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EXPERIMENTAL ASSESSMENT OF RC WALLS AND BUILDING CORES IN REGIONS OF LOWER SEISMCITY

S. J. Menegon[1], J. L. Wilson[2], N. T. K. Lam[3] and E. F. Gad[4]

[1] *PhD Candidate, Swinburne University of Technology, Melbourne, Australia, smenegon@swin.edu.au*
[2] *Professor and Executive Dean, Swinburne University of Technology, Melbourne, Australia, jwilson@swin.edu.au*
[3] *Associate Professor and Reader, University of Melbourne, Melbourne, Australia, ntkl@unimelb.edu.au*
[4] *Professor and Department Chair, Swinburne University of Technology, Melbourne, Australia, egad@swin.edu.au*
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Abstract

This paper presents the preliminary results of a recent experimental study testing the lateral cyclic displacement capacity of limited ductile reinforced concrete (RC) walls and building cores. In regions of lower seismicity, such as Australia, the majority of low, mid and high-rise buildings are constructed using RC walls as the predominant lateral load resisting system and typically fall into the category of limited ductile RC construction. The walls are generally configured to form box shaped configurations surrounding lift cores and stairwells. While considerable research efforts have been devoted to testing rectangular RC walls, in both regions of low and high seismicity, very few tests have been performed on geometric, i.e. box shaped, building cores. As such the experimental study predominately consisted of near full scale cast in-situ and precast RC building cores. The test program also included one full scale rectangular wall. The tests were performed using the state-of-the-art MAST hybrid system at Swinburne University of Technology, which can apply full six degree of freedom loading to a test specimen. Using the MAST system the specimens were tested as highly slender flexure controlled cantilever elements with a shear span ratio of 6.5.

Keywords: reinforced concrete walls; RC experimental testing; RC walls; RC building cores.

1. Introduction

In regions of lower seismicity RC walls are a popular and widely used lateral load resisting element in many different types of buildings. In Australia, for example, the majority of low, mid and high-rise buildings ultilise RC walls – of various cross sections and configurations – as the primary lateral load resisting system of the building. This generally consists of individual isolated rectangular walls, a central building core or a combination of both. The buildings typically then have an RC beam and column gravity frame, which can be in the form of a traditional two-way beam and slab system, band beams with one-way slabs or a flat slab.

In Australia RC walls are typically constructed using – what would be considered in regions of higher seismicity – as poor detailing practices, they're most commonly detailed and constructed without any conferment reinforcement in the end regions and with lap splices located at the base of the wall in the plastic hinge zone. The detailing is typically performed in accordance with the main body of the Australian standard for concrete structures, AS 3600 [1], which results in a 'limited ductile' RC structure in accordance with the Australian standard for earthquake actions, AS 1170.4 [2].

The walls are typical very slender, flexure controlled elements with shear-span ratios greater than four and are configured around lifts and emergency exit stairwells to form box shaped building cores. The majority of experimental testing programs of RC walls have typically consisted of rectangular walls with either ductile reinforcement detailing or detailing practices not commonly used in Australia. This experimental testing program was initiated to assess the seismic performance of RC walls matching current construction practices in Australia.

The initial phase of the testing program consisted of one rectangular RC wall and three box shaped RC building cores. The building core specimens consisted of one monolithic cast in-situ core specimen and two jointed precast core specimens. Jointed precast cores are becoming increasing popular in the Australian construction industry and are being adopted in lieu of tradition cast in-situ building cores in many low and mid-rise buildings. The jointed precast cores typically consist of individual rectangular panels, cast off site and later transported and erected on site and joined together vertically using dowel connections and horizontally using stitch plate connections. This initial phase of the testing program is only considering in-plane unidirectional response of the test specimens.

2. Experimental Testing Program

2.1 Cast In-Situ Specimens – S01 and S02

The experimental testing program consisted of four test specimens. The first two test specimens were monolithic cast in-situ RC elements. The first specimen (S01) was a rectangular wall and the second specimen (S02) was a box shaped building core. The rectangular wall has a cross section of 1200x200mm and a height of 2600mm. It was detailed with two layers of vertical and horizontal reinforcement, one per face, with 'U' bars at the end regions of the wall. The vertical and horizontal reinforcement consisted of N20 and N12 bars respectively, as shown in Fig. 1. N20 and N12 denotes 20mm and 12mm diameter normal ductility grade D500N reinforcing bar respectively. D500N rebar is produced in accordance with AS/NZS 4671 [3] and has a characteristic yield stress, ultimate stress and ultimate strain of 500MPa, 540MPa and 5 per cent respectively. The mean properties of D500N rebar are 550MPa, 660MPa and 9.5 per cent respectively [4]. The details and properties of test specimen S01 are summarised in Table 1.

