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Civil Engineering Research Report

Lateral load resistance of AFS wall panels

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2.0 SUMMARY

A series of five test specimens were produced from the AFS150 structural wall system. The specimens were designed to investigate the influence of wall length, longitudinal reinforcement content, and horizontal reinforcement content. The walls were subjected to an increasing level of reverse cyclic loading. A theoretical analysis was undertaken to predict the behaviour of the reinforced concrete walls.

The experimental response of the test specimen behaved in a ductile manner. All of the test specimens exceeded displacement ductility of 6 before failure occurred. Failure of the specimen was defined as a significant drop in the lateral load carrying capacity of the test specimen. Failure occurred in every specimen due to rupture of the starter bars at the interface between the wall panels and the foundation block. The results from the experimental investigation indicated that no shear deformation occurred in the test specimen and that the AFS wall system behaved in a predictable, ductile manner.

A strong correlation was achieved between the experimental results and the theoretically derived response. Based on this finding it was concluded that the flexural performance of the AFS150 panels could be predicted using conventional reinforced concrete theory and analysis techniques.

The shear reinforcement requirements for the AFS wall panel systems were adequately predicted using the AFS design method (modified Australian) and the requirements of the New Zealand Concrete Structures Standard, NZS3101: 1995. It was recommended that conservative estimates of β_4 and β_5 developed by AFS be adopted to account for temperature derived shrinkage and tensile effects in the wall panels.

Test specimen AFS5 was found to undergo minor shear failure at a lower load that estimated by either the New Zealand Concrete structures standard or the AFS design approach. It is believed this was due to the large height to length ratio of specimen AFS5. It is recommended that the height to length ratio of the wall panels does not exceed 1.0

3.0 INTRODUCTION

The Department of Civil Engineering at the University of Canterbury was contracted by Architectural Framing Systems (AFS) to evaluate the performance of the AFS Structural wall panel system. The University of Canterbury acted as an independent testing facilitator and conducted tests on the specimens provided by AFS.

The AFS structural wall system is a permanent formwork structural wall panel. It consists of lightweight sandwich panels created by bonding fibre cement sheets to galvanised steel stud frames. The panels are reinforced with conventional reinforcing steel and in-filled with concrete. The wall panels are intended to be used as a lateral load resisting, structural wall system in New Zealand for both domestic and commercial construction. The performance of the wall systems under reverse cyclic loading is unknown. In addition, it was uncertain as to the effect the vertical steel members in the wall panels would have on the shear performance of the walls.

This report provides a summary of the testing completed on the AFS wall system. It presents and analyses the experimental results and provides a comparison of the results with theoretical models.

Section 4 of this report outlines the design and detailing of the test specimen. Included in the chapter is a summary of the testing equipment used and the reverse cyclic loading scheme imposed on the specimen. Section 5.0 of this report presents the experimental results and provides a comparison of the results to the theoretical modelling. A detailed discussion and analysis of the test results is presented in Section 6.0. This includes a comparison of the performance of the AFS test specimen to the requirements of the New Zealand Concrete Structures Standard, NZS3101: 1995. A series of conclusions and recommendations are presented in Chapter 7.0.

4.0 DESIGN AND DETAIL OF TEST UNITS

A series of five wall units were constructed from AFS150 permanent formwork wall panels and subjected to an applied horizontal load in the plane of the wall. The dimensions, reinforcement contents and layouts, and material strengths of the wall units were designed to evaluate the flexural and shear performance of the wall panels for use in New Zealand.

4.1 AFS Permanent Wall Panels

The AFS structural wall system is a permanent formwork structural wall panel. It consists of lightweight sandwich panels created by bonding fibre cement sheets to galvanised steel stud frames. The panels are reinforced with conventional reinforcing steel and in-filled with concrete.

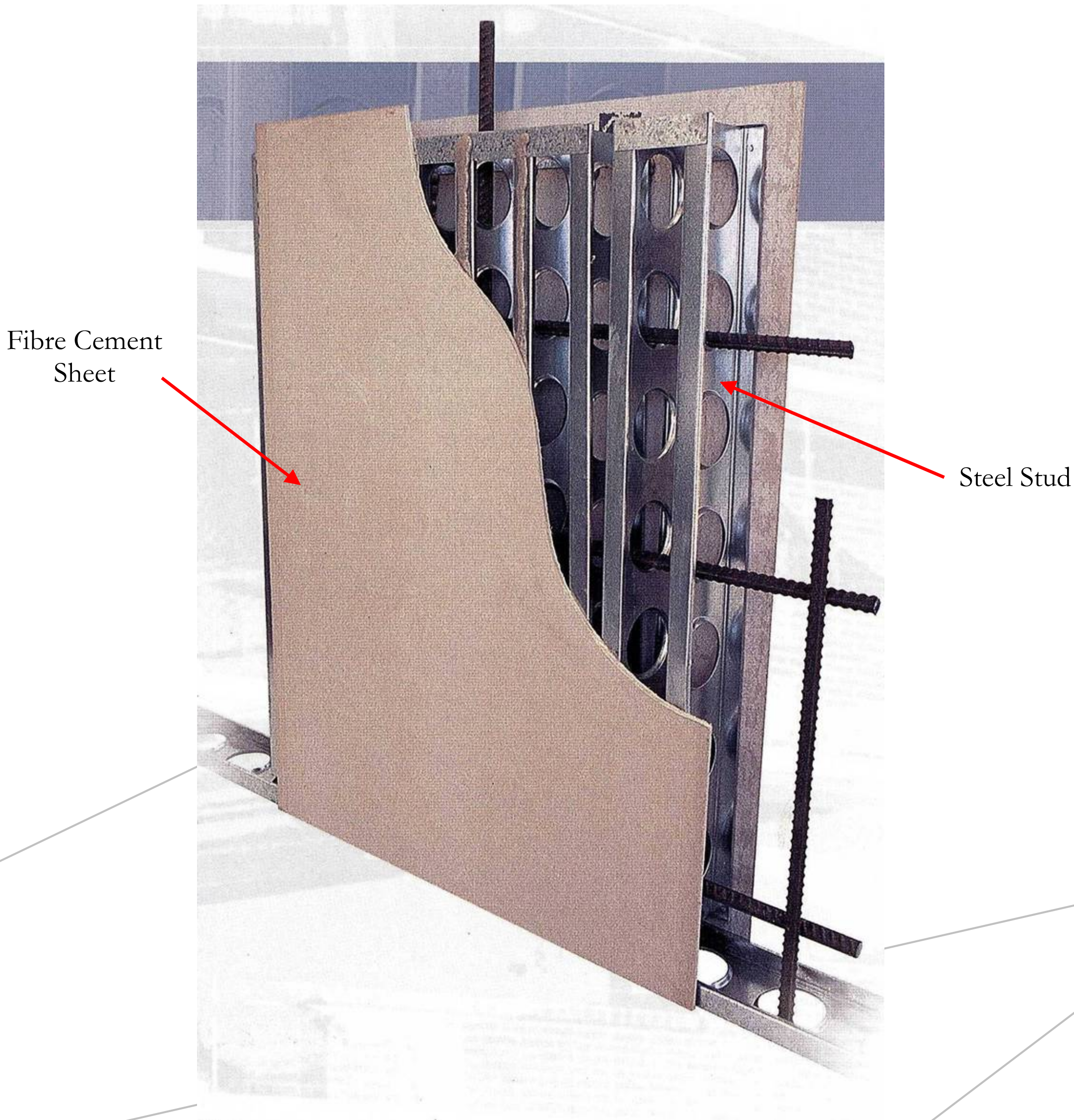

Figure 1 *Elevation of the AFS permanent formwork wall panel system*

The fibre cement sheeting is 6 mm thick with recessed edging to allow for ease of finishing. The sheets are bonded to the galvanised steel studs using a proprietary adhesive. No additional information was provided regarding the material or mechanical properties of the adhesive or fibre cement sheets.

The steel studs used in the panels are custom made from Grade G300 in accordance with AS1397. The steel has a nominal yield strength of 300 MPa and a nominal ultimate yield capacity of 340 MPa. The studs have punched holes through the webs to allow for placement of horizontal reinforcement and for the concrete to flow between stud lines. The pattern, size and location of the holes are varied depending on the thickness of the wall panel.

Testing was undertaken on the AFS150 wall panel system. The AFS150 has a nominal thickness (measured to the outside of the fibre cement sheets) of 150 mm and has vertical steel studs at 110 mm centres. The cross sectional properties of the steel studs used in the wall panels are shown below in Table 1.

Table 1 *Properties of the steel studs used in the AFS150 wall panel*

Type	BMT (mm)	t_w (mm)	A_{stud} (mm^2)	x_c (mm)	y_c (mm)	I_{xx} (x 10^3 mm^4)	I_{yy} (x 10^3 mm^4)	r_x (mm)	r_y (mm)
AFS150	0.55	136	117.2	7.64	68.0	306.4	15.85	51.13	11.63

Where

BMT = thickness of steel used for the stud

t_w = thickness of the wall measure to the inside of the fibre cement sheets

A_{stud} = cross sectional area of the steel stud

x_c = distance to the centroid of the steel stud in the strong axis

y_c = distance to the centroid of the steel stud in the weak axis

I_{xx} = second moment of area of the steel stud in the strong axis

I_{yy} = second moment of area of the steel stud in the weak axis

r_x = radius of gyration in the strong axis

r_y = radius of gyration in the weak axis

The webs of the steel studs were punched with a nominal 90 mm diameter circular hole at 150 mm centres. The punching profile produced a flared hole which is designed to increase the shear friction coefficient between the steel stud and the concrete. A profile of the flare is shown in Figure 2 below. The punching in the steel studs of the AFS150 wall panels produces a concrete contact area through the stud of approximately 30.5%.

