13th International Brick and
Block Masonry Conference
Amsterdam, July 4-7, 2004

CAPACITY OF SINGLE AND DOUBLE WYTHE UNREINFORCED CONCRETE BLOCK WALLS

Cameron Baker[1], Jim K. Marr[2], R.G. Drysdale[3]

Abstract

The cavity wall, or double-wythe wall, is a common form of masonry construction. Compared to single wythe construction, there intuitively should be an increased stiffness of the wall due to the additional wythe. Treatment of the structural design of the cavity wall differs between countries and, in Canada, between engineered and empirical design. It was of interest to investigate whether or not the capacity of a wall is significantly increased due to the additional wythe.

An experimental program was conducted to determine whether a cavity wall has additional capacity over a single-wythe wall. Concrete block walls 4 m in height and 0.8 m in length were constructed with standard 20 cm hollow units. A total of 21 unreinforced hollow concrete block walls were used to construct 7 single-wythe walls and 7 cavity walls. Walls were tested with equal end eccentricities of either t/12, t/6, t/4, t/3 or 5t/12 and the results compared to code provisions.

Generally there was little structural benefit of the additional wythe.

Keywords

Bending, cavity walls, compression, concrete block, double-wythe walls, eccentric loading, masonry, unreinforced.

1 Introduction

Cavity walls are a standard form of construction throughout the world, consisting of two wythes of masonry tied together and separated by an air space. In Canada, a typical cavity wall consists of a clay brick outer wythe tied to a concrete block inner wythe. When no sharing of lateral and/or gravity load is anticipated the block wythe (the backup wall) is designed to resist both loads and the brick wythe is referred to as veneer. The addition of flashing across the air space at the bottom of the wall, weep holes and air vents give the familiar open rain screen system. The well-documented building science benefit of reduced moisture transportation through the wall is the reason for creating the air space (cavity) between the wythes.

[1] Current M.A.Sc. Student, McMaster University, Hamilton, ON, Canada
[2] Former M. Eng. Student, McMaster University, Hamilton, ON, Canada
[3] Professor of Civil Engineering, Martini, Mascarin and George Chair in Masonry Design, Director of Centre for Effective Design of Structures, McMaster University, Hamilton, ON, Canada

Because sharing of gravity load is very difficult to achieve in practice, loads from floors and roof are usually applied to the block wythe as shown in Figure 1a. Also, as shown, the exterior wythe often does not have the same top and bottom support conditions. From structural mechanics, the use of ties to connect the two wythes ensures that lateral wind or seismic loads will be shared between the two wythes in a manner depending on the relative stiffnesses of the wythes, support conditions, and stiffness and spacing of the ties. In Canada, the main difference between what is known as a cavity wall versus a veneer wall is whether the design is based on sharing the lateral load or not. For the construction shown in Figure 1b, the only way to know for sure whether it is intended to be a cavity wall or a veneer wall is to review the designer's calculations.

The 1965 Edition of the National Building Code of Canada contained the first real provisions for engineering design of masonry and it was in the "detailed structural analysis" section for loadbearing masonry that the "two-thirds Rule" first appeared as

"For cavity walls loaded on both wythes the effective thickness shall be determined from the following formula:

$$T = \tfrac{2}{3}\left(T_0 - W_c\right)$$

where

$T_0 =$ overall thickness of wall, including width of cavity

$W_c =$ width of cavity"

For cavity walls loaded on one wythe only, the effective thickness was taken as the thickness of the loaded wythe. Over the intervening codes, this '2/3 rule' migrated to the empirical design part of the masonry code and was changed to:

"The effective thickness of a cavity wall used to determine the height or length to thickness ratio shall be taken as two thirds of the sum of the thickness of the wythes, but not less than the thickness of either wythe. The… ."

