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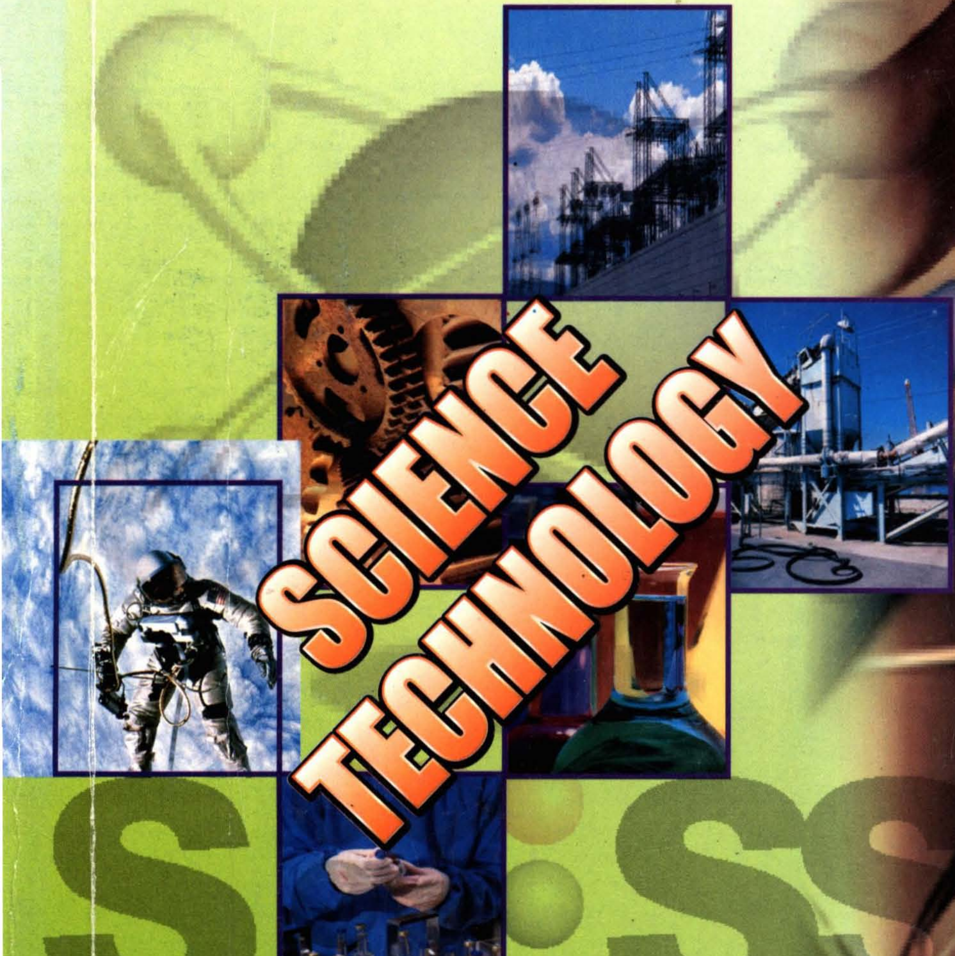

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Experimental Seismic Performance of Multi- Panel Precast Hollow Core Walls in Warehouse Buildings

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ABSTRACT

The seismic resistance of a superassemblage of precast hollow core wall units for warehouses is investigated. The superassemblage consists of six prestressed concrete 1.2m wide hollow core units. Two of the units are tied to the foundation via unbonded vertical tendons while the other four units primarily act as "non-structural" cladding. The superassemblage represents the wall of a single storey warehouse type structure. The longitudinal unbonded prestressing tendons consist of regular thread-bars with an in-series portion of those bars possessing a reduced diameter to act as "fuses". Prior to testing, the fuse-bars are prestressed to 50% of their yield capacity. The multi-panel wall is tested under several different conditions: in-plane quasi-static reverse cyclic loading with different sizes of fuse-bars; and with and without rubber block spacers and sealant between units. Experimental results demonstrate that smaller diameter fuses lead to superior behaviour, as foundation uplift is inhibited. No structural damage occurs up to the experimental ± 4% drift limit. Some minor non-structural distress is observed to commence with sealant failure at 3% drift. This damage, however, is inexpensive to repair. Results also show that the hysteretic energy absorption that arises from the yielding tendons as well as the interacting rubber spacers and panel sealants provides an equivalent viscous damping factor of 10% at design drift amplitude of 2%. The overall good performance of the multi-panel wall system well satisfies the requirements of an emerging seismic Damage Avoidance Design (DAD) philosophy.

