INTERPRETATION OF THE COMPRESSIVE STRENGTH OF MASONRY PRISMS

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ABSTRACT This paper draws attention to and suggests solutions to problems associated with using compressive strengths of masonry determined by prism tests. In this regard, examination of the treatment of compressive strengths in current building codes shows that they incorporate simplistic provisions which are not appropriate for applications. In particular, reference is made to the influence of the geometric characteristics of the units, the direction of loading, the direction of eccentricity, methods for testing the individual materials, and the influence of filling voids with grout.

1. INTRODUCTION

Designers accustomed to working with concrete and steel tend to visualize structural behaviour as an isotropic phemonema. This simple and easy to work with concept has been extended to masonry design. However, even casual inspection of masonry units or masonry construction reveals that it must have definite orthotropic characteristics. This is obvious because of the predominance of coring in the vertical direction, the aspect ratio of the units, and the potential directionally dependent material characteristics.

1.1 Determination of Compressive Strength

At the present time, two methods are commonly used to determine the compressive strength of masonry for design purposes. Tests of prisms made from similar units, mortar, and grout (if applicable) are used to establish the mean compressive strength and variability from which the characteristic compressive strength, f_m is determined (1). However, to relate prism strength to strength of masonry in structures, it has been shown that the height of the prism and the potential influence of loading platen restraint can have significant effects on strengths (2). To avoid this problem, use of prisms at least 4 units high are recommended. In addition, it has been shown that the mortar bedded area and the bond pattern influence strength (3-6). Therefore, prisms should represent actual construction. In addition to the above, it has been found that for grout filled masonry, the strength contribution of the grout is much less than would be predicted by superposition of the grout strength times its area (9, 10). Therefore, testing of the combined masonry assemblage is necessary.

The second method for determining compressive strength is from compressive strengths of the units and the mortars. Here it has been shown that the condition of the block (dry or wet) and the capping conditions can have significant effects on the apparent strength of the block (5, 11). This procedure is based on derivation of a table based on test results. Therefore, the adequacy of this approch is dependent on the range of block types and materials to be covered by single values. As a result, this approach tends to have to be quite conservative to encompass the large variations which exist.

2. EXPERIMENTAL PROGRAM

2.1 Objectives and Scope

The experimental program was designed to provide data which would illustrate the influence of direction of loading (Normal (N) or Parallel (P) to the bed joints) and direction of eccentricity (either in plane or out-of-plane). For the latter, eccentricity of one sixth of the height (H), length (L), and thickness (t) were chosen to provide a significant influence while maintining the entire section in compression. [Previus research (12, 13) illustrated behaviour for a larger range of out-of-plane eccenticities.] In addition to the above, it was decided to include grout filled and, to a lesser extent, solid block masonry.

2.2 Materials

2.2.1 Mortar. The type S mortar used had mix proportions of Portland Cement : Lime : Sand of 1:0.5:3.33 by volume. Batching was done using corresponding weight proportions of 1:0.21:4.24 with a constant amount of water being added to give an average flow of 122%. Three 2 in. (51 mm) mortar cubes were made for each 43.5 kg batch. No retempering was permitted and mortar was thrown out after 30 minutes. At the age of 17 months the mean compressive strength of the 54 air cured cubes was 18.8 MPa with a coefficient of variation of 14.2%.

2.2.2 Grout. The medium strength grout had mix proportions of Portland Cement : Lime : Sand of 1:0.044:3.55 by weight or 1:0.1:3.0 by volume. The water to cement weight ratio of 0.70 produced a very fluid grout nominally measured at 280 mm. At the time of testing the prisms, the compressive strength from 150 mm diameter non-absorbent cylinder molds was 21.8 MPa. However, the more representative compressive strengths from block molded control prisms was 34.0 MPa for the standard 75 x 75 x 150 mm prisms and 30.4 MPa for 120 x 120 x 390 prisms. The latter have a surface area to volume ratio very close to grout in the cells of 190 mm hollow blocks.

2.2.3 Brick. The nominal 90 x 190 x 57 mm bricks used had properties typical of Canadian manufacture. The average actual size was 88 x 190 x 56 mm for thickness, length, and height. The bricks were cored along the mid thickness with three 35 mm diameter holes located at mid length and 45 mm from each end. Based on net area, compressive strength was found to be 118.4 MPa and the in-plane flexural tensile strengths and splitting tensile strengths were 10.9 MPa and 5.3 MPa respectively. The initial rate of absorption was found to be 0.28 $kg/m^2/min$.

