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LATERAL LOAD TESTS OF SLENDER POST-TENSIONED MASONRY WALLS

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Abstract

Recent research at the University of Minnesota indicates that existing design formulas for moment capacity of post-tensioned masonry (PTM) sections may not be applicable to slender walls. In order to address this issue, an experimental program is underway to investigate the resistance of slender PTM walls to transverse loading. Twelve 4.27-m (14-ft) tall walls with 810 x 100 mm (32 x 4 in.) cross-sections are being tested under monotonically increasing transverse loads. The first six walls were built using hollow concrete block, and the remaining six were constructed using cored clay brick. The walls are post-tensioned using high-strength threaded steel rods, and one-half of the wall specimens feature *unrestrained* tendons, while the other six have *restrained* tendons. Three different magnitudes of effective prestress are investigated.

Key Words

Design formulas, flexural strength, load tests, out-of-plane bending, prestressed masonry, slender walls, tendon restraint.

Notation

A_{ps} Area of prestressing steel
A_s Area of reinforcing steel
M_n Nominal moment capacity
P_u Factored axial load
a Depth of rectangular stress block
b Section width
d Effective depth of prestressing steel
d_b Diameter of post-tensioning bar
d_p Allowance for protrusions between tendon and inside surface of masonry
f'_m Masonry compression strength
f_{ps} Tendon stress at nominal strength
f_{pu} Specified tensile strength of tendon
f_{se} Effective prestress after losses
f_y Specified yield strength of steel
l_p Clear span of member
t_{fmax} Maximum face-shell thickness

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1 Introduction

Loadbearing masonry, unlike steel and concrete frame construction, has the capability of incorporating architectural elements, such as partitions and non-load bearing walls, into structural design without adding significant material or cost to the structure. This feature eliminates the coupled need in modern construction for a load-resisting frame and architectural elements, thus offering structural advantage and cost effectiveness. However, the structural success of masonry depends on the ability to overcome its intrinsic low tensile strength, either by reinforcing or by prestressing.

Prestressed masonry utilizes a compressive force applied by steel tendons (either bar or strand) placed within the masonry cells. The compression stress arising from this force counteracts the tensile stress resulting from service loads. Two procedures can be used to supply the prestressing force to the steel: pre-tensioning (tendons are tensioned between external abutments before stress transfer to the masonry) or post-tensioning (tendons are tensioned against the masonry after it achieves target strength). Between these two methods, post-tensioning is preferred for masonry over pre-tensioning due to construction ease, and because stress loss from elastic deformation during post-tensioning process is minimized (Schultz and Scolforo 1991).

Engineered use of post-tensioning for masonry was first implemented in the United Kingdom in the 1960s. Throughout the last few decades, research has helped to evolve more economic and efficient systems (Ganz 1990, Dur-O-Wal 2001, DYWIDAG 2002). Prestressed walls require anchorage to a reinforced concrete footing. In some cases, galvanized steel or plastic ducts connected by threaded sleeves and filled with grease are used to provide corrosion protection. Tendons that are not placed in ducts can be protected from corrosion through galvanic coatings or other corrosion protection systems (Denso 2002).

Masonry units are laid following standard procedures with the ducts or prestressing bars protruding from the cavities in the masonry. Within the cavities, various levels of tendon restraint can be used, which results in different types of behavior. Once the blocks are laid to a specified height, a capping unit, typically in the form of a precast component, or a bearing plate hidden in a masonry unit, is used to provide anchorage at the top of the wall. After the wall has achieved a target value of masonry compression strength, the tendons are stressed using a hydraulic jack or torque wrench (Schultz and Scolforo 1991, Ganz 1990, Dur-O-Wal 2001, DYWIDAG 2002).

2 Influence of Construction Details on Wall Behavior

In the application of post-tensioning technology to masonry structures, there are three common construction methods used for placing the tendons, namely grouted, guided, and unguided. These three methods, in turn, affect the manner in which the tendons are restrained from transverse movement relative to the surrounding masonry.

