SHAKE TABLE TESTING OF POST-TENSIONED CONCRETE MASONRY WALLS

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INTRODUCTION

Six single-storey unbonded post-tensioned concrete masonry (PCM) walls were tested on a single dimension shake table at North Carolina State University. The principal intent of this study was to validate the use of PCM for residential construction, before the first PCM house is built in New Zealand. Three rectangular walls were tested to demonstrate the seismic performance of post-tensioned rocking walls, followed by walls containing a door and window opening and a shrinkage control joint. A detailed account of wall construction, test setup, testing procedure and test results are provided in this paper.

INTRODUCTION

A research team from the University of Auckland has been investigating the in-plane seismic performance of PCM walls since 1995. This investigation has demonstrated the favourable characteristics that such walls have over their conventionally reinforced equivalents, such as increased in-plane strength and the absence of residual post-earthquake wall displacements. Wall damage typically involves masonry crushing of the wall bottom corners, and this can be easily repaired, thereby reinstating original wall strength and stiffness [1]. A number of additional performance enhancers have been trialled with encouraging results. For example the use of confinement plates improves the strain capacity of the masonry, resulting in increased maximum lateral wall drifts. High strength masonry units in the lower wall corners also enables greater drifts to be achieved before masonry crushing, and the addition of conventional steel reinforcing provides supplementary hysteretic damping to a rocking system that would otherwise have very low levels of damping [2].

The intent from project conception was to develop a post-tensioned concrete masonry wall system that provided improved seismic performance, with respect to conventional reinforced masonry construction. The initial focus was on residential construction for reasons such as less involvement required by consulting engineers and the potential for rapid market growth. This paper contains the results of the first series of tests conducted as a research collaboration between the University of Auckland, New Zealand and North Carolina State University (NCSU), USA which resulted in a number of wall specimens being subjected to a series of ground excitations on a shake table.

HISTORIC APPLICATION OF PCM

Though the concept of prestressed masonry has been around for decades, the codification of prestressed masonry as a standard construction technique has only begun in more recent years. This does not suggest an absence of prestressed masonry applications earlier in the twentieth century. Many projects using clay brick and calcium silicate masonry, typically in Europe, have been documented for example see Ganz [3]. One of the first known applications of post-tensioned concrete masonry was in bridge girders for the North Carolina Department of Transportation in the early 1950s. These were in service until 2003, at which time they needed replacing due to the deteriorating substructure [4]. Several other one-off projects have been completed in more recent years, such as a sound wall for the Colorado Department of Transportation [5] and repairs of existing structures using prestressing technology [6].

The PROTO-II™ wall system uses post-tensioning to provide out-of-plane strength for fences and retaining walls and according to company literature the system has been used in over ten thousand miles of wall over the past two decades [7]. The Dur-O-Wal Sure-Stress™ system is a post-tensioning system that can be used with any mass-produced concrete masonry unit and applied to any situation where the benefits of prestressing are required [8]. The Integra wall system is probably the most widely used concrete masonry post-tensioning system in the US and has been implemented in approximately 15,000 homes since production began in 1984 [9].

Though it would seem that the number of projects involving post-tensioned concrete masonry is vast, the use of PCM in highly seismic areas, such as

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California or New Zealand, is much more limited. One such example was the use of PCM in New Zealand by Hanlon [10], who began experimenting by building several one and two storey structures before constructing a six-storey apartment building in Christchurch.

DESIGN OF WALL TESTS

A series of meetings were held between the authors and a number of industry personnel to identify the key requirements of a wall system for the residential market. This market is very price driven, but currently in New Zealand there are clients who are willing to pay a small premium for the benefits that masonry houses offer, such as improved thermal characteristics and the use of durable materials. It is not known at this stage if prestressed masonry will be cost equivalent to conventionally reinforced masonry, however early indications suggest it will be similar due to the reduction of grout and mild steel required.

NZS 4229, the New Zealand Standard for concrete masonry buildings not requiring specific engineering design [11], provides simplified guidelines for the design and construction of reinforced concrete masonry residential structures. The intent of this study was to utilise the design strategy of NZS 4229, including evaluation of gravity and seismic demand, and publish these guidelines in the form of a supplement.

CONSTRUCTION DETAILS

Wall dimensions are listed in Table 1 and shown in Fig. 1, where the hatched area indicates cells that were grouted and the prestressing ducts are depicted as a clear space inside a hatched cell. This system requires the cell containing the tendon and the two end cells to be grouted. Due to dimension limitations caused by the size of the shake table, this resulted in some walls being fully grouted. Wall 3 was derived from wall 2 after completion of earlier testing. Upon completion of testing wall 4, the 800 mm wide panel was removed and the remaining panel tested as wall 5.