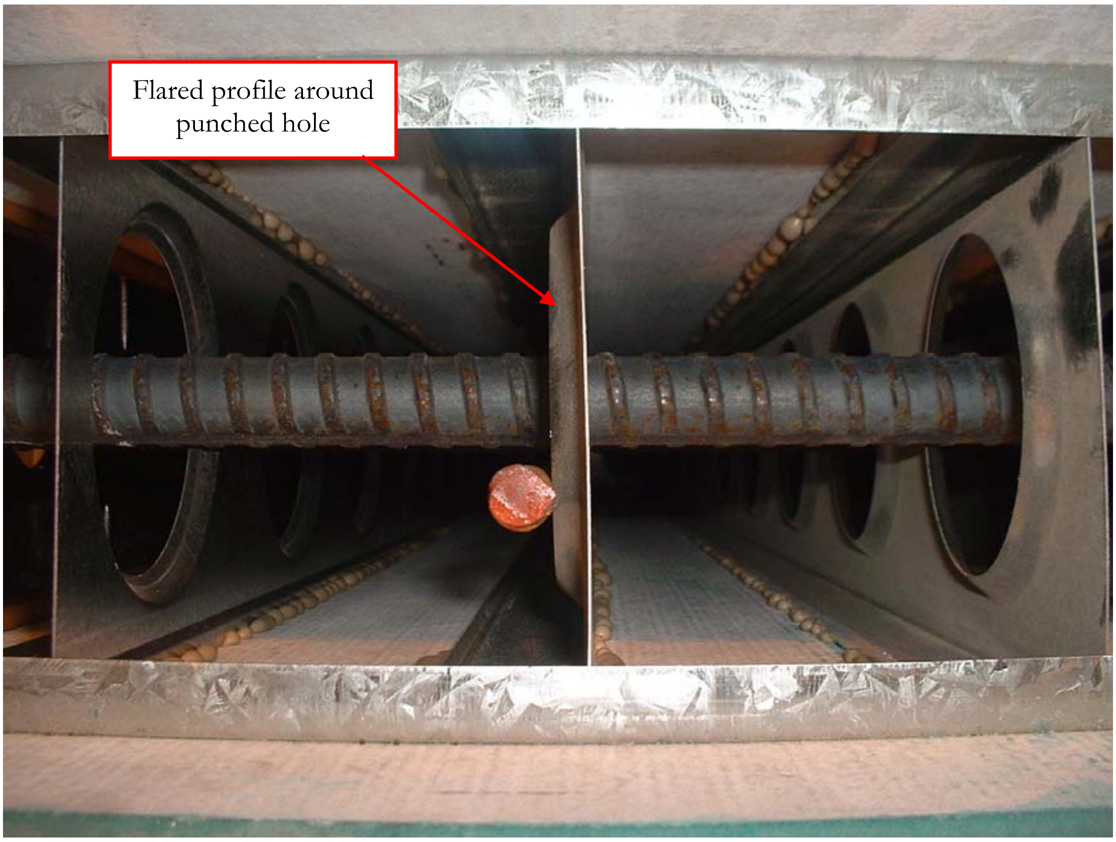

Figure 2 *Flared profile in the steel stud*

The AFS permanent formwork panels were reinforced with conventional deformed reinforcing bars before being filled with concrete. The details of the reinforcement and concrete are outlined in the following sections.

Further information regarding the AFS permanent formwork wall panels is available in the *AFS Structural Wall Technical Manual* [A1].

4.2 Design and Construction of Test Specimen

A series of five AFS permanent formwork concrete walls were constructed from AFS150 wall panels. All of the wall panels were designed as 2400 mm tall, to represent the height of conventional walls used in light commercial and domestic construction in New Zealand. The overall width of the walls was 150 mm, with an internal concrete core of 135 mm. The as-built dimensions of the walls and reinforcing steel contents of the walls are shown in Table 2.

Table 2 *Dimension of the AFS wall specimen*

Label	Wall Height (mm)	Wall Length (mm)	Starter Bar Reinforcement (mm)	Vertical Reinforcement (mm)	Horizontal Reinforcement (mm)
AFS1	2400	3600	XD12 @ 600	XD12 @ 600	XD12 @ 600
AFS2	2400	2500	XD12 @ 300	XD12 @ 300	XD12 @ 600
AFS3	2400	2500	XD12 @ 300	XD12 @ 300	-
AFS4	2400	2500	XD12 @ 300	XD12 @ 300	XD12 @ 300
AFS5	2400	1500	XD12 @ 300	XD12 @ 300	XD12 @ 300

The walls were constructed on a foundation base block to simulate the effect of being attached to a reinforced concrete beam in a building or a to foundation beam. The foundation base blocks were designs with a width of 900 mm and a height of 400 mm. The dimensions of the foundation blocks were sized to allow attachment to the strong frame used to apply the load to the walls. Each of the foundation blocks were reinforced with four XD20 longitudinal reinforcing bars top and bottom and XR12 stirrups at 100 centres. An XD reinforcing bar is a 20 mm diameter deformed reinforcing bar with a lower characteristic yield strength of 500 MPa. XR12 reinforcing bars are a plain round bar of 12 mm in diameter with a lower characteristic yield strength of 500 MPa. The reinforcing in the foundations beams was designed to remain elastic during the testing of the wall panels.

A series of 12 mm diameter starter bars were cast into the foundation beams at the desired centres. Each of starter bars extended 560 mm from the top surface of the foundation beam in accordance with the non-contact lap splice requirements of the New Zealand Concrete Structures Standard, NZS3101: 1995 [N1]. The spacing of the starter bars for each of the test specimen are shown in Table 2 above.

The walls were constructed by firstly securing a metal tray section to the top surface of the foundation beams using ramset power driven nails. Care was taken to ensure the starter bars extending from the top surface of the foundation beams were located in the holes of the tray and that the tray was aligned correctly with the foundation beam. A proprietary adhesive product was placed on the up-stands of the tray and then the AFS150 wall panels were lowered onto the trays. The AFS150 panels were supplied in maximum lengths of 1200 mm and were joined to form the required wall lengths. The vertical joints between the 1200 mm long panels were both glued using a proprietary adhesive and posi-drive screws.

Horizontal reinforcement was placed inside the wall panels, at the desired centres, by feeding the bars through the voids in the steel studs, as shown in Figure 3. Once all of the horizontal reinforcement was installed, solid end caps were glued and screwed to the ends of the wall panels to prevent concrete from leaking out during pouring.

Figure 3 *End view of the panels, showing voids in which the horizontal reinforcing was placed*

The vertical reinforcement was placed in the walls by feeding the bars into the wall panels from the top. Care was taken to ensure the vertical reinforcement was held centrally in the wall by capturing it between the horizontal reinforcement, as shown in Figure 4.

Two 25 mm diameter threaded reinforcing bars were placed at 150 mm nominal centres in the top of the wall panels. These rods extended 300 mm outside of the wall length and were used to attach the hydraulic actuator to the wall panels. This method of attachment was chosen as it simulated the effect of the top of the wall being attached to a floor or roof diaphragm with the load being applied to the end of the wall.

Figure 4 *Vertical Reinforcing bar being held central in wall width by the horizontal reinforcing*

The AFS150 permanent formwork wall panels are usually filled with concrete in maximum lift heights of 1500 mm. Due to time constraints it was decided to pour the test specimens to the full height of 2400 mm in a single pour. As a precaution a layer of 15 mm plywood was attached to the outside of the wall panels to prevent bursting of the formwork due to the additional hydraulic pressure resulting from the increased height of wet concrete. The 15 mm plywood was removed from the specimen the day after pouring.

A specifically designed 13 mm aggregate concrete mix was in-filled into the AFS150 wall panels using a concrete pump. The concrete was mixed by a local ready-mix concrete supplier. A pencil vibrator was used to vibrate the top of the wall panels to remove any air voids. A series of 12 concrete test cylinders, 100 mm diameter x 200 mm tall, were constructed from the concrete.

The mould work was removed from the cylinders after 24 hours and the specimens placed in the 'fog room' to be wet cured at a temperature of 21°C and relative humidity of 100% until required for testing.

4.3 Material Properties

A total of three steel coupons were randomly selected from each type of reinforcement used during the experimental testing programme. The individual steel coupons were approximately 900 mm long and were subjected to a quasi-static tensile load. The stress-strain response from the individual steel coupons was recorded using a modified version of the UDL data acquisition system, which was developed at the University of Canterbury. All axial strains and displacements were recorded over a gauge length of 50 mm.

The results from the three coupons taken from each type of reinforcement were found to be very consistent, with no results differing by more than ±4%. The recorded values of yield strength, f_y, yield strain, ε_y, ultimate tensile stress, f_u, and the ultimate uniform strain, ε_u, were averaged for each type of reinforcement and the average values were taken as the true material properties of the reinforcement. Table 3 represents the averaged material properties of the individual reinforcements used in each type of tested concrete column. The reported percentage elongation is a total percentage elongation measured over a set gauge length in the ruptured potion of the reinforcing bar. The Steel I.D is related to the manner in which reinforcing bars are labelled in the construction industry in New Zealand.

Table 3 *Main Characteristics of the Steel Properties*

Steel I.D	f_y (MPa)	ε_y (x1000)	Modulus of Elasticity (GPa)	f_u (MPa)	ε_u (x1000)	f_y/f_u
XD12	548	3.58	193	638	193	1.16

The concrete mix used for the AFS150 wall panel testing is shown in Table 4 below. The mix was developed by AFS and produced by a local ready mix concrete supplier. The target compressive strength of the mix was 30 MPa at 28 days.

A total of three 100 mm diameter concrete test cylinders, constructed from the concrete used in the wall panels were tested on the same day as the wall units to determine the strength of the concrete in the walls. The results from the three tests were averaged to determine the strength of the concrete.

Table 5 presents the average concrete compressive strength for each of the test specimen and the age at which the cylinders were tested.

Table 4 *Concrete Mix Design*

Component	Quantity
13 mm aggregate	620 kg
Medium Sand	1020 kg
Cement	335 kg
Fly ash	135 kg
Super plasticiser	1.25 litres (ADVA 125)
Water	145 litres

Table 5 *Concrete Compressive strength determined from 100 mm diameter test cylinders*

Specimen I.D	Concrete Age (Days)	Compressive Strength, f'_c (MPa)
AFS1	46	27.0
AFS2	53	30.0
AFS3	53	30.0
AFS4	56	31.0
AFS5	58	32.0

4.3 Instrumentation

The load was applied to the AFS150 wall units by a 300 kN double acting actuator with approximately ± 500 mm of travel. A 300 kN load cell was attached to the actuator providing a digital record of the load applied to the wall units. The displacement at the top of the wall was recorded using 200 mm travel rotary potentiometer. The location of the potentiometer and load cell is shown in Figure 5.

An array of 30 mm travel linear potentiometers was placed on one face of the wall units to determine the location of any deformations that occurred in the wall. In addition, a 50 mm travel linear potentiometer was mounted on the foundation block at either end of the wall to record any relative horizontal movement that may have occurred between the wall and the foundation block. Figure 5 provides a graphical representation of the location of the potentiometers on the test specimen.

Three 16 bit serial boxes were used to log the information provided from the instrumentation on the AFS150 wall units. A Compaq Evo N800v laptop computer was installed with the Universal Data Logger (UDL) software and used as the logging computer.