The grouted condition involves full grouting of the core containing the prestressing tendon after it is stressed. This case is fully *restrained* since the tendon is not free to move transversely relative to the masonry at any point along its length. This is true even if the tendon is unbonded, i.e., all grout is placed outside of the duct containing the tendon and no grout is injected into the space between the duct and the tendon.

The second condition is the guided tendon; where the core is left ungrouted, but the tendon is *restrained* at discrete points along its length by devices that are inserted in, or constitute a part of, the masonry assemblage. The only method for guiding tendons that is currently recognized by the Masonry Standards Joint Committee (MSJC) in the United States (US), the MSJC code (2002), is to place the masonry such that it is in contact with the tendon, or to provide grout plugs, at periodic intervals along

the length of the tendon. From the viewpoint of structural performance, the grouted and the guided conditions are considered to be the same, i.e., both represent tendons that are *restrained* relative to the masonry, as long as at least three uniformly spaced restraints are provided for the latter.

The last restraint category comprises tendons that are *unrestrained* relative to the masonry. This condition is achieved by not grouting the cavities containing the tendons, and not using mechanical devices as restraints. The primary motivation for this form of construction is one of economy in materials, labor and construction time. In this case, the tendon is able to move freely relative to the masonry (Graham and Page 1995). For the *unrestrained* case, the moment arm for the internal couple providing flexural resistance decreases as tendon movement is directed towards the compression face of the masonry member. Furthermore, if the tendon deflection relative to the masonry is large, the tendon force may lead to the eventual instability of the wall for slender members.

3 Proposed Modifications to Prestressed Masonry Provisions

The past two decades have witnessed the development of design provisions for prestressed masonry structures in many countries, including the United Kingdom (UK), Switzerland, Australia, Canada and the US. In most cases, these design provisions address both pre-tensioned and post-tensioned members, but only the latter system has seen widespread application in practice. Moreover, post-tensioned masonry (PTM) members with unbonded tendons are more common than those with bonded tendons. Recent research at the University of Minnesota indicates that design provisions in the US and UK for determining the flexural strength of PTM sections can be overly conservative if unbonded tendons are used, and, especially, if the tendons are restrained and the tendon stress is low.

Based on the analysis of a database assembled from test data of eight experimental investigations that are documented in technical literature (Bean 2002, Bean 2003), it was determined that the MSJC procedures for calculating out-of-plane flexural strength of post-tensioned wall sections needed to be revised. Analysis of the experimental database enabled the identification of improvements to the MSJC formulas that significantly increase the accuracy of moment capacity estimates for stocky post-tensioned walls (i.e., walls for which second-order effects due to slenderness are negligible). Using the modifications described below, estimates of moment capacity may undergo an increase or a decrease, relative to capacity calculated using the existing MSJC provisions (Bean 2002, Bean 2003).

3.1 Tendon Stress at Flexural Capacity, f_{ps}

The accuracy of the MSJC method for calculated moment capacity (M_n) showed a dependency on tendon effective stress (f_{se}), as well as tendon stress at nominal capacity (f_{ps}). As the tendon stress increased, the tendency for the MSJC analysis procedure to underestimate the flexural capacity increased. The problem appeared to be associated not only with the estimation of tendon stresses at ultimate, but also with the characterization of cross-sectional flexural resistance.

One of the difficulties in determining the out-of-plane flexural capacity of PTM members concerns the estimation of the tendon stress increase as the member (e.g., a wall) deforms under transverse loads (e.g., wind pressure). As the member bends under these conditions, the masonry along the tension face cracks (i.e., opening of bed joints), and this action tends to elongate the tendons. Thus, flexural deformation is usually accompanied by an increase in tendon stress, even for the case of tendons that are placed concentrically. Moreover, this phenomenon occurs even if a tendon has little effective prestress (i.e., after losses), as long as the tendon is in a “snug tight” condition at the beginning of loading.

Phipps (1992) developed an expression to approximate the prestressing stress in a tendon and proposed it for prestressed masonry design practice in Great Britain. Phipps' equation is given by

$$f_{ps} = f_{se} + \frac{\chi d}{l_p}\left[1 - 1.4\left(\frac{f_{pu}A_{ps}}{bdf'_m}\right)\right]^n \quad (1)$$

where χ =690,000 MPa (100,000 psi) and the exponent, n=1. Equation (1) was adopted by the MSJC in the 2002 standard.