The bond beam in all walls consisted of 2×16 mm (No. 5) bars, with one bar located in each of the top two block courses on opposite sides of the centrally located post-tension duct. Stirrups were spaced at 600 mm centres except when the bond beam formed a lintel as was the case for wall 2

and wall 6, where they were spaced at 165 mm on centre over the opening. Wall 4 contained two dowel bars crossing the control joint with a total length of 800 mm. One half of the bar was greased and coated with a plastic sleeve to ensure bond was not formed with the surrounding grout. ASTM-706 mild reinforcing longitudinal steel, which is the seismic grade mild reinforcing steel in the USA, having a specified yield stress of 414 MPa (60 ksi), was used in all walls tested. The stirrups used in the bond beam were ASTM-615, 6 mm round wire stirrups, with the same specified yield stress.

The walls were constructed in running bond by experienced blocklayers under supervision. Ordinary concrete masonry blocks, having a nominal width of 152 mm (6 in.) giving an actual width of 143 mm (5 5/8 in.), and S-type trade mortar were used in construction. ASTM C91 requires S-type mortar to have a 28 day mortar cube compressive strength equal to or greater than 14.5 MPa. Grout used in all walls was batched at the laboratory using a mechanical mixer, and provided an average 28 day masonry prism strength of 22.7 MPa. Dramix ZL 30/50 zinc-plated steel fibres were used which had a length of 28 mm, hooked ends and a dosage of 30 kg/m^3 and Sika Grout Aid was added to the mix for all walls except the first, providing a slow controlled expansion prior to grout hardening. Vibration of the grout was carried out for all grout pours, using a long steel rod. Wash-out ports were provided at all wall ends and in the cells that contained tendons. Following testing of wall 2, repair of lower wall corners was instituted in order to create wall 3. SikaRepair® 224, a one-component, cementitious, sprayable mortar, was used for the structural repairs.

All walls were prestressed with Dywidag threadbar, from the Dywidag (DSI) Formtie product range, with a nominal diameter of 16 mm, an effective cross-sectional area of 177 mm^2, a specified tensile rupture strength of 195 kN and a yield strength of 163 kN. All tendons remained unbonded over the entire wall height and had a typical unbonded length of 3000 mm, accounting for the foundation, mass blocks and load cell. The initial prestress force for all tendons was approximately 75 kN or $0.46f_{py}$ (f_{py} = prestress tendon yield stress). This allowed wall 1 to achieve a drift of approximately 1% before tendon yielding was expected.
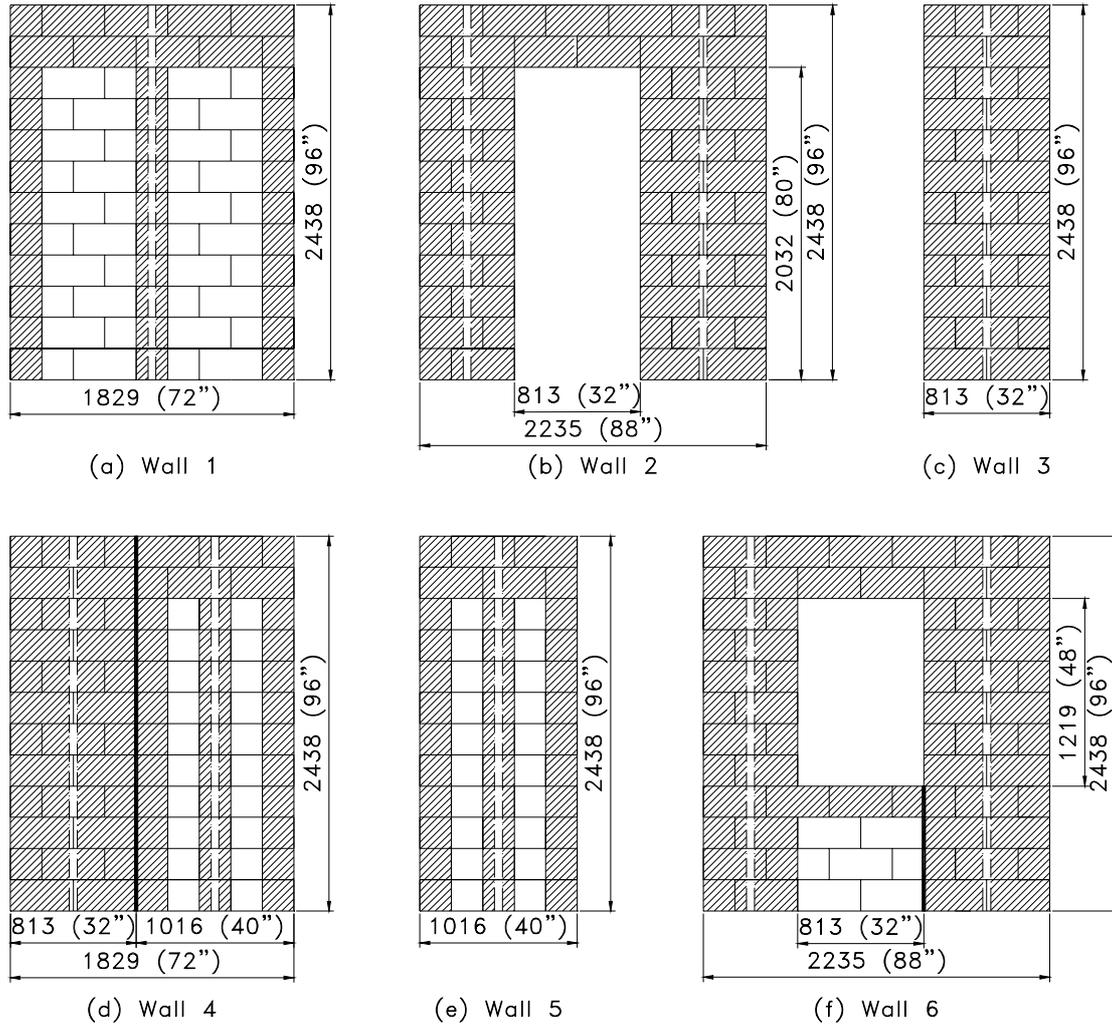
Figure 1 Wall dimensions

