

POST-TENSIONING OF A SINGLE-WYTHE CLAY BRICK MASONRY WALL

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ABSTRACT Vertical reinforcement embedded in grout improves the flexural behaviour of a load bearing masonry wall, but does not increase the cracking moment significantly. The cracking moment depends primarily on the tensile strength of the masonry construction. As with concrete, the most effective method to develop full flexural and shear capacity is prestressing. In this paper, we present results from a single-wythe storey-high clay brick wall constructed of cored units, post-tensioned in both the vertical and horizontal directions with the Dywidag Threadbar System with unbonded tendons.

1. INTRODUCTION

Load bearing masonry walls have to be capable of resisting forces produced by nature, man and general operating conditions. In order to minimize the devastating results of earthquakes or explosions, designers of masonry use vertical reinforcement embedded in grout. However, this practice introduces significant problems of workmanship arising from grouting heavy reinforcing bars in the restricted cores of masonry units. The result is often a lack of continuity in the grout resulting in a reduction of the effective structural action of the vertical reinforcement. Although properly placed reinforcement and grout does improve the flexural strength of the masonry wall, there is little increase in the cracking moment because the tensile and shear strength of the masonry is not significantly increased. As in concrete, a suitable method for achieving an increase in the tensile and shear properties of a masonry wall is the application of a prestress.

With the selection of suitable masonry units, established post-tensioning systems may be used to stress walls immediately after the mortar has gained sufficient strength. In addition to vertical prestressing, forces can also be applied horizontally through use of bond-beam units. The cracking moment is thus increased in both the vertical and the horizontal directions.

If satisfactory attention is paid to corrosion and fire, the tendons could be unbonded: grouting problems would be eliminated with the additional bonus of a reduction in weight of the wall.

Vertical and horizontal post-tensioning was applied to a single storey-high clay brick wall constructed of extruded core units, 90 X 190 X 390 mm, utilizing the Dywidag Threadbar System. The wall incorporated a window opening with a post-tensioned lintel and sill, Figure 1.

The experimental measurements concentrated on the stress distribution resulting from the post-tensioning forces and the loss of prestress.

2. TESTING PROGRAM

The wall was constructed from wirecut extruded clay units produced by I.XL Industries of Medicine Hat, Alberta. Actual dimensions are shown in Figure 2, but are tabulated nominally as 100 X 200 X 400 mm. The unit complies with CSA A82.8M 1978 and ASTM C652.

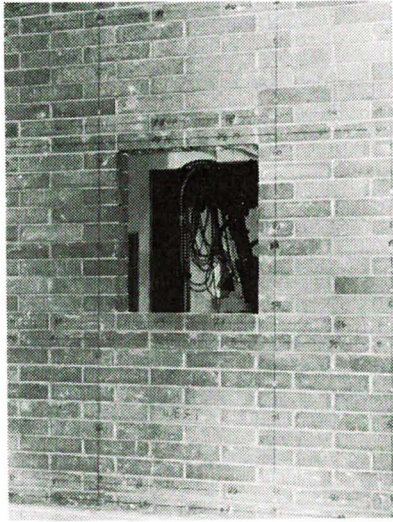


Figure 1

Wall

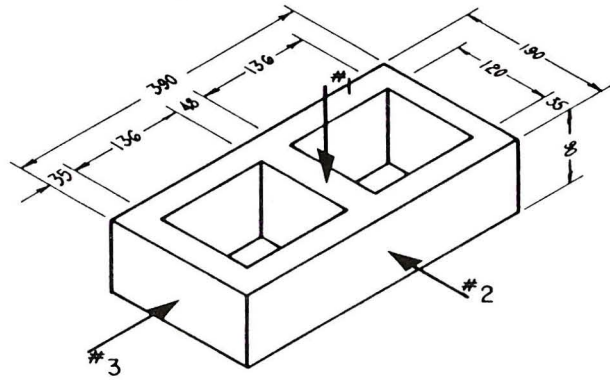


Figure 2

Monarch 200 Unit

The wall was constructed 29 courses high and 6 stretchers wide, with a window opening of 9 courses high and 2 stretchers wide. The bricks were laid with type N mortar and 10 mm thick joints. The wall was positioned on a simulated concrete foundation in which 4 Dywidag 15 mm diameter prestressing rods had been anchored, Figure 3.

