Auckland UniServices Limited

Our Ref: 11569

IN-PLANE CYCLIC TESTING OF FORMBLOCK® MORTARLESS CONCRETE MASONRY WALLS

INTERNATIONAL REPORT

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August 2006

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1 Executive Summary

In August 2005 W Stevensons & Sons Ltd, on behalf of Global Building Systems Ltd, approached the University of Auckland to test a fully grouted mortarless concrete masonry wall system referred to as Formblock®. Two Formblock® mortarless concrete masonry walls were tested to determine their in-plane cyclic performance. These walls are referred to herein as Commercial Wall 1 and Commercial Wall 2. This was a modification to the originally scheduled testing where the second wall test was to have imitated residential construction. Instead it was determined that Commercial Wall 1 incorrectly lacked consistency in the vertical reinforcement used as starter bars at the base of the wall. Consequently, due to the importance of commercial design, the second test was conducted as a repeat of the first test in order to rectify the reinforcement details.

This report outlines the procedure used to test the walls. At the client's request, only the results of the second test are reported herein, and are reported with reference to four masonry standards, NZS 4230:2004, BS 5628-2:2005, AS 3700:2001 and US masonry standard MSJC (2005). Testing indicated that the Formblock® wall system exhibited excellent displacement ductility capacity, substantially in excess of the maximum value of $\mu = 4$ that is stipulated in NZS 4230:2004. No significant slip was observed in the non-contact lap splices at the base of the wall, and the use of Formfill concrete was found to ensure masonry compression strengths well in excess of the values specified in NZS 4230:2004. Overall, the Formblock® concrete masonry system performed in a manner compliant with NZS 4230:2004, BS 5628-2:2005, AS 3700:2001 and US masonry standard MSJC (2005).

2 Introduction

Mortarless masonry is fast becoming an accepted form of masonry construction worldwide. The principal advantage of using this mortarless technology is the reduction in numbers and quality of registered masons needed during construction. The international trend of a declining number of young people electing to become registered masons is putting pressure on the availability of expertise in the masonry trade. This has underscored the growing opportunity for mortarless masonry. Construction time is reduced due to the absence of mortar joints, consequently reducing costs for both building contractors and building owners.

Mortarless masonry construction is not included in most concrete masonry design standards. The purpose of the following reported testing was to establish whether mortarless concrete masonry can be reliably designed using NZS 4230[1], US masonry standard MSJC[2], AS 3700[3], and BS 5628[4], with particular emphasis given to the grouting operation and to performance of the non-contact lap splice at the base of the Formblock® wall system.

A series of two mortarless concrete masonry wall tests were conducted, including the testing of six masonry prisms. The concrete masonry units (CMU's) used for the wall tests were Stevenson 20 Series Formblock® components which are shaped to provide a smooth surface at any interface. Fully grouted in-plane wall tests were conducted for the two walls. The results from the testing program on Commercial Wall 2 are reported herein.

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[1] NZS 4230:2004, *Design of Reinforced Concrete Masonry Structures*, Standards New Zealand, Wellington, 2004

[2] *Building Code Requirements for Masonry Structures* (ACI 530-05/ASCE 6-05/TMS 602-05), Masonry Standards Joint Committee, 2005

[3] AS 3700:2001, *Masonry Structures*, Standards Australia, Australia, Sydney, 2001

[4] BS 5628-2:2005, *Code of Practice for the use of masonry – Part 2: Structural use of reinforced and prestressed masonry*, British Standards Institution, London, 2005

3 Construction and Tests of Masonry Prisms

A total of six masonry prisms were constructed in two sets of three, referred to as Set 1 and Set 2, using 200 mm by 398 mm (7.89 in. by 15.67 in.) Formblock® CMU's filled with Formfill concrete. No shrinkage compensating admixture was needed and rodding provided sufficient compaction without the need for vibration. The Formfill concrete was pumped through a grout pump, see Figure 3-1.

3.1 Formfill Concrete Specifications

Formfill concrete is a structural 20 MPa (2900 psi) concrete with 10 mm (0.39 in.) aggregate specifically designed to fill Formblock® walls. It can be poured to a maximum height of 1.8 m (71 in.), after which the previous pour must stiffen before any additional Formfill can be added. It requires a spread of 500 – 600 mm (19.69 – 23.62 in.) as shown in Figure 3-1. Superplasticiser is added to help flow and to reduce water. Formfill concrete is designed to be sufficiently fluid to fill all voids, to bond together adjacent Formblock® units, and to bond reinforcement.

Figure 3-1 Spread test of Formfill and construction of prisms for testing

3.2 Testing of Masonry Prisms

The Set 2 prisms were tested at 28 days using the testing apparatus shown in Figure 3-2. It was observed that the masonry face shells did not delaminate from the inner concrete cores, but that cracking instead occurred across the prisms. This suggested that the superplasticiser was successful in reducing plastic settlement, eliminating the need and cost associated with using a shrinkage compensating admixture. The failure mechanism can be seen in Figure 3-3.

