SEISMIC PERFORMANCE OF MONOLITHIC WALL PANEL SUBJECTED TO QUASI-STATIC LATERAL CYCLIC LOADING

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ABSTRACT

A half-scale of monolithic precast wall panel (4000x1350x125) together with foundation beam (3000x500x465) is designed to emulate the behaviour of a ductile cast-in-place concrete wall and designed accordance to New Zealand Standard (NZ3101). The slenderness ratio of wall panel is 30 and its aspect ratio is 3. The specimen is constructed on strong floor and tested under lateral quasi-static reversed cyclic loading from 0.1% drift up to 3% drift. The flexural strength of monolithic wall depends on the spacing of transverse and longitudinal bars in the potential plastic zone which located one-fifth height of wall. The percentage of longitudinal and transverse reinforcement ratios in this type of wall are 0.84% and 0.25%, respectively. These percentages of reinforcement bars are exceeding the minimum percentage of 0.24% as specified in BS 8110 which provide in both directions. Experimental results show that wall panel starts to crack at 0.25% drift and fully cracks on the surface of the wall at 1.0% drift. Spalling of concrete cover at bottom corner of the wall starts at 2.0% drift and become worse at 2.5% drift. The specimen reduces the strength degradation at 2.5% and longitudinal bars fractured at 3.0% drift. Overall results showed that monolithic precast reinforced wall panel performs well up to 1.0% drift (the wall behaving in elastic region) but performed badly after 1% drift until 3 % drift (under elasto-plastic and plastic regions) where strength degradation occurs.

Keywords: *Emulate, Flexural Strength, Monolithic Wall, Quasi-static Lateral Cyclic Loading, Seismic Design*

1.0 INTRODUCTION

Monolithic conventional reinforced concrete wall panels experienced moderate and severe damage during the past earthquakes. Poor seismic performances of monolithic wall panels were observed during the 1985 Michoacan Earthquake in Mexico [1], the 1995 Hyogo-Ken Nanbu (Kobe) earthquake in Japan [2] and the 1999 Chi-Chi earthquake in Taiwan [3]. In these earthquakes, it was observed that reinforced concrete buildings which were built using monolithic wall panels were partially or fully collapse of wall panel, spalling of concrete, buckling and fractures of reinforcement bars between wall-foundation interfaces. Massive damages of these walls are due to the formation of plastic hinge zones (PHZ) at wall-foundation and wall-beam interfaces. Plastic hinge zones also occurred in beam-column interfaces and wall-foundation interfaces in reinforced concrete buildings. The soft-story mechanism occurs in reinforced concrete buildings also attributes to the collapse and damage of these structures.

Beside the formation of plastic hinge zones during earthquake, most of the wall panels are constructed with poor workmanship and did not follow the seismic provisions in their design. Moreover, these types of wall panels are commonly used in the construction of multi-storey reinforced concrete buildings in low and high seismic regions such as Malaysia, Thailand, Sumatra, Sulewasi, Java Islands, Pacific Islands, New Zealand, Japan and others countries. In order to improve the seismic performance of reinforced concrete buildings, precast wall panels are assembled together with moment-resisting frames which can reduce mechanism of plastic hinge. Past earthquakes such as the 1971 San Fernando Earthquake [4], the 1985 Chilean Earthquake [5] and the 1988 Armenia Earthquake [6] showed that precast wall panels performed better than monolithic wall panel with minimum structural damage.

A huge structural damage is expected to occur during earthquake due to large residual displacements especially in monolithic reinforced concrete wall. In the plastic hinges zones of wall panel, repair work can vary from epoxy injection of 1mm wide cracks or less, to concrete replacement. The longitudinal reinforcement bars could buckle and fracture during earthquake. Therefore, the process of retrofitting of reinforced concrete wall panel and structures will take from 2 weeks to 6 months depending on the severity of structural damages in the buildings.

Customarily, cast-in-situ reinforced concrete wall panel is used in the construction of medium and high-rise reinforced concrete building. However, the trends are changing from traditional method to ready-made method where precast structural components are prepared in plant/industry and assemble at site. Recently, most of medium and high-rise reinforced concrete buildings are constructed using steel-tunnel form and behaving as ductile monolithic wall panel. Thus, the usage of IBS (Industrialised Building System) in reinforced concrete buildings such as load-bearing wall panels and floor slabs is keep increasing from years to years. Nevertheless, most of load-bearing wall panel using fabric wire mesh as a substitution of longitudinal and transverse reinforcement bars could reduce the material cost and percentage of steel in concrete. It is worth to point out that the reduction of steel in concrete could decrease the flexural strength and stability of wall panels under earthquake loading. Therefore, it is recommended to properly design precast wall panel as shear wall.

It is worthwhile to investigate the seismic performance of monolithic wall panels under lateral cyclic loading which designed according to New Zealand Code of practice (NZ 3101: Part 1: 1995). Visual observations on structural damage and recorded data during experimental work are very useful to predict the behaviour of monolithic wall panels in reinforced concrete buildings under long-distant earthquake in Malaysia which designed according to BS 8110 (Part 1:1995). The amount of energy dissipated during cyclic loading can be measured using equivalent viscous damping equation.

2.0 DESIGN MONOLITHIC WALL PANEL

The most challenging part in designing wall panel is to design the connections at interface wall-foundation beam interfaces and its detailing of reinforcement bars in the plastic hinge regions so that the wall can resist earthquake loading with minimum damage. When the longitudinal reinforcement bar is lapped with the protruding bar from foundation beam, it is required to achieve continuity of longitudinal bars by tied them with wires. Plastic hinge zone is normally located within one-fifth of wall's height from base of wall. Short plastic hinges result in the development of tensile strains in the concrete that are significantly larger than those anticipated from conventional cast-in-place construction.