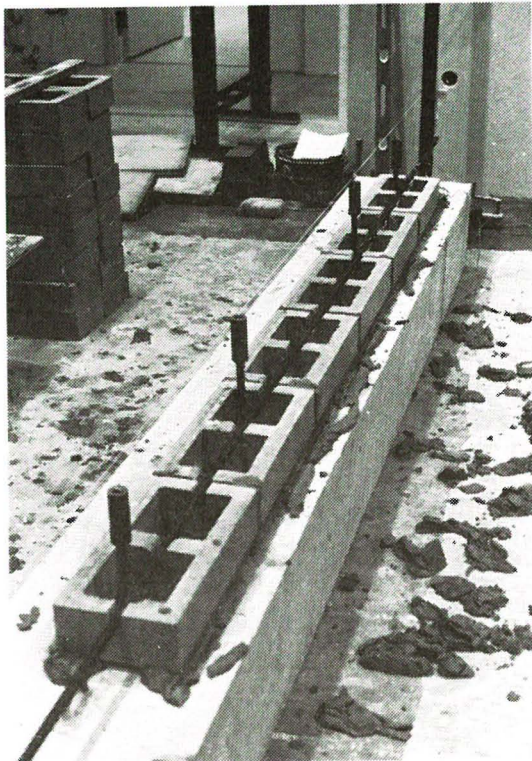


Figure 3

Foundation

Since the brick properties required to investigate a prestressed wall are more extensive than for a conventional brick wall, the unit used for this wall was tested in three directions, shown in Figure 2 and the results obtained are shown in Table 1.

Table 1

Direction #1	430 kN
#2	178 kN
#3	86 kN

The wall was stressed using the Dywidag post-tensioning system. This system uses hot rolled alloy steel bars for tendons. The bars are rolled with rib deformations which act as threads to which anchorages and couplers can be screwed. Anchor Plates are provided with conical seats ensuring accurate centering and proper bearing of the anchor nut. Tendons of any length can be assembled by means of couplers, Figure 4, a and b.

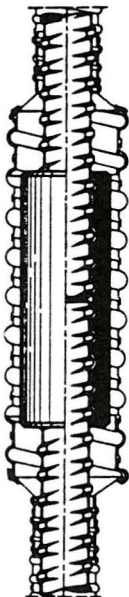


Figure 4a
Coupler

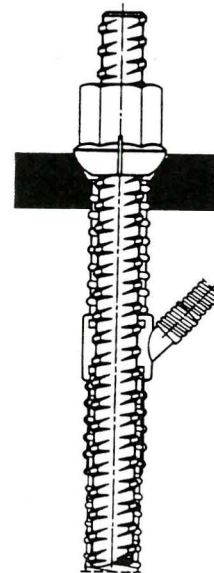


Figure 4b
Anchorage

The nominal tendon diameter was 15 mm with an ultimate strength of 191 kN, (1000 MPa). In order to ensure proper coupling to the anchorages cast in the concrete foundation, part of the brick at the level of connection was temporarily removed so that the rod guided down the cores of the wall could be securely placed on the anchorage and fully tightened. The brick part was then replaced in the unit with mortar. All tendons had been instrumented with two axially placed electrical resistance strain gauges and calibrated. After a curing period of 14 days, the wall was post-tensioned using a hydraulic jack and an accurate loadcell. Although the stressing stages had been carefully planned, problems with anchor plates for the horizontal tendons resulted in a delay of about 20 hours after forces of 70 kN had been applied to the vertical tendons, Figure 5.