Equation (1) is used to compute tendon stress for the case of unbonded, *restrained* tendons, when the cross section reaches its nominal moment capacity (i.e., flexural strength calculated on the basis of cross-section dimensions and material strengths assumed in design). Whereas, for the unbonded, *unrestrained* tendon condition, a moment strength check is not required according to the MSJC code (i.e., only a check of service stresses is needed), so the tendon stress at nominal moment strength is not needed. This is, perhaps, an oversight of the MSJC, and if a moment strength check is sought, then the tendon stress at ultimate must be estimated. For unrestrained tendons, it is commonly, and conservatively, assumed that the tendon stress at nominal moment strength (f_{ps}) is equal to the effective stress in the tendon (f_{se}).

Two changes were recommended (Bean 2002, Bean 2003) and accepted by the MSJC to improve the accuracy of Equation (1). The first of these concerns the magnitude of the coefficient χ, which was recognized as the most likely source of error in the MSJC provisions. On the basis of analysis of the database of previous experiments, it was determined that two values for χ are needed: 4,800,000 MPa (700,000 psi) for walls with unbonded, *unrestrained* tendons, and 6,900,000 MPa (1,000,000 psi) those with unbonded, *restrained* tendons. These values are larger than the current value of 690,000 MPa (100,000 psi) in the MSJC standard.

The second change to Equation (1) addresses the exponent, n. By changing the magnitude of n to an optimal value equal to1/2, the overall accuracy of tendon stress predictions for the experimental database increased markedly, while variability of the data set decreased (Bean 2002, Bean 2003).

3.2 Compression Stress Block Depth, *a*

The depth of the masonry compression stress block, a, and the flexural strength, M_n, of the section can be calculated using Equations (2) and (3), respectively,

$$a = \frac{f_{ps}A_{ps} + f_y A_s + P_u}{\gamma f'_m b} \quad (2)$$

where the coefficient γ = 0.85, and

$$M_n = (f_{ps}A_{ps} + f_y A_s + P_u)\left(d - \frac{a}{2}\right) \quad (3)$$

where A_s and f_y are the area and yield strength, respectively, of any non-prestressed reinforcement in the wall. For walls with restrained concentric post-tensioning, which is the most likely application, the effective depth, d, is equal to one-half of the wall thickness, t.

With respect to these provisions, research at the University of Minnesota indicates that the coefficient γ in the denominator in Equation (2), which defines the magnitude of uniform stress in the compression block, is another source of inaccuracy when predicting moment strength. With γ = 0.85, Equation (2) underestimates the actual depth, *a,* of the compression stress block for the masonry walls included in the experimental database. Experimental research in Germany with eccentric compression tests of masonry prisms corroborates this observation (Jager and Pflucke 2001). A more accurate representation of stress block depth in the experimental

database was found when the coefficient γ was taken equal to 0.62. Here it is noted that most of the walls included in the database were made using clay units, so this observation is biased towards clay masonry, which tends to have less curvature in the ascending branch of the stress-strain diagram in compression than concrete masonry does. Thus, a value for γ equal to 0.80, which is specified for strength design of all types of masonry in the MSJC code (2002), was found to produce acceptable results for the wall tests in the database.

3.3 Effective Depth, *d*, for Unguided/Unrestrained Walls

Analysis of walls with unrestrained tendons in the database of existing experimental investigations (Bean 2002, Bean 2003) indicated a need to redefine the effective depth, *d*. This distance, which spans from the extreme compression fiber of the cross section to the centroid of post-tensioning steel, changes as the wall is loaded. If unrestrained, tendons have the freedom to move towards the compression face of the masonry, and the moment arm for the internal couple providing flexural resistance decreases. In the limit, the tendon can come into contact with the inside surface of the cavities in the masonry. For unguided walls under such conditions, the depth *d* should be defined as

$$d = t_{f\,\max} + \frac{1}{2} d_b + d_p \qquad (4)$$

where d_p=13 mm (0.5 in). The dimension d_p is an allowance made for the thickness of mortar droppings and other protrusions between the tendon and the inside surface of masonry. The term t_{fmax} is the maximum face-shell thickness of the masonry unit.