Table 1 Wall Properties

Wall	Wall Type	Length	Height	Thickness	Wall Self-Weight	Additional Seismic Mass
1	Rectangle	1829	2438	143	12	2880
2	Door Opening	2235	2438	143	12	2880
3	Rectangle	813	2438	143	6	1440
4	Control Joint	1829	2438	143	14	2880
5	Rectangle	1016	2438	143	7	1440
6	Window Opening	2235	2438	143	14	2880
		mm	mm	mm	kN	kg

TEST SETUP

Fig. 2 shows wall 4 ready for testing. The shake table has surface dimensions of 2438 mm × 2438 mm (8 ft × 8 ft) and a displacement capacity of ±127 mm (±5 in.). It provides an acceleration of up to 1G to a mass of 18,143 kg (40,000 lbs). The walls were built on reusable concrete foundations then moved onto the shake table using an overhead crane. The foundations contained the lower prestress anchorages and were bolted down to the table using a series of 16 mm (5/8 in.) threaded rods. The additional seismic mass blocks were placed on the wall top with a layer of high strength mortar placed in between to provide an even bearing surface. The top prestress anchorage was installed on top of the mass blocks, to hold the structure together. The additional seismic mass applied to the wall top on walls 1, 2, 4 and 6 was 2880 kg, which represented a loading of 3 kPa over an area of 9 m^2. The remaining walls had an additional seismic mass of 1440 kg installed.

Support frames were erected to provide out-of-plane support to the walls, with a number of rollers used to allow for the table and wall displacements. Instrumentation was installed to record accelerations and displacements at key locations, such as the wall top. The data acquisition system recorded 200 scans per channel per second to provide accurate time history data. The walls were post-tensioned using a hollow core jack and hand pump, and the prestress force was measured using load cells.

Figure 2 Wall 4 ready for testing

TEST PROCEDURE

All walls were subjected to a variety of ground excitations on the single degree of freedom shake table at NCSU. Table 2 shows the variety of ground excitations used, which includes both pulse and reversal type records with long and short durations. To provide a range of peak ground accelerations (PGA) a number of records were scaled by applying a scaling factor to the acceleration trace. In some cases the peak displacement was greater than the table displacement capacity, therefore a time scale was applied, resulting in an implied length scale, which is noted in Table 2. The records were run in such an order as to achieve larger wall displacements with each subsequent run. Table 2 shows a typical example of the order used for testing the wall specimens.

TEST RESULTS

Fig. 3 depicts the crack patterns and wall damage for each specimen at the conclusion of testing. Fig. 4 shows the force-displacement backbone curves for the six walls, which were derived using the equation of motion for a SDOF system subjected to an external force. Recognising that the velocity at the peak displacement must be zero, the force at peak displacement is simply the seismic mass multiplied by the total acceleration. By plotting the force at the peak displacement of each wall oscillation throughout the duration of an earthquake record, a backbone curve can be fitted which encompasses all these points. Fig. 4 contains backbone curves for a number of ground accelerations run for each wall. It should be noted that any base sliding was subtracted from the top of wall displacements prior to plotting, and this was seen to be small compared with the top of wall displacements for all dynamic tests. The backbones curves captured for the three rectangular walls are compared with the analytical predictions of Laursen and Ingham [12] showing an acceptable match for all three walls.