To overcome this problem, the lower portion of the precast wall panel is embedded within the recess cast into the foundation. Figure 1 shows the typical detail connection between monolithic wall panel and foundation beam in New Zealand. The gap left between the foundation and wall panel can be grouted using mortar. A minimum recess depth was calculated from the basic deformed bar development length for hooked bars in tension zone.
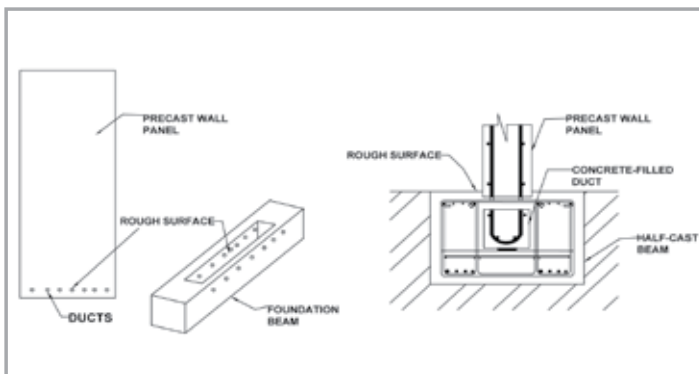

Figure 1: Connection detail for walls embedded in the foundation beam

Figure 2 shows the schematic arrangement of reinforcement bars in the wall panel and foundation beam. The dimensions of monolithic wall panels are 4000mm height, 1350mm wide and 125mm thickness. The lateral quasi-static cyclic loading is applied at 3500mm height of the wall. It is expected that plastic hinge zone occurs at 800mm (one-fifth of wall's height). 2HD10 (f_y=460N/mm^2) were used as longitudinal reinforcement and 5.5mm diameter of hoops (f_y=260N/mm^2) were used as transverse reinforcement. The longitudinal and transverse reinforcement ratios for this specimen were 0.84% and 0.25% respectively. The longitudinal reinforcement ratios at the wall ends and at centre were 1.26% and 0.57%.
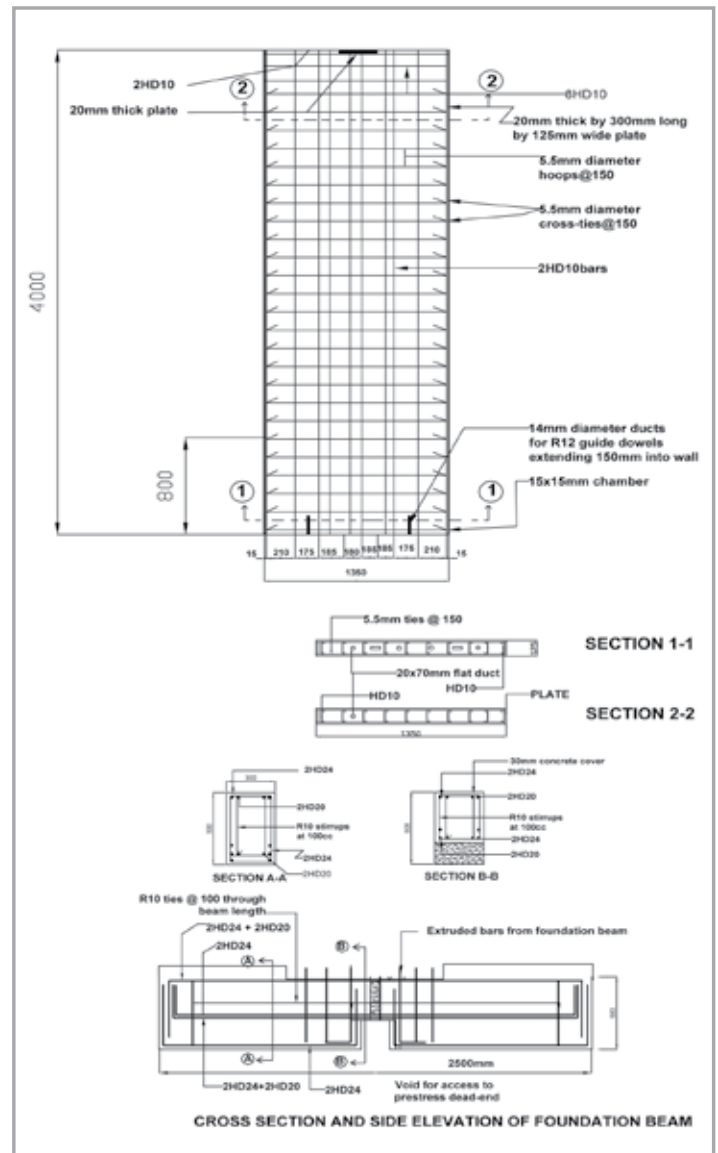

Figure 2: The arrangement of reinforcement bars in wall panels and foundation beam

The volumetric reinforcement ratio at the wall ends was 1.90%. The foundation beam was designed as a pad footing with additional longitudinal and transverse reinforcement bars which designed to resist gravity load and seismic loading. Monolithic wall panel is connected to foundation beam by tied up the extruded bars from foundation beam and longitudinal reinforcement bars in the wall using wires.