Table 1 – Test specimen S01 and S02.

Specimen	Depth (mm)	Test wall height (mm)	Real wall height (mm)	Shear span ratio	Vertical reinf. ratio	Horizontal reinf. ratio
S01	1200	2600	10400	6.5	0.018	0.005
S02	1200	2600	10400	6.5	0.014	0.005

The building core specimen has a cross section of 1200x1200mm, a wall thickness of 130mm and a height of 2600mm. The specimen has a 500mm long x 1300mm high opening on one side, as shown in Fig. 1 and Fig. 2. The specimen is detailed with N12 and N10 vertical and horizontal reinforcing bars respectively. The specimen has 'U' bars at the corners which lap with primary horizontal reinforcement and closed ligatures around the vertical reinforcement in the return sections of wall adjacent to the opening (Fig. 1). The details and properties of test specimen S02 are summarised in Table 1. The specimens were constructed using grade N40 concrete, i.e. concrete with a characteristic compressive strength of 40MPa at 28 days.

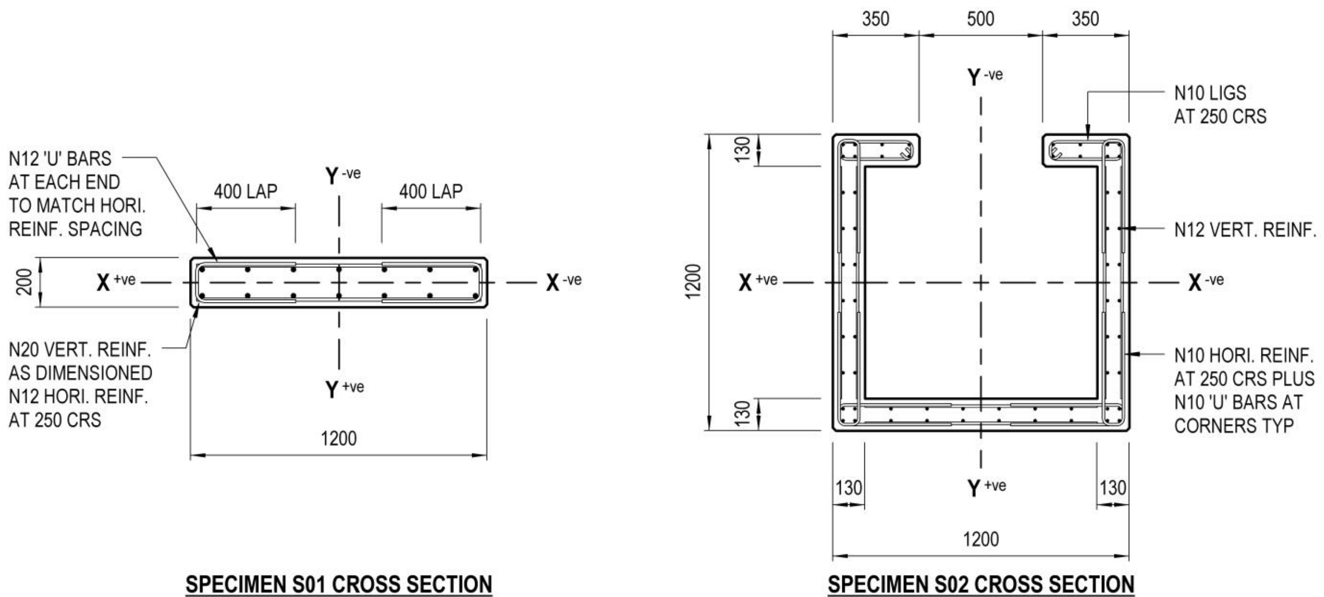

Fig. 1 – Test specimen cross section.