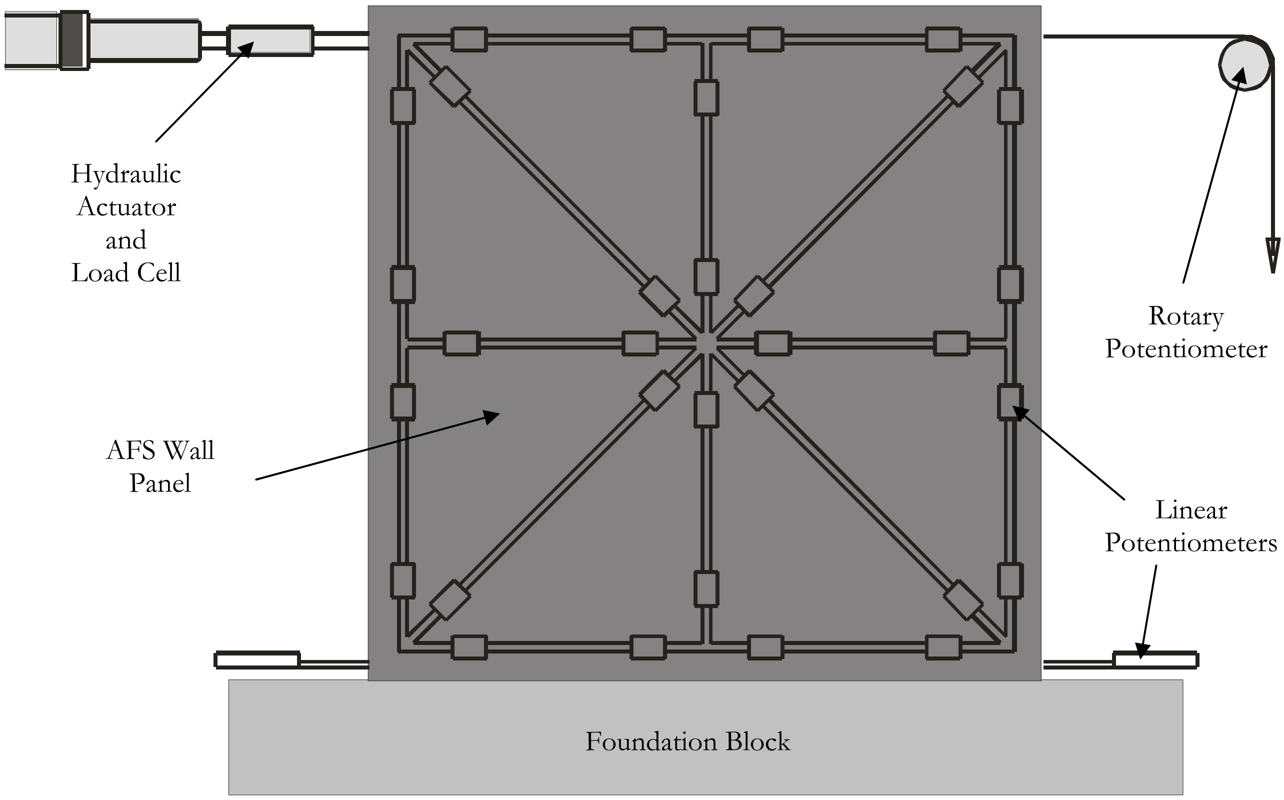

Figure 5 *Instrumentation location on AFS150 wall units*

4.4 Test Setup

A self equilibrating reaction frame was used to brace the hydraulic actuator when applying the load to the AFS150 wall units. The foundation block of the test specimen was bolted to the beam of the reaction frame using eight x M24 threaded rods. Bottle jacks were placed horizontally at either end of the foundation block to prevent it from sliding during testing.

The hydraulic actuator was attached between the column of the reaction frame and the top of the AFS wall units. The attachment of the hydraulic actuator to the walls was achieved by using a fixing bracket and the 25 mm diameter threaded reinforcing bar which were cast into the wall units, as shown in Figure 6. An electronic load cell was inserted in line with the hydraulic actuator to record the load applied to the wall units.

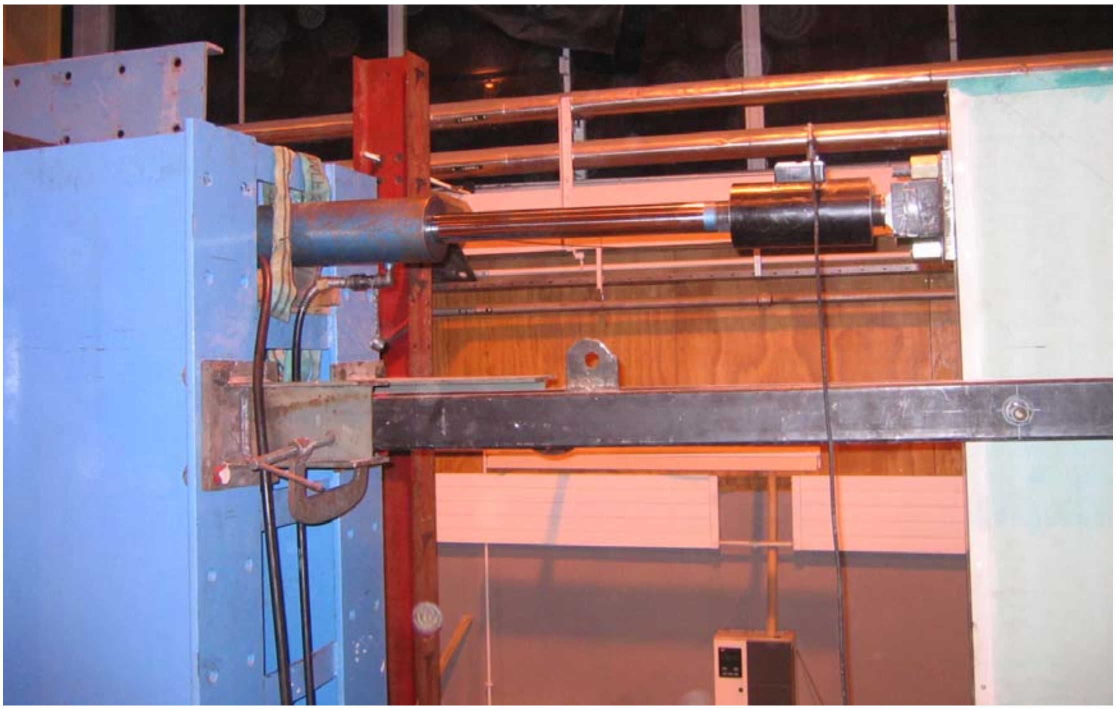

Figure 6 *Connection of the load cell and hydraulic actuator to the AFS150 wall unit*

A steel bracing beam with Teflon sliders was placed either side of the AFS wall units to prevent any other of plane deformation occurring during the testing sequence. Care was taken to ensure a clearance of 20 mm was maintained between the Teflon coated sliders, which were attached to the bracing beams, and the walls.

The rotary potentiometer, which recorded the deformation at the top of the wall, was attached to a secondary self equilibrating reaction frame. The reaction frame was kept separate from the main load resisting reaction frame to ensure no load induced deformations of the frame were recorded.

Figure 7 *Test unit AFS2 located in testing rig*

4.5 Loading Pattern

The AFS150 wall units were tested under a regime of reverse cyclic loading with increasing magnitude, based on the standard University of Canterbury simulated seismic testing scheme, shown in Figure 8 [P1]. In this scheme two cycles of load equivalent to 75% of the specimen nominal yield strength are imposed in each direction. The mean displacement magnitude at these four increments is multiplied by $^4/_3$ to estimate the equivalent bilinear yield displacement, Δ_y. Figure 9 represents graphically the procedure to determine the yield displacement of the wall units. All subsequent cycles of load were controlled by target displacements. Each of the target displacements was determined to corresponding to different displacement ductility values, where displacement ductility is defined as the ratio of maximum displacement to the yield displacement of the wall.

The theoretical strength of the wall units was derived using the computer programme "MC-Concrete" developed at the University of Canterbury [A1]. The analyses were undertaken using the characteristic strengths of the reinforcement (reported in Section 4.3). It was assumed an axial load was applied to the wall units in the analyses, equivalent to the self weight of the AFS150 wall.

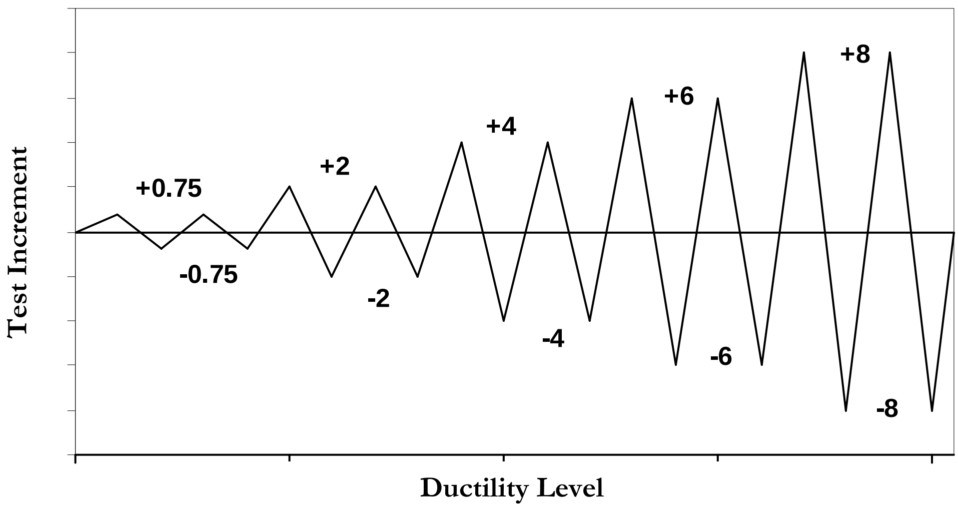

Figure 8 *Loading Pattern Applied to the AFS150 Wall Units.*

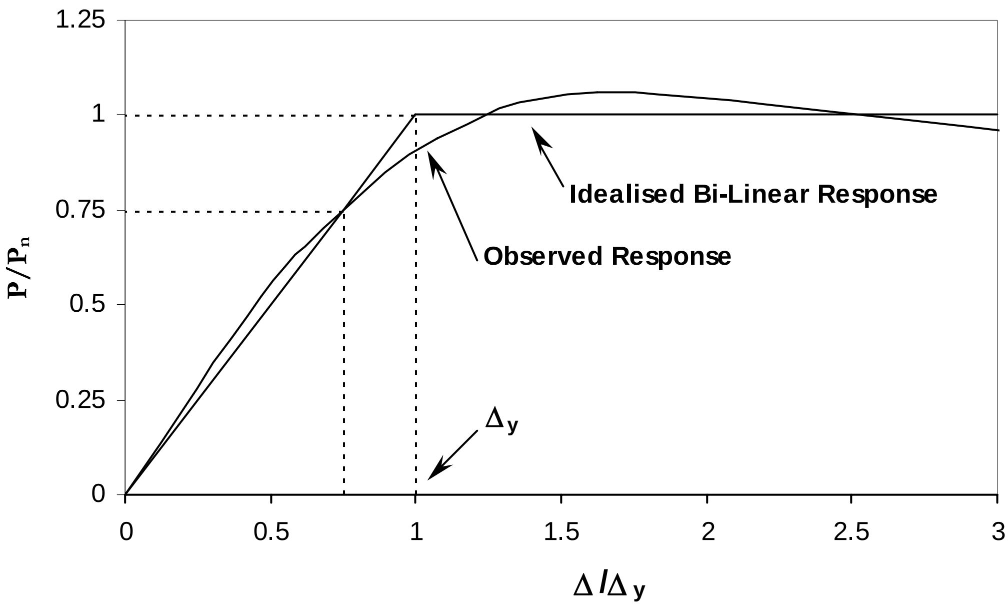

Figure 9 *Determination of the Equivalent Bilinear Stiffness and Yield Displacement.*

Testing of the AFS150 wall units was terminated when the horizontal load resistance of the walls significantly reduced. The failure load was defined to have been reached when load carrying capacity of the wall dropped to below 80% of the maximum, or when the first longitudinal reinforcing bar fractured. This failure criterion was originally derived by Park et al [P1].

5.0 EXPERIMENTAL RESULTS

The purpose of the experimental testing programme is to establish the performance of the AFS wall panel systems when subjected to lateral loading. The experimental test set up and loading regimes were developed to simulate realistic boundary conditions. It was felt the lateral load resistance of the AFS permanent wall panels would be reliant on the shear friction that develops between the concrete and the surface of the vertical steel studs. To ensure the performance of the wall panels was similar to wall panels in real buildings it was necessary to test the panels when the concrete was as mature as practicable. This would allow the concrete to begin drying and shrink away from the steel studs. As a result the testing was undertaken on the panels when they were 46 and 58 days old.