2.2.4 Blocks. Different capping conditions were used to show the influences of using half versus full blocks, hard versus soft capping, and face shell versus full capping. For the 190 mm hollow blocks, full hard capping of blocks gave a mean compressive strength of 19.2 MPa based on the minimum cross section area. Similar tests on half blocks resulted in a compressive strength of 17.3 MPa. The compressive strength for hard capping on the face shells of half blocks was 21.1 MPa based on the minimum face shell area. Soft capping of fully capped half blocks resulted in a strength of 15.6 MPa. In addition, compression loading parallel to the bed joint using hard capped ends resulted in a strength of 16.6 MPa based on the minimum face shell area. The compressive strength of hard capped half block solid units was 15.6 MPa. As a measure of tensile strength, the splitting tensile strength of the face shells was 2.06 MPa.

2.3 Fabrication and Test Procedures

2.3.1 Brick. Small brick walls measuring 7 bricks high and 4 bricks long were constructed in running bond. Mortar joints were compressed on both sides using a 15 mm diameter cylindrical jointer. The walls and corresponding mortar cubes were stored in the laboratory until testing. Then the walls were cut into prisms either 7 bricks high by 1 brick long or 3 bricks high by 2 bricks long for testing normal and parallel to the bed joint respectively.

The brick prisms were capped with a thin layer of hydrostone between the bricks and 50 mm thick steel end plates. To these were attached 25 mm thick steel positioning plates drilled to be bolted on at the required eccenticities. Steel plates 150 mm square by 50 thick with a milled seat for a 50 mm diameter ball were fastened to the positioning plates to provide pinned end conditions. The dimensions of each specimen were recorded and the load was positioned in relation to these actual dimensions.

2.3.2 Blocks. Small block walls measuring 2 blocks long by 4 blocks high were constructed in running bond. For the hollow units, only the face shells were mortared except that mortar was placed on the end webs for walls to be filled with grout later. Full mortaring of the bed and head joints was required for the walls built with solid units. At the time of testing, the walls were cut in half to provide 2 prisms, 1 block long by 4 blocks high for loading normal to the bed joints. Similar walls 2 blocks long by 2 blocks high were fabricated for loading parallel to the bed joints. Within two weeks of fabrication, the grout was placed and consolidated using a 30 mm poker vibrator.

The block prisms were tested at an average age of 17 months. Each prism was capped with a thin layer of hydrostone between the blocks and the 75 mm thick steel end plates. To these were attached 25 mm thick steel plates drilled to be bolted on at the required eccentricities. These were milled to accommodate a 38 mm diameter roller. The rollers provided the top and bottom pinned end conditions in the direction of eccentricity while the spherical seat in the head of the test machine permitted adjustment in the other direction.

3. DISCUSSION OF THE TEST RESULTS

3.1 Brick Tests

3.1.1 Failure Loads and Strengths. As would be expected, the mean failure loads recorded in Table 1 show decrease in capacity for eccentric loading. However, to get a better idea of the effects, strengths have been calculated assuming linear elastic behaviour. In each case the actual net and gross cross sections were used in the calculations. The last two columns contain the ratio, K, of strength to strength for concentric loading normal to the bed joints.

3.1.2 Failure Modes. For loading normal to the bed joints, the failure under concentric load was preceded by some splitting originating in the head joints with final failure being quite explosive and following formation of additional vertical cracks in the bricks. For eccentricities of t/6 and L/6, large, brick-high spall zones were indicative of vertical cracking through the bricks on the compression side of the prisms. The prisms loaded parallel to the bed joints exhibited vertical cracking along a line joining the holes in each brick. Failure was by spalling and was accompanied by some additional vertical cracking. For the concentric load case, vertical cracking along the bed joints

usually preceded this failure mechanism.

3.1.3 Comments. Since the failure mechanism for compression involves tensile splitting of the bricks (4, 12, 13, 22), it seems obvious, and it is confirmed by observation, that cracks tend to follow the path of least resistance parallel to the load. The area of this failure plane is therefore often partially composed of mortar joints and voids created by the coring in the bricks. Therefore, relative to the compressive strength of brick, compressive strengths of brick assemblages should be sensitive to tensile/compressive strength ratios, size and location of coring, and location of mortar joints. For the bricks used in this study, the perforations reduced the bed joint area by about 20% whereas they reduced the vertical cross section through the brick by nearly 40% and the vertical lengthwise cross section along the mid thickness by 55%.

For concentric loading, the prism strength, based on minimum net area, is nearly the same for compression normal and parallel to the bed joints. There is no particular reason why this should be so nor is there any correlation with the fact that the compressive strength of the brick tested lengthwise is normally between 50 to 60% of the strength tested flatwise (12) with each strength based on its net area. If the abnormally high strength ratio for the t/6 eccentricity is replaced by the more usual 1.3 to 1.5 values (6, 12, 14), it can be shown that application of plastic theory and concentric strengths will provide reasonable predictions of capacity for eccentric loading.

