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# INFLUENCE OF BLOCK GEOMETRY AND GROUT TYPE ON COMPRESSIVE STRENGTH OF BLOCK MASONRY

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## ABSTRACT

Thirty-nine prisms were prepared and tested under uniaxial compression to investigate the effect of grout on the compressive strength of grouted concrete block masonry. The variables considered included the block geometry as well as the grout type and strength. Blocks with pear or rectangular shaped cells that had flared or non-flared face shells and webs were used in preparing the specimens. Both fine and coarse grout were used and the grout strength was changed by altering the mix proportions. Grouting masonry has been found to increase the load carrying capacity of the grouted masonry assemblage, but based on the increased area resulting from grouting, not always its strength ($f'_m$). Using coarse aggregate in the grout mix resulted in higher compressive strengths than when grout containing only fine aggregate was used. Choosing strong grout emphasized this effect. Furthermore, improving the alignment of the grout columns formed in the cells of the blocks and increasing their cross-section by using non-flared face shells also added to the contribution of grout.

## INTRODUCTION

### *Background*

Filling the cells of hollow concrete block masonry with grout is a very effective technique for increasing the loadbearing cross sectional area of a wall. Because hollow units weigh less than solid units, and because it is easier to lay hollow blocks on face shell

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beds of mortar than it is to lay solid blocks on full mortar beds, the productivity of masons is much higher for hollow block masonry. In addition, the presence of the cells allows vertical reinforcement to be grouted in place in the wall. For these reasons, grouted block masonry is the accepted method used to provide sufficient loadbearing area without increasing the wall thickness.

Use of grout with a compressive strength equal to the block strength or the masonry strength (ACI 1992) has been accepted as sufficient to ensure that the entire area of a grouted member could be assigned the same compressive strength as hollow masonry. However, during the 1970s many researchers observed that grouted masonry had lower compressive strengths than comparable hollow masonry. That is, although the compressive capacity of a grouted prisms is much higher than for the comparable prism made using ungrouted hollow blocks, the much larger effective cross-sectional area of the grouted blockwork results in a lower stress at failure than for the ungrouted blockwork. The effective area for hollow masonry is at most equal to the cross sectional area of the block and often only the area of the face shells.

*Possible Explanations for Lower Strengths of Grouted Masonry*
A test program (Drysdale et al. 1979) showed that calculated compressive capacities, based on superposition of the area of hollow masonry times its strength plus the area of grout times the compressive strength of the grout, far exceeded the actual compressive capacity. In fact, very high grout strengths were required to achieve a strength of grouted masonry equal to the ungrouted strength. Many hypotheses have been put forward to explain this phenomenon (Drysdale et al. 1994). These include:

*Material Incompatibility*. The stress-strain properties of the hollow concrete block and mortar assemblage are different from the columns of grout in the cell spaces. As a result, superposition based on plastic behaviour is not valid (i.e. one of the materials fails before the strength of the other is reached). Examination of failure modes, which generally show initial cracking and eventual failure of the face shell area with the grouted cells remaining relatively intact after failure, provides support for this hypothesis.

*Effects of Block Geometry*. For demoulding of blocks during manufacture, it is necessary that the face shells and webs of the block be slightly tapered. In addition, to aid in construction, the tops of the webs and face shells are often flared out to provide a better hand hold for lifting the block and to provide a larger platform for spreading the mortar. This geometry results in the grout having a wedge shape over each course of masonry. Coupled with the lower axial stiffness at the mortar bed joints, it appears that the columns of grout can act as wedges inside the masonry (Hamid et al. 1986). The failure of the block-mortar assemblage described above is also consistent with this explanation.

*Bond Pattern Geometry*. In running bond, the webs of the blocks generally do not align vertically. This results in sudden changes in the centroids of the columns of grout from one course to the next as well as sudden changes in the net section at the mortar bed joints. If the block-mortar assemblages and the grout columns do not combine to form a homogeneous material, it seems sensible to assume that the discontinuities in the grout columns could reduce the effectiveness of the grouted areas.

*Initial Plastic Shrinkage and Flaws in Grout Columns.* As very fluid grout is poured into the cells of concrete block, water is gradually absorbed by the block resulting in a decreased volume of the grout. Even with reconsolidation of the grout while it is still workable, some volume change will continue until the grout hardens due to water absorption. Signs of volume change include horizontal pulling of the grout column away from one or more sides of the cells in the block and some tendency for vertical separation of parts of the grout columns at the bed joints. The flaws sometimes observed in the grout columns at the bed joint location (Miller et al. 1978) can also result from a tendency for coarse grout to bridge over the cells where the nonalignment of the webs provides an obstruction to uniform flow of the grout during consolidation. Any voids in the cross-sectional area of the grout would be expected to negatively affect the contribution of the grout column to compressive strength.