Currently, in engineered design, each wythe of a cavity wall is designed independently for its share of the applied axial load and/or lateral load. Any inherent benefit of lateral stiffening due to coupling of the walls is neglected. In empirical design in Canada (CAN-CSA S304.1-94), and in other parts of the world, the 'two-thirds rule' is used, where the effective thickness, t_e, of the wall used in slenderness considerations is taken as $\tfrac{2}{3}\left(t_1 + t_2\right)$, where t_1 and t_2 represent the actual thickness of each wythe, but t_e need not be taken as less than the larger of t_1 or t_2.

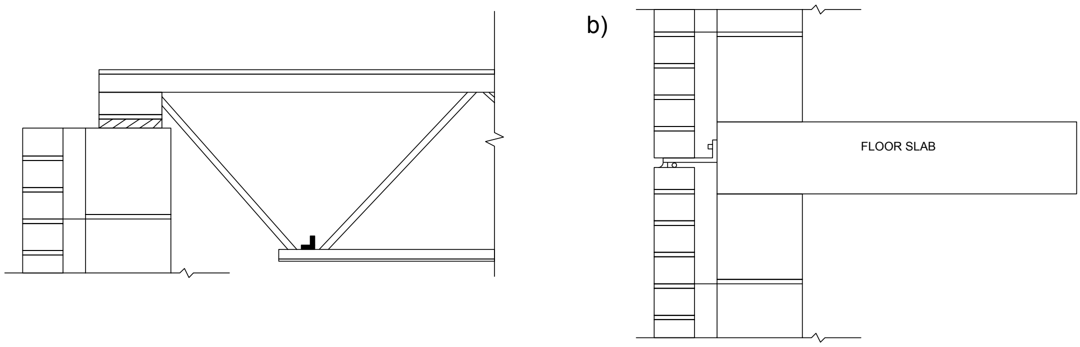

Figure 1 Detail of slab and wall support conditions.

These inconsistencies in design treatment prompted an investigation into the capacity of these double wythe walls.

2 Previous Research

Most of the previous cavity wall research dealt with the response to lateral loading and the effects of ties to couple the separate wythes. It is generally accepted that two equally stiff wythes of a cavity wall will share the bending moment caused by out-of-plane loads equally. In fact tests have shown that the out-of-plane capacity of a cavity wall is slightly higher than simply twice that of a single wythe (West, 1982). This indicates that there is some coupling action between the two wythes.

Research into the axial load capacity is more limited. Brick single wythe and cavity walls were tested under partially fixed conditions to investigate the influence of wall ties on the axial load capacity (Fisher, 1970). Little or no additional capacity was found for the cavity wall over the single wythe.

James (1972) investigated the axial behaviour of storey-height brick single and cavity walls. Both wythes of the cavity walls were loaded in this investigation, with results showing no difference in axial capacity between the two wall types.

Kumar and de Vekey (1989) axially tested single wythe and cavity walls consisting of AAC units as the loaded wythe and ordinary quality Fletton clay brick as the unloaded wythe. For concentrically loaded walls, the mode of failure for each wall type was similar. For walls tested with an eccentricity of ¼ of the wall thickness, the mode of failure changed slightly but the presence of the unloaded brick wythe did not significantly increase the vertical load capacity.

Davey and Thomas (1950) tested 113 mm brickwork wythes. The second wythe of the cavity wall was constructed as an unloaded wythe. The walls were built to a height of 2.74 m, giving a slenderness ratio of 24. Loading was applied eccentrically at 2/9 of the wall thickness. The single wythe and cavity walls were found to have similar capacities.

The Structural Clay Products Institute (1969) tested brick single wythe and cavity walls with a wythe slenderness ratio of 37.5 and loaded eccentrically at 1/6 of the wythe thickness at the top. Differences in capacities were found to be only about 3% which is negligible considering scatter of results for 3 test repetitions.

Laird (1983) carried out an evaluation of the empirical design method versus the engineered design method, based on the 1978 Canadian code. After clarifying errors in the empirical method, Laird questioned whether the engineered design method was correct because it disregarded any lateral stiffening effect that the unloaded wythe contributed to the cavity wall. It was suggested that the slenderness effect should be reduced.