5.1 Test Specimen AFS1

Test specimen AFS1 was constructed on the 6th of September and tested on the 22nd of October, at an age of 46 days. The compressive strength of the concrete was determined to be 27 MPa on the day of testing.

The theoretical yield strength of test specimen AFS1 was calculated by undertaking a pseudo cyclic moment curvature analysis using the computer programme "MC-Concrete" [A1]. The actual measured material properties of the concrete and steel were used in the analysis and it was assumed the wall was subject to an axial compressive load equal to the self weight of the wall. The theoretical yield moment was calculated as 636 kNm, which equates to an applied horizontal load of 265 kN at a height of 2.4 m.

Test specimen AFS1 was subjected to a monotonically increasing level of lateral load. This loading regime was used to investigate the performance of the wall under a simulated seismic event which is characterised by a large initial series of accelerations.

Prior to testing of specimen AFS1 no visible cracks were observed in the wall panel or at the interface between the wall panel and the foundation beam.

5.1.1 Description of Test Performance

Test specimen AFS1 was initially loaded in the positive (push) direction to ¾ of the theoretical yield strength, corresponding to an applied horizontal load of 200 kN. During the application of the load a fine crack developed between the base of the wall and the foundation beam. The wall panel itself remained uncracked during the application of the initial loading cycle. A displacement of 1.3 mm was recorded at the top of the wall unit when the full 200 kN was applied horizontally. The lateral load was then slowly removed from the specimen until no lateral load was being applied. The displacement at the top of the wall was observed to drop to 0.8 mm when the load was removed. The residual displacement indicated that the wall had undergone minor plastic deformation.

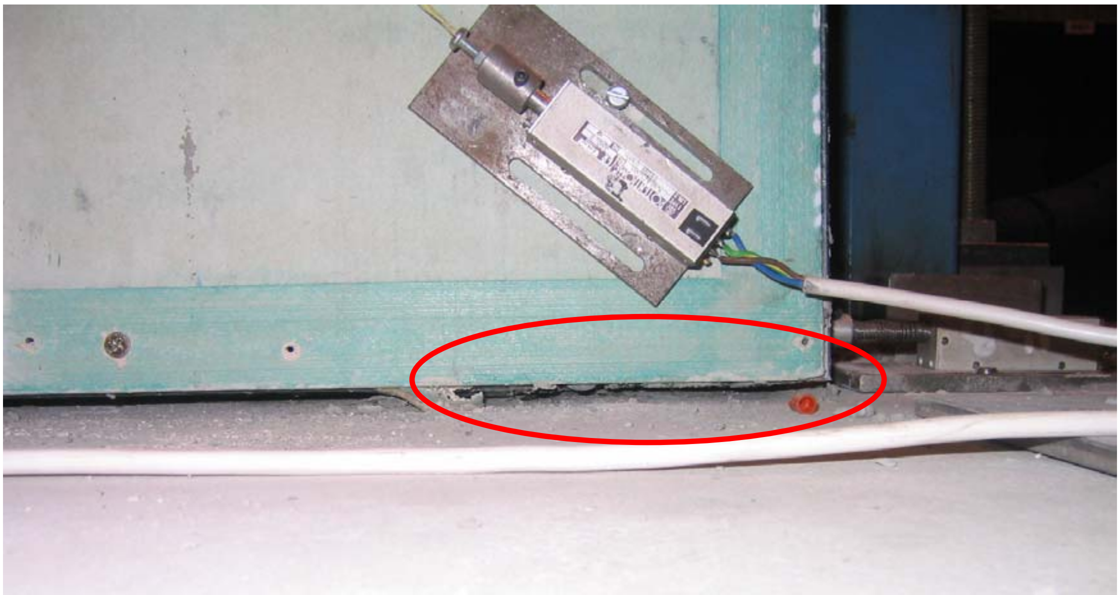

Figure 10 *Crack at the base of AFS1 during load cycles to 200 kN.*

As the load was applied to the wall panel a crack developed at the junction between the wall and the foundation beam. The crack width was observed to increase as the level of applied lateral load increased. The wall panel remained free from visible cracks during the entire test. Damage did occur to the compressive edge of the wall panel during the later stages of the testing. The damage was characterised by a bulging of the metal end cap of the wall panel, which appeared to be confining the crushing concrete.

The lateral (push) load was then reapplied to the wall in a continuous manner until the failure occurred, at an applied lateral load of 326 kN. This level of lateral load equates to an applied moment of 787 kNm. Failure of the wall was defined by a sudden loss of lateral load resistance and was caused by rupturing of the three starter reinforcing bars. Three distinctive noises were heard as the reinforcing bars ruptured, which indicated that the bars ruptured sequentially. This form of failure appears to denote that the AFS wall panel was acting as a rigid block above the foundation beam.

A short cycle of reverse loading was applied to the failed wall to determine if the crack which developed at the base of the wall would close. A load of 50 kN was applied to the wall in the reverse direction which resulted in complete closure of the crack. The testing of specimen AFS1 was terminated at the end of this loading cycle.

No significant deflection was recorded within the wall panel during the testing procedure. This indicated that the wall behaved as a rigid block. Minor horizontal displacements were recorded between the ends of the wall panel and the foundation beam. On inspection it appeared that the linear potentiometers located at the ends of the wall were measuring the bulging of the metal end caps of the wall due to the crushing of the concrete. It was concluded that only minor slipping of the wall unit along the foundation beam had occurred during the testing.

Figure 11 *Bulging of the end cap of the wall unit AFS1*

5.1.2 Load verse Deflection Response

The overall lateral load versus deflection response recorded from test specimen AFS1 is shown in Figure 12. The computer programme "MC Concrete" was used to determine the theoretical nominal strength, P_y, and ultimate strength, P_u, of the member. A detailed description of this computer package is defined elsewhere [A2].

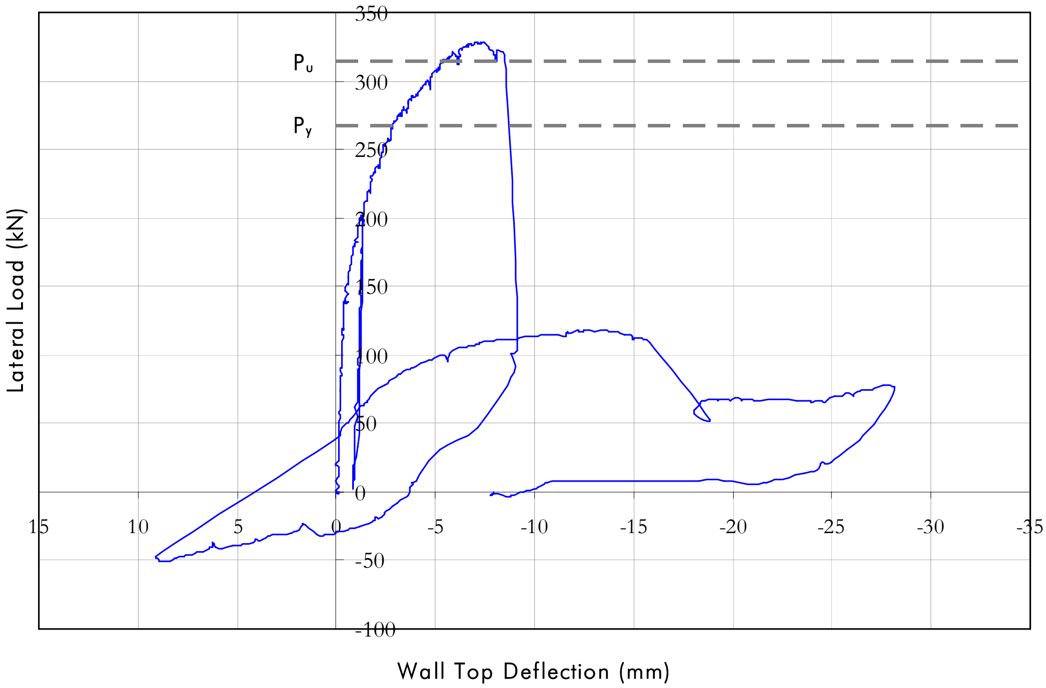

Figure 12 *Load Deflection Performance of test specimen AFS1*

The results shown in Figure 12 indicate that the theoretical model underestimated the maximum lateral load resisted by the member by approximately 4%. The theoretically derived nominal moment capacity of the test specimen was calculated to be 267 kN. This load corresponded with a decrease slope of the recorded load deformation response. The change in slope was representative of the onset of yielding in the wall panel.

5.2 Test Specimen AFS2

Test specimen AFS2 was constructed on the 6th of September and tested on the 29th of October, at an age of 53 days. The compressive strength of the concrete was determined to be 30.0 MPa on the day of testing.

The theoretical yield strength of test specimen AFS2 was calculated by undertaking a pseudo cyclic moment curvature analysis using the actual measured material properties of the concrete and steel. The results from the analysis indicated that the theoretical yield moment of the wall was 400 kNm, which equates to an applied horizontal load of 167 kN.

Test specimen AFS2 was subjected to increasing levels of reverse cyclic loading, as described in section 4.5. No visible cracks were observed in specimen AFS2 or at the interface between the wall panel and the foundation beam prior to testing.

5.2.1 Description of Test Performance

During the pre-yield load cycles a small crack was observed to have developed at the base of the wall, in the junction between the wall and foundation beam. The crack completely closed upon removal of the lateral load. The displacements recorded at the attainment of the four cycles to the pre-yield load were -1.45, 1.3, -1.45 and 1.38 mm. Based on these displacements the yield displacement of the specimen was calculated to be 2.09 mm. The wall panel was observed to have remained elastic in each of the pre-yield load cycles with no deformation recorded in the wall mounted potentiometers.

During the post yield load cycles the crack which had previously formed at the base of the wall was observed to increase in size. The maximum extension of the crack was recorded to be 6 mm during the second cycle of load to ductility 2 ($\mu_\Delta = 2$).

As the level of lateral displacement imposed on the wall was increased it was observed that the wall began to slide horizontally along the face of the foundation beam. A horizontal displacement of ± 2.5 mm was observed during the first cycle of loading to ductility 3. The level of sliding displacement was observed to increase as the laterally imposed displacement was increased.

Figure 13 *Crack at the base of Specimen AFS2 during loading cycle to a displacement ductility of 2*

Bulging of the metal end caps was observed at the base of the compressive end of the wall. It is believe the bulging was caused by crushing of the concrete on the corners of the wall. Examination of the test specimen AFS2 at the end of testing indicated that the concrete in these zones was badly cracked and was being confined by the fibre-cement sheets and metal end cap used on the ends of the walls.

Through out the loading sequence it was observed that the load achieved during the second cycle of loading to a specified level of displacement ductility was less than first cycle of load to the same displacement.

The testing of specimen AFS2 was stopped when the lateral load resistance of the wall significantly reduced. This was caused by fracture of five starter bars which extended from the foundation beam. The starter bars were observed to fractured in sequence from the tension end of the wall towards the compression zone in the wall. The starter bars fracture slightly below the surface of the foundation beam, indicating that sufficient bond had been developed between the starter bars and the concrete in the wall panel, despite the existance of a not contact lap splice with the reinforcement in the wall panel.

Figure 14 *Fractured reinforcing bar in specimen AFS2*

The wall panel was found to remain elastic throughout the test sequence, with the exception of minor crushing to the corners of the wall during the compression cycles. No deflections were recorded in the potentiometers placed on the wall panel, indicating that the wall had not deformed during the testing.