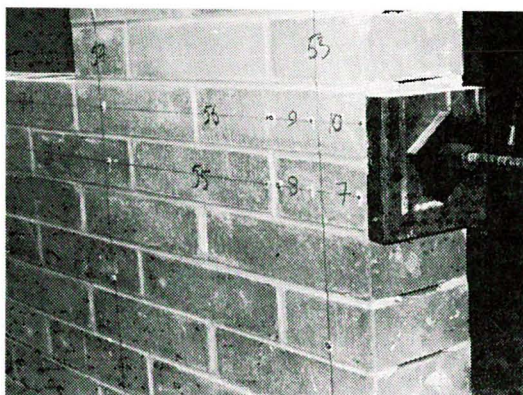


Figure 5

Anchor Plates

At the end of post-tensioning, the following forces had been applied:

Table 2

Vertical Tendons	134 kN (0.70 fpu) (756 MPa)
Lower Bond Beam	70 kN (0.37 fpu) (400 MPa)
Window Sill	70 kN (0.37 fpu) (400 MPa)
Window Lintel	50 kN (0.26 fpu) (280 MPa)
Upper Bond Beam	50 kN (0.26 fpu) (280 MPa)

Strain measurements were made on both sides of the wall, along the line of force of the vertical tendons and on either side of the line of force of the horizontal tendons, Figure 6. In an attempt to pick up peak strains in the vicinity of the bearing plates, instruments with 100 mm gauge lengths were used, while the remainder was measured with 500 mm gauge lengths.

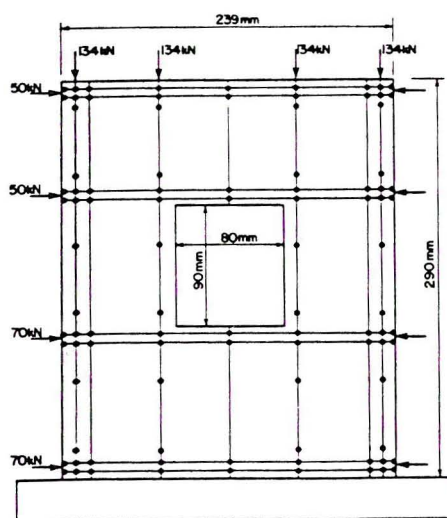


Figure 6

Surface Strain Measurement Locations

3. TEST RESULTS

Measurements recorded for the first 35 days after stressing of the wall are shown in Figures 7 through 10. Figures 7 and 8 show the strain/time behaviour of the post-tensioning tendons, while Figures 9 and 10 show the strain distribution along the line of measurement. The strain histograms correspond to the positions where they were measured on the wall.