Table 2 Suite of Earthquake Records

Acceleration Record	Location, Year	PGA (g)
El Centro NS (25%)	Southern California, 1940	0.09
El Centro NS (50%)	Southern California, 1940	0.18
El Centro NS (75%)	Southern California, 1940	0.26
Tabas (50%) *	Iran, 1978	0.47
El Centro NS	Southern California, 1940	0.35
Mammoth	California, 1980	0.92
Tabas *	Iran, 1978	0.94
Llollelo	Chile, 1985	0.71
Northridge – Sylmar *	California, 1994	0.84
Nahanni	Canada, 1985	0.98
Llollelo (140%)	Chile, 1985	0.99
El Centro NS (250%) *	Southern California, 1940	0.88

* Time scale applied to record to reduce peak displacement

Wall 1

Initially a smaller seismic mass of 960 kg was installed on wall 1 and eight ground accelerations run. The wall sustained negligible damage and reached a peak drift of only 0.29%, before additional mass was added bringing the total up to 2880 kg. A further nine records were run, with the only damage sustained shown in Fig. 3(a), being fine cracking of the mortar joint in the lower left wall corner. The force-displacement backbone curves for the nine runs with the larger mass are shown in Fig. 4(a), and indicate that all nine runs followed the same bilinear curve, suggesting that there was no loss in wall strength or stiffness. Testing ceased due to mechanical problems with the shake table.

Wall 2

For wall 2, the seismic mass was installed in two equal amounts of 1440 kg, one on top of each of the two wall panels, ensuring that the mass block did not stiffen the door lintel. The wall was subject to eleven records before testing was ceased due to considerable crushing of the wall panel lower corners, as shown in Fig. 3(b). Fig. 4(b) shows that the wall stiffness reduced throughout all the runs as a result of the accumulation of lintel cracking, but that there was no significant loss of strength until the final run in the negative direction when the lower corner was crushed.

Wall 3

Wall 3 was subjected to thirteen ground excitations before testing ceased due to splitting of the repair mortar in the lower wall corners. Fig. 4(c) shows that the wall maintained initial strength and stiffness out to a drift limit of approximately 1% before corner crushing resulted in reduced stiffness for the final two ground excitations. Wall strength was not seen to decrease until the wall reached a drift of 2.0% in the negative direction and corner splitting occurred.

Wall 4

Testing of wall 4 involved subjecting the wall to eleven ground excitations resulting in a maximum displacement, for the 1000 mm wide panel, of 15.2 mm or 0.63% drift, as shown in Fig. 4(d), demonstrating the additional stiffness achieved by tying two panels together with a control joint. The wall panels rocked independently of each other, achieving different wall displacements, but remained in phase throughout all cycles. No wall damage was observed, therefore the prestress level was reduced to approximately 24% of tendon yield and four earthquake records rerun. The 1000 mm wide panel reached a maximum displacement of 30.6 mm or 1.28% drift and fine cracking was noticed in the lower wall corners, but this was not structurally significant and no loss in wall strength was recorded. Testing of this wall was terminated at this point.

Wall 5

Initially wall 5 was subjected to ten ground excitations with a prestress level of approximately $0.46f_{py}$. Fig. 4(e) shows that the wall achieved a maximum lateral displacement of 35.9 mm or 1.50% drift, resulting in minor masonry cracking in the compression toe zones and no loss in wall lateral strength. Fig. 5 shows the relative top of wall displacement history for the Sylmar ground excitation, demonstrating the large displacement capacity and lack of residual displacements. Fig. 6 shows the prestress history, indicating that the tendon remained within the elastic range throughout the entire run and there was no loss in prestress.

Eleven additional runs were conducted in an attempt to gather data at different prestress levels within the range of $0.46f_{py}$ to $0.64f_{py}$. There was no increase in the previously achieved wall displacements and no further damage observed. Testing ceased due to completion of all desired earthquake runs and not due to wall damage.

Wall 6

Wall 6 was subjected to twelve ground excitations at a prestress level of approximately $0.46f_{py}$ and then a further three records at a prestress level of approximately $0.28f_{py}$. At the initial prestress level, minor cracking of the bond beam occurred, allowing the two side panels to rock at the base. A maximum wall displacement of 8.7 mm (0.36% drift) was recorded with minimal loss in strength, as indicated by Fig. 4(f). The non-symmetric shape of the force-displacement backbone curve is a result of the control joint below the window opening, which allowed the left panel, as shown in Fig. 1(f), to rock as an 'L' shape, resulting in a large tendon eccentricity and considerably greater wall strength in the negative direction.