3.2 Blocks

3.2.1 Failure Loads and Strengths. The strengths in Table 2 are based on gross area for the grout-filled and the solid prisms. For the hollow prisms the "effective mortared area" is used. In the case of loading normal to the bed joints, this area incorporates the overlap of the bottom section of one block on the top section of the block below. For running bond, the lack of vertical alignment of the webs means that this will be less than the 51% of gross area applicable to the minimum net area of the blocks. A simplistic approach using the 32 mm minimum face shell thickness throughout would result in an area less than 34% of gross. However, if the actual shapes of the cells and the degree of overlapping of the larger top surface of the blocks with the webs are included, this results in an effective net area of just over 41% of gross. To simplify calculation, this has been converted to an equivalent uniform face shell thickness of 39 mm. For loading parallel to the bed joints, the "effective mortared area" or simply the effective face shell area has been taken as equivalent to a uniformly 35 mm thick face shell. This is the average as the face shell tapers from 32 to 38 mm but it ignores the added area due to the flared profile near the top of the block.

3.2.2 Failure Modes. Initial cracking occurred vertically through the webs for all loading normal to the bed joints. For the grouted cases, a vertical crack also formed in the head joint and extended into the blocks above and below. For concentric load and out-of-plane eccentricity, the extension of additional web cracking near the face shells resulted in spalling off of whole block heights of face shells. For the in plane eccentricity the web cracking was restricted to the compression end and face shell spalling was limited to this zone by formation of a diagonal crack across the face shells. For loading parallel to the bed joints, the hollow specimens failed by shearing through the face shells. The previously formed cracks along the bed joints did not seem to affect this failure mode. The grouted prisms had cracking through the cross webs followed

by a shearing through the face shells.

3.2.3 Comments. It is interesting to note that, except for Series 8 and 9, web cracking initiated the prism failures. This has also been observed in wall tests (15). Block tests do not exhibit this behaviour except where face shell capping is used. In this regard, elastic finite element analyses show formation of this tension field in the webs for face shell loading. This observation leads to the hypothesis that the strength of the webs can control the compressive strength of face shell mortared block masonry. Nonetheless, the unaccounted for participation of the webs in carrying the load can result in prism strengths (based on effective mortared area) as large as or larger than block strengths calculated for full capping conditions (2, 5). Therefore, it would seem that the geometry of the block, the effective mortared area, and the method for compression testing of blocks will all influence the relationships between block strength and prism strength.

For the hollow prisms loaded normal to the bed joints, the capacity for an eccentricity of t/6 is greater than for L/6 because of the more favourable placement of the face shells for out-of-plane bending (i.e. t/6 is much less than the kern distance). The apparent 25% increase in strength normally associated with a strain gradient effect could in fact be partially accounted for by less tendency for web cracking.

The capacity for grouted prisms under axial load normal to the bed joints is much greater than for corresponding hollow prisms. Howevever, strengths based on effective area are about 25% lower. As suggested previously (9, 10), this may be attributed to incompatibility between the blocks and grout. The significantly higher strength for out-of-plane bending can be explained by the fact that the effects of incompatibility of the grout are lessened because of lower strains in the central grouted part of the wall. Therefore the compression face shell can develop higher stresses prior to splitting of the wall. For in-plane bending, the grout and the block will be strained equally in the high compression region and, therefore, in this region, behaviour will be similar to the concentric loading case.

It is interesting to note that the strength for concentrically loaded solid prisms is nearly as large as the block strength. Eccentric loading and loading parallel to the bed joints were not included because of the difficulty in interpreting the influence of a frogged end (small indentation) at one end of the blocks.

For loading parallel to the bed joints, the strength and failure mode for concentric loading of hollow prisms are similar to those observed for end loading of hollow blocks. The mortar in the head joints or the greater slenderness may account for the slight decrease in strength. For the grouted prisms, the grout filled cells appear to have caused web cracking which precipitated failure at lower strengths albeit at much larger capacities. The much higher strengths associated with eccentric loading can be explained by the fact that only the outermost parts of the prisms were at the critical stress whereas failure had to propogate through the entire face shell or the entire cross web of the blocks.