Keywords: *multi-panel precast walls, superassemblage, fuse-bars, unbonded prestressing tendons and damage avoidance design philosophy.*

Introduction

The application of precast hollow core units without transverse reinforcement as wall panels is common in certain non-seismic regions like Malaysia. Precast hollow core walls offer several advantages compared to monolithic conventional reinforced walls: design flexibility; faster construction; improved economy; no formwork; a load-bearing ability without the need for columns; and a variety of concrete finishes. The research presented herein seeks to extend the use of precast hollow core walls so that they can be constructed in moderate to high seismic regions. It has been demonstrated that single hollow core walls are capable of resisting substantial lateral loads in spite of their lack of transverse/shear reinforcement, providing the connection details are modified. But a study on a single wall panel alone is insufficient to assess the overall building performance under earthquake ground shaking. This research, therefore, utilizes an assemblage of precast hollow core wall panels to form a rocking wall system that would be representative of a prototype warehouse building. The superassemblage is tested under in-plane quasi-static reversed cyclic lateral loading. The objective of this experimental work is to investigate the relative contributions of strength and equivalent viscous damping of various components that make up a multi-panel wall system. In addition to the post-tensioned seismic wall panels, components included are rubber block spacers, sealant and bearing pads, and a steel channel cap beam that is used to tie the panels together.

The main criteria in designing multi-panel walls are the diameter of the fuse-bars and the initial level of prestress. The fuse-bar capacity must be sufficient to resist seismic and wind loads, but at the same time there should be no tensile uplift of the foundation. The most suitable initial prestress of fuse-bar is about 50% of its yield capacity; this gives the best trade-off between energy dissipation and displacement capacity. The main reason for choosing fuse-bars as the only means of energy dissipation is because they are easy to be restressed or replaced after a strong earthquake. Moreover, the fuse-bars operate in tension only, thus they are not prone to buckle, nor do they tend to "soften" the structure as tension-compression bonded fuses or external mechanical energy dissipators such as used by Holden et al. (2003).

The absence of transverse reinforcement in precast hollowcore wall units is not a major problem when using hollowcore units in seismic regions. But the base of each seismic resisting wall unit needs to be "damaged protected". The basic hypothesis of this research is to combine the self-centering concepts of rocking, together with Damage Avoidance Design (DAD) armouring details (Mander and Cheng 1997). The multi-panel wall system consists of seismic and non-seismic wall panels which are designed to rock on their foundations; the system can be implemented and constructed in high seismic regions. This paper first reviews important findings from associated research, and then goes on to present a concept development for single storey warehouse type structures. An experimental study is presented next and finally the results are discussed in terms of seismic behaviour attributes.

Findings from Previous Research

To date, minimal research has been conducted on multi-panel wall systems as compared to single wall panels. Much of the past research has focused on the performance of single precast wall panels under quasi-static cyclic lateral loading, dynamic loading and biaxial loading (McMenamin 1999; Rahman and Restrepo 2000; Holden et al. 2003; Surdano 2003; and Liyanage, 2004). A number of studies also have focused on the shear slip and opening gap occurred in a stack of horizontal panels by incorporating unbonded post-tensioning in precast multi-storey buildings (Kurama et al. 1997; Kurama et al. 1999; Kurama 2000; Kurama 2001, Furutani et al. 2000; Ile and Reynoud 2004). The PRESSS (Precast Seismic Structural Systems) research programme has carried out experimental work on a 60% scale five-storey precast building with two vertical precast wall panels joined to each other using U-shaped Flexure Plate mechanical energy dissipating connectors (Nakaki et al. 1999; Priestley et al. 1999; Conley et al. 1999; Wallace and Wada 2000). However, these previous studies (apart from some work by Holden et al. (2003), Surdano (2003) and Liyanage (2004)) did not integrate and protect the bottom part of precast wall using Damage Avoidance Design (DAD) approach developed by Mander and Cheng (1997) for bridges.