During the third run at the lower prestress level, significant cracking of the wall occurred, resulting in the masonry below the opening breaking away from the left hand rectangular panel, as shown in Fig. 3(f). A maximum displacement of 31.6 mm (1.32% drift) was achieved and testing ceased due to substantial wall damage.
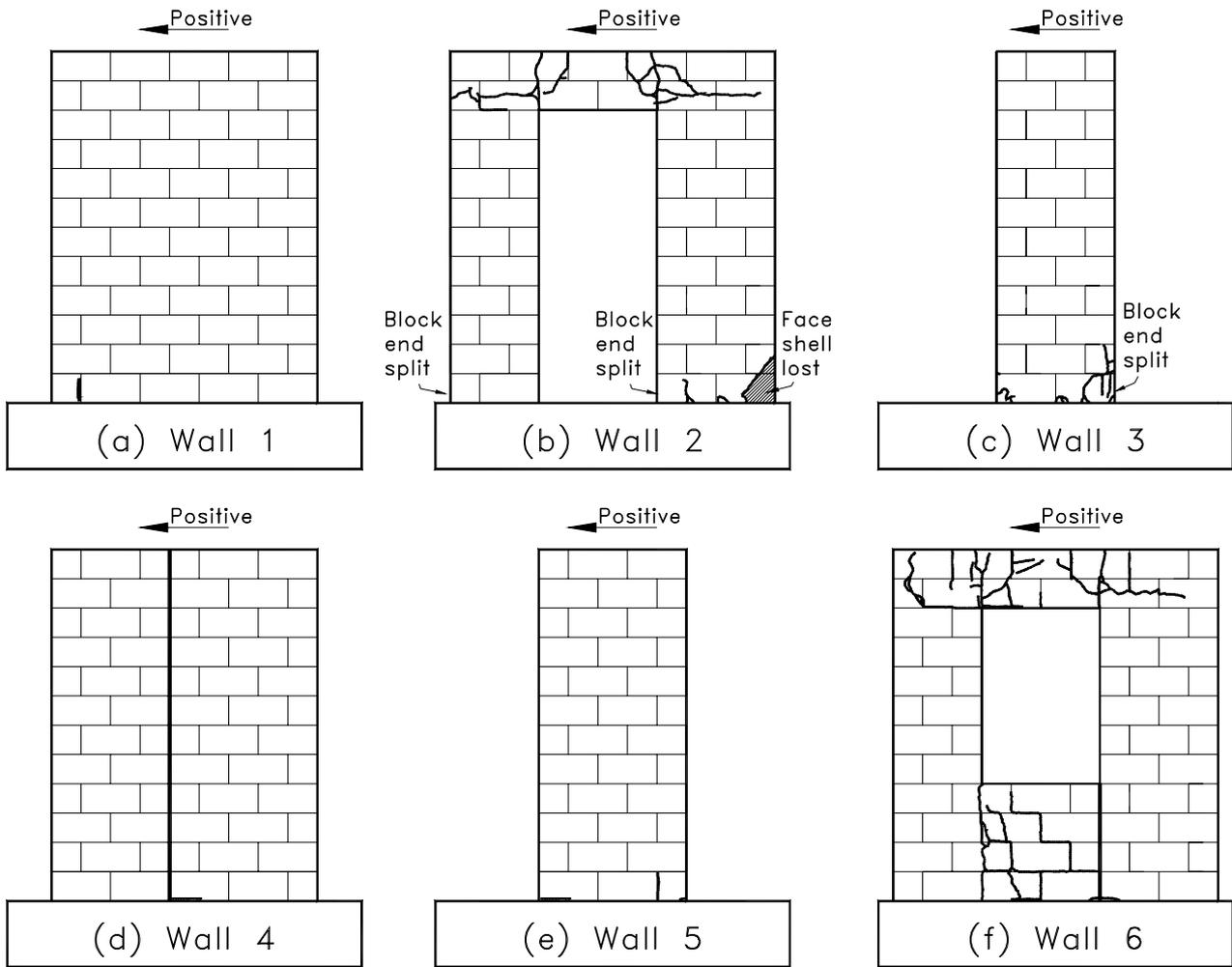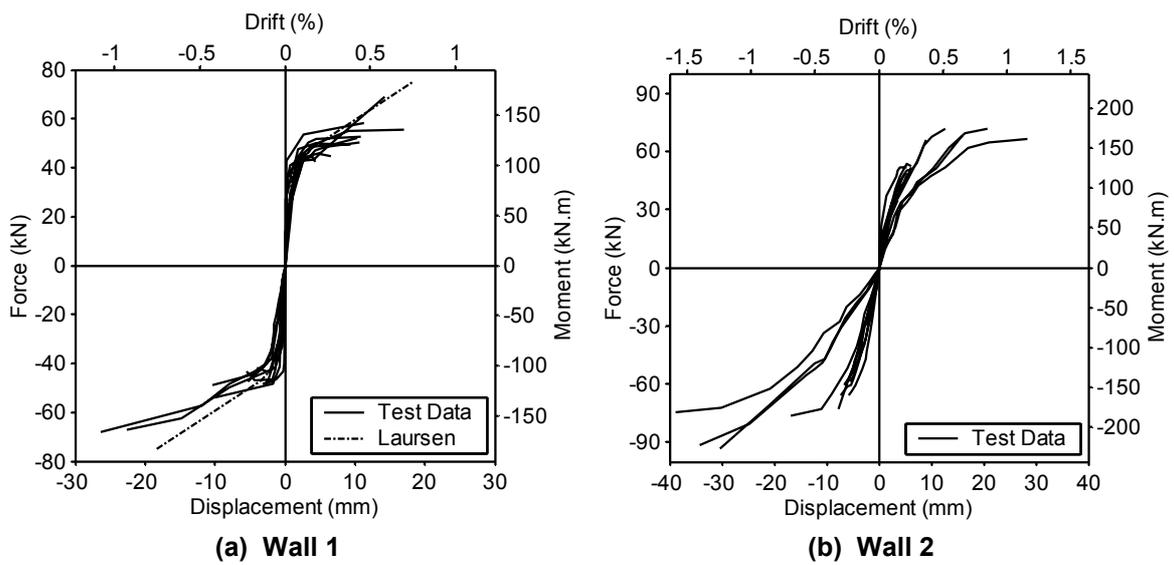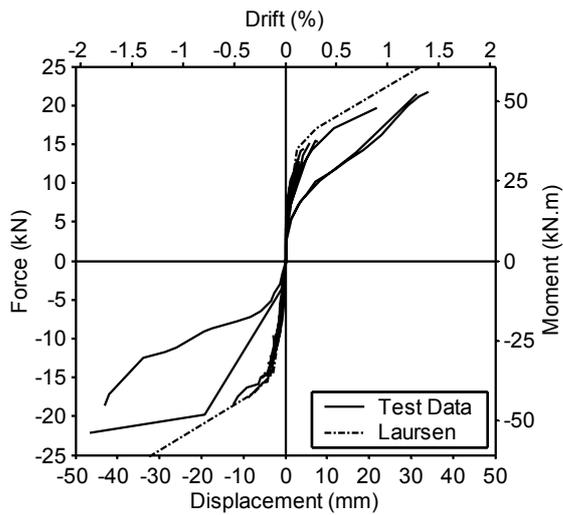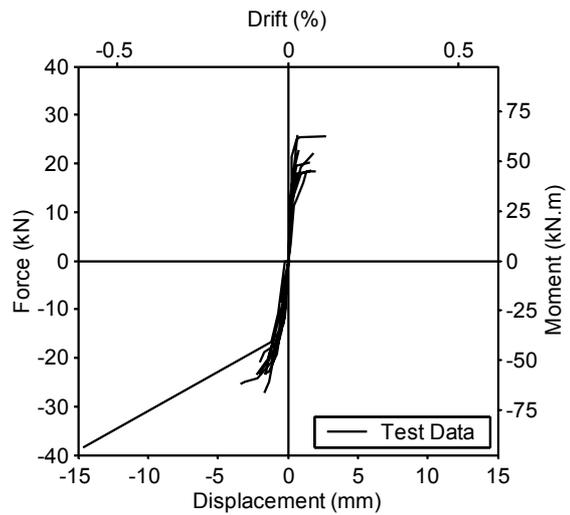
Figure 3 Wall crack patterns at end of testing