4 Experimental Program

The equations proposed above for calculating the moment capacity of post-tensioned walls were verified against a large database of experiments on stocky walls (i.e., their slenderness did not affect applied moments through second-order effects). In order to investigate the applicability of these equations to slender walls, an experimental program is underway on the resistance of slender PTM walls. The emphasis of the study is on the influence of slenderness, level of prestress, tendon restraint, and masonry type on wall behavior.

4.1 Experimental Variables

One of the concerns regarding the flexural capacity of masonry walls lies in the differences among the various types of masonry. Questions were raised earlier concerning the applicability of code provisions single-valued coefficients that address both clay and concrete masonry walls. As part of the current program, twelve 4.27-m (14-ft) tall walls are being tested under monotonically increasing transverse loads. The first series comprises six cored clay brick walls, and the second set comprises six concrete block walls. The brick walls have fully-bedded mortar joints, while the concrete block walls are face-shell bedded. All walls were made using Type S Portland cement-lime mortar, with the compressive strength of the masonry being 25.9 MPa (3760 psi) for the clay brick and 13.3 MPa (1930 psi) for the concrete block masonry. The net area of the brick and block walls was 503 cm^2 (78 in^2) and 535 cm^2 (83 in^2), respectively.

Another important parameter in quantifying the capacity of PTM walls is the influence of the amount of prestressing. Analysis of previous experiments indicated that moment capacities were underestimated in certain cases by current code procedures when low magnitudes of prestress were applied to the wall sections. Accordingly, the current research plan considers three levels of prestressing on the masonry, namely, 0.24, 0.52, 1.03 MPa [35, 75, and 150 psi].