3 Experimental Program

3.1 Material Properties

It is noted here that the walls were constructed in two series where some block used in the second series were new but of the same type and dimensions as those initially used. The walls built with these new block~~s~~ were only used for the nonloadbearing wythe of the cavity walls.

The standard 20 cm hollow concrete stretcher blocks had an average mass of approximately 16.4 kg. Based on ten compression tests of hard capped block, the average failure load of 855.7 kN results in a block strength of 20.6 MPa with 7.8% coefficient of variation.

Five four-block high prisms were tested with Hydrostone capping and 75 mm thick end plates to determine compressive strength. Four-block high prisms were used to avoid the confining effect of the end plates and provide a truer measure of compressive strength of the masonry in the wall. Pin-pin end conditions were used and the failure mode was consistently vertical splitting through the webs of the face shell mortared blocks. The mean compressive strength, f'_m, for the prisms made from the

first series of blocks was 16.96 MPa, whereas for the second set of blocks it was 13.83 MPa. The corresponding elastic moduli were found to be 12 590 MPa and 12 970 MPa based on the secant method (ASTM, E447-92b).

The bond wrench test (CSA 369.1) was used to test 8 five-block high prisms to determine the flexural tensile bond strength of the masonry. Four prisms were constructed from the first set of blocks used for the loadbearing walls and were found to have average tensile bond strength of 0.302 MPa with a COV of 21.3%. Four prisms were similarly constructed from the second set of blocks only used in several of the nonloadbearing walls, and were found to have average tensile bond strength of 0.292 with a COV of 25.8%. For the variability measured, these are essentially identical results so that the combined tensile bond strength is 0.297 MPa.

Type S, Portland cement: lime: sand mortar was mixed with volume proportions of 1:½:4 to give an average flow value of 127.5%. The average mortar cube strength was 14.2 MPa for all of the walls.

The wall ties were made of 200 mm long 4.76 mm diameter steel Z-ties with one end cut off. Ties were placed in each wall, and then cavity walls were formed by placing two wythes together with a 50 mm cavity and brazing the overlapping ends of the ties together over a length of 30 mm. Tensile tests were conducted for both individual ties and brazed ties, resulting in average failure stresses of 592 MPa and 416 MPa respectively (COV both <2%). It is of noted that failure never occurred in the brazed connections, and the lower strength of the brazed ties was attributed to bending moment developed by the eccentricity of the axial load.

3.2 Wall Testing

A total of 21 wythes 4 m high by 0.8 m long were constructed by an experienced mason. The constant height was achieved with 20 cm block. The actual height to thickness slenderness is 4.0/0.19=21.1 but in the original height to thickness limit of 20 for empirical design, the nominal wall thickness of 0.20 m was used. This maximum slenderness was investigated because it represents the most critical condition. Only the eccentricity of load varied between tests and pin ends with symmetric eccentricity provided the most critical of possible slenderness effects.

3.2.1 Test Apparatus

The test frame consisted of two W360x147 steel columns prestressed to the laboratory floor by four 63 mm diameter bolts. A W310x107 steel beam was connected to the columns by M24 bolts and supported a 900 kN capacity hydraulic jack and load cell used to apply axial compression load.

The bottom pin-end roller system for the loaded wythe consisted of two 190x1200x38 mm steel plates with a 13 mm deep cylindrical groove machined at a 20 mm radius to receive a 38 mm diameter steel roller. The roller system was similar for the unloaded wythe except the plates were only 75 mm wide as opposed to 190 mm. This was acceptable as bearing was not critical for the unloaded wythe. Twenty millimetre plates were tack welded to the top plate of the roller system for both the loaded and unloaded wythes to provide the intended eccentricity at the bottom.

To distribute load onto the loaded wythe, a 190x800x20 mm capping plate was used in conjunction with a 75 mm wide by 38 mm thick roller plate with a cylindrical groove machined to receive a 50 mm diameter steel roller bar. The roller plate was tack welded to the capping plates at the intended eccentricity. A W200x59 beam with a 50 mm bar welded to its underside was used to distribute load along the wall length and provide a hinged top end condition. A roller on top of the load distributing beam was perpendicular to the top roller and prevented in-plane rotational restraint or induced in-plane bending.