Resulting masonry crushing strengths from Set 2 prism tests were 18.4 MPa, 18.0 MPa and 17.2 MPa (2670 psi, 2610 psi and 2490 psi) for the three prisms respectively. The characteristic compressive strength of masonry (identified as f'_m in NZS 4230, AS 3700 and MSJC, and as f_k, in BS 5628) was calculated using nominal block dimensions specified in the Formblock® literature of 398 mm × 200 mm (15.67 in. × 7.89 in.). The mean masonry compressive strength for Set 2 was determined to be 17.9 MPa (2600 psi) for use in equations in NZ, US and British masonry standards. This was comfortably in excess of f'_m = 12.0 MPa (1740 psi) as stipulated in NZS 4230:2004 for observation type B masonry construction.

For comparison, the MSJC contains three different levels of quality assurance; A, B, and C, with A being the least stringent and limited to structures of lower importance and to non-structural masonry (i.e. veneers). Quality Assurance level B (of MSJC) is analogous to Observation Type B (of NZS 4230), however the former has no lower limit on the minimum masonry design compressive strength.

Using AS 3700[5], f'_m is calculated using a k_m value of 1.4, k_h of 1.3 (as there is no mortar, the ratio of masonry unit height to mortar bed thickness is >19 mm), and f'_{uc} was adopted as the measured mean masonry crushing strength (17.0 MPa). See AS 3700:2001, section 3.3.2.

k_m = a factor used to derive the characteristic compressive strength of masonry

k_h = a factor affecting the influence of the ratio of masonry unit height to mortar bed joint thickness

$$f'_m = k_h \times k_m \times \sqrt{f'_{uc}}$$
$$f'_m = 1.3 \times 1.4 \times \sqrt{17.9}$$
$$f'_m = 7.7 MPa$$

Figure 3-2 Test set-up of masonry prisms

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[5] $f'_m = k_h \times k_m \times \sqrt{f'_{uc}}$, where f'_{uc} is the characteristic unconfined compressive strength of masonry.

Figure 3-3 Failure mechanism of Set 2 prisms

4 Construction Details of Test Walls

Both test walls had a length of 2600 mm (102 in.) and a height of 2400 mm (94 in.), as shown in Figure 4-1. Vertical reinforcement was at unconventional centres and horizontal reinforcement of 3-D12 @ 600 mm c/c (3 deformed 12 mm (0.47 in.) diameter bars at 23.62 in. centres) was used. Commercial Wall 2 was vertically reinforced with 5-D12 bars (5 deformed 12 mm (0.47 in.) diameter bars).

5-D12 (5 deformed 12 mm (0.47 in.) bars) vertical starters were threaded into the precast concrete bases and a non-contact lap splice was used between these and the primary vertical reinforcement. The non-contact lap splice was not compliant with any of the masonry standards covered in this report, such that performance of this detail was an important test observation.

Masonry Standard:	Section on lap splices:
NZS 4230:2004[1]	C 6.3.9.1(d)
MSJC[2]	C 27 2.1.10.7.1.2
AS 3700:2001[3]	sections 4.11.4 or 12.3.5.3.1
BS 5628-2:2005[4]	C 8.6.7

Table 4-1 Masonry standard lap splice clauses

A conventional bond beam was constructed at the top of the wall. The vertical and horizontal reinforcement details, of $p_n + p_v = 0.2\%$[6], corresponded with the minimum reinforcement requirements of NZS 4230:2004, section 7.3.4.3(a), assuming an elastic or nominally ductile structure design philosophy. However, it is noted that for limited ductile or ductile structures in NZS 4230, the maximum spacing of reinforcement allowed is at 400 mm

6 $p_n = p_v = \frac{A_{sh}}{A_m} = \frac{\pi 12^2 / 4}{600 \times 190} \times 100 = 0.1\%$, where A_{sh} is the area of one D12 reinforcing bar and A_m is the associated cross-sectional area of masonry.

centres, (sections 7.4.5.1 and 7.4.5.2 of NZS 4230:2004). Hence the reinforcement details used in Commercial Wall 2 were conservative when considering design complying with these more demanding design philosophies. The geometry and reinforcement details of Commercial Wall 2 are shown in Figure 4-1.

The reinforcement details also corresponded with MSJC Seismic Design Categories E & F, C-18, 1.14.6.3, with AS 3700:2001 section 8.4(c), and with BS 5628-2:2005 section 8.6.3.

