the reinforcing steel, and (2) when specimen failure was initiating by crushing of the concrete prior to yielding of the reinforcing steel.
2. A statistical evaluation of 62 specimens reported in the available literature showed that a χ-factor equal to 0.7 was more appropriate than 0.5 for assessing the failure mode and flexural capacity of specimens in which block webs intercepted the depth of the compressive stress block and loading was applied normal to the head face.
3. An experimental investigation is currently underway at the University of Saskatchewan to further investigate the failure mode and resistance of pairs of beam and wall specimens with similar geometries and reinforcing steel arrangements. A total of 24 specimens are being constructed and tested under four-point loading.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge financial support as provided by the Natural Science and Engineering Research Council of Canada, the Canada Masonry Design Center, and the Canadian Concrete Masonry Producers Association. Scholarships support for the first author has also been provided by the American Concrete Institute, and the University of Saskatchewan.

REFERENCES

[1] Suter, G. T., and G. A. Fenton. (1986). "Flexural Capacity of Reinforced Masonry Members." *Journal of the American Concrete Institute* 83 (1): 127-136.

[2] Lee, R., J. Longworth, and J. Warwaruk. (1981). “Behavior of Restrained Masonry Beams.” Structural Engineering Report No 99, Department of Civil Engineering, University of Alberta. Edmonton, AB.

[3] Wong, H. E., and R. G. Drysdale. (1985). "Compression Characteristics of Concrete Block Masonry Prisms." Masonry: Research, Application and Problems, ASTM STP 871, J.C. Grogan and J.T. Conway, eds.. *American Society for Testing and Materials*, Philadelphia, 167-177.

[4] Khalaf, F. M. (1997). "Blockwork Masonry Compressed in Two Orthogonal Directions." *ASCE Journal of Structural Engineering*, 123 (5): 591-596.

[5] Hamid, A. A., and R. G. Drysdale. 1980. "Concrete Masonry Under Combined Shear and Compression Along the Mortar Joints." *Journal of the American Concrete Institute* 77 (5): 314-320.

[6] Khalaf, F. M., J. I. Glanville, and M. EI-Shahawi. (1983). "A Study of Flexure in Reinforced Masonry Beams." Concrete International, Design and Construction 5 (6): 46-53.
[7] Drysdale, R. G. and Hamid, A. A. (2005). *Masonry Structures Behaviour and Design*, Canada Masonry Design Centre, Mississauga, ON, Canada.
[8] Canadian Standards Association. (2014). *CAN/CSA S304.1-14: Design of Masonry Structures*. Mississauga, Ontario.
[9] Kaaki, T. (2013). "Behaviour and Strength of Masonry Prisms Loaded in Compression." M.Sc. thesis, Dalhousie University. Halifax, NS.
[10] Ring, T., S. Das, and D. Stubbs. (2012). "Compressive Strength of Concrete Masonry Beams." *ACI Structural Journal* 109 (3): 369-376.
[11] Ring, T. (2009). "The Influence of Horizontal Grout Continuity on the Compressive Strength [sic] of Concrete Block Masonry." M.Sc. thesis, Department of Civil and Environmental Engineering, University of Windsor. Windsor, ON
[12] Zohrehheydariha, J., S. Das, and B. Banting. (2017). "Behaviour of Stack Pattern Masonry Beams." *Proceedings of the 13th Canadian Masonry Symposium*. Halifax, NS. Canada Masonry Design Center.
[13] Samy, B. A., S. R. Sarhat, and E. G. Sherwood. (2012). "Comparing Flexural Capacity of Reinforced Masonry Beams Using Different Codes." *Proceedings of the 12th North American Society Masonry Conference.* Denver, Colorado.
[14] Canadian Standards Association. (2004). *CAN/CSA S304.1-04: Design of Masonry Structures*. Mississauga, Ontario.
[15] Roberts, J. J. (1975). "The Behaviour of Vertically Reinforced Concrete Blockwork Subject to Lateral Loading." *Cement and Concrete Association*. Technical Report 506.
[16] Roberts, J. J. (1980). "Further Work on the Behaviour of Vertically Reinforced Concrete Blockwork Subject to Lateral Loading." *Cement and Concrete Association*. Technical Report 531
[17] Rathbone, A. J. (1980a). "Behaviour of Reinforced Concrete Blockwork Beams." *Cement and Concrete Association*. Technical Report 540.
[18] Rathbone, A. J. (1980b). "The Behaviour of Single-Course Reinforced Concrete Blockwork Beams." *Proceedings of the Second Canadian Masonry Symposium*, Ottawa, June, 259-274
[19] Rathbone, A. J. (1985). "Further Work on the Behaviour of Reinforced Concrete Blockwork Subject to Lateral Loading." *Cement and Concrete Association*. Technical Report 558.
[20] Edgehill, R. L. V. (1971). "An Investigation of Block-Formed Structural Members." M.Sc. thesis, University of Manitoba, Winnipeg, MB.
[21] Heydraiha, J. Z., S. Das, and B. Banting. (2017). "Effect of Grout Strength and Block Size on the Performance of Masonry Beam" *Construction and Building Materials* 157 (2017): 685-693.
[22] Neis, V. V. and R. J. Loeffler. (1983). "Results of Ultimate Flexural and Shear Tests on Reinforced Masonry Beams." *Proceedings of the 3rd Canadian Masonry Symposium.*, Edmonton, AB. Canada Masonry Design Center. 13/1-13/16
[23] Sharhat, S.R. and E.G. Sherwood. (2013). "The Prediction of Compressive Strength of Grouted Hollow Concrete Block Masonry Based on the Contributions of its Individual Components." *Proceedings of the 12th Canadian Masonry Symposium.* Vancouver, BC. Canada Masonry Design Center.