3.2.2 Wall Construction

The 21 wythes formed 7 single wythe specimens and 7 cavity wall specimens. Three batches of mortar were used to construct each wall. Small batches were used to maintain a consistent level of workability without retempering the mortar. Mortar joints were tooled on the outside faces of the single wythe walls, but not on the interior of the cavity walls, to reflect industry practice. The 4.76 mm diameter steel ties spanning the 50 mm air space between the two wythes of the cavity walls were placed at 400 mm spacing both horizontally and vertically.

The unloaded wythe was made of the same size block as the loaded wythe and, as mentioned earlier, flexural tensile bond and modulus of elasticity values were virtually identical. This choice of similar wythes removes the material as a variable and allowed a more direct interpretation of the effects of the unloaded wythe in the cavity wall system.

To reduce the chance of premature bearing failure, the loadbearing wythes were built with solid units for the top and bottom courses.

Finally 12 mm diameter bars were loosely placed in the end cells to contain the masonry after failure.

3.2.3 Wall Test Set-up

Hydrostone was used to bond the bottom roller system to the lab floor, and to bond the bottom of the wall to the top and bottom plates. An exception to the use of Hydrostone occurred with the 5t/12 eccentricity walls, where high strength epoxy was used to distribute the load to the wall ends to avoid local crushing under the bearing plates. After placement in the test rig, the wall was vertically aligned using a threaded rod and nut system, and a small vertical load was placed on the wall for stability.

At the top of the loaded wythe, the load distribution beam was restrained laterally to prevent lateral deflection or rotation which would result in deflection of the top of the loaded wall. Two pin end threaded bars braced the top flange from one column but allowed unrestrained vertical movement. Two cables fitted with turn buckles for alignment adjustment braced the bottom of the spreader beam from the other columns and also permitted free vertical movement. Although small but measurable top deflections did occur, these were considered to be insignificant and have not been reported individually in this paper.

3.2.4 Instrumentation

Lateral deflections were measured during each test with either Linear Potentiometer Displacement Transducers (LPDTs) or dial gauges at 9 locations over the wall height. Strains were measured with a mechanical strain indicator using pairs of measurements on both sides of the walls. Also displacement readings across the cavity were taken to determine cavity behaviour.

Lateral sway was taken into account by measuring the small top deflection, and using a linear variation to zero at the bottom to determine the sway profile. This profile was then subtracted from the measured deflection values to attain the deflection relative to a line joining the ends of the wall.

4 Discussion of Results of Experimental Program

The results of the axial load tests are summarized in Table 1. Tests are identified as SW for single wythe and CW for cavity wall. Number identifiers only indicated the order of wall testing. The failure mode is also briefly described.

4.1 Initial Crookedness

The maximum additional eccentricity of axial load due to initial crookedness was 6 mm for specimen SW 11 which added 12% to the initial eccentricity for that wall. For most of the walls the crookedness tabulated in Table 1 caused additional bending

but, since this magnitude is in the range of acceptable construction tolerance, the effect has not been considered separately from the applied eccentricity. Crookedness was not measured for some of the walls tested and is recorded as not available.