Stanton and Nakaki (2002) used self-centering concepts for four precast wall-panels by utilizing unbonded tendons on each wall, with shear connectors between the walls. Rocking took place on a grouted bed. They proposed unbonded post-tensioning steel and gravity loads located at center of each wall with one initial prestressing tendon. They primary considered one limit state at the onset of yielding the post-tensioning tendons.In a recent study, Perez et al. (2004a) investigated the seismic performance of three two-storey, full-height precast concrete panels using two groups of post-tensioning steel tendons with additional limit states such as loss of initial prestress, crushing of confined concrete and fracture of the prestressing steel. They used the same vertical joint shear connectors for jointing two pieces of wall panels. Two unbonded post-tensioning steel tendons were used for each wall across the horizontal joints which were not located at the center of wall. Spiral reinforcement was employed to confine each bottom corner of the wall to sustain large compressive strains during closing and opening gap of the wall. Following this study, Perez et al. (2004b) developed a fiber-based analytical model for three panel walls under monotonic pseudo static lateral loads. They recommended that the lateral load behaviour of this wall can be controlled by adjusting the total area of post-tensioning steel tendons, the initial prestressing and total shear yield force of vertical joint connectors. Despite of the usefulness of this model in seismic design, it has not been validated with experimental work.

Prototype Design of Multi-Panel Walls

The new design approach is employed in this study seeks to demonstrate that no transverse or spiral reinforcement is required for a seismic resistant multi-panel precast concrete hollowcore wall system. This is achieved through permitting individual panel units to be free to rock on the foundation. The multi-panel wall system is divided into "seismic" and "non-seismic" panels – the former carrying the gravity (inertia) loads, while the latter eventually becomes non-structural cladding. This research will seek to determine whether only a limited number of the wall panels (say 15 to 20 percent) are sufficient to be prestressed to provide seismic resistance. By dividing the wall into seismic and non-seismic panels it is important to understand the interaction between the two, and what the weathertightness (sealant) needs should be under both normal (service) and extreme (seismic loading) conditions.

Based on the foregoing criteria, a prototype structural system has been conceived. Figure 1 shows a warehouse type industrial building that consists of a series of multi-panel precast concrete hollowcore walls. Figure 1(a) shows longitudinal and transverse lateral seismic (or wind) loading acting on the single-storey structure. A roof truss diaphragm system is used to transfer these loads to an edge member that is shown as a steel channel in Figure 1(b). The channel is attached at each rafter location via post-tensioned prestressing tendons which in turn are anchored into the foundation. Thus the "seismic walls" are clamped to the foundation under normal service loads. Under high lateral loading the walls are free to rock, but they are also restrained by the elastically elongating tendons which permit re-centering at the termination of seismic shaking. Under uplift during earthquake excitation, seismic energy can be dissipated by using in-line fuses that restrict the amount of force that can be transmitted to the foundation.

Between these "seismic wall" panels, non-seismic panels are placed and seated on a continuous rubber bearing pad. In order to permit large in-plane movement between these "non-seismic" panels it is necessary to provide a "seismic gap" and detail the vertical joints between individual panels with care. Figure 1(c) shows the design joint width for the installations of sealant and rubber block spacers between the walls' gaps. For an upper target design drift the shear strain on the rubber spacer blocks is given by

$$\gamma = \frac{\delta_h}{t_{gap}} = \theta \frac{B_w}{t_{gap}} \qquad (1)$$

in which $\delta_h =$ uplift displacement; $t_{gap} =$ the thickness between wall panels; $B_w =$ panel width; and θ = the target design drift.