3.0 THEORETICAL IN-PLANE RESPONSE WALL PANEL

Figure 3 plots the idealised lateral force versus lateral displacement response of monolithic wall panels when subjected to in-plane lateral loading. The responses of all these systems are somehow related and can be established using a single mathematical formulation. The lateral force (H_r), causing decompression at the extreme fibre at one of the lower corners of the wall which can be determined using elastic theory as derived by [7] is defined as

$$H_r = (P + W)\, \frac{l_w}{6H_w} \tag{1}$$

where P is the gravity load from the roof, W is self-weight of wall panel, l_w is the width of wall and H_w is the height of the wall. The lateral displacement (Δ_r) at the top of the wall is due to lateral force (H_r). Lateral displacement can be obtained by combining flexural and shear deformations occurring within the panels. Using elastic theory [7], assuming the wall panel is uncracked and the shear modulus of concrete is $G_c = 0.4E_c$, the following expression is obtained for Δ_r,

$$\Delta_r = \left\{ 1 + \frac{3}{4}\left(\frac{l_w}{H_w}\right)^2 \right\} \frac{H_w}{3E_c I_g} H_r \qquad (2)$$

in which E_c is Young modulus of concrete (MPa) and I_g is the gross section second moment area of the wall panel. At point B, the development of yield in the longitudinal reinforcement bars were obtained. Thus, the equation for in-plane lateral force (H_y) and lateral displacement (Δ_y) are given in Equations 3 and 4,

$$H_y = (K_{ed})(\Delta_y - \Delta_r) + 3H_r\left(1 - \frac{a}{l_w}\right) \qquad (3)$$

$$\Delta_y = 2\frac{f_y}{E_s}L_{ed}\frac{H_w}{(l_w - a)} + 3 \qquad (4)$$

where K_{ed} is the stiffness of wall panel due to longitudinal reinforcement bars, L_{ed} is the length of longitudinal reinforcement bars and is the position of resultant compression force under base of the wall.

At point C, the development of the ultimate tensile strength in reinforcement bars in idealised stress-strain relationship of steel bars. The ultimate lateral force after yielding (H_u) and ultimate lateral displacement (Δ_u) are derived in Equations 5 and 6.

$$H_u = (K_{ed})(\Delta_p - \Delta_y) + H_y \qquad (5)$$

$$\Delta_u = 2\left(\frac{f_{su} - f_y}{E'_s}\right)L_{ed}\frac{H_w}{(l_w - a)} + \Delta_y \qquad (6)$$

where f_{su} and f_y is the ultimate and yield strength of reinforcement bars and E'_s is the secant modulus of longitudinal reinforcement bar. The validation of this theory will be proven in the experimental work which will be discussed in the following section.
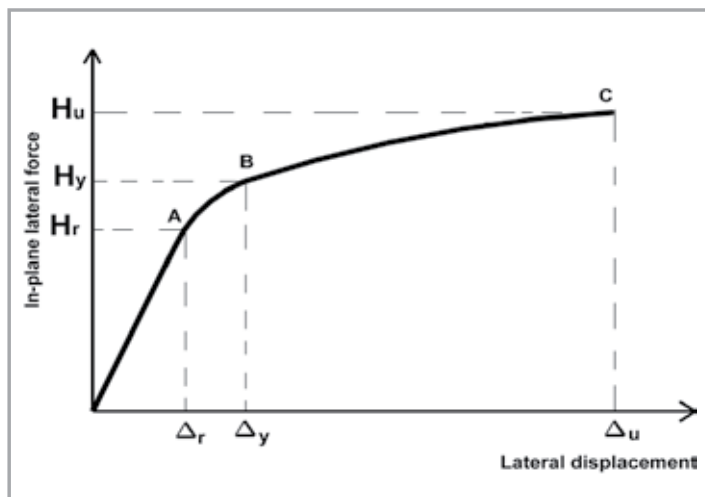

Figure 3: Theoretical in-plane behaviour for monolithic wall panel

4.0 CONSTRUCTION OF MONOLITHIC WALL PANEL

The caging of foundation beam was constructed using Y16 (high yield) longitudinal bars and welded to a 10mm thick base plate. R6 (mild steel) transverse reinforcement hoops were added and spaced equally at 100mm for wall panel. Figure 4(a) shows the completed caging foundation beam which seated on two benches of welded angle. Then the foundation cage was placed inside the formwork before connected to longitudinal reinforcement bars in wall panels [see Figure 4(b)]. Concrete strength of 30MPa was poured into the formwork and after 3 days the formwork was stripped off. Foundation beam was constructed separately from wall panel.
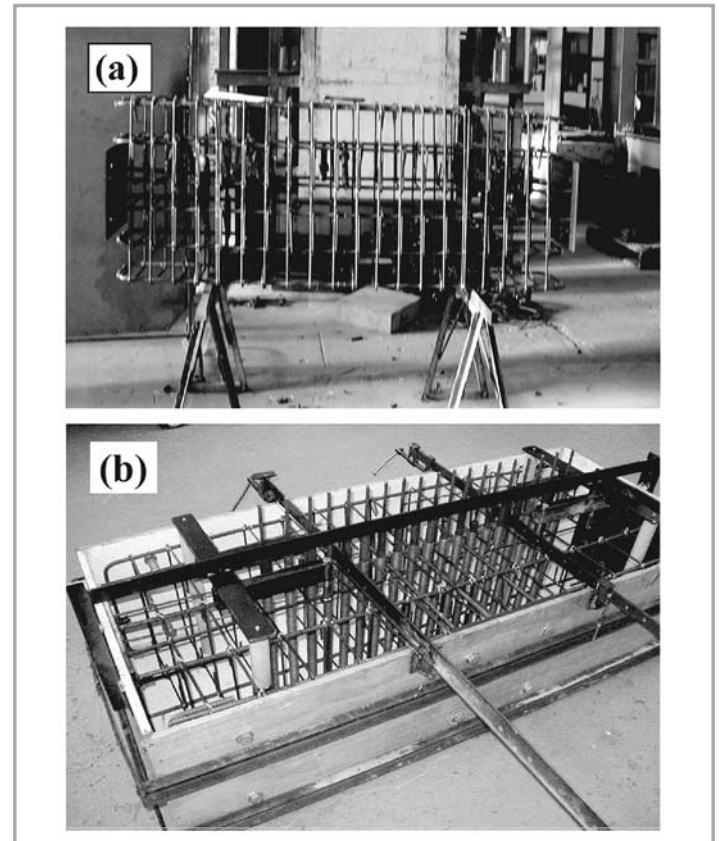