*Drying Shrinkage.* Freshly hardened grout will tend to shrink more than the surrounding concrete blocks. Blocks are manufactured with very low cement and water contents and are typically reasonably dry when they are put into the wall. Conversely, the higher water content (even after absorption by the block) and lower aggregate content in the grout will result in comparatively large shrinkage strains. Because the grout and block are bonded together, the tendency for larger grout shrinkage will logically result in initial internal tensile stresses in the grout as the concrete blocks resist shortening due to grout shrinkage. Compatible initial compressive stresses will likewise be introduced into the blocks.

Any of the above hypotheses, and likely some combination of these, may explain the observed lower compressive strengths of grouted masonry. In research projects, where lower strengths of grouted masonry have not been observed, it is possible that the use of two block high prisms for compression tests has obscured this behaviour. The existence of only one mortar bed joint and the confining effect of solid end platens result in failure modes different from those observed in four block high prisms and in walls. For this reason, tests of two block high prisms, although useful in quality control testing, have limited value in research. In addition, when grout strength is very high, the theoretical capacity of the grout columns is often close to the anticipated assemblage capacity based on gross area times the compressive strength of the ungrouted masonry.

*Scope of the Research Project*

CSA Standard CAN3 S304, "Masonry Design for Buildings" (CSA, 1984) and the limit states design edition (CSA, 1995) differentiate between the compressive strengths of hollow concrete block masonry and grout filled concrete block masonry. The differences between the specified strengths are significant where, for instance, the 9.8 MPa compressive strength for type S mortar and 15 MPa hollow block drops to 7.5 MPa when this combination is grouted solid. A similar 23 to 25% decrease in compressive strength is applied to other block strength−mortar type combinations. Although the much larger cross sectional area created by grouting still results in significant increases in the load carrying capacity of grouted blockwork, the possibility of regaining all or part of the above reduction in compressive strength is still a very worthwhile objective.

A comprehensive research program is planned to include the following features:

*Influence of Grout Type:* It is anticipated that the properties of coarse grout will be more compatible with the properties of the concrete in blocks and will undergo less shrinkage than fine grout.

*Influence of Block Geometry:* There is some evidence that flared face shells and webs do result in decreased compressive strength of grouted masonry.

*Alignment of Grout Columns:* Another aspect related to block geometry is the degree to which the columns of grout are continuous in blockwork built in running bond. Changing the number, thickness and location of the webs in concrete blocks, to achieve better vertical alignment of the webs for running bond construction, are steps that can be taken to create more uniform columns of grout. Use of rectangular cells instead of the pear shaped cells will also reduce sudden changes in the grout section.

*Use of Workability Additives and Expansion Additives in Grout:* Both plastic and drying shrinkage are affected by the high water content in the grout. Use of water reducing agents which maintain workability for lower water contents will reduce both types of shrinkage. Prewetting of the inside faces of the cells of blockwork just prior to placing the grout will reduce plastic shrinkage and may allow a somewhat less fluid grout to be used. Also, inclusion of an expansion agent in the grout can be used to compensate for grout shrinkage.

The experimental results reported in this paper are from the preliminary stage of this research project. This preliminary stage was carried out to help establish details of the main test program. It was hoped that we could avoid the costs of unproductive changes in block geometry and that we could minimize the number of combinations of variables to be tested. The preliminary tests include type of grout (coarse and fine), grout strength, face shells and webs with and without increased thickness due to flared shapes, pear versus rectangular shaped cells, and stack versus running bond patterns.

## EXPERIMENTAL PROGRAM

### *Materials*

*Concrete Block.* Hollow 190 mm concrete blocks left over from previous research programs were selected to provide different geometries of the face shells and webs and different shapes of the cells. The geometry of the most commonly available block in Ontario is shown in Fig. 1. The webs and face shells have tapered sections with an added flare at the top which increases the thickness by 4 mm and 8 mm, respectively, for one side of the face shells and end webs and on both sides of the central web. This is designated as Block Type I and the properties for the specific manufacturer are listed in Table 1. Tests were done in accordance with CSA Standard A165 (1994).