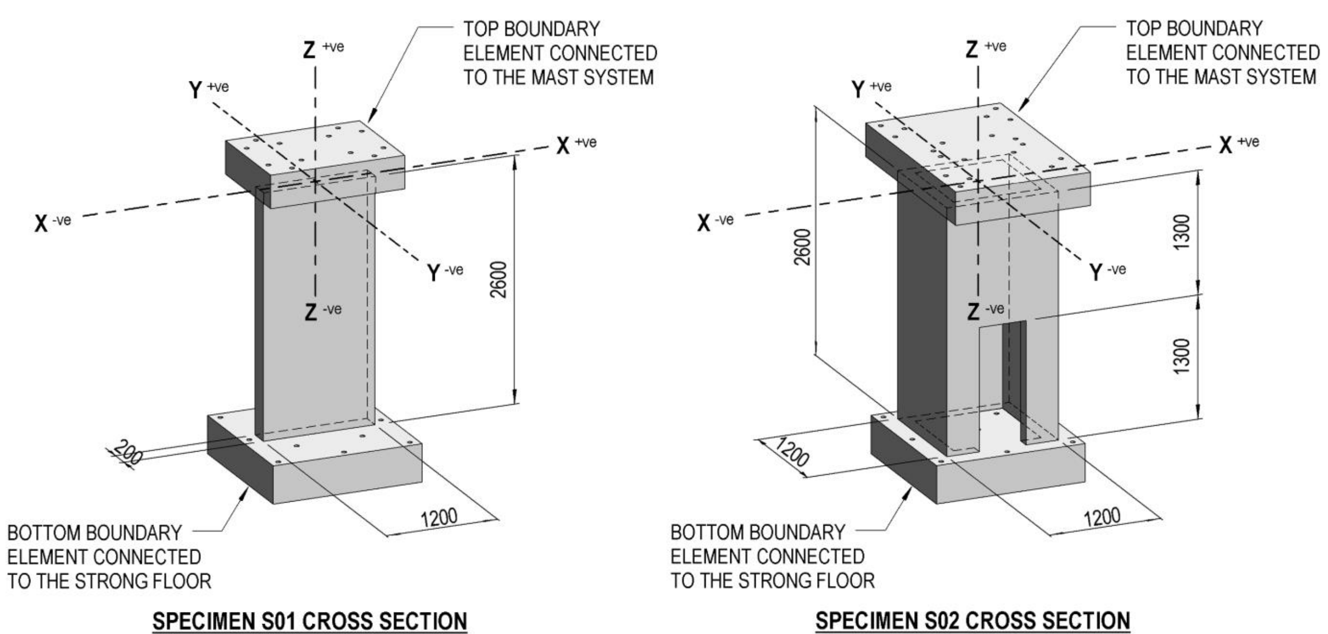

Fig. 2 – 3D view of test specimens.

The test specimens are meant to represent the ground floor component of a four storey wall and building core respectively, as shown in Fig. 3. The bending moment and shear force response of the taller four storey wall is being simulated on the one storey, ground floor component, test specimen using an applied lateral force and moment at the top of the specimen. To simulate the equivalent response the moment is applied as a function of the applied lateral force multiplied by a constant ‘k’. The constant ‘k’ is dependent on (i) the number of stories in the building, (ii) the inter-storey height of the building and (iii) the profile of the lateral load. For a four storey element with an inter-storey height of 2600mm and a triangular lateral load profile the constant ‘k’ equals 5.2. This results in the test specimens having a shear-span ratio of 6.5. The formulae for calculating the equivalent force and moment on the one storey test wall for a triangular lateral load profile are presented in Eq. (1), Eq. (2) and Eq. (3).