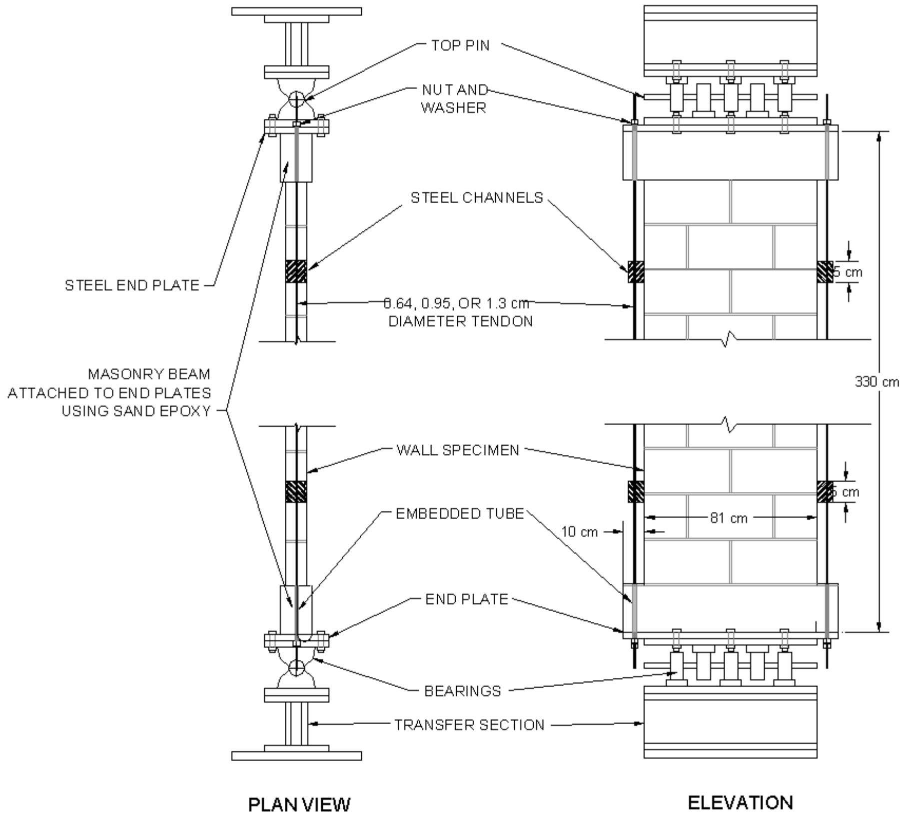


Figure 1. Detail of Wall Specimens

All twelve walls were post-tensioned using ASTM Grade B7 threaded steel rods (Figure 1). These rods had a tensile strength of 862 MPa (125 ksi) and a yield strength of 724 MPa (105 ksi). For walls with a desired effective stress of 0.24 MPa (35 psi), a 6.4-mm (1/4-in) diameter tendon was used, and for an effective stress of 0.52 MPa (75 psi) rod diameter was 9.5 mm (3/8 in). The larger 12.7-mm (1/2 -in) diameter tendon was used for walls with a desired effective stress of 1.03 MPa (150 psi). The net area is 0.52 cm^2 (0.08 in^2), 1.11 cm^2 (0.17 in^2), and 2.01 cm^2 (0.31 in^2) for the 6.4-mm (1/4 -in), 9.5-mm (3/8 -in), and 12.7-mm (1/2 -in) diameter tendons, respectively. Losses were minimized by restressing the tendons immediately before testing.

A target value of 40 was selected for the slenderness ratio (h/t) of the wall specimens, (Figure 1) which required units of small nominal thicknesses (102 mm or 4 in.) and even smaller cavities (38 mm or 1.5 in.). In order not to restrain the post-tensioning tendons in the small cavities of such narrow units, the threaded bars were placed externally on either side of the masonry wall. Short lengths of steel tubes were bolted and adhered with epoxy on either side of the wall specimen to simulate the inside surface of the masonry cavities. The dimensions of the tubes were selected to represent the scaled dimensions of a cavity in a wall with a 203-mm (8-in.) nominal thickness. To simulate the restrained condition using these steel tubes, wooden blocks were fitted inside the tubes to restrict the relative movement of the tendon.

Table 1. Estimated Moment Capacities for PTM Wall Specimens

		UNRESTRAINED (U)		RESTRAINED (R)	
Specimen Designation	Effective Prestress, f_{se} (MPa)	Tendon Stress, f_{ps} (MPa)	Moment Strength, M_n (kN-mm)	Tendon Stress, f_{ps} (MPa)	Moment Strength, M_n (kN-mm)
PB#-35	0.24	270	290	320	710

PB#-75	0.52	260	650	310	1490
PB#-150	1.03	280	1300	330	2790
PC#-35	0.24	280	330	340	780
PC#-75	0.52	270	720	330	1590
PC#-150	1.03	290	1380	340	2880

Knowing the details of the walls and the levels of prestress, moment capacities for wall specimens were calculated using the modified procedure proposed described above (Bean 2002, Bean 2003). The results are given in Table 1 including effective prestress levels, estimated tensile stress in the tendon at nominal strength and the nominal moment capacity.

5 Wall Construction

The wall specimens were constructed on top of a concrete footer beam with a larger width than the block or brick used for the masonry walls panels. The depth of the footer beam was equal to that of a concrete block, as well as that of three clay brick units. These footer beams were attached to a steel end plate by means of a sand-epoxy mortar. For testing, the steel end plates were connected to the bearings in the test setup using steel bolts.