Figure 4: Preparation of foundation beam; (a) foundation cage consists of transverse and longitudinal reinforcement bars; and (b) foundation cage was placed inside the formwork before pouring the concrete

Figure 5(a) shows the horizontal orientation arrangement of longitudinal and transverse reinforcements bars for a monolithic single wall panel before pouring the concrete. The braces are used to hold in the position of ducting and lifting anchors. Prior to placing the cage onto the casting bed, strain gauges were placed on both external most longitudinal bars. The foundation beam was placed correctly locating through which the horizontal threaded bars were to be fed through. Ready-mixed concrete was poured into formwork of wall panel. Then the process of connecting between foundation beam and wall panel took place. Figure 5(b) shows front view of monolithic wall after the completion of the lifting operation where the wall is connected vertically together to foundation beam. Figure 6 shows the isometric view of monolithic wall panels seated on strong floor which is ready for instrumentation. The welded steel bracket was holding the foundation beam to the strong floor so that beam did not move

during testing. It is also prevent the foundation beam from lifting once lateral loading was applied to the wall panel. A pair of steel channel is bolted to reaction frame as safety required in case if the wall toppled in out-of-plane direction. The monolithic wall panel was ready for instrumentation and experimental set-up as described in the following section.
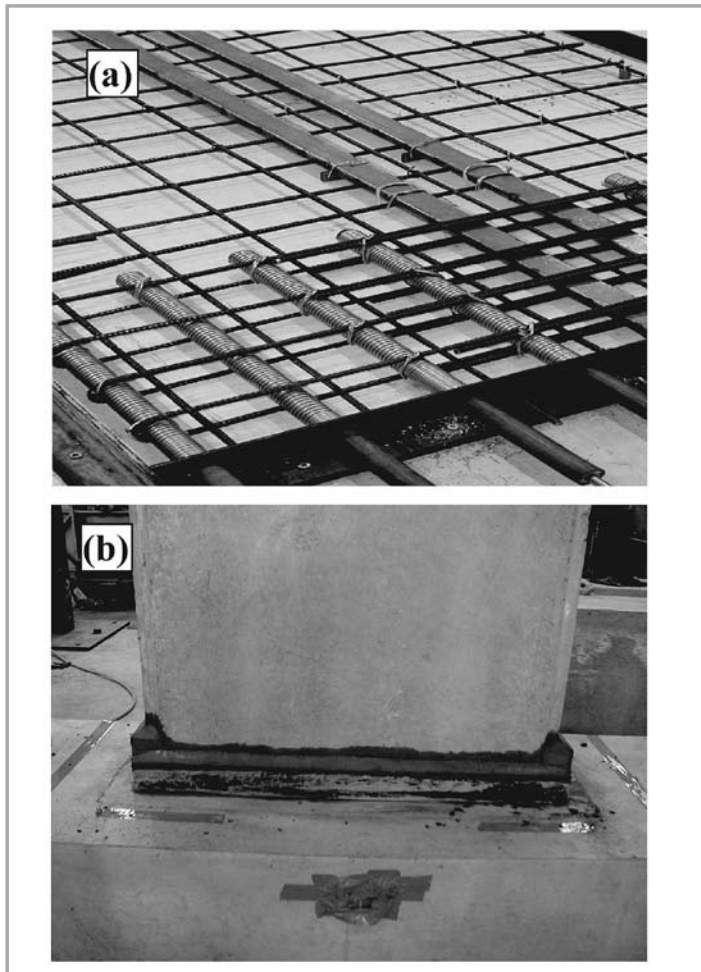

Figure 5: Construction of monolithic wall panel; (a) systematic arrangement of reinforcement bars and its connection at base of wall; and (b) tilt-up monolithic wall panel is connected to foundation beam after concreting and curing
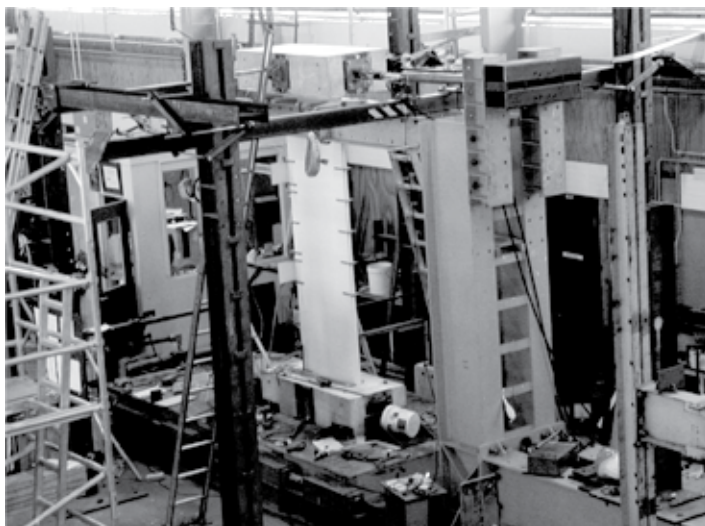

Figure 6: Isometric view of monolithic wall panel on strong floor

5.0 INSTRUMENTATION AND EXPERIMENTAL SET-UP

Figure 7 shows the reaction frame together with monolithic wall panel. The main loading frame consisted of four steel columns bolted down to a 750 mm thick strong floor. Two RHS were bolted either side of each unit spanning the loading and restraint columns of the test rig at two thirds the height of the wall. These provided resistance to both out-of-plane deformations and twist. The foundation beam were secured to strong floor through the use of two large double channel section and 32 mm high-strength hold down bolts, spaced 1800 mm apart. These sections provided restraint against overturning of the foundation beam once lateral force was applied to the wall.