(a) Wall 1

(b) Wall 2

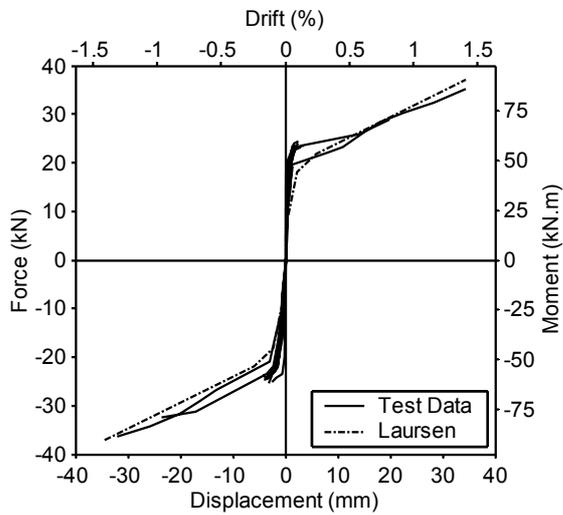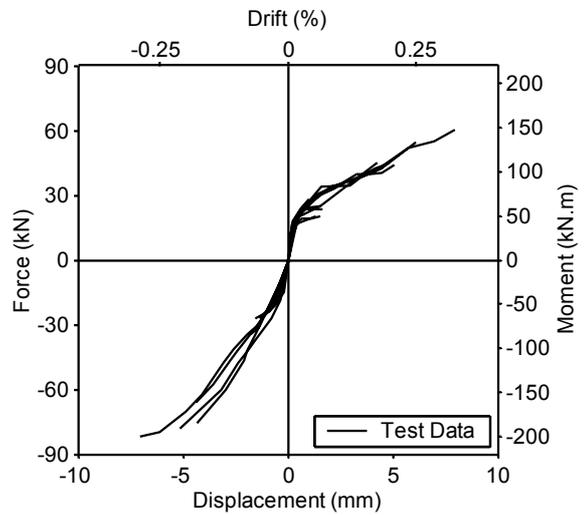**(c) Wall 3**   **(d) Wall 4**