5.2.2 Load versus Deflection Response

The overall lateral load versus deflection response recorded from test specimen AFS2 is shown in Figure 15. The computer programme "MC Concrete" was used to determine the theoretical nominal strength, P_y, and ultimate strength, P_u, of the member. A detailed description of this computer package is defined elsewhere [A2].

The maximum level of lateral load resited by test specimen AFS2 was determined to be 258 kN in the negative (pull) direction and 231 kN in the positive (push) direction. The theoretical model underestimated the peak lateral resistance of specimen AFS2 by an average of 18%.

The lateral load versus deflection plot shown in Figure 15 shows that the peak lateral load resisted by the concrete member increased for each successive level of displacement ductility (μ_Δ). It is believed that the increase in load capacity was caused by strain hardening occurring the longitudinal reinforcing steel.

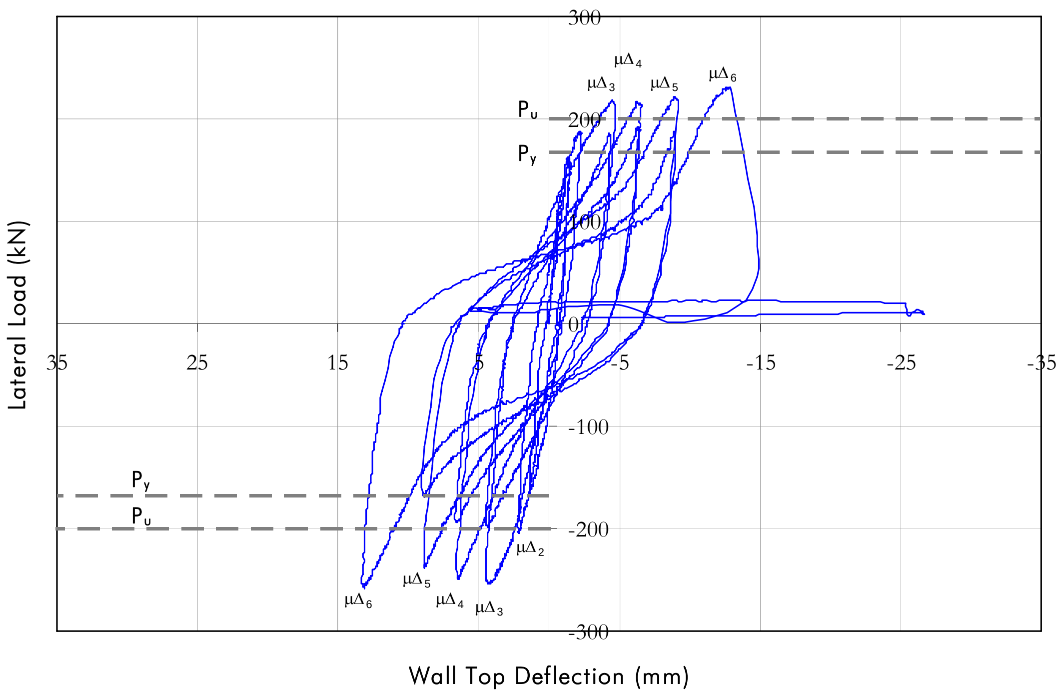

Figure 15 *Load Deflection Performance of test specimen AFS2*

The test specimen was found to have a decreasing level of stiffness as the level of applied load was increased. This can be observed in Figure 15 by the pinching of the hysteresis loops, and the flattening of the slope of the load versus deflection chart during the low level of applied load.

The reduced stiffness is believed to have been caused by the closing of the crack which occurred at the base of the wall. A very low level of force was required to close the crack, equivalent to the compressive yield capacity of the longitudinal reinforcement. This low level of required force and associated large displacement resulted in the reduced stiffness. Once the crack had closed the stiffness of the test specimen was found to increase rapidly, characterised by an increase in slope of the load versus deflection chart shown in Figure 15.

5.3 Test Specimen AFS3

Test specimen AFS3 was constructed on the 6th of September and tested on the 29th of October, at an age of 53 days. The compressive strength of the concrete was determined to be 30.0 MPa on the day of testing.

The theoretical yield strength of the test specimen AFS3 was calculated by undertaking a pseudo cyclic moment curvature analysis using the actual measured material properties of the concrete and steel. The results from the analysis indicated that the theoretical yield moment of the wall was 400 kNm, which equates to an applied horizontal load of 167 kN.

Test specimen AFS3 was subjected to increasing levels of reverse cyclic loading, as described in section 4.5. Prior to testing of specimen AFS3 no visible cracks were observed in the wall panel or at the interface between the wall panel and the foundation beam.

5.3.1 Description of Test Performance

During the pre-yield load cycles a small crack was observed to develop at the base of the wall, in the junction between the wall and foundation beam. The crack completely closed upon removal of the lateral load. The displacements recorded at the attainment of the pre-yield load were -2.0, 1.1, -2.1 and 1.1 mm in the first and second cycles of the push and pull load cycles respectively. Based on these displacements the yield displacement of the specimen was calculated to be 1.58 mm. The deflection imposed on the test specimen AFS3 was observed to return to zero upon removal of the lateral load. This indicated that the wall was behaving in an elastic manner.

Figure 16 *Pre-test photo of specimen AFS3*

During the post yield load cycles the crack which had previously formed at the base of the wall was observed to increase in size. A crack width of 8 mm was recorded during the second cycle of load to ductility 2 ($\mu_\Delta = 2$).

No horizontal sliding deflection was record between the wall panel and the foundation beam during the test sequence. The potentiometers located at the ends of the wall did record minor displacements; however this was determined to be due to the bulging of the steel end caps. Examination of the test specimen AFS3 at the end of testing indicated that the concrete located in the end of the walls was badly cracked and was being confined by the fibre-cement sheets and metal end cap used on the ends of the walls, which resulted in bulging of the end caps..

Through out the loading sequence it was observed that the load achieved during the second cycle of loading to a specified level of displacement ductility was less than first cycle of load to the same displacement.

Figure 17 *Bond failure with end reinforcing bar*

The testing of specimen AFS3 was stopped when the lateral load resistance of the wall significantly reduced. This was caused by fracture of three starter bars which extended from the foundation beam. The starter bars were observed to fracture in sequence from the tension end of the wall towards the compression zone in the wall. The starter bar closest to the tension end of the wall did not fractured but suffered a bond failure with the concrete in the wall panel, as shown in Figure 17. The bond failure was thought to be due to the reinforcing bar being located in close proximity to the end of the walls (within 60 mm). The concrete in this end zone was badly cracked during the compression load cycles.

The wall panel was found to remain elastic throughout the test sequence, with the exception of minor crushing to the compression corners of the wall. No deflections were recorded in the potentiometers placed on the wall panel, indicating that the wall had not deformed during the testing.

5.3.2 Load versus Deflection Response

The overall lateral load versus deflection response recorded from test specimen AFS3 is shown in Figure 18. The computer programme "MC Concrete" was used to determine the theoretical nominal strength, P_y, and ultimate strength, P_u, of the member.

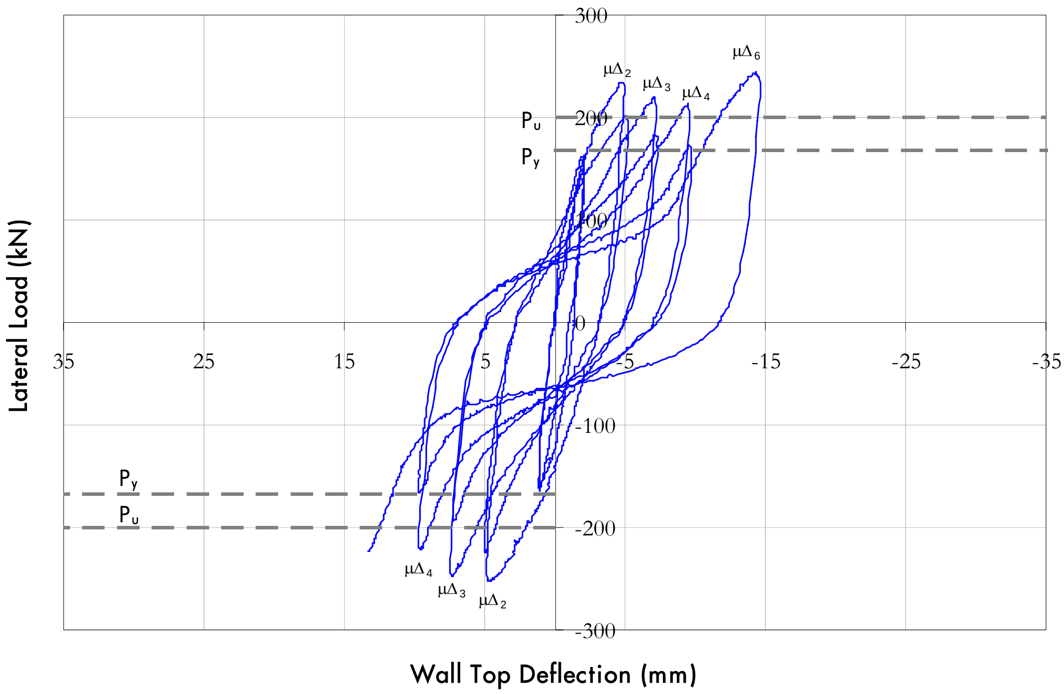

Figure 18 *Load Deflection Performance of test specimen AFS3*

The maximum level of lateral load resited by test specimen AFS3 was determined to be 249 kN in the negative (pull) direction and 238 in the positive (push) direction. The theoretical calculation for the ultimate capacity of the wall panel was 200 kN. The theoretical model underestimated the peak lateral resistance by an average of 17.5%.

The lateral load versus deflection plot shown in Figure 18 indicates that the peak lateral load resisted by the concrete member increased for each successive level of displacement ductility (μ_Δ). It is believed that the increase in load capacity was caused by strain hardening of the longitudinal reinforcing steel.

The test specimen was found to have a decreasing level of stiffness as the level of applied load was increased. This can be observed in Figure 18 by the pinching of the hysteresis loops. A similar observation was made for test specimen AFS2.

Failure occurred during the first cycle of loading to ductility 6 in the negative load (pull) direction. The failure resulted in a sudden loss of load carrying ability.

5.4 Test Specimen AFS4

Test specimen AFS4 was constructed on the 6th of September and tested on the 2nd of November, at an age of 56 days. The compressive strength of the concrete was determined to be 31.0 MPa on the day of testing.

The theoretical yield strength of the test specimen AFS3 was calculated by undertaking a pseudo cyclic moment curvature analysis using the actual measured material properties of the concrete and reinforcing steel. The results from the analysis indicated that the theoretical yield moment of the wall was 405 kNm, which equates to an applied horizontal load of 169 kN applied at a height of 2.4 m.

Test specimen AFS4 was subjected to increasing levels of reverse cyclic loading, as described in section 4.5. Prior to testing of specimen AFS4 no visible cracks were observed in the wall panel or at the interface between the wall panel and the foundation beam.

5.4.1 Description of Test Performance

During the pre-yield load cycles a small crack was observed to develop at the base of the wall, in the junction between the wall and foundation beam. The crack completely closed upon removal of the lateral load. The displacements recorded at the attainment of the pre-yield load were -2.0, 1.6, -2.1 and 1.6 mm respectively. Based on these displacements the yield displacement of the specimen was calculated to be 2.7 mm. The wall panel was observed to remain elastic in each of the pre-yield load cycles with no deformation recorded deformation occur in the wall mounted potentiometers.