Hydraulic rams and actuators were utilised for restraint loading and control in-plane displacement. The lateral load applied to the wall was applied through the use of a ± 450 mm stroke double-acting hydraulic actuator (330 kN compression and 440 kN tension). This actuator was bolted to the steel column of the main loading frame at a height 2830 mm from the strong floor. Connection to the wall was provided via load cell through a pin connection. This consisted of a 20 mm thick plate cast into the specimen to which 10 mm deformed bars extending the width of the wall were welded. Equivalent gravity load was simulated via a mass concrete block of 34 kN placed on top of wall and externally post-tensioning two 23 mm diameter threaded high-strength bars. A constant axial load was achieved through the use of computer controlled hydraulic valves.
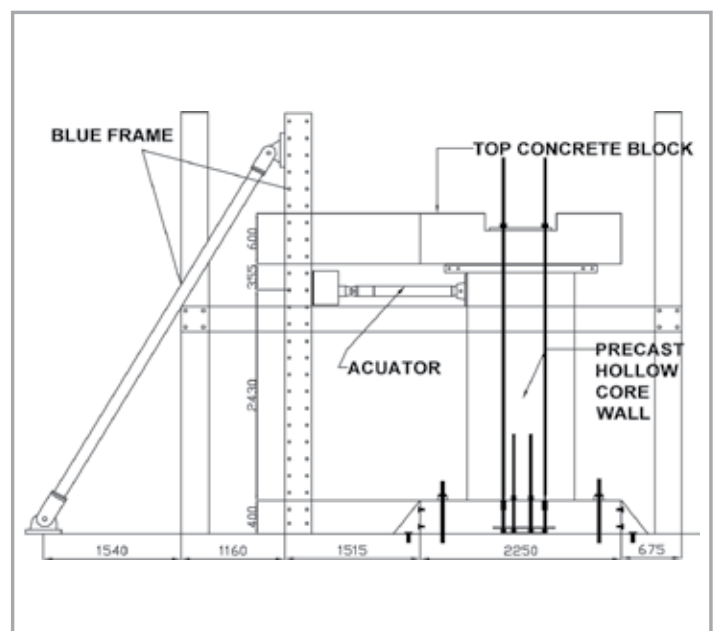

Figure 7: Schematic representation of test rig for monolithic wall panel

Figure 8 shows the locations of linear potentiometers on top surface and side of wall panel. These potentiometers were used to measure deflection, rotation and average curvature of the wall when in-plane lateral loading was applied at top of the wall. The in-plane lateral displacement of the specimen was monitored by one linear potentiometer aligned with the actuator providing the lateral force located at P5.
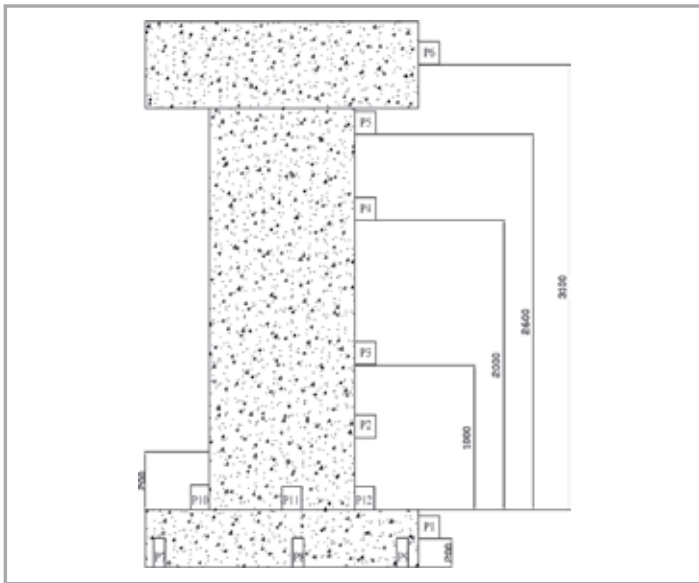
Figure 8: Schematic arrangement for locations of linear potentiometers

6.0 TESTING PROCEDURE AND LOAD REGIME

The loading applied on top of monolithic wall panel comprises of quasi-static lateral displacement controlled based cycles. The loading test regime is shown in Figure 9. Initial cycles to start with ±0.25% and ±0.5% lateral drift are completed for two cycles for each drift. These cycles were within the elastic range which allowed the stiffness and yield displacement of the wall to be established. Subsequent cycles composed of two large amplitudes with increment of ±0.5% drift. Lateral displacement applied should be slowly and steady so that the pattern of hysteresis loops can be obtained. It is very important to detect the behaviour of steel during elastic, yield plateau, elasto-plastic and ultimate or failure load during experimental work. This schedule was completed up to 3% drift lateral drift. The damage was identified based on amount of cracks, spalling of concrete, buckling of reinforcement bars and fracture of bars. The application of lateral displacement and loading were at a height of 3500 mm above strong floor.
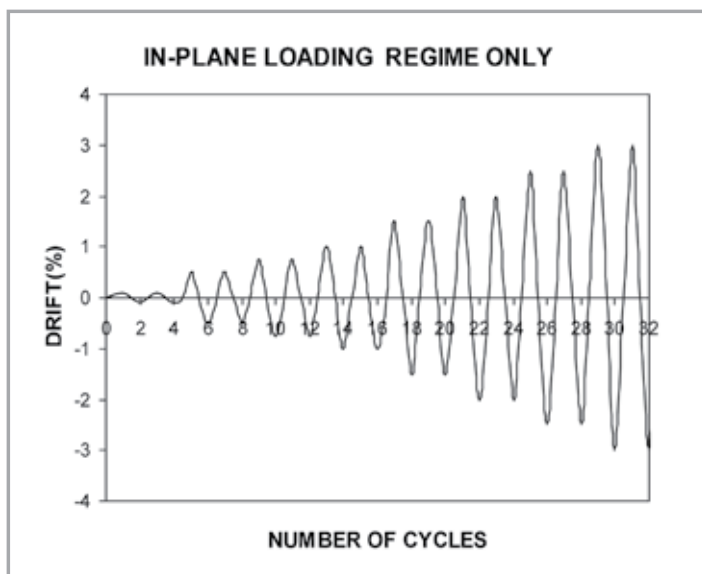