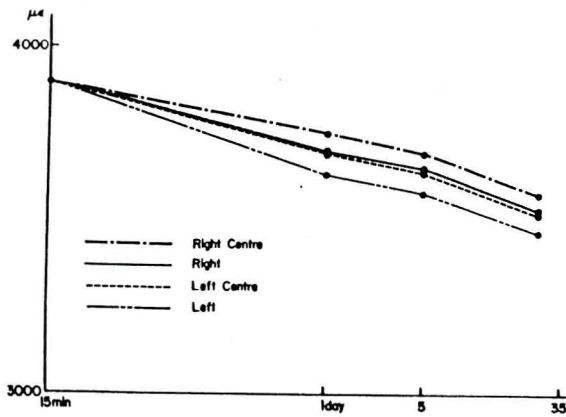


Figure 7
Vertical Tendons

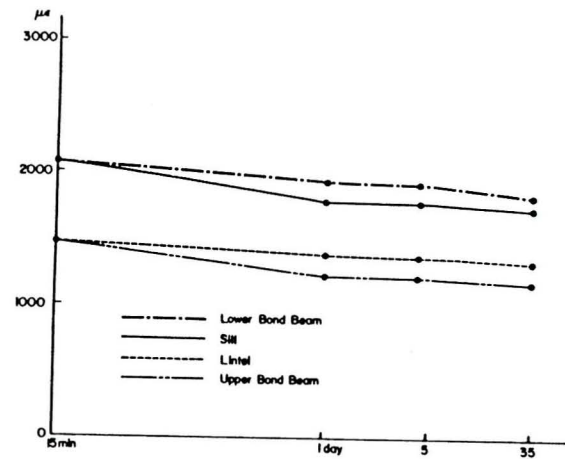


Figure 8
Horizontal Tendons

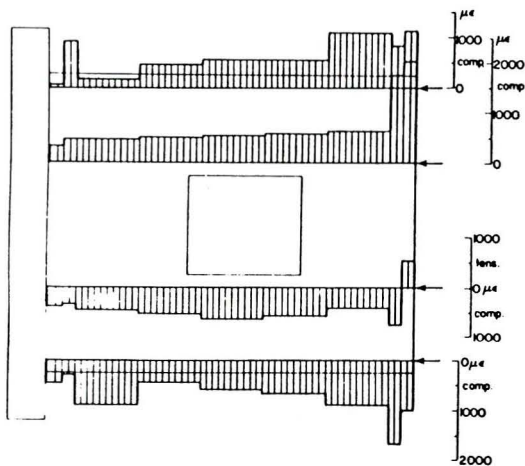


Figure 9
Vertical Strain Distribution

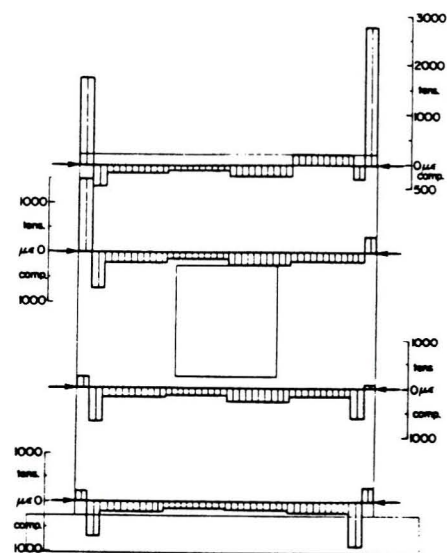


Figure 10
Horizontal Strain Distribution

4. DISCUSSION

The prestressing system used enables the retention of the tensioning force applied without the anchorage loss experienced by systems which employ barrel and jaw anchorages, (1,2). The anchorage nut can be tightened easily to full force and no slippage occurs when the prestressing jack is released. There was virtually no loss after about 15 minutes and the losses recorded for the period measured may be attributed largely to creep in the wall materials. Figure 7 shows that the prestress loss in the vertical tendons is fairly consistent, 8-11%, for a force of 134 kN each. Horizontally, Figure 8, although both the lower bond beam and the sill were equal at 70 kN, the loss for the first was 10% and the latter, 15%. For the lintel and upper bond beam where the load was 50 kN, the values were 8% and 20% respectively. These differences could be due to the lower bond beam being stiffened by the concrete foundation compared with the sill: the upper bond beam is not restrained.

Figure 2 and Table 1 show why the horizontal post-tensioning forces were reduced from those applied vertically. When tested on-end, the strength of the units used is only about a quarter of the vertical compressive strength. The upper bond beam in particular had to be very carefully tensioned. The measured surface strains are shown as histograms in Figures 9 and 10. The shorter gauge lengths near the anchor plates give some indication of the higher strains experienced near these plates. The strain distribution in the vertical tendons appears to be relatively similar in all four tendons. Within expected variations, they increase gradually to peak values at the top. The average of the vertical strain is higher than of the horizontal, which is because of the higher forces applied and the much higher number of joints covered. Vertically 28 joints are loaded against 5 horizontally, and since the major portion of the creep is expected to take place in the mortar, this result appears to be reasonable. The lower horizontal strains show some very high peak values. These values are quite probably caused by cracks in the bricks, although this could not always be visually confirmed.