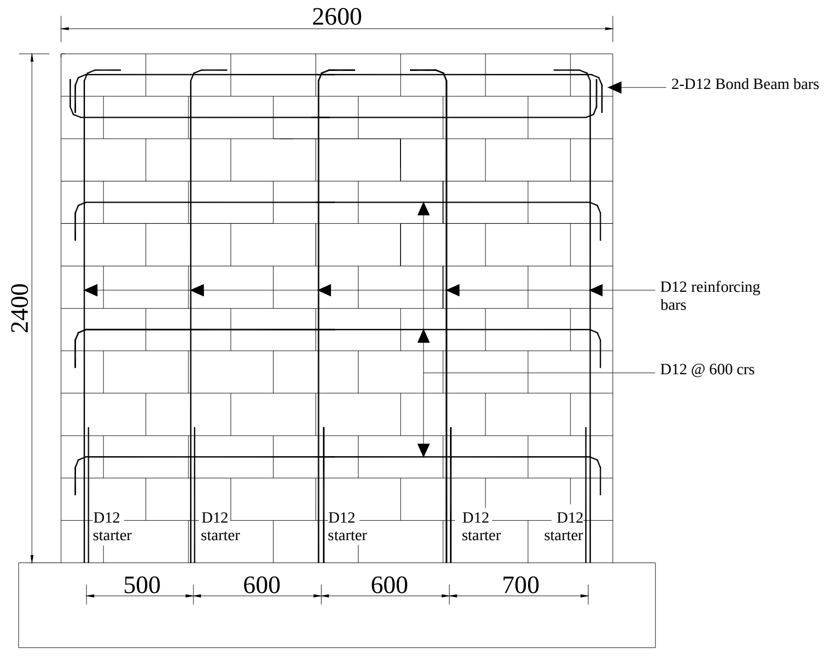

Figure 4-1 Geometry and reinforcement details of Commercial Wall 2.
(All dimensions in mm. 1000 mm = 39.37 in.)

Commercial Wall 2 was poured on Thursday November 17 2005. The wall was grouted using Formfill concrete as previously specified in section 3-1 of this report. Wall grouting proceeded without event, with no rupture of face shells or excess leakage of grout occurring during the operation.

The lower course of CMU's were laid on a bed of mortar. Plastic bridges, termed Form Bridges, were used in the construction of the wall. Their purpose was to join any two blocks together at their perpendicular ends and to ensure that the reinforcing steel was kept accurately and securely in position. This can be seen in Figures 4-2 and 4-4.

Figure 4-2 Wall construction showing Form Bridges and plastic shims

Small plastic shims were installed where required during construction to maintain the vertical alignment of the wall. They can be seen in Figures 4-2 and 4-3. No bonding agent was provided to any joints.

Figure 4-3 Plastic shims positioned at joints

Figure 4-4 Reinforcement alignment using Form Bridge

5 Test Set-up and Instrumentation

The set-up employed to test the mortarless concrete masonry walls in-plane was the same as has been used for numerous concrete masonry tests at the University of Auckland for public-domain research[7]. Lateral force was provided by a hydraulic actuator mounted horizontally at the height of the top surface of the wall. The actuator reacted against the laboratory's strong wall, and was attached to the test wall via a steel channel section. This channel was secured to the test wall by five D20 (deformed 20 mm (0.79 in.) diameter) threaded rods cast into the bond beam. Out-of-plane movement was restricted by two steel supports attached by pinned joints to the channel and a support frame. The wall was supported by a specially constructed base which was stressed to the strong floor of the laboratory. Details of the test set-up are shown in Figure 5-1.

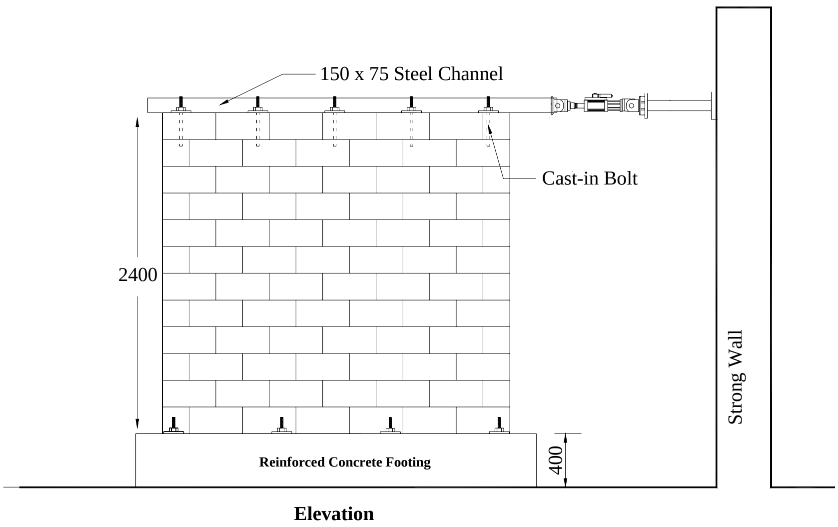

Figure 5-1 Test set up for Commercial Walls 1 & 2
(All dimensions in mm. 1000 mm = 39.37 in.)

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[7] Brammer, D.R., *The Lateral Force-Deflection Behavior of Nominally Reinforced Concrete Masonry Walls*, ME Thesis, University of Auckland, 1995, 271 p.

The walls were instrumented with multiple devices to measure overall force vs. displacement response, as well as relative deformations associated with sliding and uplift of the wall. The instrumentation used in the testing is indicated in Figure 5-2 and 5-3.