Figure 9: Testing schedule for monolithic wall panel

7.0 EXPERIMENTAL RESULTS AND VISUAL OBSERVATIONS

During the experimental work, crack widths were measured on the wall panel at a number of locations especially within the plastic hinge zone. Measurements of loadings and displacements were taken at both peak and unloaded semi-cycles up until widths in excess of 4mm were observed. Positive semi-cycles are those loading from East to West direction. Cracking of the wall was observed from the very first cycle to +0.25% drifts. These cracks were only 0.1mm wide at the peak load and closed completely once the unit was unloaded. The extent of these cracks covered approximately 2/3 the width of the wall and extended 1.4m up from the foundation. Figure 10(a) shows the crack distribution on the surface of wall panel when in-plane loading was applied at top of the wall at 0.5% drift. A second cycle of 0.5% drift resulted in the extension of old cracks. At this drift level cracks from loading in one direction began to prominently connect with cracks formed through loading from opposite direction. Figure 10(b) shows a wider crack began to develop within the wall panel when the drift was increased to 1.0% drift. The widest crack was measured at 2.2mm and residual drift could be observed through naked eye. Repeated semi-cycles to ±1.0% drift produced few new cracks within the plastic hinge zone.

Complete failure of the specimen was determined at an imposed drift of 3.0%. The eastern outermost bars fractured at the semi-cycle to +3.0% drift. Extremely large cracks ran through the plastic hinge zone in particular, with the majority remaining open upon the unloading structures [see Figures 11(a) and (b)]. Extensive spalling at the corners of the wall caused much of the longitudinal steel to buckle resulting in the unit being virtually unrepairable.
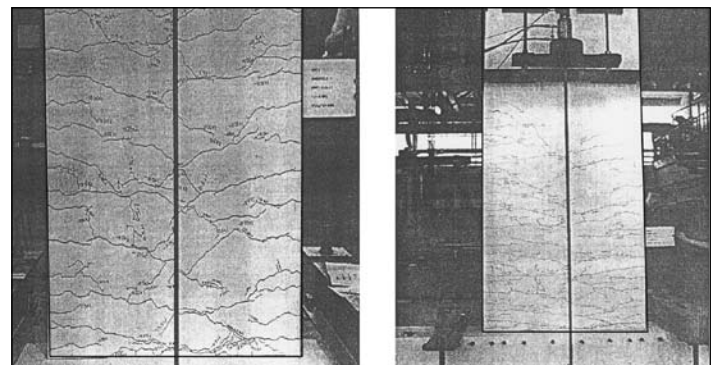

Figure 10: Experimental observation of cracks on surface of specimen; (a) North face of specimen at -0.5% drift; and (b) North face of specimen at +1.0% drift
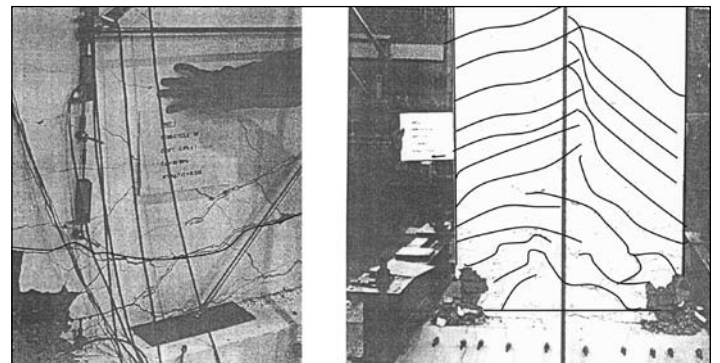

Figure 11: Visual observation at damage stage; (a) fractured of longitudinal reinforcement bars at -3.0% drift; and (b) view of North face of the specimen at the end of the test (after 3.0% drift)

8.0 ANALYSIS OF EXPERIMENTAL RESULTS

Load versus displacement graphs are measured and plotted in this study. The lateral cyclic displacements are measured using linear potentiometers which labeled as P6, P5, P4, P3 and P2 (see Figure 8). In this experiment, the controlled displacement is located at P5 where it is 3500 mm above strong floor. The applied loads are recorded using load cell and double actuator for three cycles for each drift. Since the overall data for hysteresis loops for load versus displacement are huge, all the data were exhibited in the Appendix 2A [8]. Only the graph for load versus displacement at position P5 is plotted and discuss in this paper. The load versus displacement graphs for different potentiometers are plotted in the thesis [8].

Figure 12 illustrates the overall hysteretic behaviour of wall panel under in-plane lateral loading up 3.0% drift at location of P5. The individual solid light line shows a hysteresis loop for one cycle of loading when wall panel moved from original position to East to West direction and go back to its original position. The loads are applied at a height of 3500 mm above strong floor with incremental percentage of 0.5% drift. Drift is defined as the ratio of lateral displacement divide by the height of the wall panel multiplied by 100%. The drifts for each cycle are 0.05%, 0.1%, 0.5%, 1.0%, 1.5%, 2.0%, 2.5% and 3.0% drift (see Figure 12). The solid light lines show the movement/displacement of wall panel in-plane directions (East to West) with respect to the applied load. The shape and size of the loops are typical of the expected for a cast-in-place monolithic wall panel. In addition, the theoretical backbone curve (marked as solid heavy line) plotted on the experimental results. The theoretical analysis is plotted based on the mathematical equation derived from Equations (1) to (6) and Figure 3.