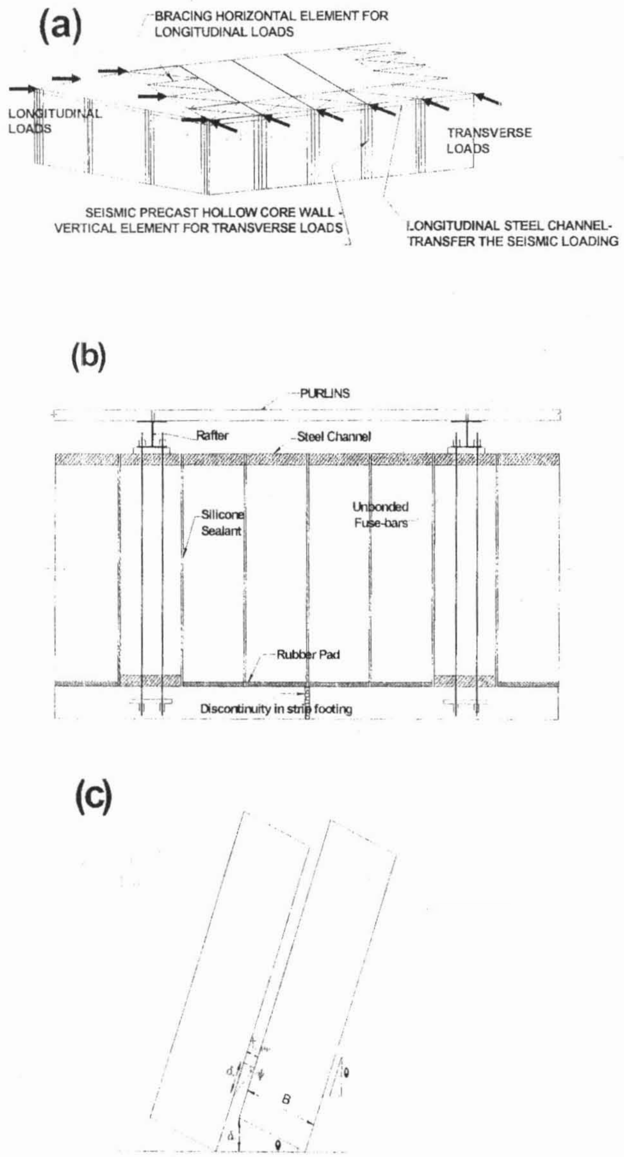


Fig. 1: The Superassemblage Multi-Panel Wall is representing as part of a Prototype Warehouse; (a) the Isometric Warehouse showing the directions of loading and schematic arrangement of Seismic Walls; (b) Side Elevation of Multi-Panel Wall consisting of Seismic and Non-Seismic Wall; and (c) the Design Joint Width between the Gap of the Walls

Resistance Mechanisms in a Multi-Panel Wall System

There are four principal components that contribute to the overall resistance of a multi-panel wall system, as shown in Figure 3.2, these are given by

$$F_H = F_{SW} + F_{NS} + F_V + F_{CH} \tag{2}$$

where $F_H =$ total lateral force applied at the eaves level, $F_{SW} =$ resistance provided by the post-tensioned seismic wall including the effects of fuses and mechanical energy dissipators (if any); $F_{NS} =$ resistance arising from the self-weight of the non-seismic walls; $F_V =$ shear resistance contribution arising from the sealant compound between the walls; and $F_{CH} =$ contribution of the plastic mechanism of the steel channel. Figure 2(a) shows the principal resistance mechanism arising from the post-tensioned walls. By taking moments about the toe of the rocking wall unit

$$F_{SW} = \frac{B_w}{2H}\left(W_r + W_W + T_1 + T_2\right) + \frac{e_p}{H}\left(T_1 - T_2\right) \tag{3}$$

in which $B_w =$ panel width; $H =$ wall height; $W_r =$ reaction load from the rafter; $W_W =$ self-weight of the wall panels; T_1 and $T_2 =$ respective forces in the first and second tendons; $e_p =$ the eccentricity between the unbonded post-tensioned tendons.

Figure 2(b) shows the resistance of one non-seismic wall panel as a result of self-weight

$$F_{NS} = \frac{B_w}{2H} W_W \tag{4}$$

The lateral resistance provided by imposing shear deformations along each vertical wall joint as shown in Figure 2(c) can be found from

$$F_v = \frac{B_w}{H} V_r \tag{5}$$

where $V_r =$ total shear resistance provided by rubber spacing blocks plus the sealing compound.
Figure 2(d) shows the seismic walls connected by the steel channel along with Figure 2(e) which depicts the deformed plastic mechanism that leads to plastic hinges in the channel. Using virtual work principles it can be shown that the resistance contributed by the mechanism is

$$F_{CH} = \frac{2M_p}{H}\left(1 + \frac{1}{n_{ns}}\right) \tag{6}$$

where $n_{ns} =$ number of non-seismic walls placed in between the seismic walls; and $M_p =$ plastic capacity of the reduced channel section.