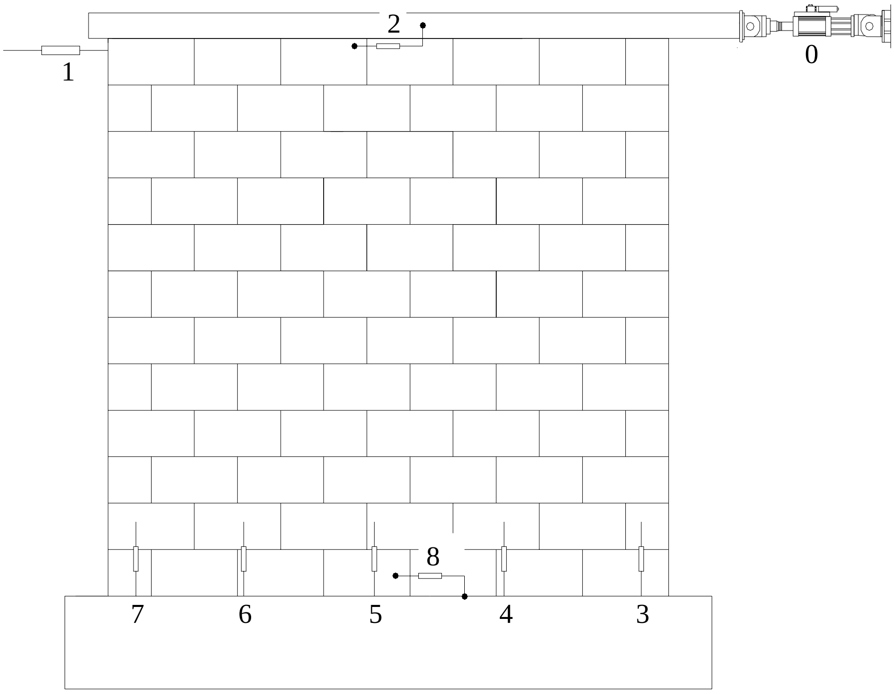

Figure 5-2 Instrumentation used for Commercial Wall 2

Figure 5-3 Instrumentation used to measure uplift at the base of the wall

6 Method of Testing

6.1 Review of Structural Seismic Design

For the purposes of designing structures to withstand lateral loads induced by earthquakes, a computational model may be developed where the characteristics of the structure can be identified using an equivalent single degree of freedom oscillator, which accurately captures the structure's mass at its centre of height, the structural period, and the plastic strength of the structure. Using such models, it is well established that the acceleration that the structure is subjected to during the design level earthquake is dependent on the period and ground conditions, which can be used to develop design spectra. An example of a seismic design spectrum is shown in Figure 6-1.

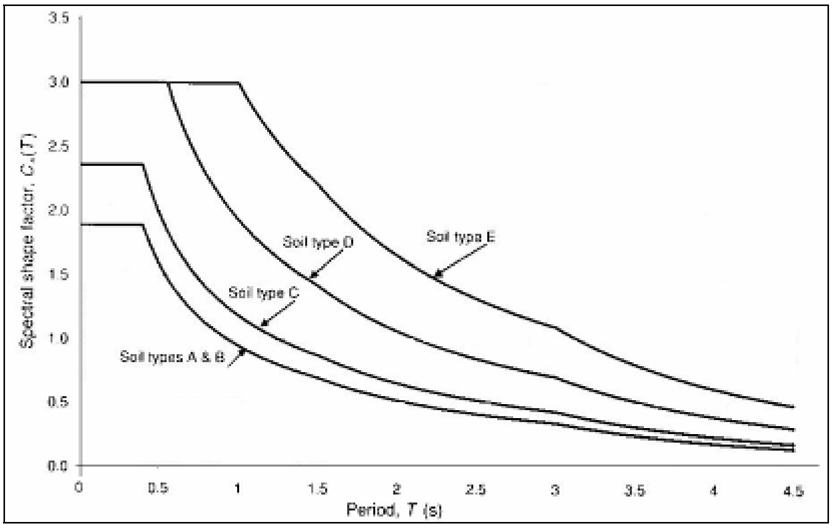

Figure 6-1 Spectral Shape Factor, $C_h(T)$ - General[8]

It is equally well understood that in order to perform satisfactorily during an earthquake, two different design approaches may be considered. Using an elastic design philosophy, the structure can be designed so that lateral forces

[8] NZS 1170.5:2004, *Structural Design Actions, Part 5: Earthquake Actions – New Zealand, p.12*, Standards Association of New Zealand, Wellington, 2004.

never exceed those required to cause yielding of structural components, which will generally result in designing for large lateral forces for structures located in highly seismic regions. However, if the structure can be shown to be able to reliably deform plastically (inelastically) following the onset of yielding, it is possible to design for substantially reduced lateral loads, resulting in more cost effective design.

The important parameter to quantify when assuming the structure may be able to deform plastically is to have confidence in the selected maximum lateral deformations that can be sustained before the strength of the structure begins to reduce. It is customary to consider lateral displacements, and to use the term ductility to describe the ratio of the maximum displacement measured before loss of strength was encountered, divided by the displacement at which yielding of the structure occurred.

The purpose of the study reported here was to experimentally verify the lateral displacement characteristics of the Formblock® system to confirm that it may be used with confidence to complete structural seismic designs consistent with the philosophy of NZS 4230:2005, US masonry standard MSJC, AS 3700:2001, and BS 5628-2:2005.