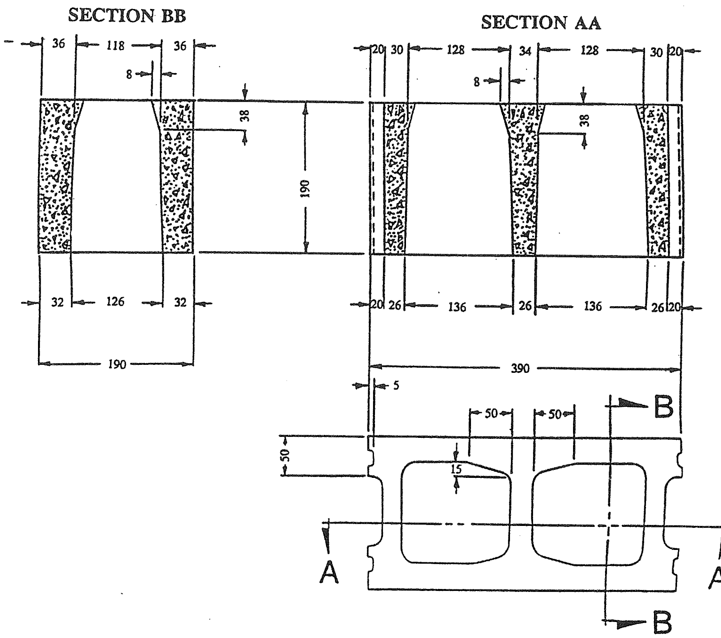


Fig. 1 **Dimensions of a Hollow 190mm Concrete Block with Pear Shaped Cells and Flared Face Shells and Webs**

The average values from tests of five blocks are shown. Compressive capacity of a block (kN) rather than strength (MPa) is shown because use of average cross-sectional area of the block, based on volume of the block, may not be a meaningful indicator of strength where the extra volume due to the flared sections may have little effect. Based on minimum area of the block, the strength is 27.8 MPa (4000 psi). The second last column in Table 1 shows the effective mortared area for a one block length (390 mm) laid in running bond with only the face shells mortared. The last column is the minimum area of continuous grout in the block, taking into account misalignment of the webs and the thickened block sections. Block Type II has a shape similar to Type I except that the webs and face shells are only tapered and do not have the added thickness at the top due to the flares. The thickness at the tops of the face shells and end webs is about 4 mm more than at the bottom and the central web is just over 8 mm thicker at the top than at the bottom. This was a very strong block with a compressive strength of 40.8 MPa (5900 psi), based on the same minimum cross sectional area as Block Type I. Because the block is not thickened as much at the top, the effective mortared area is slightly less than for Block Type I and the minimum continuous grout area is significantly larger.

Block Type III is similar to Block Type II except that the pear shaped cells are replaced

**Table 1 Properties of Concrete Blocks**

| Block Type | Block Description | Density (kg/m$^3$) | Absorption (%) | Compressive Capacity (kN) | Effective Areas in Running Bond (mm$^2$) | |
|---|---|---|---|---|---|---|
| | | | | | Hollow Face Shell Mortared | Minimum Continuous Grout |
| I | Pear shaped cells with flared face shell and web | 2156 | 5.72 | 1050 | 30708 | 17454 |
| II | Pear shaped cells with non-flared face shells and webs | 2053 | 6.90 | 1540 | 30028 | 22088 |
| III | Rectangular cells with non-flared face shells | 2054 | 7.13 | 1260 | 27304 | 22184 |

**Table 2 Properties of Grouts**

| Grout Type | Grout Mix Proportions (by weight) | | | | | Compressive Strength* (MPa) |
|---|---|---|---|---|---|---|
| | Portland Cement | Sand | Pea Gravel | Lime | Water | |
| Weak Coarse | 1 | 4.48 | 2.73 | | 1.02 | 22 |
| Normal Fine | 1 | 2.66 | | 0.04 | 0.60 | 38 |
| Normal Coarse | 1 | 3.69 | 2.25 | | 0.80 | 31 |
| Strong Fine | 1 | 2.00 | | | 0.45 | 52 |
| Strong Coarse | 1 | 3.12 | 1.90 | | 0.57 | 39 |

*From tests of 75 × 75 × 150 mm block moulded grout prisms.

with rectangular cells. Based on the minimum block area, the compressive strength is 34.8 MPa (5,000 psi). The change to rectangular cells results in a significantly lower effective mortared area and a slight increase in minimum continuous area of grout.