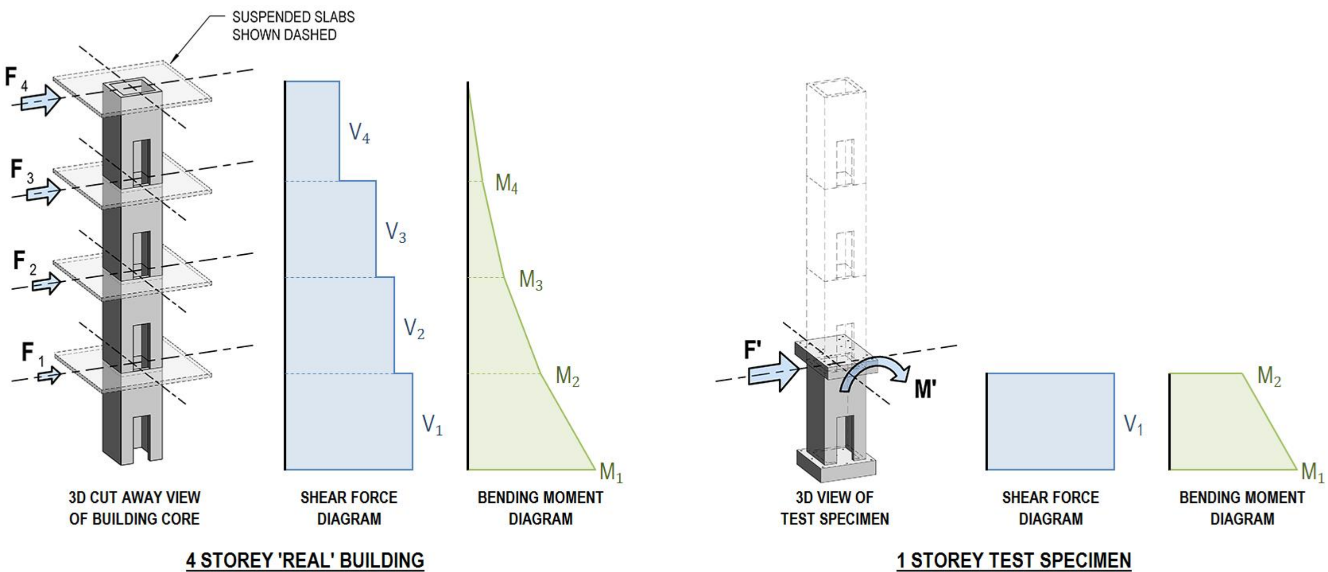

Fig. 3 – Simulation of four storey building using a one storey test specimen.

$$F' = \sum_{i=1}^{n} F_i \tag{1}$$

$$M' = kF' \tag{2}$$

$$k = \frac{h\sum_{i=2}^{n} i(i-1)}{\sum_{i=1}^{n} i} \tag{3}$$

2.2 Jointed Precast Specimens – S03 and S04

The last two specimens were jointed precast building core specimens. These specimens were constructed out of six individual elements, which consisted of a top and bottom boundary element block and four individual 130mm thick panels (Fig. 4 and Fig. 5). The panels were connected to the top and bottom boundary elements using standard dowel connections. The dowel connection consisted of an N24 reinforcing bar that had one end cast into the top or bottom boundary element block and the other end slotted into a hollow grout tube cast in at the top or bottom of the panel respectively. After the panels were erected the grout tubes were filled using high strength grout. Adjacent panels were connected together using standard stitch plate connections. The stitch plate connection consisted of a structural steel angle welded to cast in steel plates in each respective panel (Fig. 6).

The precast specimens were constructed to generally match the same specifications as the cast in-situ building core. Specimen S03 was constructed using low ductility L grade reinforcing mesh to AS/NZS 4671 and specimen S04 was constructed using normal ductility N grade reinforcing bars to AS/NZS 4671. Low ductility reinforcing mesh is commonly specified in precast panels in Australia because of ease and speed of construction. However its use in RC walls is questionable due to its brittle material properties. The characteristic yield stress, ultimate stress and ultimate strain to AS/ZS 4671 is 500MPa, 515MPa and 1.5 per cent respectively. While the mean values are somewhat higher than these values – 585MPa, 620MPa and 3.3 per cent respectively [4] – the ultimate stress to yield stress ratio and ultimate strain values are still very low, likely resulting in the ductility of the wall being severely limited if any yielding of the mesh occurred.

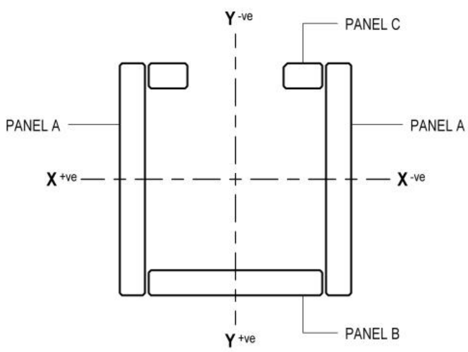

Fig. 4 – Joint precast specimen.

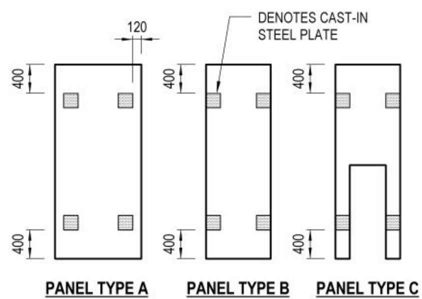

Fig. 5 – Panel elevations.

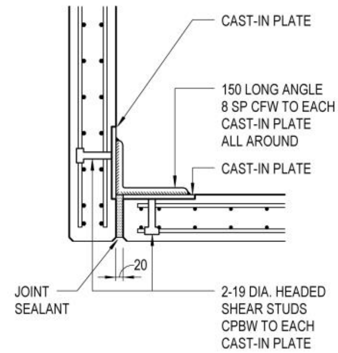

Fig. 6 – Stitch plate detail.