6.2 Adopted Method of Testing

The adopted method of testing was selected to ensure comparability with past masonry tests conducted at the University of Auckland, using the procedure established by Park[9]. The basis of this method is determination of the strength of the system being tested.

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[9] Park, R., *Evaluation of Ductility of Structures and Structural Assemblies from Laboratory Testing*, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol 22, No. 3, September 1989.

The nominal strength of the walls was determined using procedures outlined by Paulay and Priestley[10] as further detailed in sections 7 of this report. A serviceability earthquake loading was calculated. This was determined using the equation:

$$F_{service} = \phi . \frac{\mu}{1} . \frac{L_s}{L_u} \cdot F_n = 0.28 F_n \,,$$

where $\phi = 0.85$ is the strength reduction factor, $\mu = 2$ is the typical structural ductility factor, $L_u = 1$ is the ultimate limit state factor and $L_s = 1/6$ is the serviceability limit state factor. This serviceability earthquake load was determined using NZS 4230:2005 only, but is also satisfactory in an international context.

The loading cycle used for the tests is illustrated in Figure 6-1. This loading cycle was adapted from that presented by Park[9] and consists of several stages.

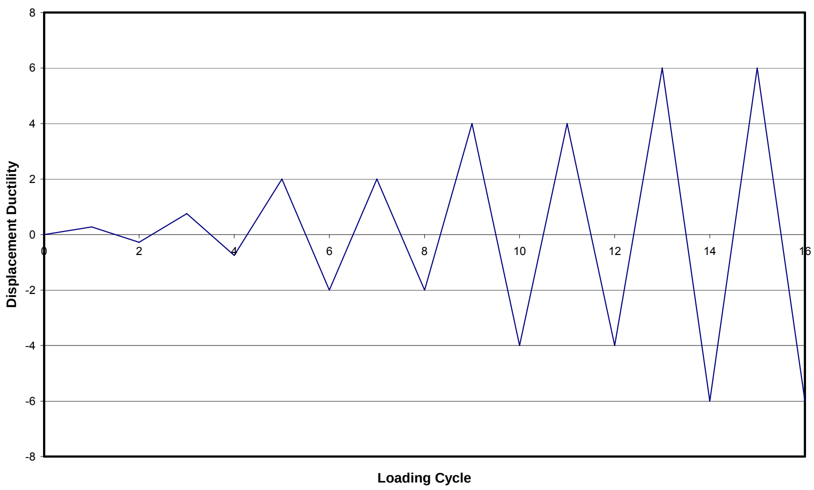

Figure 6-2 Planned loading cycle

1. The system was first taken to the serviceability level load in each of the push and pull directions.

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[10] Paulay, T. and Priestley, M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, 1992, John Wiley & Sons Inc

2. A load of $0.75F_n$ was applied in each direction. From this cycle an average displacement was extrapolated to find the ductility one displacements.
3. The structure was then taken through two full ($\pm$) cycles to ductility 2, then two to ductility 4, proceeding to increase the displacement in increments of ductility 2 until failure occurred. Failure was defined as having occurred when the lateral force sustained by the structure falls below 80% of the peak value sustained by the structure or when rupture occurred.

6.3 Definitions Used in Test Descriptions

Push/Pull Cycle

Describes whether the actuator was pushing or pulling the test wall relative to the reaction wall.

Positive/Negative Cycle

A positive cycle has been defined as one in which the actuator is pushing the test wall away from the reaction wall. It follows that a negative cycle is one in which the actuator was pulling the test wall towards the reaction wall. Tests were begun with a positive (push) cycle.

Actuator End/Gauge End

The actuator end is defined as the end of the wall to which the actuator is attached. The gauge end is therefore the end to which the actuator is not attached. During a push cycle, the base of the test wall at the gauge end was in axial compression and the base of the test wall at the actuator end experienced axial tension.

7 Commercial Wall 2

This section describes the testing of Commercial Wall 2, which was a fully grouted wall that was 2400 mm (94.5 in.) high and 2600 mm (102.3 in.) long. The wall was subjected to in-plane pseudo-static cyclic loading that simulated earthquake effects. Commercial Wall 2 was tested on Thursday December 15, being 28 days after grouting. The ductility one displacements were determined to be $\mu_{1\ [Push]}$ = 0.8 mm (0.031 in.), and in the pull direction, $\mu_{1\ [Pull]}$ = 0.7 mm (0.028 in.). Unfortunately the development of a crack at the top of the wall influenced the displacements that the wall was subjected to, such that the intended load history was not followed. It is emphasised that this did not influence the validity of the findings from this test.

7.1 Nominal Wall Strength

The masonry compression strength was determined from prism test results as detailed in section 3, as 17.9 MPa (2600 psi) for the Formblock® concrete masonry, according to NZS 4230, MSJC and BS 5628. f'_m was determined to be 7.7 MPa under the provisions of AS 3700. The reinforcement had a nominal yield strength of f_y = 300 MPa (43,500 psi).

For a wall height of 2400 mm (94.5 in.), a wall length of 2600 mm (102.3 in.), a wall thickness of 190 mm (7.5 in.) and a wall density of 3.6 kN/m^3 (130.8 × 10^6 lb/in.^{3}) the self weight of Commercial Wall 2 was calculated as W_t = 22.46 kN (5050 lb).