*Mortar.* Type S mortar composed of 1.0 : 0.5 : 4.0 parts by volume (or 1.0 : 0.21 : 4.39 parts by weight) of portland cement, lime and sand, conforming to CSA Standard A179 (1994) was used throughout the test program. A water to portland cement ratio of 1.08 was established to satisfy the mason's requirements for workability. This resulted in an average flow of 125%. Small batches of mortar were produced so that retempering was not required.

From tests of the three 51 mm (2 in.) mortar cubes prepared for each mortar batch, the average compressive strengths of the mortar varied from 17.7 MPa to 23.6 MPa with an overall average of 19.8 MPa based on tests of 36 cubes. The cubes were air cured and tested at the same time as the prisms.

*Grout.* To study the effects of grout type and grout strength, fine and coarse grouts, with the mix proportions shown in Table 2, were prepared. The mix proportions were controlled by weight. The volume proportions of the normal fine and coarse grouts fall within the specified range in CSA A179 (1994). The "strong" grouts were mixed with less aggregate and the "weak coarse" grout had extra aggregate added. Slump values in the range of 230 to 250 mm were measured.

The test specimens used to determine grout strength were cut from grout filled blocks. The prism specimens cut from the grouted cells were 75 mm square by 150 mm long. The grout filled blocks were stored with the test prisms and the grout specimens were cut from the blocks and tested at the same time as the prisms were tested.

The block moulded grout specimens were used because it had been determined previously that these specimens gave the most accurate representation of actual grout strength (Guo, 1991). The reason is that the amount of water absorbed from the grout and the curing conditions exactly reproduce the conditions in the prisms.

*Prism Test Specimen*

Four block high by one block long prisms were built in running bond where the head joints were introduced in alternate courses by cutting standard stretcher units in half and facing the two ends together. This avoided introducing an extra web in the courses with head joints which is what happens when splitter units are used. Face shell mortar bedding was used except that end webs had mortar placed on them to provide a dyke to prevent leakage of grout out of the cells during grouting. There were two sets of prisms built in a stack pattern. The face shells and webs were fully mortared for these prisms. Grout was placed in lifts and consolidated using an internal 29 mm diameter poker type vibrator.

Prisms were hard capped using a thin layer of gypsum capping compound between the prism and 51 mm (2 in.) thick steel plates. The bottom of the prism was placed on a

38 mm diameter steel roller and the top was loaded through the 229 mm diameter spherical seat in the hydraulic test machine. Mechanical strain indicators were used to measure horizontal and vertical strain on the faces and ends of the prisms at regular load increments.

*Test Results*

*Modes of Failure.* In the case of the ungrouted prisms with face shell mortared joints, vertical cracks began to develop in the webs of the blocks at loads as low as 50 percent of the failure load. At failure, these cracks became inclined and passed through the face shells of the blocks. Conversely, for the grouted prisms, vertical cracks tended to develop first in the face shells, often associated with the head joints, and later in the block webs at the ends of the prisms. At failure, spalling off of large parts of the face shells and some damage to adjacent grout cells over the mid-height region of the prism was typical.

*Strength Characteristics.* The average failure loads from tests of the three specimens for each type of prism are listed in Table 3. For most of the tests, the three results were grouped quite closely but for Prism Type PF5, one very low result, which would have reduced the average capacity to 1405 kN, was discarded. Similarly, for Prism Type PN2, one very high result, which would have increased the average capacity to 1874 kN was not included. Inclusion of these two test results would not change the general trends or overall conclusions but their arbitrary exclusion does give a more consistent response compared to other results.

Analysis of the test results is complicated by the use of blocks having very different compressive strengths but there are some very obvious observations that can be made. For Block Type I, the strength of 23.6 MPa for the hollow face shell mortared prisms (Prism Type PF1) would equate to a prism capacity of 1664 kN if the grout provides a similar compressive strength for the remaining part of the cross-section. Despite using grout types with compressive strengths up to 52 MPa, most of the prism strengths were well below this value. Only the prism filled with strong coarse grout exceeded this strength and the stack pattern prism with normal coarse grout had an average compressive strength nearly equal to the 23.6 MPa strength for hollow masonry. Therefore, grout strengths exceeding the assemblage and the block strengths did not provide sufficient additional capacity so that the compressive strengths of grout filled masonry could be equated to the compressive strengths of hollow masonry. Although the information is less extensive, similar results are noted for Block Types II and III. Another way to look at this relationship is to compare the ratios of capacities of grouted prisms versus hollow prisms, $P_{u_{Grouted}}/P_{u_{Hollow}}$, to the corresponding ratios of effective areas. For the prisms built in running bond, the ratios of effective areas of grouted prisms to hollow prisms are 2.30, 2.35 and 2.58 for Block Types I, II and III, respectively. Again, the only Prism Type to exceed these values was PF6 with Block Type I and strong coarse grout. What is very apparent is that coarse grout is far more