During the post yield load cycles the crack which had previously formed at the base of the wall was observed to increase in size. The maximum extension of the crack was 8 mm during the second cycle of load to ductility 2 ($\mu_\Delta = 2$).

Figure 19 *Crack at the interface of specimen AFS4 and foundation beam during ductility 2 loading cycle*

As the level of lateral displacement imposed on the wall was increased the wall began to slide horizontally along the face of the foundation beam. A horizontal displacement of ± 1.5 mm was observed during the first cycle of loading to ductility 2. The level of sliding displacement was observed to increase as the laterally imposed displacement was increased.

Bulging of the metal end caps was observed at the base of the compressive end of the wall. It is believe the bulging was caused by crushing of the concrete on the corners of the wall. This effect was observed in all previously tested wall panel specimen.

The testing of specimen AFS3 was stopped when four starter bars which extended from the foundation beam fractured. The starter bars were observed to fracture in sequence from the tension end of the wall towards the compression zone in the wall. The location of the fracture was found to be slightly below the surface of the foundation beam, indicating that sufficient bond had been developed between the starter bars and the concrete in the wall panel.

The wall panel was found to remain elastic throughout the test sequence, with the exception of minor crushing to the compression corners of the wall. No deflections were recorded in the potentiometers placed on the wall panel, indicating that the wall had not deformed during the testing.

Figure 20 *Measured slip of specimen AFS4 along foundation beam*

5.4.2 Load versus Deflection Response

The overall lateral load versus deflection response recorded from test specimen AFS4 is shown in Figure 21. The computer programme "MC Concrete" was used to determine the theoretical nominal strength, P_y, and ultimate strength, P_u, of the member.

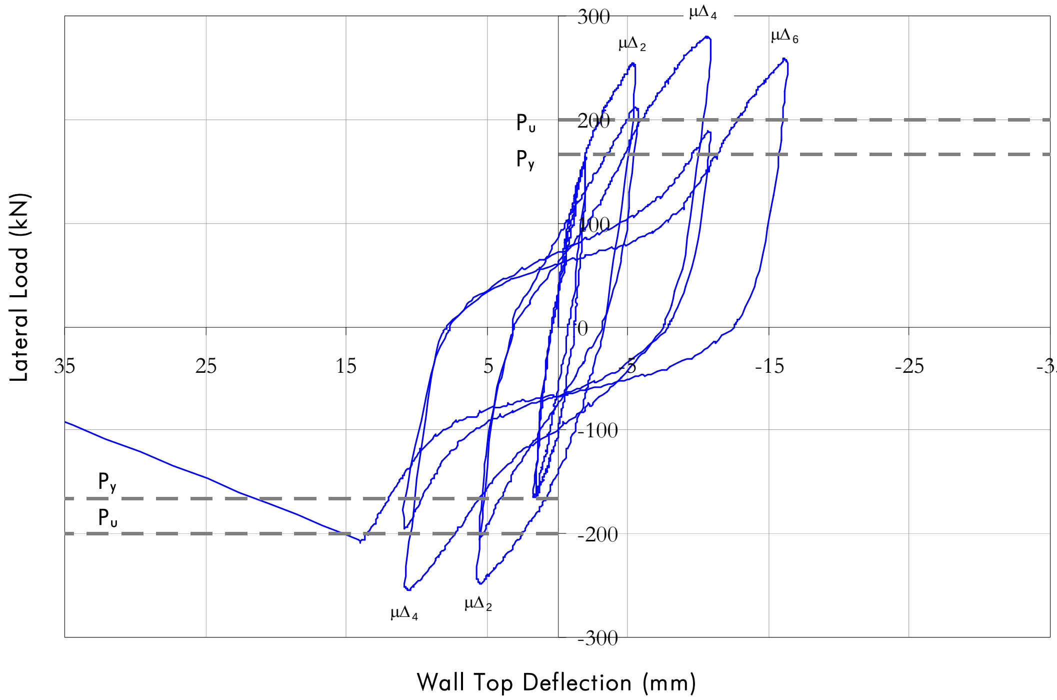

Figure 21 *Load Deflection Performance of test specimen AFS4*

The maximum level of lateral load resited by test specimen AFS4 was determined to be 276 kN in the negative (pull) direction and 248 in the positive (push) direction. The theoretical calculation for the ultimate capacity of the wall panel predicted a lateral load resistance of 200 kN, and underestimated the peak lateral resistance by 27.5% and 19.4% in the pull and push directions respectively.

The lateral load versus deflection plot shown in Figure 21 indicates that the maximum load occurred during the loading cycle to a displacement ductility of 4 ($\mu_\Delta = 4$).

The test specimen was found to have a decreasing level of stiffness as the level of applied load was increased. This can be observed by the pinching of the hysteresis loops, and the flattening of the slope of the load versus deflection chart during the low level of applied load. The reduced stiffness is believed to have been caused by the closing of the crack which occurred at the base of the wall. A very low level of force was required to close the crack, however this low level of force resulted in large displacement and a reduced stiffness. Once the crack had closed the stiffness of the test specimen was found to increase, characterised by an increase in slope of the load versus deflection chart at higher levels of applied load.

Failure occurred during the first cycle of loading to ductility 6 in the negative load (pull) direction.

5.5 Test Specimen AFS5

Test specimen AFS5 was constructed on the 6th of September and tested on the 29th of October, at an age of 53 days. The compressive strength of the concrete was determined to be 32.0 MPa on the day of testing.

The theoretical yield strength of the test specimen AFS5 was calculated by undertaking a pseudo cyclic moment curvature analysis using the actual measured material properties of the concrete and reinforcing steel. The results from the analysis indicated that the theoretical yield moment of the wall was 200 kNm, which equates to an applied horizontal load of 83 kN.

Test specimen AFS5 was subjected to increasing levels of reverse cyclic loading, as described in section 4.5. Prior to testing of specimen AFS5 no visible cracks were observed in the wall panel or at the interface between the wall panel and the foundation beam.

5.5.1 Description of Test Performance

During the pre-yield load cycles a small crack was observed to develop at the base of the wall, in the junction between the wall and foundation beam. The crack completely closed upon removal of the lateral load. The displacements recorded at the attainment of the pre-yield loads were -1.3, 1.1, -1.5 and 1.1 mm respectively. The yield displacement of the specimen was calculated to be 1.9 mm.

During the post yield load cycles the crack which had previously formed at the base of the wall was observed to increase in size. A crack width of 5 mm was recorded during the second cycle of load to ductility 2 ($\mu_{\Delta} = 2$).

Through out the loading sequence it was observed that the load achieved during the second cycle of loading to a specified level of displacement ductility was less than first cycle of load to the same displacement.

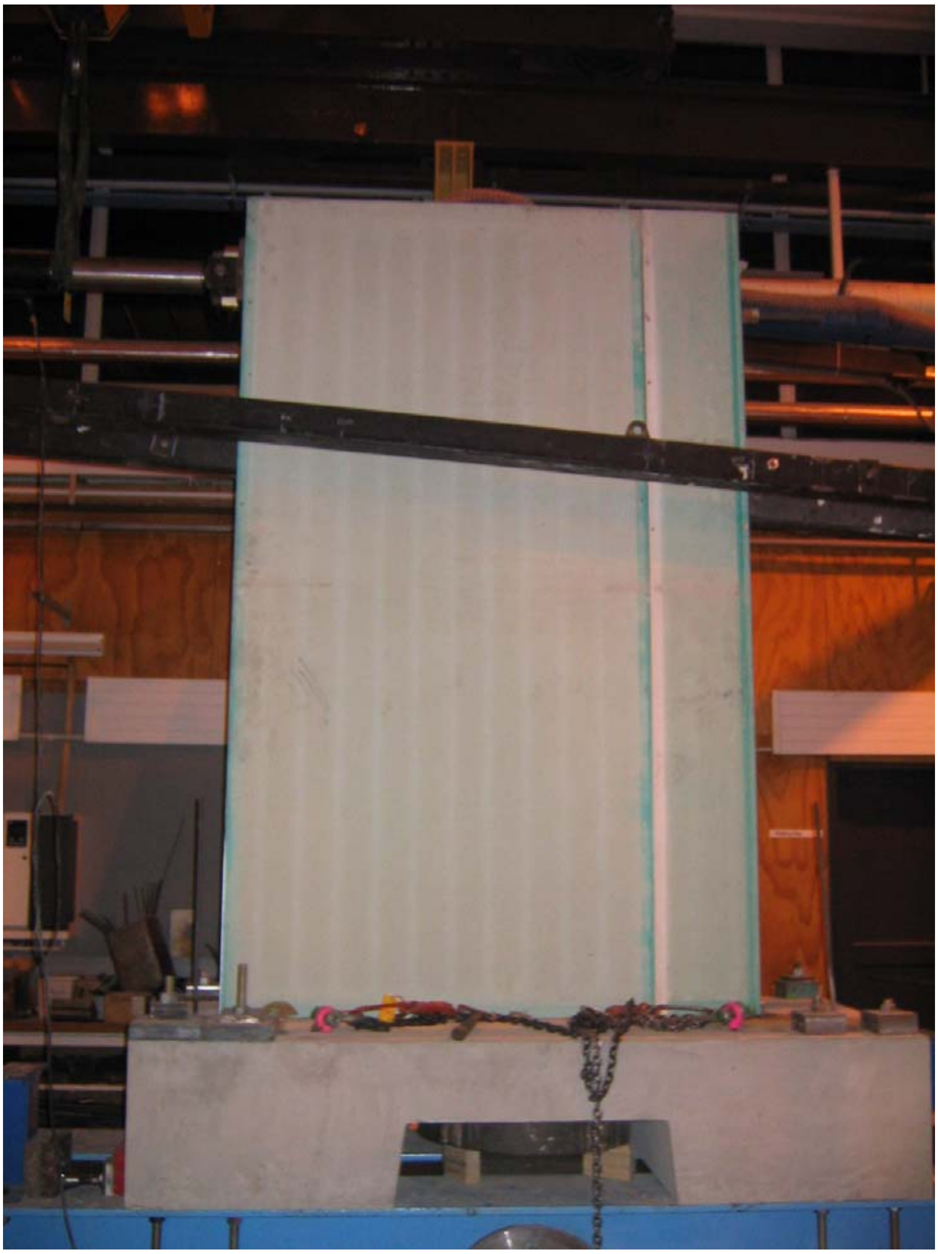

Figure 22 *Specimen AFS5 prior to testing*

A significant amount of horizontal sliding was recorded between the wall unit and the foundation beam. The degree of sliding increased as the level of imposed load increased. A maximum displacement of ± 15 mm was recorded during the load cycle to a displacement ductility of 5 ($\mu_\Delta = 5$).

The testing of specimen AFS5 was stopped when three starter bars fractured at the top surface of the foundation beam. The starter bars fractured in sequence from the tension end of the wall. The starter bar closest to the tension end of the wall suffered a bond failure with the concrete in the wall panel. The bond was broken due to the large degree of crack which occurred in the end zones of the wall panel.