Under the provisions of NZS 4230, the calculated nominal flexural strength in the push direction was M_n = 231.1 kNm, corresponding to a lateral force of F_n = 96.3 kN. The corresponding serviceability lateral force was 27.0 kN and the lateral force at $0.75F_n$ was 72.2 kN. In the pull direction the calculated flexural strength was M_n = 251.5 kNm, with a corresponding lateral force of F_n = 104.8 kN. The serviceability lateral force was 29.3 kN and the lateral force at $0.75F_n$ was 78.6 kN.

Under the provisions of AS 3700:2001, the calculated nominal flexural strength in the push direction was M_n = 237.3 kNm, corresponding to a lateral force of F_n = 98.8 kN. The lateral force at $0.75F_n$ was 74.1 kN. In the pull direction the calculated flexural strength was M_n = 258.2 kNm, with a corresponding lateral force of F_n = 107.5 kN. The lateral force at $0.75F_n$ was 80.6 kN.

Under the provisions of BS 5628-2:2005, the calculated nominal flexural strength in the push direction was M_n = 206.4 kNm, corresponding to a lateral force of F_n = 86.0 kN. The lateral force at $0.75F_n$ was 64.5 kN. In the pull direction the calculated flexural strength was M_n = 237.4 kNm, with a corresponding lateral force of F_n = 98.9 kN. The lateral force at $0.75F_n$ was 83.55 kN.

Under the provisions of MSJC (2005), and using a strength based design approach (chapter 3), the calculated nominal flexural strength in the push direction was M_n = 2076 kip-in., corresponding to a lateral force of F_n = 22.1 kips. The lateral force at $0.75F_n$ was 16.6 kips. In the pull direction the calculated flexural strength was M_n = 2226 kip-in, with a corresponding lateral force of F_n = 23.7 kips. The lateral force at $0.75F_n$ was 17.8 kips.

Appendix A shows how the calculated nominal flexural strength was derived for each masonry design standard.

7.2 Commercial Wall 2 Testing

Prior to Testing

A preliminary investigation of the wall revealed that numerous joints were open to a width of up to 3 mm (0.12 in.) as can be seen in Figure 7-1. Both wall faces appeared to be in a similar condition.

Figure 7-1 Openings in joints on wall before testing

Serviceability

F = 33.0 kN (7418 lb), Δ = 0.07 mm (0.0027 in.):

There was nothing to report.

F = -37.1 kN (-8340 lb), Δ = -0.04 mm (0.0016 in):

There was nothing to report.

¾ F_n

F = 72.5 kN (16298 lb), Δ = 0.61 mm (0.024 in.):

There was nothing to report.

F = -78.1 kN (-17557 lb), ∆ = -0.53 mm (0.021 in.):

A very small base crack had formed, having a width of less than 1 mm (0.039 in.).

First Inelastic Cycle

F = 74.5 kN (16748 lb), ∆ = 0.74 mm (0.029 in.):

A diagonal hairline crack had formed at the top corner of the wall on the gauge end which can be seen in Figure 7-2. A base crack had formed, less than 1 mm (0.039 in.) wide.

Figure 7-2 Crack at top corner on gauge end

F = -112.4 kN (-25268 lb), ∆ = -1.58 mm (0.062 in.)

The base crack had extended to almost ¾ of the length of the wall.

Second Inelastic Cycle

F = 35.2 kN (7913 lb), ∆ = 0.04 mm (0.0015 in.)

There was nothing to report.

F = -114.5 kN (-25740 lb), Δ = -2.06 mm (0.081 in.)

There was nothing extra to report except that the base crack was now open 1 mm (0.039 in.).

Third Inelastic Cycle

F = 48.7 kN (10948 lb), Δ = 0.08 mm (0.0031 in.)

The top crack had extended to the second block down and was still less than 1 mm open.

F = -127.3 kN (-28618 lb), Δ = -5.53 mm (0.217 in.)

The base crack was now open about 4 mm (0.157 in.). However, all joints above looked to be in a satisfactory condition.

Fourth Inelastic Cycle

F = 72.0 kN (16186 lb), Δ = 0.63 mm (0.025 in.)

The base crack now ran the full length of the wall in both directions, as shown in Figure 7-3.

Figure 7-3 Base crack along total length of wall from both directions

The diagonal crack at the top corner of the wall had opened to approximately 4 mm (0.157 in.). This can be seen in Figure 7-4.

Figure 7-4 Crack at top corner now opened to 4 mm

F = -116.7 kN (-26235 lb), Δ *= -6.69 mm (0.263 in.)*

There was nothing to report.

Fifth Inelastic Cycle

F = 98.1 kN (22053 lb), Δ *= 8.00 mm (0.315 in.)*

Cracking at the base now appeared approximately symmetric in both directions.

F = -73.9 kN (-16613 lb), Δ *= -6.96 mm (0.274 in.)*

Sixth Inelastic Cycle

F = 62.2 kN (13983 lb), Δ = 10.17 mm/13.67mm (0.400 in./0.538 in.)