Table 1 Wall Test Results

Wall	End Eccentricity (mm)	Initial Crookedness at Mid-height (mm)	Failure Load (kN)	Mid-height Deflection Near Failure (mm)	Initial Failure Mode and Location
SW10	$t/12$ = 16	+0.3	720	9	Compression Second Course
CW7	$t/12$ = 16	-2	727	10	Compression Top and Bottom 3rd Course
SW6	$t/6$ = 32	+1.5	695	N.A.	Compression at Mid-height
CW3	$t/6$ = 32	-2	690	19	Compression at Mid-height
SW11	$t/4$ = 48	+6	508	13.5	Compression at Mid-height
CW 8	$t/4$ = 48	+5	510	13.5	Compression at Mid-height
CW 4	$t/3$ = 63	+2	405	15	Flexure at Mid-height
SW 2	$t/3$ = 63	N.A.	400	N.A.	Compression at Mid-height
SW 7	$t/3$ = 63	+0.5	390	14.7	Flexure at Mid-height
CW1	$t/3$ = 63	-0.5	385	N.A.	Flexure at Mid-height
SW 8	$5t/12 = 79$	+4	105	2.5	Flexure at Mid-height
SW 9	$5t/12 = 79$	+2.5	138	4.5	Flexure at Mid-height
CW 5	$5t/12 = 79$	+4	165	9	Flexure at Mid-height
CW 6	$5t/12 = 79$	+2.5	210	N.A.	Flexure at Mid-height

4.2 Wall Deflections

For walls which had deflection measurement continued to near failure, the deflection at failure was extrapolated and is recorded in Table 1. This provides an indication of the $P \cdot \Delta$ effect for slenderness. It is noteworthy that the deflection behaviour of the single wythe wall and the loaded wythe of the cavity wall were very similar especially near failure.

4.3 Cracking of Unloaded Wythe of Cavity Walls

All of the unloaded wythes of the cavity walls cracked at loads far less than the recorded capacity for the loaded wythe. For walls with eccentricity of t/4 and greater, tensile strains near mid-height of the loaded wythe were observed similar to the

comparable single wythe wall. At eccentricities of t/3 and 5t/12, cracks were visible in these loaded wythes prior to failure.

4.4 Failure Mode and Location

The use of a grouted course at the top and bottom of preliminary tests proved to be an ineffective precaution against premature bearing failure due to the use of relatively thin bearing plates. Use of solid units in these courses prevented premature failure and, except for the walls with t/12 eccentricity, walls failed near mid-height in the region of maximum moment due to deflection. At the t/12 eccentricity, the load was much closer to concentric and moment effects were not as great so that failure was similar to prism compression tests and occurred near the ends.

4.5 Single Wythe and Cavity Wall Capacities

At the t/12 = 16 mm end eccentricity, the failure loads and final deflections for SW 10 and CW 7 were almost identical. Initially, the cavity wall was 60 % stiffer than the single wythe wall but this difference was negligible at failure.

At end eccentricity of t/6 = 32 mm, walls SW 6 and CW 3 failed at nearly the same load with compression failure at the mid-height on the compression side of the loaded wythes. The initial stiffness of the cavity wall (CW 3) was approximately double that of the single wythe wall (SW 6) but, again, this difference diminished with increased load. For these walls, the $P \cdot \Delta$ effect added about 60% to the primary moment.

The walls with end eccentricity of t/4 = 48 mm failed in an explosive compression failure mode at mid-height of the walls at essentially the same load. The wall deflections caused an additional 28% moment.

At the t/3 eccentricity, the pair of single wythe walls (SW 2 and SW 7) gave an average capacity of 395 kN with the mid-height failure being identified in one case as compression and the other as flexure. The cavity walls also gave an average capacity of 395 kN for flexural mid-height failures. The failures occurred at deflections of about 15 mm which adds about 23% to the moment due to the $P \cdot \Delta$ effect. The flexural description of failure means that as cracking occurred at mid-height, deflection increased rapidly and load decreased in a post-peak descending branch behaviour leading to complete collapse of the wall without local compression failure in the masonry.

For the walls loaded at 5t/12 = 79 mm end eccentricities, there was much larger scatter in the test results consistent with capacity basically controlled by flexural tensile bond strength. The maximum deflection prior to onset of failure was about 9 mm for the cavity walls but only 2 mm for the single wythe walls. For this eccentricity, the axial load is applied just outside of the centroid of the compression faceshell. Another way of looking at this is that at a total eccentricity of 95 mm, the internal resultant compression force would have to exist at the face of the block, leaving no area under compression. Although the scatter of results is large, the average 122 kN capacity for the single wythe walls is obviously much less than the 188 kN average for the cavity wall specimens. This indicates significant benefit of the unloaded wythe.