4. CONCLUSION

4.1 Simple Solutions to Complex Problems

The essence of the acceptability of simple rules to govern design is that an added degree of conservativeness must be incorporated to assure that some aspect of normal practice does not result in an insufficient level of safety. A corollary is that normal practice must be explicitly defined to limit the ranges of properties of materials (relative and absolute) and geometric characteristics. While the compression requirements for most building codes seem to be sufficiently conservative to cover a very wide range of material and geometric characteristics, the corresponding material standards do not appear to provide adequate limitations on the ranges of these characteristics. Therefore, it is suggested that for simple design rules to be viable for modernized masonry building codes, the masonry industry must adhere to stricter material standards which will result in more uniform properties. Reduction of the potential ranges of behaviour will then permit implementation of simple design provisions which are less conservative and therefore more economically viable.

For brick masonry, this may only require development of a relationship between strengths for loading parallel to the bed joint as a function of the normal to the bed joint strength. As is currently the case, these strengths may be related to brick strength and mortar properties. Then the higher apparent strengths associatd with bending could be accounted for either using higher allowable stresses or possibly by application of plastic behaviour.

Block masonry provides a more complex situation. However, if the test results reproduced in this paper were assumed to be typical, the following types of relationships would be possible. It is likely that testing of only fully capped blocks is feasible. Therefore, prism strengths under concentric load will depend on whether joints are fully mortared or only face shell mortared. For face shell mortaring and for solid blocks, 4 course high prism strengths of 90-95% of the block strengths would be appropriate. (Other references show strengths for full mortared hollow blocks to be in the range of 80-85% of block strengths.) For grouted prisms, a simple arbitrary formula which takes the effective mortared area at 100% of f'_m from hollow prisms and the remaining area at 50% of f'_m will provide only slightly conservative values if the appropriate f'_m and the effective area values are used for compression normal and parallel to the bed joints. For eccentric loading and pure bending, the apparent increases in compressive strength can be accommodated either by using higher allowable stresses or by application of the rectangular stress block associated with plastic behaviour. [However, this will have to be checked for in-plane eccentricities in walls where low strain gradients could result in large portions of the walls being subject to high stresses thereby reducing apparent strain gradient benefits.] Finally, adjustments will need to be made for 2 course versus 4 course high prisms and to correlate prism strength to wall strength. However, these two effects seem to nearly cancel each other.

4.2 Alternative Solutions

It may not be possible to sufficiently limit the ranges of properties of masonry materials so that application of simple design rules and improved efficiency of masonry are compatible. In this case, an alternative design procedure based on test values and performance requirements may encourage some masonry manufacturers to further optimize the design of their units. In such

situations, the potential structural benefits should increase the weighting of this aspect of design of the masonry units.

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TABLE 1 -- COMPRESSION TESTS OF BRICK PRISMS

Series	Loading[a] Direction	Ecc.[b]	Failure[c] Load (KN)	C.O.V.[d] %	Mean Strength (MPa) Net Area	Gross Area	K[e] Net Area	Gross Area
A	N	0	826.3	9.1	57.8	48.1	1.0	1.0
B	N	t/6	797.8	4.2	103.2	92.9	1.79	1.93
C	N	L/6	565.0	9.1	76.1	65.8	1.32	1.37
D	P	0	597.3	18.7	59.4	35.9	1.03	0.73
E	P	H/6	406.6	3.5	81.2	48.9	1.40	1.02

TABLE 2 -- COMPRESSION TESTS OF BLOCK PRISMS

Series	Loading[a] Direction	Type[f]	Ecc.[b]	No. of Tests	Mean Capacity (KN)	C.O.V.[d] (%)	Mean Strength[g] (MPa)	K[e]
1	N	H	0	5	571.4	8.4	18.8	1.00
2	N	H	t/6	5	465.3	9.1	23.2	1.23
3	N	H	L/6	3	358.2	15.0	23.6	1.26
4	N	G	0	3	1060.1	1.5	14.3	0.76
5	N	G	t/6	4	760.6	12.6	20.5	1.09
6	N	G	L/6	3	575.1	4.1	15.5	0.82
7	N	S[h]	0	3	980.0	5.8	14.7	0.78
8	P	H	0	3	431.7	7.4	15.8	0.84
9	P	H	H/6	4	315.9	5.2	23.1	1.23
10	P	G	0	3	784.3	15.6	10.6	0.56
11	P	G	H/6	4	599.8	10.5	16.2	0.86

a) N = Normal to the Bed Joint, P = Parallel to the Bed Joint.
b) Ecc. = Eccentricity of applied load as fraction of thickness (t), Length (L), and Height (H).
c) Average of 4 tests.
d) C.O.V. = Coefficient of Variation.
e) K = Mean Strength - Mean Strength for Series A or 1.
f) H = Hollow, G = Grouted, S = Solid.
g) Based on effective mortared area.
h) A small frogged end reduces the area slightly.