Figure 23 *Expansion of the end plate in test specimen AFS5*

5.5.2 Load versus Deflection Response

The overall lateral load versus deflection response recorded from test specimen AFS5 is shown in Figure 24. The computer programme "MC Concrete" was used to determine the theoretical nominal strength, P_y, and ultimate strength, P_u, of the member.

The maximum level of lateral load resited by test specimen AFS5 was determined to be 110 kN in the negative (pull) direction and 93 kN in the positive (push) direction. The theoretical calculation for the ultimate capacity of the wall panel predicted a lateral load resistance of 100 kN. The theoretical model over estimated the peak lateral resistance by 7% in the positive direction and over estimated the peak resistance by 10% in the positive direction.

The lateral load versus deflection plot shown in Figure 24 indicates that the peak lateral load resisted by the concrete member was approximately the same for each successive level of displacement ductility (μ_Δ).

The test specimen was found to have a decreasing level of stiffness as the level of applied load was increased. This can be observed in Figure 18 by the pinching of the hysteresis loops. A similar observation was made for all other test specimens.

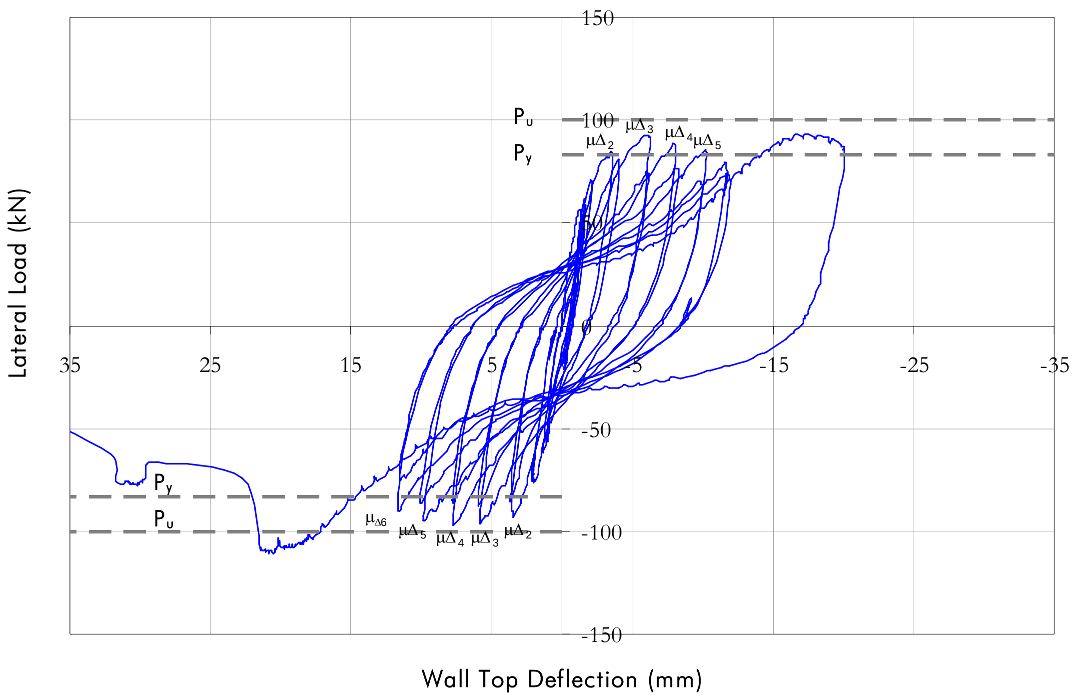

Figure 24 *Load deflection performance of test specimen AFS5*

6. DISCUSSION AND ANALYSIS

The purpose of the experimental testing programme was to establish the performance of the AFS wall panel systems to applied lateral loading. The experimental test set up and loading regime was developed to simulate realistic boundary conditions. It was felt the resistance of the AFS permanent wall panel systems to the applied loadings would be reliant on the shear friction that develops between the concrete and the surface of the steel studs. To ensure the performance of the wall panels were as close at practicable to wall panels in real buildings it was necessary to test the panels at as later date as possible after pouring. This would allow the concrete to begin to dry out and shrink away from the steel studs. As a result the testing was undertaken on the panels when they were between 46 and 58 days old.

6.1 Flexural Response of Wall Units

In regions of high seismic activity reinforced concrete walls are required to perform in a ductile manner when subjected to lateral loading. Ductility is defined as the ability to undergo inelastic deformations with little or no decrease in load carrying ability. The most desirable form of ductile behaviour in reinforced concrete walls is flexural deformation.

The flexural performance of conventional reinforced concrete walls has been extensively researched and is well understood. Numerous researchers have developed analytical models to predict the behaviour of the concrete and reinforcing steel [A2]. Strong correlations are often derived between the theoretical models and experimental results obtained from testing undertaken on walls panels. In this report the theoretical models developed by Mander et al. were used to model the performance of the AFS wall panel systems [A2]. A series of theoretical pseudo-cyclic moment curvature analyses was completed on the AFS wall panels using the Mander models. A detailed description of the Mander models and the analysis technique used is defined elsewhere [A2]. The results from the analyses were used to predict the nominal yield strength and ultimate load resisting capacity of the wall panels.

The correlation between the ultimate load capacity of the experimentally tested wall panels and the theoretical model are shown in Table 6. For all of the wall panels the pull load cycles were the direction of the first cycle of loading. The push load cycles were the reverse loading cycles, and applied to the wall directly after the pull cycles. As a result the ultimate load in the push cycles were always reached after the walls had undergone similar levels of displacement, and corresponding damage, in the pull loading direction.

Table 6 *Dimension of the AFS wall specimen*

I.D	Ultimate load during the PULL cycles			Ultimate load during the PUSH cycles		
	Experimental (kN)	Theoretical (kN)	Diff (%)	Experimental (kN)	Theoretical (kN)	Diff (%)
AFS1	326	314	3.8	-	314	-
AFS2	258	200	29.0	231	200	15.5
AFS3	249	200	24.5	238	200	19.0
AFS4	276	200	38.0	248	200	24.0
AFS5	110	100	10.0	93	100	(7.0)

The results shown in Table 6 indicate the theoretical model over estimated the ultimate load developed in all of the AFS test specimens during the pull cycles. The level of lateral load resistance of test specimens AFS2, AFS3, and AFS4 were also found to exceed the theoretical prediction. Test specimen AFS5 was achieved a low ultimate load than predicted using the theoretical model during the push cycles.

It is believed that the inability of test specimen AFS5 to achieve the theoretically predicted ultimate load during the push cycles was due to the damage that the wall had previously undergone during the pull cycles. It was noted in section 5.5.1 that the wall panel of test specimen AFS5 underwent significant horizontal displacement relative to the foundation block during the testing. To this extent it is believe that the level of displacement measured at the top of test specimen AFS5 was comprised of both sliding and rocking displacements. This would have resulted in the wall panel being subjected to less rocking motion than assumed in the theoretical model. The load deformation response of test specimen AFS5 indicated that the specimen was still increasing in strength when the desired displacement was reached. It is believed that had the measured displacement been corrected for the sliding displacement the specimen would have achieved an ultimate capacity in the push direction which was greater than that derived from the theoretical model.

The ultimate load resisting capacities of test specimens AFS2, AFS3, and AFS4 during the pull loading cycles were found to exceed the theoretically predicted ultimate load by 29%, 24%, and 38% respectively. It is believed that the improved performance of the test specimen above that of the theoretical model was the result of steel end plates in the wall providing a level of confinement to the extreme compression edge of the wall. It was noted in Chapter 5 that the steel end plates were bulging during the testing due to the pressure being applied from the cracked concrete. The influence of passive confinement on reinforced concrete members has been extensively investigated [A2, P1, P2, P3, Z1]. It has been shown that passive confinement of concrete by steel members, such as stirrups, increases the effective strength of the concrete which in turn results in a decreased neutral axis depth and an increased moment capacity [A2]. It is believed that steel end plates in the AFS wall panels were acting to confine the concrete and increased the strength of the specimen. This influence is beneficial to the performance of the wall when subjected to lateral loads for both flexural performance and ductile behaviour. Further research should be undertaken to quantify this performance.

The theoretical analyses undertaken on the AFS wall panels indicated that the failure mechanism of the walls would be fracture of the starter bars at the extreme tension edge of the wall. This failure mechanism was observed to have occurred in all of the test specimens. As a result it is concluded that the failure mechanism of the AFS wall panels was adequately predicted using conventional reinforced concrete theory.

The failure load of the wall panels was achieved during or after the first cycle of loading to a displacement ductility of 6 ($\mu_\Delta = 6$). Achievement of this level of ductility is defined as a "fully ductile". The load deformation responses of the five AFS test specimens showed moderate degree of pinching, due to the horizontal sliding of the test specimen on the foundation block. This form of sliding is common in all reinforced concrete walls. No deformations were recorded in the wall panels which indicated that the walls were adequately reinforced and lightly stressed. It is believed the flexural performance of the wall panels would not have been adversely affected by the removal of the longitudinal reinforcement, due to the presence of the vertical steel members in the panel construction. Panels with no vertical reinforcement were not tested, as it was felt important to test wall panels which had reinforcement contents compliant with the New Zealand Concrete Structures Standard, NZS3101: 1995.

Based on the results shown in Table 6 it appears that the flexural performance of the AFS wall panels can be adequately modelled using conventional reinforced concrete theory. The experimental tested AFS wall panels behaved in a manner compliant with the flexural requirements of the New Zealand concrete structural standard, NZS3101: 1995, for fully ductile walls.

6.2 Shear Response of Test Specimen

Reinforced concrete walls have been observed to exhibit brittle behaviour when detailed to fail under shear loads. The brittle failures result in significant and often explosive loss of load carrying capability. As a result, reinforced concrete walls are designed to prevent shear failures.

The AFS wall panel systems have vertical steel members at 110 mm centres which are used to support the exterior sheets of fibre cement. The steel members have holes punched through them to allow concrete to flow through and to allow horizontal reinforcement to be placed.

The New Zealand concrete structures standard, NZS3101: 1995 has particular requirements for shear reinforcement in concrete wall panels. The contribution of the concrete to the shear performance of the wall (v_c), when subject to no axial load, is determine as shown below:

$$v_c = 0.2\sqrt{f_c^{'}} \qquad \text{Eqn. 1.0}$$

Where

v_c = shear stress resisted by the concrete, MPa

f_c = 28 day concrete compressive strength, MPa

Based on Eqn. 1.0 above, the requirements for shear reinforcement are determined as shown below:

$$V^* = V_c + A_v f_y \qquad \text{Eqn. 2.0}$$

and

$$A_V = \frac{V^* - V_c}{f_y} \qquad \text{Eqn. 3.0}$$

then

$$A_V = \frac{V^* - 0.2\sqrt{f_c^{'}}\,b_w d}{f_y} \qquad \text{Eqn. 4.0}$$

where

A_v = area of shear steel required to resist the applied shear force, mm²

f_y = lower characteristic yield strength of the shear reinforcement, MPa

V^* = shear force applied to the wall, kN

b_w = width of the wall, mm

d = depth of the wall which is defined as 80% of the wall length, mm

A limitation has been placed in the New Zealand Concrete Structures Standard as to the minimum amount of shear reinforcement which is allowable in a concrete wall. Further restrictions are provided for the allowable spacing of the shear reinforcement. These requirements governed the design of the shear reinforcement in the experimental test series undertaken in this report.