(e) Wall 5   **(f) Wall 6**

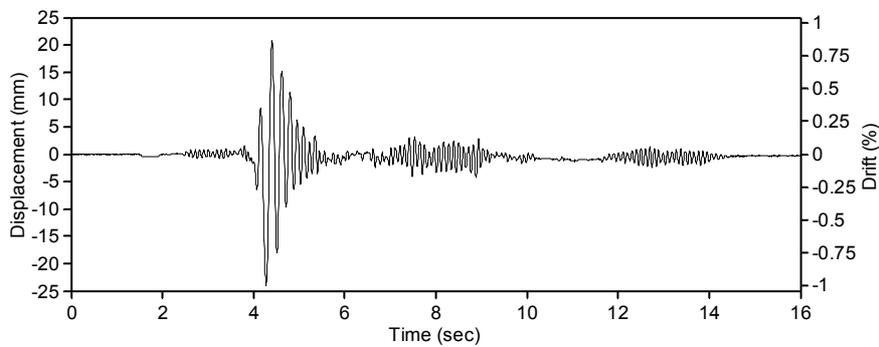**Figure 4 Force-Displacement backbone curves**

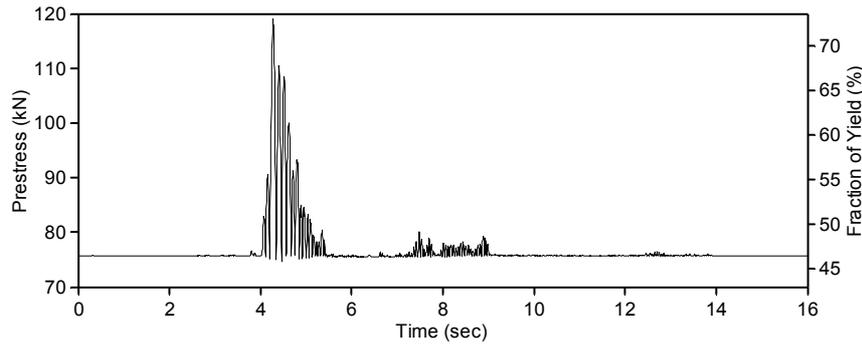**Figure 5 Wall 5 Top of Wall Displacement History (Sylmar)**

Figure 6 Wall 5 prestress history (Sylmar)

DISCUSSION

Testing of three rectangular post-tensioned concrete masonry walls demonstrated the desirable characteristics of a rocking wall when subjected to seismic excitation, achieving drift levels of at least 1% and resulting in no loss in strength and negligible residual displacements. The only rectangular wall to undergo masonry crushing in the lower corners, was wall 3 when it achieved lateral drifts in excess of 1.5%. This failure mode is desirable due to the ease in which repair can be conducted. The tendons remained within the elastic range throughout all tests, which is a result of designing walls with an appropriate level of initial prestress. The shake table testing of rectangular walls further validates the equations derived by Laursen and Ingham [12], which had previously been compared with cyclic test results.

Testing of wall 4 demonstrated the additional damping properties of a doweled joint, which can be observed by comparing the force-displacement curve with that of walls 3 and 5 which received the same suite of ground accelerations. Due to the low level of wall drifts, masonry crushing was not expected.

Testing of walls 2 and 6 resulted in more significant cracking, largely confined to the bond beam. Though the cracking was extensive, the crack widths did not exceed a couple of millimetres and the post-tensioning ensured that the residual displacements, as a result of lintel elongation, were minimal. Both walls were subjected to ground excitations in excess of the largest design demand in the IBC [13], before substantial wall damage occurred. In the case of wall 2, wall damage was due to crushing of the lower wall corners that could be easily repaired as was demonstrated by testing wall 3. Wall 6 did not show any significant loss in strength, until the prestress was reduced to a level that would result in an inefficient design, and the wall was subjected to an excitation greater than IBC design spectra.

FUTURE WORK AND CONSTRUCTION OF PCM HOME

Following the series of shake table tests reported here, a single storey simple PCM structure was built and tested on the shake table at NCSU, as shown in Fig. 7. The 2 m square building, made up of four external walls and one internal wall, consisted of two door and four window openings and two shrinkage control joints. The test objective was to investigate the performance of L and T corners and further investigate opening and wall connection performance. The structure was subjected to a series of ground motions at different prestress levels then rotated 90 degrees and the testing procedure repeated. Preliminary data analysis indicates that the structural elements performed as expected based on results from the tests presented in this paper. A more comprehensive account of the experimental results will be published in the near future.