**Table 3 Prism Test Results**

| Block Type | Prism Type | Grout Type | Failure Load, Pu (kN) | $f'_m$ † (MPa) | $\frac{P_{u_{Grouted}}}{P_{u_{Hollow}}}$ | $\frac{P_u}{P_{block}}$ | $K_1$ | $K_2$ | $K_3$ |
|---|---|---|---|---|---|---|---|---|---|
| I | PF1 | None | 724 | 23.6 | 1.00 | 0.69 | – | – | – |
| | PF2 | Normal Fine | 1289 | 18.3 | 1.78 | 1.23 | 0.37 | 0.42 | 0.85 |
| | PF3 | Strong Fine | 1492 | 21.2 | 2.06 | 1.42 | 0.37 | 0.42 | 0.85 |
| | PF4 | Weak Coarse | 1290 | 18.3 | 1.78 | 1.23 | 0.65 | 0.73 | 1.47 |
| | PF5 | Normal Coarse | 1493* | 21.2 | 2.06 | 1.42 | 0.62 | 0.70 | 1.42 |
| | PF6 | Strong Coarse | 1703 | 24.2 | 2.35 | 1.62 | 0.63 | 0.71 | 1.44 |
| | PF7S+ | None | 875 | 23.2 | – | 0.83 | – | – | – |
| | PF8S+ | Normal Coarse | 1637 | 23.2 | 2.26 | 1.56 | 0.74 | 0.74 | 1.38 |
| II | PN1 | None | 930 | 31.0 | 1.00 | 0.60 | – | – | – |
| | PN2 | Normal Fine | 1739* | 24.7 | 1.87 | 1.13 | 0.52 | 0.59 | 0.96 |
| | PN3 | Normal Coarse | 2010 | 28.5 | 2.16 | 1.31 | 0.86 | 0.97 | 1.58 |
| III | RN1 | None | 800 | 29.3 | 1.00 | 0.63 | – | – | – |
| | RN2 | Normal Fine | 1440 | 20.4 | 1.80 | 1.14 | 0.39 | 0.43 | 0.76 |

* Average of 2 prism tests
\+ S = stack pattern
† Based on effective mortar bedded area for hollow prisms and the total solid area for grout filled prisms.

effective than fine grout even though the strengths of the coarse grouts are generally lower than for the fine grouts.

For the prisms built in the stack pattern using Block Type I, the capacity of the hollow prism is larger than for the prisms built in running bond, because the webs are mortared. Using the different effective areas, the compressive strengths are similar. However, the grouted stack pattern prisms (PF8S) has about 10% additional capacity compared to the corresponding grouted running bond prisms (PF5). This is likely due to the existence of more uniform columns of grout in walls built in a stack pattern. Figure 2 shows partial vertical sections in the plane of walls built in stack pattern and running bond. As can be seen, alignment of the webs in the stack pattern produces much more uniform columns of grout over the height of the wall.

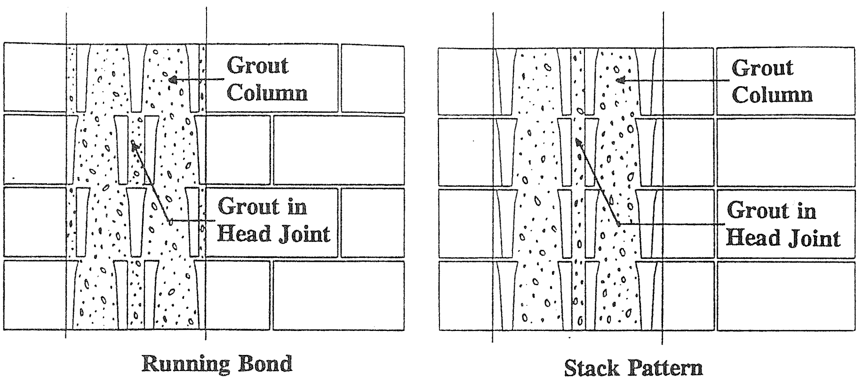


**Fig. 2 Vertical Sections Through Grouted Concrete Block Walls**

In an attempt to reduce the influence of different block strengths on comparisons of prism capacities, the prism capacities were also normalized by dividing by the appropriate block capacity. For the stronger concrete in Block Types II and III, these ratios are slightly lower than for Block Type I. In addition, even though the grout area is larger for Block Types II and III, the relative capacities of the grouted prisms appear to be slightly lower rather than the expected higher capacities. Therefore, an alternate method for looking at the influence of grout on prism capacity was tried. The following simple equation was used:

$$P_{u_{Grouted}} = P_{u_{Hollow}} + K_i \, A_{i_{grout}} \, f'_{grout}$$

where $P_{u_{Grouted}}$ = capacity of the grouted prism

$P_{u_{Hollow}}$ = capacity of the corresponding hollow prism

| | | |
|---|---|---|
| $A_{i_{grout}}$ | = | effective cross-section area of the grout based on criteria i |
| $f'_{grout}$ | = | compressive strength of the grout |
| $K_i$ | = | grout efficiency factor for the grout area determined using criteria i. |

The first analysis (i = 1) designated $A_1$ as equal to the gross area minus the effective mortar bedded area. On this basis, the fine grout is seen to be only about 37% effective whereas the coarse grout is about 64% effective for prisms built in running bond. For Block Type II, the lack of flared tops on the webs and face shells of the block may be the reason for the higher efficiency factors, although this argument is not supported by the tests using Block Type III which also did not have flared webs and face shells.

For running bond, Figure 2 clearly illustrates that the pocket of grout formed in the head joints between the frogged ends of blocks is completely discontinuous from course to course. Experience has shown that this region may not be filled effectively and also, even if it is filled with grout, its effectiveness as a load carrying part of the section is questionable. Therefore, the next analysis (i = 2) was done using $A_2$ equal to the gross area minus the effective mortar area minus the area of the grout cell formed in the frogged ends of the block. The reduced area of grout (except for the stack pattern prism) resulted in higher calculated efficiency factors but, with the exception of PN3, these were still well below 100% efficiency.

At the extreme, the minimum continuous area of the grout columns can be calculated. These values are listed in the final column of Table 1. Using these values for $A_3$ (i = 3), we can see that the efficiency factors for fine grout increase significantly but are generally still well below 100%. For coarse grout, the calculated efficiencies greater than 100% indicate that the effective areas of grout are likely somewhere between criteria 2 and 3.

## CONCLUSIONS

1. Filling the cells of concrete masonry with grout results in substantial increases in capacity but the average stress at failure (i.e., strength) is generally less than for ungrouted masonry except where very high strength grout is used.

2. Grout made with coarse aggregate is more effective than fine grout in increasing the compressive capacity of grouted block masonry.

3. From the tests of stack pattern prisms and the prisms made with blocks not having flared webs and face shells, it appears that creation of larger and/or more continuous columns of grout enhances the increases in capacity due to grouting. However, the influence is not as large as originally anticipated. The use of blocks with different concrete strengths also creates some uncertainty regarding some aspects of this comparison.

4. This preliminary test program has helped finalize details for the main test program where blocks with different cell and web configurations will be produced by the same plant using the same concrete mix design. Influences of expansion agents, grout to block strength ratios, grout strength and type and continuous horizontal grout sections will also be investigated in this next phase.

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# CREEP TESTS ON CLAY MASONRY PRISMS: APPARATUS AND SOME INITIAL RESULTS

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## ABSTRACT

The apparatus for a comprehensive set of creep tests on clay masonry prisms is described. The apparatus was designed to be self-sufficient should components fail in that the tests are intended to last for a minimum of fifteen years. Three series of tests have been begun examining the effect of the following variables on creep: unit, mortar, stress, moisture condition, and age at loading. The moisture condition of the unit when laid and the temperature during the test period were not investigated. Preliminary assessment of the data reveals that creep in clay masonry occurs for at least 2500 days: and creep varies with both moisture condition and age at loading. Analysis through the use of specific creep for a specific mortar/unit combination may not be possible.

## INTRODUCTION

Masonry construction has changed significantly during the twentieth century. Thick low-stressed walls have given way to other construction techniques. The changes in economics and the introduction of concrete blockwork have almost caused the demise of load-bearing brickwork. Brickwork is now mainly used for decorative purposes as veneer, or in low-rise housing: its load-bearing capabilities being heavily under-utilized or ignored.

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