A loud noise was heard and a drop in strength occurred. It was later confirmed that the starter bar had ruptured.

F = -63.99 kN (-14385 lb), Δ = -12.0 mm (0.472 in.)

Rupturing of one of the bars was again encountered.

Seventh Inelastic Cycle

F = 32.25 kN (7250 lb), Δ = 13.43 mm (0.529 in.)

31.2 16.9 -37.6

Another bar ruptured. The test was terminated.

7.3 Experimental Results

Figure 7-6 shows the force-displacement history of Commercial Wall 2. Maximum strengths of 98 kN (22.03 kips) for the push direction (positive quadrant) and 127 kN (28.55 kips) in the pull direction (negative quadrant) were measured. This compares favourably with the calculated nominal flexural strengths.

Masonry design code	Measured strength		Calculated strength		Ratio of strength, measured/calculated	
	Push	Pull	Push	Pull	Push	Pull
NZS 4230:2004[1]	98.0 kN	127.0 kN	96.3 kN	104.8 kN	1.02	1.21
MSJC[2]	22.0 kips	28.6 kips	22.1 kips	23.7 kips	1.00	1.21
AS 3700:2001[3]	98.0 kN	127.0 kN	98.8 kN	107.6 kN	0.99	1.18
BS 5628-2:2005[4]	98.0 kN	127.0 kN	86.0 kN	98.9 kN	1.14	1.28

Table 7-1 Wall flexural strengths for different masonry standards

In the push direction the wall survived a cycle to a ductility of 8.00/0.80 = 10.00. In the pull direction the wall survived a cycle to a ductility of -6.69/0.70 = 9.60.

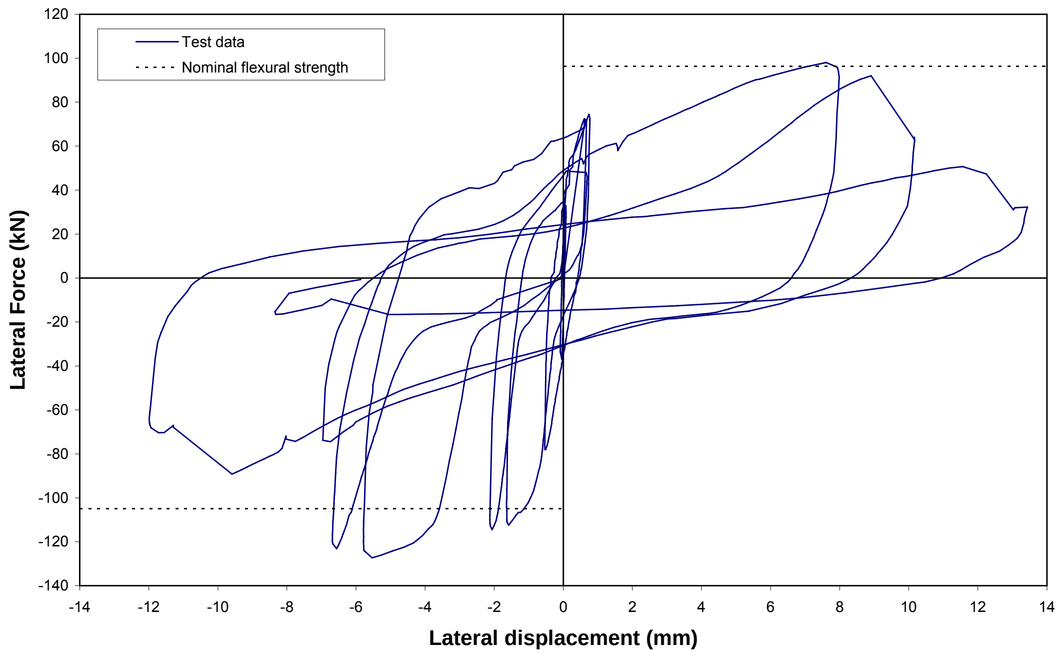


Figure 7-5 Displacement history of Commercial Wall 2 (metric)