Substituting equations (3) to (6) into (2) and normalizing with respect to the total seismic weight (W_T) gives the base shear capacity.

$$C_c = \frac{F_h}{W_T} = \frac{B_w}{2H}\left(\frac{W_r + W_W + T_1 + T_2}{W_T}\right) + \frac{e_p}{H}\left(\frac{T_1 - T_2}{W_T}\right) + n_{ns}\frac{B_w}{H}\frac{W_W}{W_T}$$

$$+ \frac{B_w}{H}\frac{Vr}{W_T}\left(n_s + 1\right) + \frac{2M_p}{HW_T}\left(1 + \frac{1}{n_{ns}}\right) \tag{7}$$

where the total seismic weight is given by $W_T = W_r + (n_{ns} + 1)W_W$.

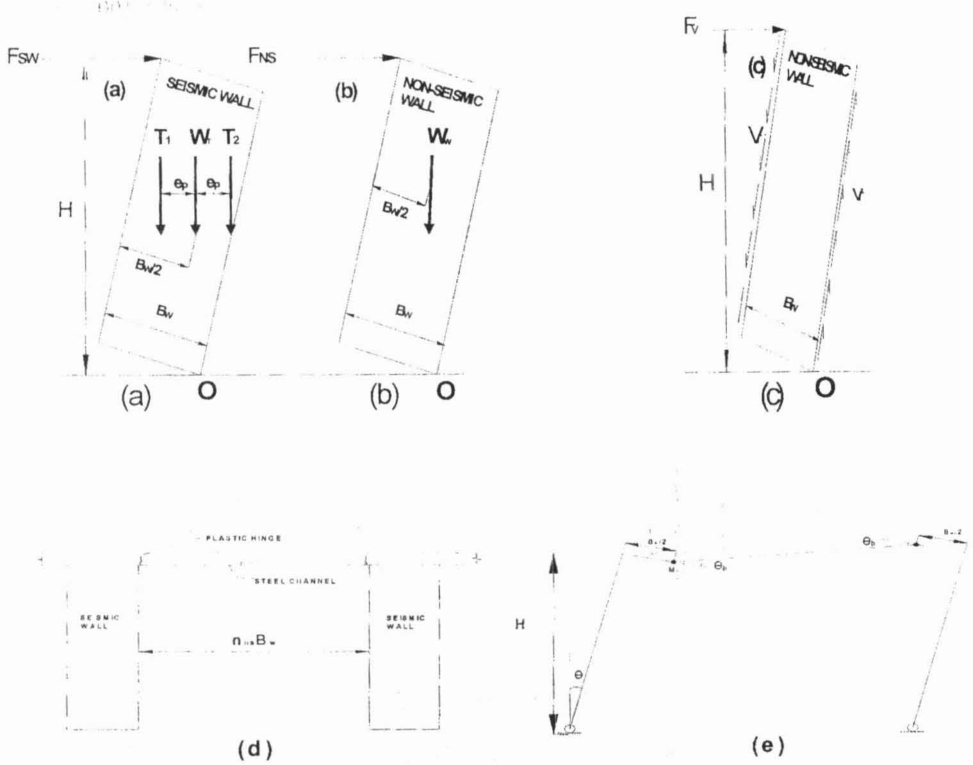
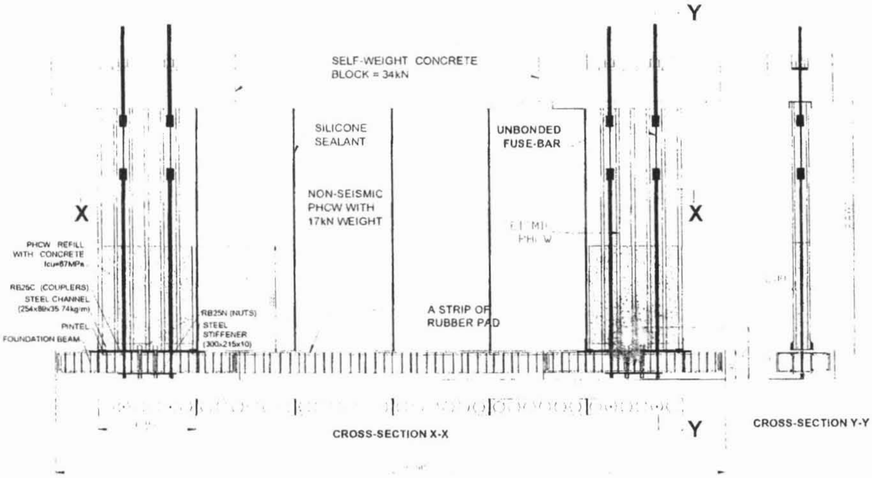