The New Zealand Concrete Structures Standard, NZ3101: 1995, has requirements for the shear friction which is developed at an interface in a concrete member, such as the base of a wall or the intermediate steel members used in the AFS wall panels. The requirement for a conventional concrete wall is shown below:

$$V^* = \mu_f \left(A_v f_y + N^* \right) \qquad \text{Eqn. 5.0}$$

therefore

$$A_v = \left(\frac{V^*}{\mu_f} - N^* \right) \frac{1}{f_y} \qquad \text{Eqn. 6.0}$$

where

μ_f = coefficient of friction (1.4 when placed against roughen concrete, 1.0 when placed against smooth concrete, and 0.7 when placed against steel)

N^* = Axial load applied to the wall (compression is positive)

With the AFS wall panels the interface between the concrete and steel member is a combination of concrete and steel surfaces, due to the holes drilled in the webs of the steel members. As a result the value of m_f must be determined based on the proportional areas of the concrete and steel. For the AFS150 wall panels the proportion of open area in the steel members is 28%, resulting in a composite shear friction coefficient value of $\mu_f = 0.90$.

The Australian concrete structures standard, AS3600, requirements for the area of shear reinforcement required in a concrete wall are shown below:

$$A_v = \frac{V^* - \beta_5 f'_c b_w d}{\beta_4 f_y} \qquad \text{Eqn. 7.}$$

where

β_5 = shear plane coefficient associated with the concrete (0.5 in monolithic concrete and 0.2 with steel members)

β_4 = shear plane coefficient associated with the reinforcement (0.9 with concrete and 0.6 with steel)

All other variables have been previously defined.

AFS recommend reducing the values and β_5 below those in the Australian standard, to allow for the effects of differential shrinkage of the concrete away from the steel studs and for tensile effects. As a result, AFS recommended using $\beta_5 = 0$ for steel interfaces and $\beta_5 = 0.4$ for concrete interfaces.

The AFS wall panels have steel members crossing the concrete core of the wall panels at 110 mm intervals. It is important to determine the coefficients of β_4 and β_5 at these locations. However, the steel studs have holes located in their webs, which allows concrete to pass through. It is therefore necessary to determine the relative ratio of concrete area and steel area along the face of steel member to determine the correct values of β_4 and β_5. The approach developed by AFS is shown below:

$$\beta_4 = 0.6(1 - A_c) + 0.9A_c \qquad \text{Eqn. 8.0}$$

and

$$\beta_5 = 0.0(1 - A_c) + 0.4A_c \qquad \text{Eqn. 9.0}$$

where

A_c = area of holes in the steel members divided by the total area of the steel members

Based on the expressions shown in Eqn. 8.0 and 9.0, the AFS150 wall panels which were tested in the experimental testing programme have shear friction coefficients of $\beta_4 = 0.69$ and $\beta_5 = 0.12$.

Table 7 Shear reinforcement requirements for AFS wall panel specimen

Specimen I.D	Actual Horizontal Steel	New Zealand Wall Shear	New Zealand Shear Friction	AFS
AFS1	XD12 @ 600	XD10 @ 400	697 mm²	663 mm²
AFS2	-	XD10 @ 400	444 mm²	332 mm²
AFS3	XD12 @ 600	XD10 @ 400	444 mm²	332 mm²
AFS4	XD12 @ 300	XD10 @ 400	444 mm²	332 mm²
AFS5	XD12 @ 300	XD12 @ 300	222 mm²	43 mm²

The horizontal steel contents used in the test specimen and the requirements of the New Zealand Concrete Structures Standard for shear reinforcement and shear friction reinforcement are presented in Table 7. The governing criteria in the New Zealand shear equations were the maximum allowable spacing and the minimal reinforcement contents. Table 7 also presents the area of shear reinforcement determined by the AFS (modified Australian) requirements. The reinforcement requirements were calculated using the theoretical nominal ultimate capacity of the specimens, the lower characteristic yield level of the reinforcement (500 MPa) and the target 28 day compressive strength of the concrete (30 MPa). This approach is used in the design of conventional concrete structures.

The level of reinforcement used in the test specimen was chosen to be representative of that used in the construction industry in New Zealand, while providing an indication of the performance of the wall panels with varying degrees of shear reinforcement. In addition to the shear reinforcement listed in Table 7 above, two 25 mm diameter threaded reinforcing bars were cast into the top of the specimens. The reinforcing was chosen to simulate the effect of the top of the wall being attached to a floor or roof diaphragm with the load being applied to the end of the wall.

The load deformation plots of specimen AFS2, AFS3 and AFS4 are shown in Figure 25. Specimens AFS2, AFS3, and ASF4 were of identical dimensions and longitudinal reinforcement contents. The concrete strength of the three specimens was also very similar. The key variable between the specimens is the volume of shear reinforcement, as indicated in Table 7.

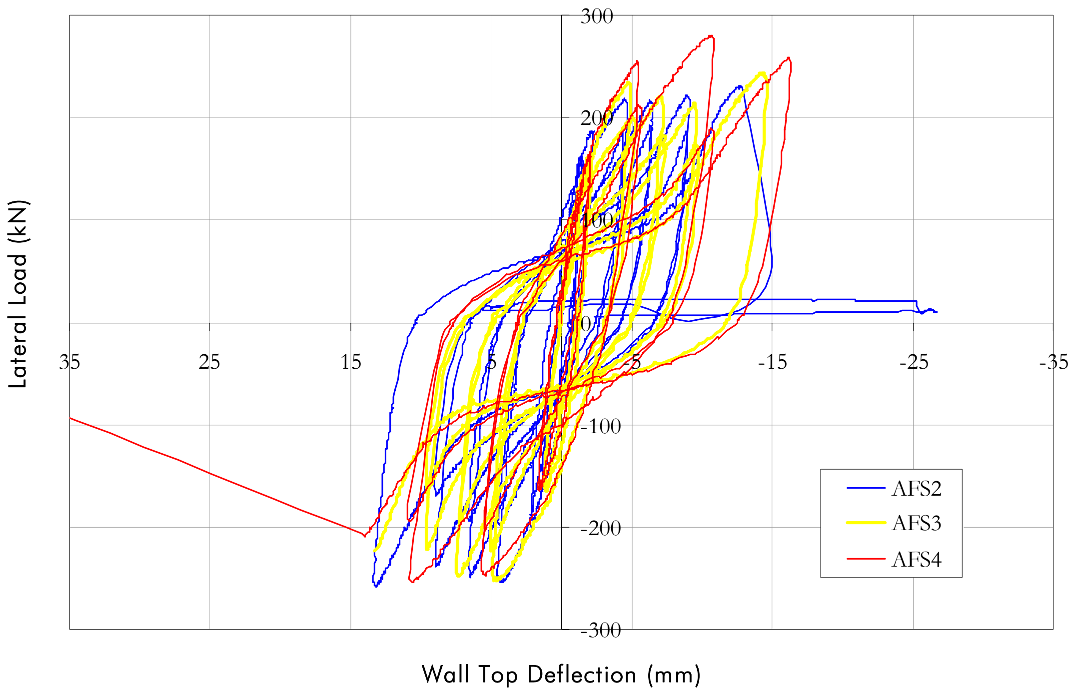

Figure 25 *Load Deflection Performance of test specimens AFS2, AFS3, AFS4*

The load deformation responses shown in Figure 25 indicate that the specimen AFS4 achieve a higher peak load than specimens AFS2 and AFS3 during the first cycle of loading in the positive direction. The peak load achieved by each specimen during second cycle of loading was comparable. The stiffness of the three test specimens remained similar throughout the testing sequence. Based on the results presented in Figure 25 there is no discernable difference between the responses of these three specimens. It would therefore appear that the overall response of the specimen was not influence by the level of shear reinforcement placed in the test specimen. It should also be notes that each of these test specimen failed under flexural loading.

The area contained under the load deformation responses provides an indication as to the level of seismic energy which was dissipated during the loading sequence. In seismically active areas it is desirable to have large energy dissipated by the load resisting elements in a building. Building elements which fail in shear are observed to have narrow and heavily pinched hysteresis loops in the load deformation responses. The load deformation responses of the five AFS wall panel specimens showed minor levels of pinching during the experimental testing. It is believed this pinching was due to the sliding of the wall panels on the foundation blocks, which is commonly found to occur in conventional concrete walls. The overall energy dissipation of the AFS wall panels specimen was good which indicates that none of the specimens was undergoing shear deformations.

It was noted in Chapter 5 of this report that no deformation was recorded in any of the wall panels during the entire series of tests. All of the inelastic deformations occurred at the interface between the wall panels and the foundation block. This indicates that the wall panels did not undergo any significant shear deformation, and that the wall panels remained essentially elastic throughout the testing.

Specimen AFS5 was found to undergo significant horizontal displacement relative to the foundation block. This indicates that the shear friction at the base of the wall had been exceeded. Specimen AFS5 was provided with five 12 mm deformed reinforcing bars as starter bars into the wall panel. One of these five bars was found to have had poor bond with the concrete in the wall panel, as discussed in section 5.5.1. Based on the design expression for shear friction in the New Zealand Concrete structures standard this wall unit should have develop a shear friction strength of 217 kN. The shear friction strength of test specimen AFS5 was found to be only 90 kN. It is believed that the experimental shear friction strength of the test specimen was lower that predicted due to the high aspect ratio of the test specimen. Specimen AFS5 had a height to length ratio of 1.6. It is recommended in the AFS manual not to exceed an aspect ratio of 1.0.

Based on the finding presented above it appears that the shear performance of the AFS wall panels is adequately predicted using the AFS method and the requirements of the New Zealand

Concrete Structures Standard. It is recommended that the conservative AFS values be adopted for β_4 and β_5, allowing for the possibility of differential shrinkage and tension cracking. It is also recommended that the height to length aspect ratio of the wall panels does not exceed 1.0.

7. CONCLUSIONS

A series of five test specimens were produced from the AFS150 structural wall system. The specimens were designed to investigate the influence of wall length, longitudinal reinforcement content, and horizontal reinforcement content. The walls were subjected to an increasing level of reverse cyclic loading. A theoretical analysis was undertaken to predict the behaviour of the reinforced concrete walls. The following conclusions were drawn from the testing and analysis of the test results.

1. The AFS wall panels systems behaved in a ductile manner, achieving a displacement ductility level in excess of 6.

2. The flexural response of the AFS wall panels was adequately predicted using conventional reinforced concrete theory and analysis techniques.

3. The vertical steel members in the AFS wall panels act as flexural reinforcement in the wall panels, limiting the length of the plastic hinge zone to the junction between the wall and foundation members. This did not adversely affect the performance of the walls in the experimental testing.

4. The shear reinforcement requirements for the AFS wall panel systems are adequately predicted using the AFS design method (modified Australian) and the requirements of the New Zealand Concrete Structures Standard, NZS3101: 1995. It is recommended that conservative estimates of β_4 and β_5 developed by AFS be adopted to account for temperature derived shrinkage and tensile effects in the wall panels.

5. No shear deformations were found to occur within the AFS wall panels during the experimental testing. This finding was applicable to all test specimens and was independent on the shear reinforcement content.

6. A poor correlation occurred between the shear friction requirements of the New Zealand Concrete structures standard, the AFS design approach the experimental results of test specimen AFS5. It is believed the lack of correlation was the result of the large height to length ratio of specimen AFS5. It is recommended that the height to length ratio of the wall panels does not exceed 1.0

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