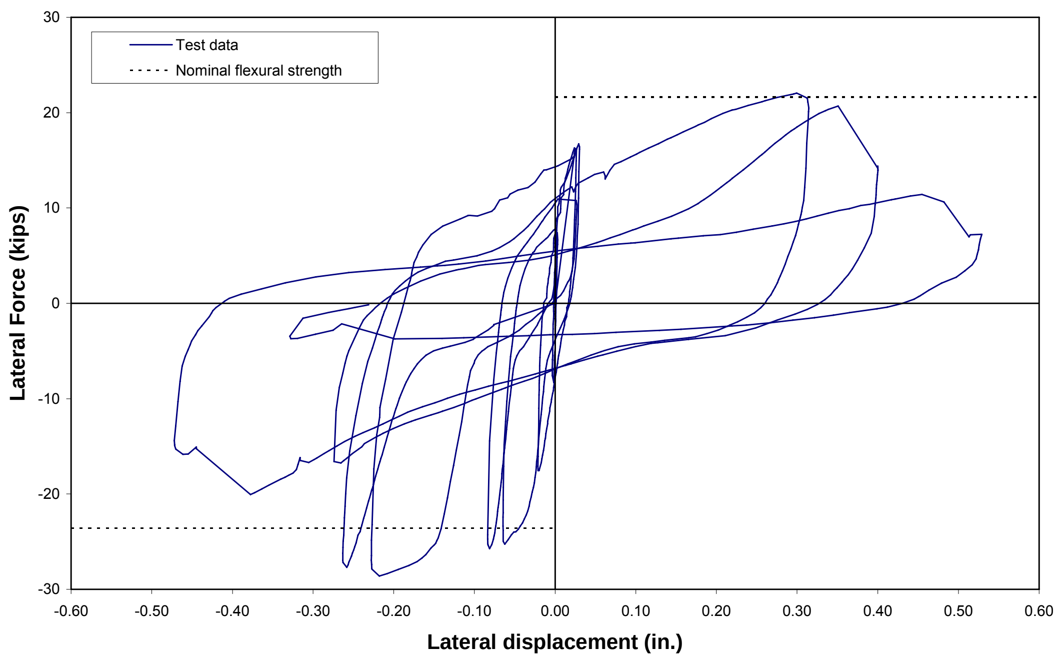


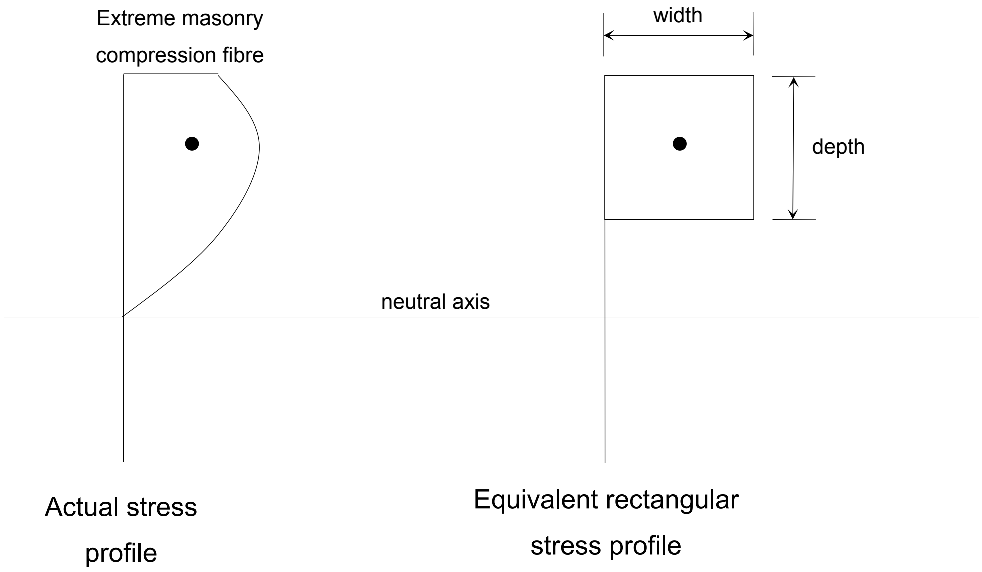
Figure 7-6 Displacement history of Commercial Wall 2 (US customary units)

8 Conclusions

1. The use of Formblock® CMU's in conjunction with Formfill concrete allowed the system to develop sufficient compression strength to satisfy all relevant material design standards.
2. Formfill concrete does not require an expansive shrinkage compensating admixture.
3. The addition of superplasticiser achieved its intended function of reducing plastic settlement.
4. Vertical alignment of wall and accurate placement of reinforcement was achieved without difficulty using Form Bridges.
5. Displacements at the serviceability limit state were insignificant, indicating satisfactory response.
6. Measured wall strength corresponded with nominal flexural strength calculations using each design code.
7. There was no evidence of slip or hairline cracking around the non-contact splice at the base of the wall.
8. Maximum displacement ductilities of approximately 10 were measured for both directions of loading prior to the onset of strength loss.
9. Overall, the Formblock® concrete masonry system performed in a manner compliant with NZS 4230:2004, BS 5628-2:2005, AS 3700:2001 and MSJC.

9 Appendix A

Equivalent masonry rectangular stress blocks:



Masonry design code	Equivalent rectangular stress block		Maximum useable strain at extreme masonry compression fibre
	Width	Depth	
NZS 4230:2004[1]	$0.85 \times f'_m$	$0.85 \times c$	0.0030
MSJC[2]	$0.80 \times f'_m$	$0.80 \times c$	0.0025
AS 3700:2001[3]	$0.85 \times 1.3 \times f'_m$[5]	$0.85 \times c$	0.0035
BS 5628-2:2005[4]	f_k/γ_{mm}	$1.0 \times c$	0.0035

Where:

f'_m = characteristic masonry compressive strength, derived according to the appropriate masonry design standard

f_k = characteristic masonry compressive strength, derived according to the appropriate masonry design standard

c = depth from extreme compression fibre to neutral axis

γ_{mm} = partial safety factor for strength of reinforced masonry in direct compression and bending[11].

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[11] Table 7, BS 5628-2:2005