During the post-tensioning procedure, some cracking was heard. On inspection, some cracks were visible in the face-shells (e.g. Figure 11), and in a large number of webs. Shrive (3) has described a deep-beam mechanism for the cracking of face-shell bedded masonry. More recently, Simbeya (4) has suggested that in most instances the deep-beam mechanism will be the limiting factor for strength of face-shell bedded masonry subject to point loading: the presence of a bond-beam being positive only in certain circumstances.

The use of unbonded tendons as shown in the sill and the lintel, Figures 12 and 13, raises the question of corrosion, a problem which is a continuing one even with conventional reinforcement. The Dywidag system enables tendons to be placed in sheathing which can then be grouted. This would of course result in some increase in total cost, but depending on the structure would probably be minimal. Another problem to be addressed is the one of fireproofing and more research is required to establish acceptable ratings for this type of construction.

In an attempt to simplify on-site post-tensioning, Curtin (5) used a torque wrench considered more appropriate for the level of expertise normally available on a construction site. However, the small diameter tendon used in this experiment was less suitable for this method because of twisting prior to proper seating of the anchorage. In addition, control of the post-tensioning force would be less accurate because of the influence of the surface condition of the bearing plate. Rusted surfaces, burrs or oil could all produce different resistances to the torque wrench, resulting in different post-tensioning forces.

But it is probably also unrealistic to expect electronic load measuring equipment as used for this experiment, to become standard on masonry sites. However, in addition to applying a direct tensile force, a centerhole jack with pump and pressure gauge could become as acceptable as a torque wrench.

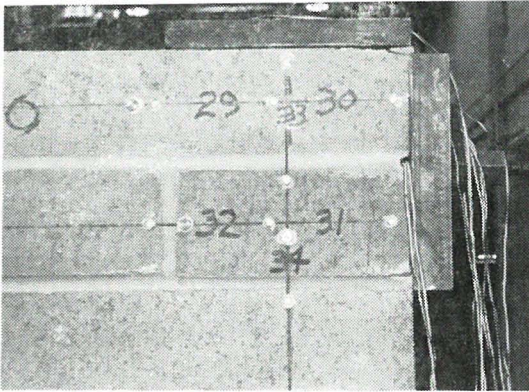


Figure 11
Tensile Crack

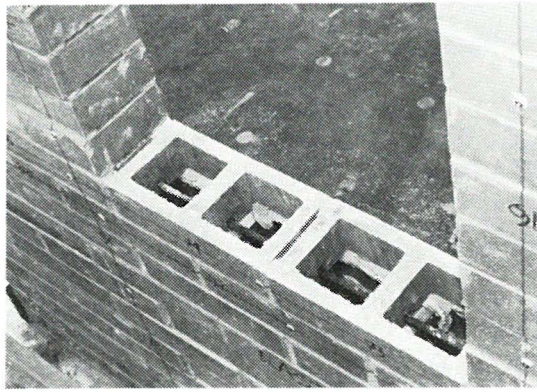


Figure 12
Sill



Figure 13
Lintel

5. CONCLUSION

The results of the work described show that it is possible through selection of suitable units, to post-tension a masonry wall in both the horizontal and vertical directions. Post-tensioning systems that minimize initial loss of prestress in relatively short tendons are available and may be utilized conveniently without too much emphasis on extra expertise. Although continuation of prestress loss can be assumed, it is expected to be within acceptable limits for design purposes. In addition to producing a wall lighter than those constructed with conventional reinforcement and grout, there is also the capability of attaining full flexural capacity with the important resulting increase in the cracking moment. These findings should merit further investigation into this method of construction. This wall will be monitored for loss of prestress and surface strain behaviour for a year and then tested in shear. The project is part of an investigation into the possible improvement of masonry walls when subjected to flexure and shear.

6. ACKNOWLEDGEMENTS

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7. REFERENCES

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