3. Test Setup and Loading Protocol

3.1 The MAST System

The specimens were tested using the Multi-Axis Substructure Testing (MAST) system in the Smart Structures Laboratory (SSL) at Swinburne University of Technology. The MAST system is a state-of-the-art test machine capable of applying full six degree of freedom (DOF) loading in mixed-mode, switched-mode, hybrid or a combination therein [5-7]. The MAST controller uses MTS control hardware, MTS 793 Degree of Freedom software and MTS TestSuite to control the six DOFs using eight individual MTS actuators (i.e. four ±1,000kN vertical actuators and two pairs of ±500kN horizontal actuators in orthogonal directions). The machine can test specimens of any material or shape with a maximum plan section of 3x3m, height of 3.35m and weight of 10 tonnes. The MAST system and its associated non-concurrent DOF force capacities and displacement limits are shown in Fig. 7 and Table 2 respectively.

Table 2 – MAST system non-concurrent DOF capacities.

Degree of freedom	Force capacity	Displacement limit
T_x – x-axis translation	±1,000kN	±250mm
T_y – y-axis translation	±1,000kN	±250mm
T_z – z-axis translation	±4,000kN	±250mm
R_x – x-axis rotation	±4,500kNm	±6.3°
R_y – y-axis rotation	±4,500kNm	±6.3°
R_z – z-axis rotation	±3,500kNm	±8.1°

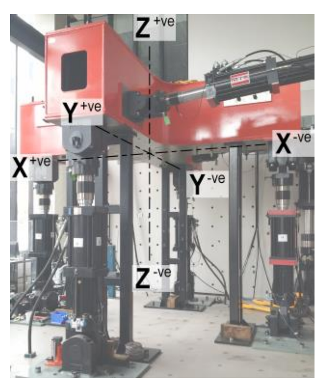

Fig. 7 – The MAST system.

The MAST system and its six DOF loading capability means is it capable of applying bi-directional in-plane lateral loads and torsions to the RC wall and building core specimens. This experimental study however is only considering unidirectional in-plane lateral loading on the test specimens, which means the systems third dimension actuators would just be being used to stabilise the two-dimensional test setup against out-of-plane reaction forces. For this reason, the specimens were designed to be tested at a 45 degree angle to the default axis of the system (Fig. 8). The control software allows the user to adjust the location and angle of the systems default axis system (i.e. as shown in Fig. 7) as required for the test. The '45 degree' test setup meant the in-plane lateral load capacity of the system for a two dimensional test was increased by a factor of root 2 from the values given in Table 2, i.e. the maximum in-plane lateral load and moment is increased to ±1,414kN and ±6,364kNm respectively and the maximum in-plane translation and rotation is increased to ±354mm and ±8.9° respectively.

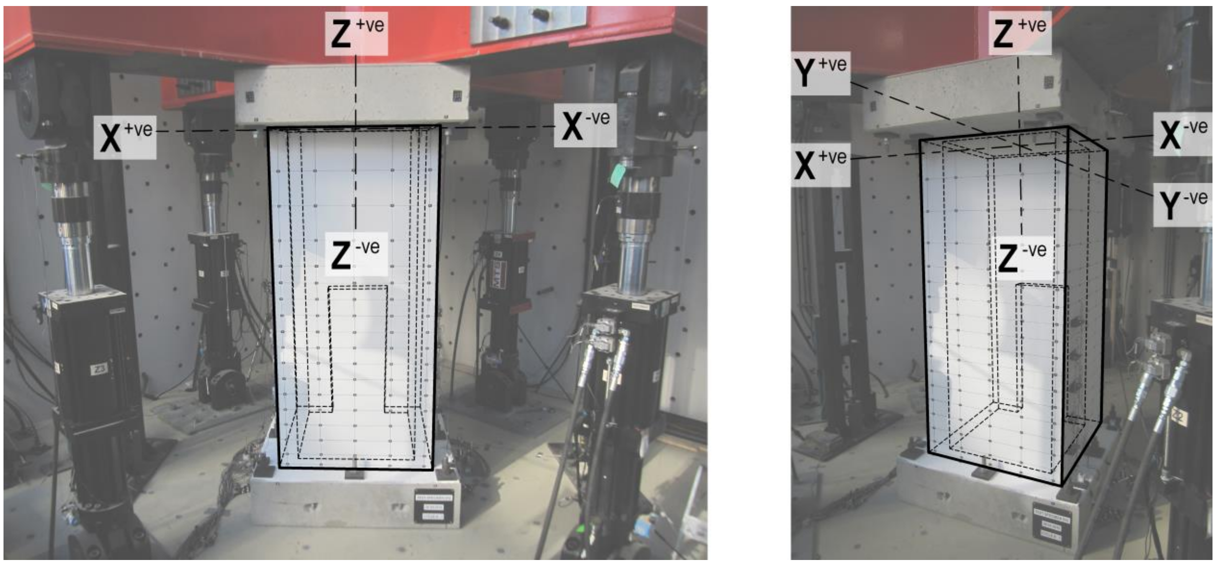

Fig. 8 – Test specimen S02 in the MAST system showing the rotated axis used during the testing.