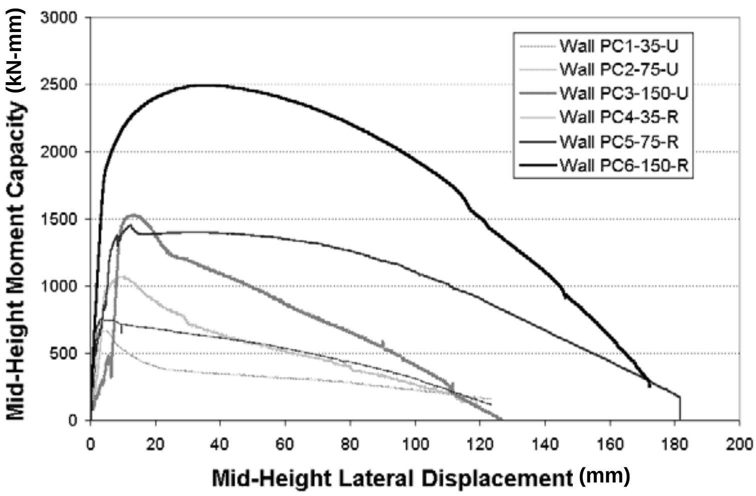
After the wall panels had cured, the header beam was attached to the top of the wall section using sand-epoxy mortar. The epoxy was prepared and, using an overhead crane, the header beam was positioned and bonded to the top of the wall. Once this step was completed, the prestressing bars were inserted through the top and bottom bond beams and then tightened, which placed the wall in slight pre-compression.

Once the epoxy cured, the walls were prestressed. The stress in the tendon was monitored by means of back-to-back strain gages on couplers placed at discrete locations along the post-tensioning bar. Immediately before each experimental test, the stress in the tendon was completely removed, and the tendon was restressed to the desired level.

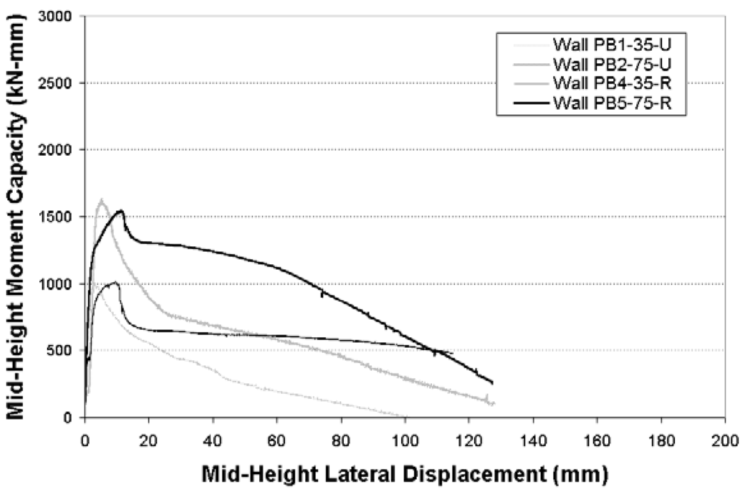
6 Loading Procedure

The masonry walls were supported top and bottom by spreader beams attached to pins, the bottom of which are bolted to the laboratory floor. The top pin was attached to a steel support beam that was restrained laterally to prevent transverse motion.

The lateral loads applied during the tests were generated by a single 155-kN (35-kip) horizontal actuator attached to a braced steel column. This loading was delivered to the wall by way of a whiffletree arrangement, which comprised threaded steel rods, spreader beams, and cylindrical washers. This whiffletree was attached to the masonry wall specimens at four locations along a vertical plane. This scheme had been used previously and it produced a lateral moment distribution that closely simulated the moment diagram for a uniformly distributed lateral load up to peak load.