The experimental result shows very similar and good agreement with theoretical results. The theoretical backbone gives a relatively close approximation to the measured response, suggesting only a slightly greater capacity at the higher drift levels. The performance of the specimen was close to that

predicted analytically as discussed in Section 3 and Figure 3 in this paper. The graph also shows a typical behaviour characteristic of a conventionally monolithic reinforced concrete structural wall. Energy was dissipated mainly through yielding of the longitudinal wall reinforcement at the plastic hinge region. The connection of the wall to the foundation beam had no influence on the overall response of the unit. Adequate detailing of reinforcement bars at plastic hinge zone which located at one-fifth from wall panel can allow the structural wall panel to deform in a ductile manner as clearly shown in Figure 12.

Figure 13 shows the closely hysteretic response of wall panel up to 1.0% drift. The yield point of the extreme longitudinal bars is plotted on this curve. Yielding was found to have taken place on the first cycle to ±0.5% drift (21mm) at about 140kN lateral force. The theoretical; backbone curve (marked as solid heavy line) fits very well with the hysteretic response of wall panel as shown in Figure 13. The elastic stiffness and reference yield displacement shown are based on the secant line passing through the point at which the first yielding was observed. The reference yield drift ratio obtained for this unit was 0.5% drift. Consequently, the ductility capacity of this unit was 2.5%/0.5% = 5.0 which is equal to the value selected in this design. The amount of energy dissipated can be measured using equivalent viscous damping equation. By breaking down the hysteretic response into its individual cycles, the level of equivalent damping for the structures can be obtained Chopra [9]. The theoretical equivalent viscous damping for a system with hysteretic behaviour is calculated by

$$\xi_{eq} = \frac{1}{4\pi} \frac{E_D}{E_{SO}} = \frac{1}{2\pi} \frac{E_D}{F_{max}\Delta_{max}} \qquad (7)$$

where E_D = the theoretical cyclic pushover curve area under a hysteresis loop and E_{so} = $1/2(F_{max}\Delta_{max})$; F_{max} = average maximum strength in forward and reverse loading directions and Δ_{max} = average maximum displacements in both loading directions.
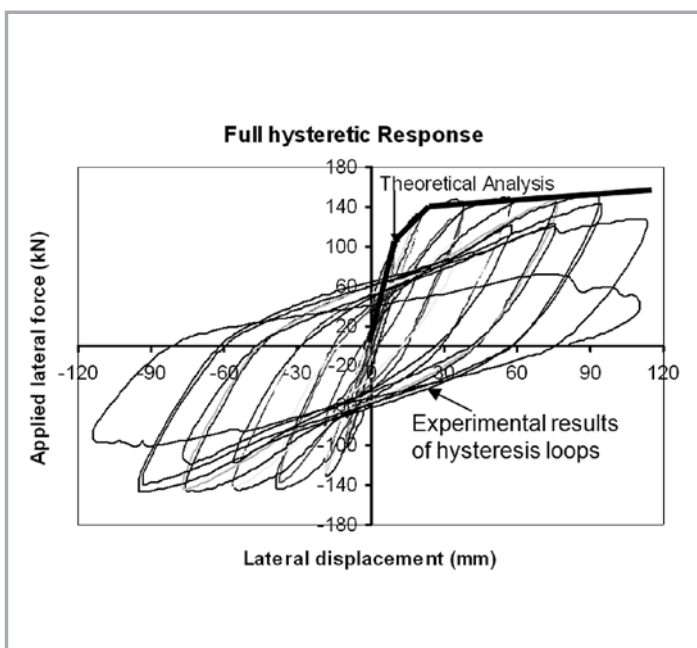

Figure 12: Full hysteretic response of wall specimen under in-plane loading
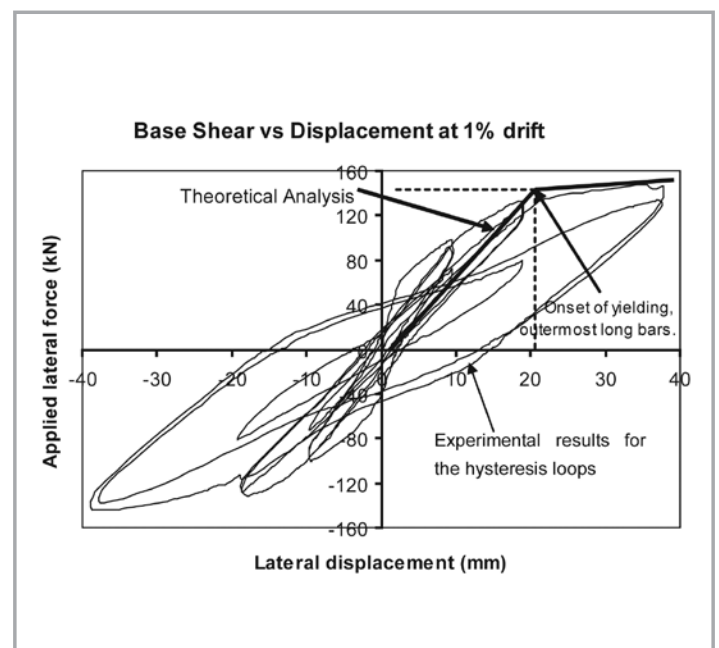