3.2 Loading Protocol

The specimens were tested under pseudo-static cyclic test conditions. Initially an axial load was applied to the test specimens to simulate the pre-compression load on the wall (i.e. the gravity load from the surrounding building). The axial load was applied in force-controlled mode in the z-axis (T_z) and maintained for the duration of the test until axial load failure of the specimen occurred (i.e. complete structural collapse). The applied axial force for specimen S01 and S02 was -585kN and -1200kN respectively.

After the axial load was applied to the specimen, the specimen was subject to incrementally increasing cyclic lateral displacements in the x-axis (T_x). For each lateral displacement increment the specimens were subjected to two positive and two negative cycles, in line with the recommendations given in ACI 374.2R-13 [8]. After the initial series lateral displacement cycles, the subsequent lateral displacement increments were determined so the next value was between 5/4 and 3/2 times the current displacement increment. This procedure for calculating new lateral displacement increments was determined with reference to ACI ITG-5.1-07 [9]. The test was paused at the second positive and second negative cycle of each increment to take photos, mark crack patterns and take photogrammetry measurements. The lateral x-axis displacement loading protocols for specimens S01 and S02 are shown in Fig. 9.

For the duration of the test a moment was applied about the y-axis in force-controlled behavior to simulate the bending moment and shear force response of a taller four storey building in the one storey test specimen (Fig. 3). The applied moment was equal to the in-plane x-axis force multiplied by a value of 5.2, as discussed in the previous section. The remaining out-of-plane DOFs were commanded to stay at zero displacement and rotation in displacement-controlled behavior for the duration of the test, i.e. T_y was equal to zero movement and R_x and R_z was equal to zero rotation. A summary of the six DOF loading protocol is presented in Table 3.

Table 3 – Loading protocol summary.

Degree of freedom	**Mode**	**Loading**
T_x – x-axis translation	Displacement	Refer to Fig. 9
T_y – y-axis translation	Displacement	Zero movement
T_z – z-axis translation	Force	Constant axial force
R_x – x-axis rotation	Displacement	Zero rotation
R_y – y-axis rotation	Force	$M_y = k * F_x$
R_z – z-axis rotation	Displacement	Zero rotation

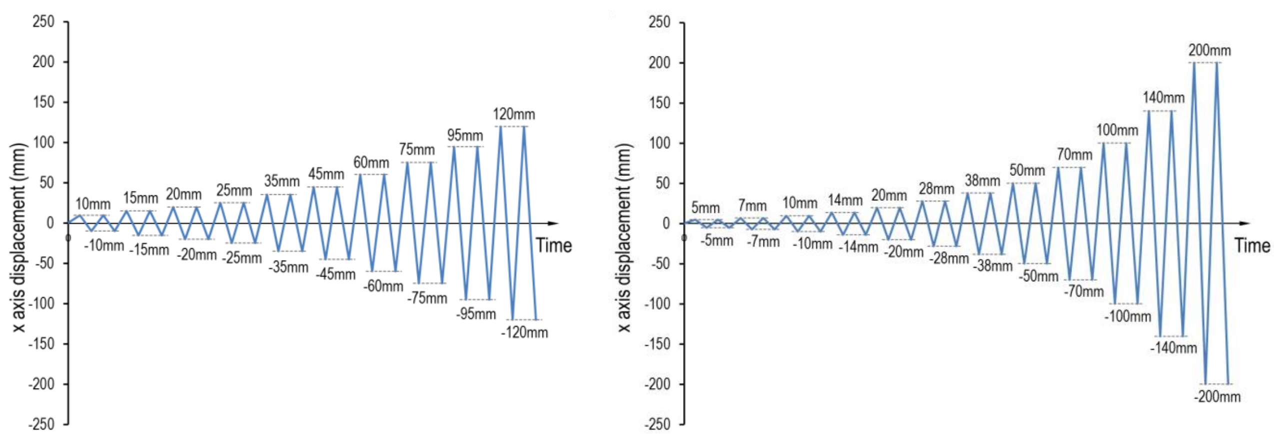

Fig. 9 – Cyclic x direction displacement increments for specimen S01 (right) and specimen S02 (left).