(a) Concrete Block Walls



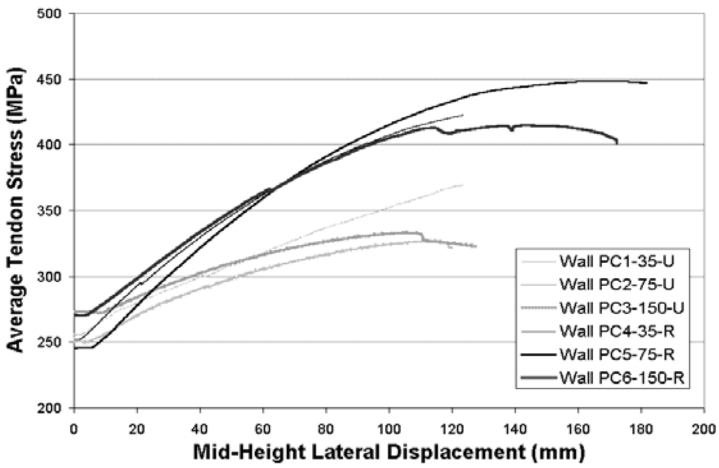
(b) Clay Brick Walls

Figure 2. Experimental Lateral Load-Displacement Behavior

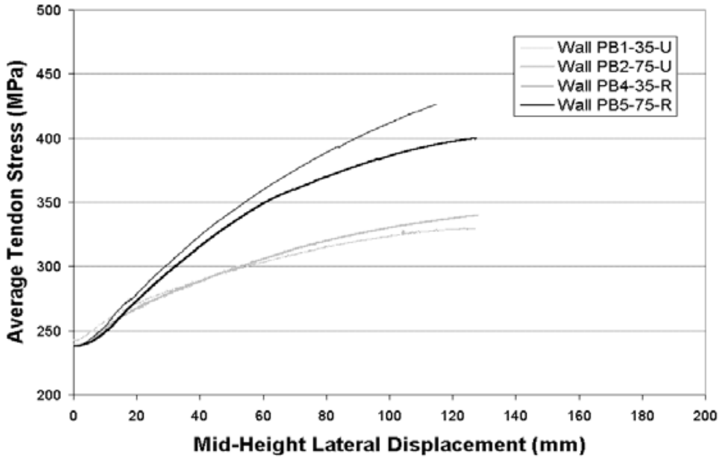
The tests were conducted by applying lateral displacement through the whiffletree. Lateral displacement at the mid-height of the wall was incremented slowly throughout the test. The loading and specimen response to loading were measured using internal load cells in the actuator, load cells on the whiffletree system, load cells on the post-tensioning rods, and LVDTs at various locations along the height of the masonry.

7 Experimental Results

At the time this document was written, all six concrete block walls and four of the six clay brick walls had been tested. Some wall featured restrained tendons, while the others had unrestrained tendons (Table 1). The behaviour of all specimens prior to cracking was linear-elastic. Horizontal cracks formed in the bed joints at the masonry-mortar interface at or near mid-height. Although the crack widths increased with subsequent loading and the lateral displacement grew quickly, the post-cracking behavior differed based on the restraint condition of the tendons. A plot of the lateral load-displacement response of the specimens can be seen in Figure 2.



(a) Concrete Block Walls



(b) Clay Brick Walls

Figure 3. Experimental Tendon Stress-Displacement Behavior

The predicted lateral load capacities of the walls were calculated using the revised equations described in Section 3, assuming that the lateral loads were uniformly distributed at the four loading levels along the height of the wall. In all cases, wall behaviour is stable up to peak moment, and peak moment is underestimated using the formulas in Section 3 except for PC3-75-R and PC3-150-R, which had a slightly lower peak moment strengths than were predicted.

After reaching peak moment, the moment capacities of the concrete block walls (Figure 2a) decreased gradually until the wall had lost all capacity at a lateral displacements on the order of 3.5% and 5.5% of wall height. The brick walls (Figure 2b) exhibited a brief post-peak region with rapid strength decay, followed by an extended period of gradual strength decay to failure. However, in all cases, post-peak behaviour was marked by some degree of instability. As noted in Figure 2, the degree of instability was influenced most by the type of tendon restraint, with the unrestrained tendons showing greater distress. The magnitude of tendon effective prestress is also seen to affect stable response after peak moment. The behaviour of the post-tensioning tendons during loading is also necessary for a complete characterization of wall response to lateral loads (Figure 3). As predicted, the rate of stress increase in the restrained cases was much larger than that of the unrestrained cases.

8 Conclusions

When this paper was written, experimental testing of six concrete block walls and four clay brick walls had been completed. The specimens were tested in a setup that induces monotonically increasing lateral displacement. Reasonable estimates of flexural strengths were obtained using the proposed equations, even though the strength of the concrete block walls with restrained tendons were overestimated slightly. Two additional clay brick wall tests, one with restrained tendons and another with unrestrained tendons, will be conducted to further investigate the influence of slenderness, level of prestress, tendon restraint, and masonry type on wall behavior.

9 Acknowledgements

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