5 Conclusions

There are several possible explanations for the origin of the “two-thirds rule”. For two wythes of solid masonry with equal thicknesses t, the sum of the moment of inertia values (related to deflection or buckling) can be replaced with the equal moment of inertia of a single wall having a thickness of 0.63 times the sum of the two thicknesses, 2t. Alternatively, for stresses due to bending, the sum of the section moduli for the two wythes can be replaced with the section modulus of a single wythe solid wall with equivalent thickness of 0.71 times the sum of the thickness, 2t. A

compromise between these two section properties is to use 2/3 or 0.67 times the total thickness, 2t.

Tests (Marr, 1992) on very slender models of cavity walls with slenderness h/t=74 and loaded concentrically on one wythe showed that the elastic buckling load for the cavity wall model was approximately double that for the loaded wythe when the unloaded wythe was detached. Similarly, it is known that the flexural capacity of two equal wythes of masonry without axial load will be double that of a single wythe provided that they are supported in the same manner and connected together so that their deflections will be identical.

From Figure 2, which contains the results of the capacity data in Table 1, it is clear that for eccentricities of load up to t/3, there is no justification for making any allowance for stiffening by an unloaded wythe. The cavity walls were unable to carry greater axial load than the comparable single wythe walls. For larger primary bending moments, it also appears that the engineering approach of determining the independent capacity of each wall to resist its share of the load will provide the most appropriate solution. In some cases of lower axial compression load on the most highly loaded wythe, the ability to share bending prior to cracking of the less highly loaded wythe may produce larger capacities of the combined system compared to designing the loaded wythe to resist all of the bending.

For empirical design of cavity walls, codes generally restrict the level of axial load to a relatively low value. For the Canadian Code, the maximum unfactored loads would be 106.4 kN and 76 kN for the single wythe and cavity walls, respectively. This compares to 121.5 kN average test capacity at eccentricity of 5t/12 for the single wythe wall. The double wythe wall data is not directly comparable because a wall height of 5.33 m would be permitted based on the “2/3 rule”. If eccentricity of axial load is restricted to less than t/3, the single wythe wall and cavity walls would be marginally adequate. However, although empirically designed walls are designed based on axial load and slenderness limits, they are also required to resist wind pressures. Engineering analysis shows that in the absence of the “benefit” of axial compression, at their limiting heights, neither the single wythe nor the cavity wall have sufficient flexural capacity to resist local wind pressures.

The main problem with empirical design of cavity walls is the lack of a fundamental basis for the empirical design to properly treat bending. Additionally, neglect of effects of using wythes of different materials and geometry are challenges to the rationality of the approach.

If this negative assessment is accepted, then why haven’t there been more failures of empirically designed walls. The answer likely lies in the unaccounted for redundancy incorporated into the construction of most masonry buildings. Two-way bending for walls supported on more than 2 edges, arching of walls between supports, toughness introduced by connecting intersecting walls, and perhaps a tendency not to push empirical design to the limit are possible explanations.

Empirical design has served the masonry industry well by reducing design cost and requiring minimal design expertise. To date it has not been identified as a major cause of safety related problems. However, as modern masonry continues to use more slender walls and walls that are more isolated from other structural components, it is only natural that this will be copied in layout of empirically designed buildings. The resulting loss of features that compensate for the lack of rationality of empirical design will make these buildings more vulnerable to failure. Perhaps it is time for a ‘new’ empirical design where table provide designs based on proper engineering analysis.