4. Preliminary Results

The preliminary results of the cast in-situ test specimens (S01 and S02) are presented in Fig. 10 and Fig. 11. The preliminary force vs. displacement and moment vs. rotation responses shown in Fig. 10 and Fig. 11 respectively were obtained directly from the MAST System controller with no corrections made for any slip or rotation at the interface between the top and bottom boundary elements and the MAST System crosshead and strong floor respectively. Laser displacement sensors were used to measure the amount of horizontal and vertical slip at the top and bottom of the test specimen and string potentiometers were used to independently measure the in-plane horizontal displacement (i.e. T_x) and rotation (i.e. R_y) of the specimen. The ensuing detailed analysis of the test results will account for any movement or slip in the boundary elements of the specimens.

Both specimens were able to achieve a displacement ductility of about 2 before serious strength degradation started to occur. The test specimens were detailed in accordance with the main body of the Australian Standard for concrete structures, AS 3600 [1] which results in a 'limited ductile' classification according to the Australian earthquake code, AS 1170.4 [2]. AS 1170.4 allows designers to adopt a displacement ductility factor of 2 for limited ductile RC structures, which is in good agreement with the preliminary test results.

The failure mechanism of the cast in-situ rectangular wall specimen (i.e. S01) was crushing of the concrete at the extreme compressive fibre of the wall. This allowed for the gradual reduction in lateral load capacity as seen in Fig. 10. This in contrast to the building core specimen (i.e. S02) where the failure mechanism was a combination of fracturing of the vertical reinforcement, unzipping of the lap splice and degradation of the concrete due to bond failure between the concrete and reinforcement, which resulted in a more sudden drop off and reduction in the lateral load capacity (Fig. 10).

The in-plane lateral load capacity of the cast in-situ building core specimen (i.e. S02) was within 80 per cent of its maximum value – a drop in lateral load capacity below 80 per cent of the maximum has traditionally been considered as 'lateral load failure' in RC column and wall testing – for displacement increments up to ±60mm. When the specimen was subjected to displacement increments of ±100mm, on the first positive cycle the lateral load capacity dropped to about 50 to 60 per cent of the maximum and then on the first reversed negative cycle the lateral load capacity dropped to below 20 per cent of the maximum. However despite the serious reduction in lateral strength, the specimen was still able to resist the initial axial load of 1200 kN. Axial load failure of the specimen (i.e. complete structural collapse) occurred after the test specimen was subjected to a displacement increment of 140mm.

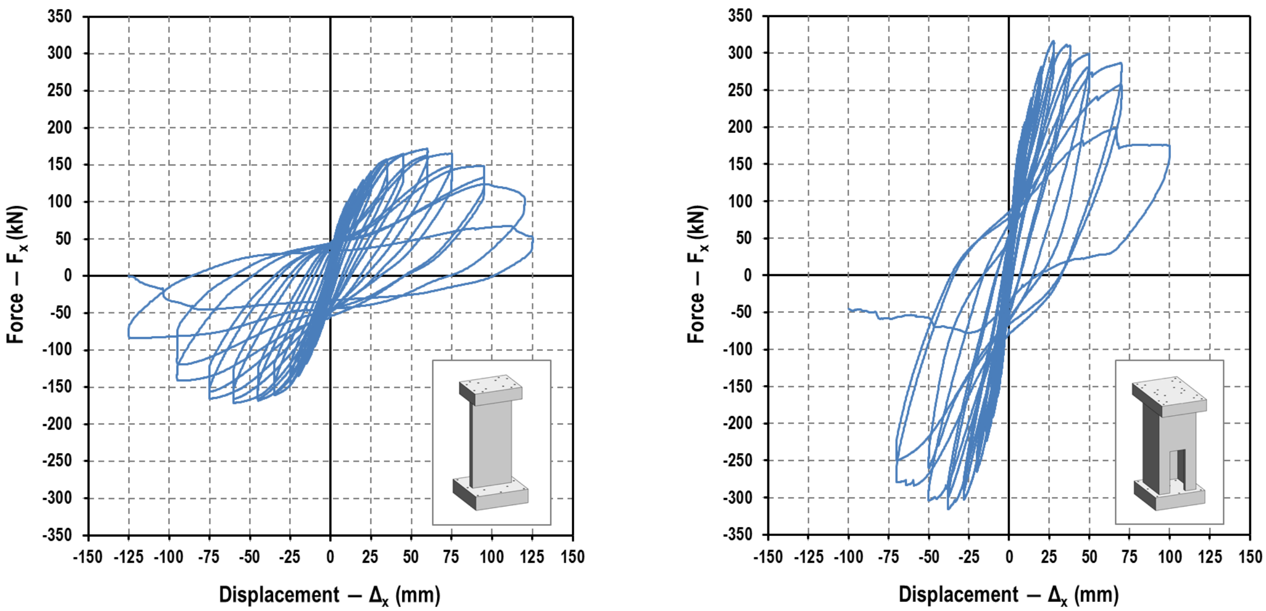

Fig. 10 – Preliminary test results, force vs. displacement response of S01 (left) and S02 (right).