Figure 7 Simple structure ready for testing

The final phase of this research program involves the design and construction of New Zealand's first PCM house. Design of the structure has not yet begun, however Fig. 8 shows the proposed example wall cross-section for a single storey PCM wall. In this situation standard 150 mm nominal width masonry units are used, placed upon an external reinforced concrete foundation beam. The steel layout in the foundation is as per NZS 4229 except for the addition of the post-tension anchorage, which consists of a Dywidag (DSI) Radiused Anchor and a coupler embedded in the floor slab to a depth of half the coupler height. The anchorage needs to be located accurately in the floor slab to ensure the tendon is located in the centre of the wall width, which is possible by tying the anchor bar on the mild reinforcing in the foundation, and by providing a means of holding the top of the coupler at the correct location. Upon completion of the floor, the wall can be constructed by stacking the blocks and ensuring that washouts are provided at the bottom of each cell that contains a tendon, to provide access to the coupler when installing the tendon. The top two courses require the installation of the bond beam reinforcement, and then upon completion of the blockwork, the PVC duct and tendon can be installed. It is recommended to provide a small amount of prestress to the wall, by way of tightening the nut using a wrench, to provide some out-of-plane bracing for the wall during the grouting process. Preliminary trials indicate that it is difficult to provide more than approximately 20 kN of prestress using a standard adjustable wrench due to the course threads. The wall is then partially grouted, filling the cells that contain tendons, around openings, at corners and in the block courses that contain the bond beam.

The wall is prestressed using a small hollow-core jack and hand pump with a calibrated in-line pressure gauge, to ensure that the required prestress force is obtained. After prestressing the additional threadbar beyond the nut can be cut off, the timber top plate installed and the subsequent construction of the remaining structure started.

Prestress losses due to creep, shrinkage and steel relaxation need to be considered in the design of PCM walls. Laursen [12] found that for a partially grouted wall, built with common New Zealand concrete masonry units, mortar and grout, prestress losses as high as 30% can be expected, of which approximately 50% of the loss occurs within the first 3 months. This can be accounted for by either restressing the wall 3 months after initial prestress or allowing for the expected losses when specifying the initial tendon prestress. In the case of prestress loss due to bar yielding during a seismic event, the wall could be restressed by

removing minimal roofing material to enable access to the top anchorage, and the previously explained stressing procedure carried out again. This is likely to be less demanding than the repair of damage to a comparable conventionally reinforced concrete masonry home.

Other considerations that still need to be addressed include the detailing for tendon corrosion protection and the performance of post-tensioned out-of-plane walls, which will be verified in part by dynamic testing but mainly through referencing of previous research on this topic.
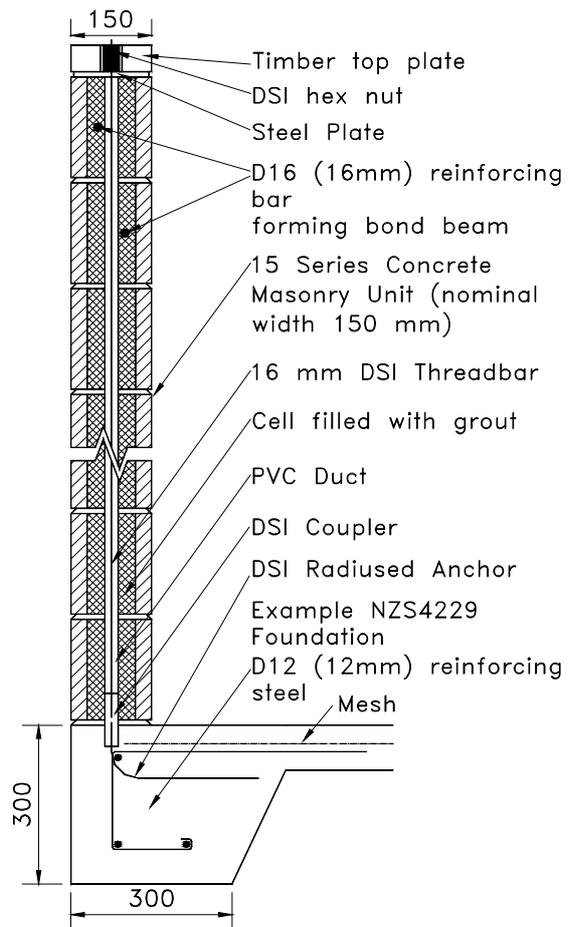

Figure 8 Proposed example wall cross-section

CONCLUSIONS

It is concluded that post-tensioned concrete masonry walls are able to withstand large seismic demands and undergo drifts up to 1% with little or no loss in strength or residual displacements.

Wall strengths can be accurately predicted using the equations previously derived by Laursen and Ingham.

The level of initial prestress is a key design parameter, not only to ensure that the tendon remains within the elastic range, but to ensure that the structure will not reach displacement levels that will result in masonry crushing.

Though energy dissipation is low for a rocking wall, the inclusion of such design features as a control joint, provide significant increases in damping, or reductions in wall displacements.

Repair of damaged walls can easily be carried out due to the typical concentration of damage in wall lower corners.

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