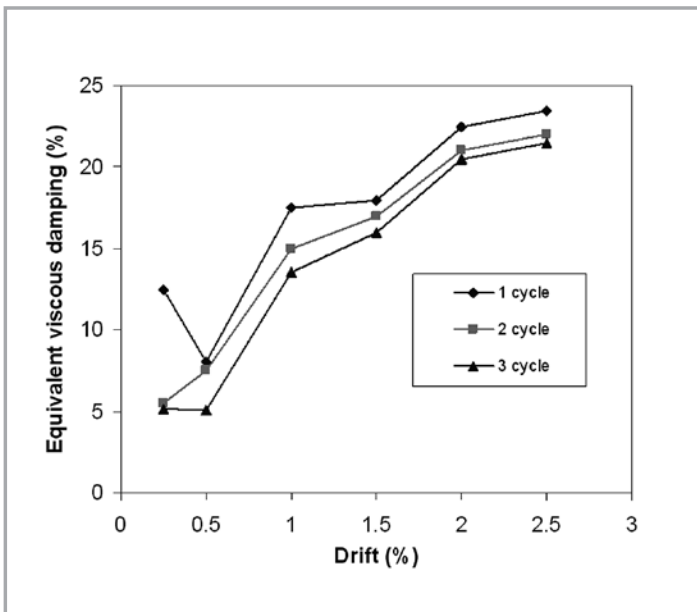*Figure 13: Hysteretic response of wall specimen up to 1% drift*

Figure 14: Equivalent viscous damping from hysteretic response of specimen

Figure 14 illustrates how equivalent viscous damping increases from approximately 5% at 0.25% drift to 23.5% at 2.5% drift for three cycles. The increase of damping in the first cycles to 0.5% and beyond is mainly due to yielding of the longitudinal reinforcement. The first cycle from 0.25% to 0.5% lateral drift shows a drop off in damping due to high levels of released energy that comes from the initial cracking of the specimen. In the cycles to large drift values the loss of damping in the second cycle can be attributed to the fact there is initial strain energy dissipated through concrete in compression. As the result strains remain in the concrete, the amount of energy dissipated by the concrete is significantly reduced in the second cycle. The equivalent viscous damping for the third cycle is lesser than second cycle.

For monolithic wall panel, the average equivalent viscous damping is approximately 23% which is somehow bigger than equivalent viscous damping for rocking wall panel [8]. The equivalent viscous damping for monolithic slender/thin reinforced concrete wall tested using shaking table is equivalent to 0.76% [10]. It was done by transferring the response into frequency domain and utilising half power bandwidth method.

Another experimental work was conducted on rocking slender/thin precast wall panel subjected to dynamic loading and the total equivalent viscous damping was measured at maximum in-plane is 7% [11]. A similar identical rocking slender wall panel was tested under biaxial loading (in-plane and out-of-plane direction) and the measured equivalent viscous damping is 10% [12]. Based on the overall comparison of equivalent viscous damping between rocking slender precast wall panel and monolithic wall panel, monolithic wall panel dissipated more energy than rocking slender wall panel. It can be observed that more structural damage were occurred in monolithic wall such as cracks, spalling of concrete and fractured of longitudinal reinforcement bars as compared to rocking slender wall. In other words, the wall panel which is designed using rocking

structure concepts can behave well under earthquake excitation then monolithic reinforced concrete wall panel.

The specimen performed as per design, its behaviour typical of a monolithic reinforced cast-in-place structural wall. Its capacity was very close to the predicted analytically. Capacity design principles ensured an elastic mechanism formed particularly well. Virtually no pull-out of the wall panel was observed during testing and providing idealised monolithic connection assumed in design. The specimen attained a displacement ductility of $\mu_\Delta = 5.0$, and a drift of 2.5% when significant strength degradation occurred. As expected from the behaviour of the conventional monolithic construction large residual cracks, exceeding 2mm in width were observed after undergoing a peak drift of 1.0% drift. Residual drifts once the unit was pushed into the inelastic domain, which is also a characteristic of monolithic construction, where marginally smaller than at peak drifts.

9.0 CONCLUSIONS

The monolithic wall panel which designed according to New Zealand Standard (NZ 3101: Part 1:1995) experiences a substantial structural damage when tested under quasi-static lateral cyclic. The structural damage were wall cracks, spalling of concrete and fractured of longitudinal reinforcement bars at both bottom corner of the wall. Energy dissipation through the formation of plastic hinge zone will cause a lot of damage to wall panel. The recess connection provided is the ideal monolithic connection assumed in design. The unit started to crack at 0.25% drift and eventually failed due to fracture of longitudinal reinforcement bars at 2.5% drift. The unit experienced strength degradation at 3.0% drift at first cycle and collapse at second cycle. Residual cracks between 1 mm and 2.2 mm in width were recorded at the onset of yielding at 1.0% drift. The equivalent viscous damping of this unit is very high in the first cycle, followed by second cycle and third cycle. This parameter is used to measure the amount of energy dissipated during ground shaking. It is meaning to say that most of the energy is dissipated in the first cycle due to the formation of plastic hinge zone in the wall-foundation beam interfaces.

Based on the past experimental results such monolithic slender wall under some earthquake excitation [10], dynamic response of rocking precast slender/thin wall panel [11] and biaxial behaviour of slender/thin wall panels [12], wall panel which designed using the concepts of rocking structures can performed better than monolithic reinforced concrete wall panels. This can be proved by measuring the amount of equivalent viscous damping where the rocking wall panel has approximately $\xi_{eq} = 7\%$ to $\xi_{eq} = 10\%$ and monolithic reinforced concrete wall has $\xi_{eq} = 23\%$. Therefore, it can be concluded that rocking precast wall performed better than monolithic wall in terms of damage index, ductility, strength and equivalent viscous damping. It is suggested that further experimental work and analysis should be conducted for monolithic reinforced concrete wall panel which designed according to British Standard (BS 8110: Part 1: 1997) with minimum percentages of 0.24% reinforcement in longitudinal and transverse directions. ∎

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