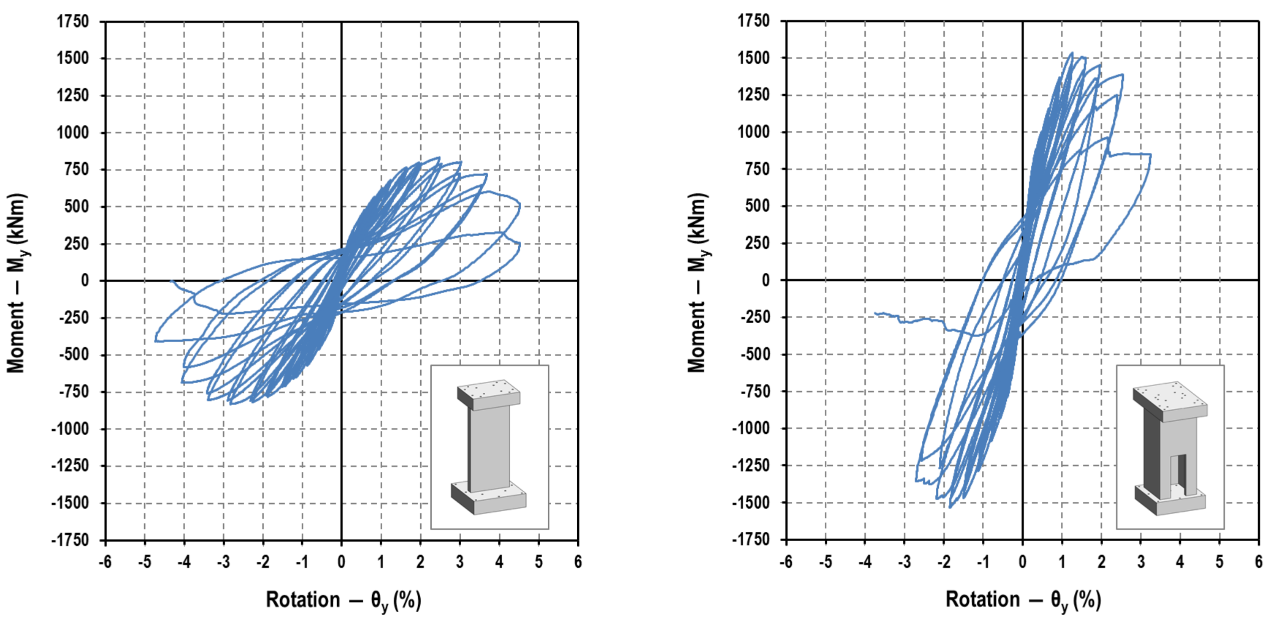

Fig. 11 – Preliminary results, moment vs. rotation response of S01 (left) and S02 (right).

5. Conclusions

This paper has presented the details and preliminary results of a recent experimental testing program into cast in-situ and precast RC walls and building cores in Australia. The testing program included one cast in-situ rectangular wall, one cast in-situ building core specimen and two jointed precast building core specimens. The jointed precast specimens were constructed using individual rectangular panels which were erected and joined together in the corners using a typical stitch plate connection and joined vertical to top and bottom boundary elements using a typical dowel connection.

The specimens were tested as slender cantilever elements with a shear span ratio of 6.5 and an axial load ratio of about 5 per cent. The cast in-situ rectangular wall and building core specimen had a vertical reinforcement content of 1.8 and 1.4 per cent respectively. The jointed precast specimens were constructed to generally match the specifications of the cast in-situ core, with one specimen constructed using low ductility L grade reinforcing mesh and the other using normal ductility N grade reinforcing bar matching the cast in-situ specimen.

The cast in-situ rectangular wall and building core both achieved a displacement ductility of about 2 before serious strength degradation started to occur, which is in good agreement with the ductility assumptions usually adopted by Australian designers when using the Australian earthquake loading standard. The ultimate failure mechanism of the rectangular wall was crushing of the concrete in the extreme compressive fibre of the wall, whereas the building core specimen failed due to the development of high tensile strains in the vertical reinforcement, which resulted in a combination of fracturing of the vertical reinforcement, unzipping of the lap splice and degradation of the concrete due to bond failure between the concrete and reinforcement.

6. Acknowledgements

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