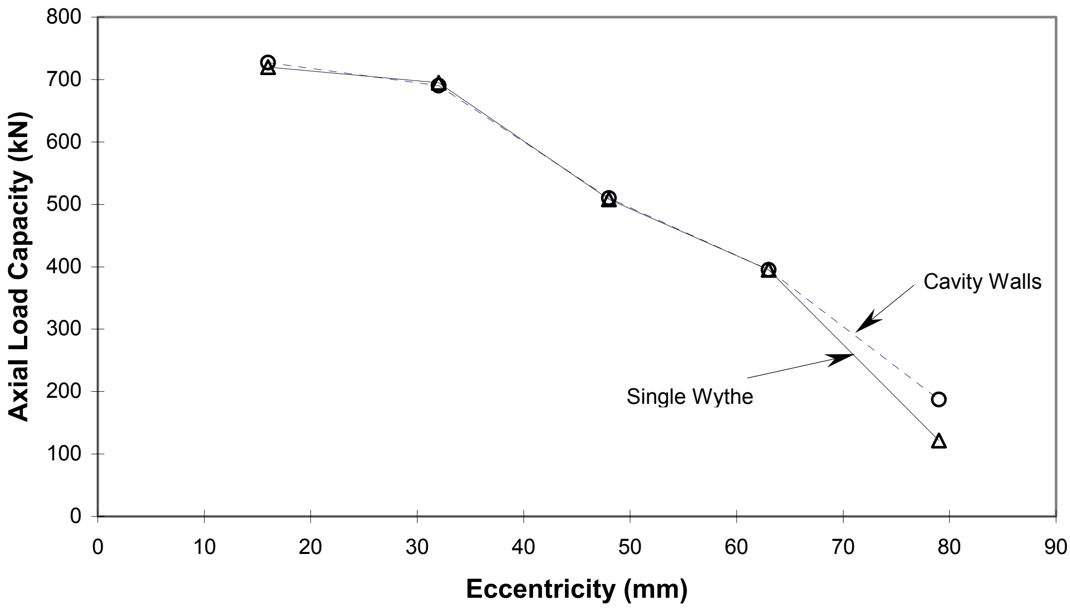

Figure 2 Effect of eccentricity on wall capacity.

6 Acknowledgements

This study forms a part of ongoing research in The McMaster Centre for Effective Design of Structures funded through the Ontario Research and Development Challenge Fund. This research was funded through grants from the Natural Science and Engineering Research Council of Canada (NSERC). The authors appreciate the contribution of the mason's time made available through the Ontario Masonry Contractors Association (OMCA) and the donation of concrete blocks by the Ontario Concrete Block Association (OCBA).

References

American Society for Testing and Materials, 1992, Standard Test Methods for Compressive Strength of Masonry Prisms E447-92b, ASTM, West Conshohocken, PA, U.S.A.

Australian Standard, 1988, Masonry in Buildings AS 3700-1988, The Standards Association of Australia Standards House, North Sydney, N.S.W.

Brick Institute of America (formerly Structural Clay Products Institute), 1969, Recommended Practice for Engineered Brick Masonry, Virginia.

Canadian Standards Association Standard CAN3-S304.1-94, Engineered Masonry Design, Rexdale, Ont.

Davey, N. and Thomas F.G., 1950, The Structural uses of Brickwork, Structural and Building Engineering Division Meeting, 28-33.

Fisher, K., 1970, The Effect of Wall-Ties on the Compressive Strength of Cavity Walls, Proceedings of the Second International Brick Masonry Conference, Stoke-on-Trent, 177-185.

James, J.A., 1972, Investigation of the Behaviour of Single Leaf, 9'' and 11'' Cavity Storey Height Walls Under Axial Load, Report No. W/3/A, Building Development Laboratories, Morley, 39 pages.

Kumar S. and de Vekey, R.C., 1989, Narrow Lightweight Block Walls: Performance Under Vertical Load, International Journal of Masonry Construction, Vol. 3, 91-101.

Laird, D.A., 1983, The Empirical Design Method: How Does It Compare To The Engineering Analysis Method?, Third Canadian Masonry Symposium, Edmonton, Canada, 3-1 to 3-12.

Marr, J.K. 1992. Capacity of Single and Double Wythe Concrete Block Walls, Ph.D. Thesis, McMaster University, Hamilton, Ontario, Canada.

West, H.W.H., 1982, The Lateral Resistance of Cavity Walls with Dissimilar Leaves, Technical Note 330, British Ceramic Research Association, 781-792.