Fig. 2: Resistance Mechanism of a Multi-Panel Wall System; (a) Lateral Resistance due Post-Tensioned Tendons and Self-Weight of Seismic Walls; (b) Lateral Resistance Coming from Self-Weight of Non-Seismic Wall; (c) Shear Resistance from Silicone Sealant ; (d) Plastic Hinge Occurred at V-cut Shape of Steel Channel Closed to Seismic Wall; and (e) the Plastic Mechanism on Steel Channel Cap Beam.

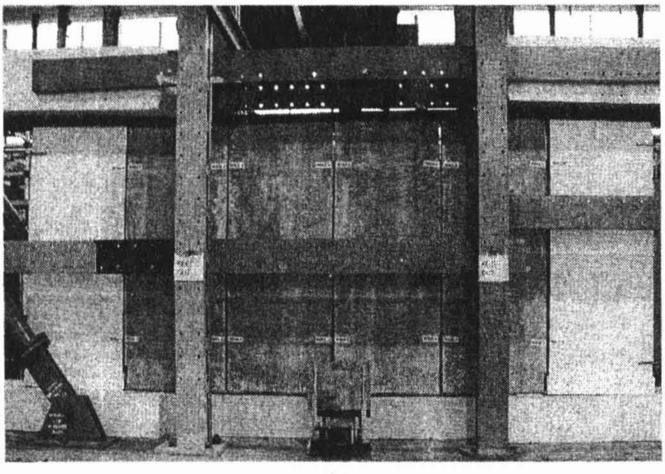
Design and Construction of the Multi-Panel Wall Superassemblage

Figure 3 shows the reinforcement details and the experimental setup of the super-assemblage. Wall 1 and Wall 2 together with their own foundation and four new hollowcore units as the non-seismic infill wall panels. Initially, 20mm diameter, 500mm long fuse-bars were used in series with the 25mm thread bar tendons. They were inserted into the second and fifth void sections of the seismic walls and screwed into couplers located at two-thirds height of the walls, as shown in Figure 3(a). An infill spread footing foundation beam (4730x350x400mm) was constructed between the original seismic foundation beams (that were located beneath the seismic panels) and connected contiguously to the seismic foundation beam. The entire foundation beam was anchored to the laboratory strong floor to inhibit sliding, but note that no uplift or hold down restraint was provided. This was to ensure the foundation beam was a true representation of a spread footing. Beneath the non-seismic wall panels a 4730x350x20mm rubber pad (IRHD55) was placed to seat those panels.

A 9000mm long 254x79x28.9kg/m steel channel cap beam was placed on top of the seismic walls to tie them together. The channel spanned across, but did not touch, the non-seismic wall panels. The channel was prestressed to the seismic walls to ensure it acted as a tie beam. A V-shape cut to the channel flanges was applied to allow the lateral load to transmit to the next seismic wall only through the steel web. The purpose of this cut was to minimize flexural bending of the channel. Mass concrete blocks, 34kN each, were placed on top of the seismic walls to represent the gravity load reaction from the roof/rafter system. The bottom concrete blocks were bolted to the top of the steel channel using 20mm bolts with six steel plates welded on each flange of the steel channel. The main reason for bolting each concrete block to the steel channel was to ensure proper transfer of lateral load from the hydraulic actuator to the first and second seismic walls. Two different sizes of rubber blocks, 100x50x40mm were placed into the inner wall gap and 100x50x25mm, were inserted into the outer wall gaps. A photograph of the overall front elevation view of multi-panel precast hollow core wall system together with the foundation beam is shown in Figure 3(b). The seismic wall and seismic foundation beam are painted white, while the grey units are non-seismic infill wall panels.



(a)

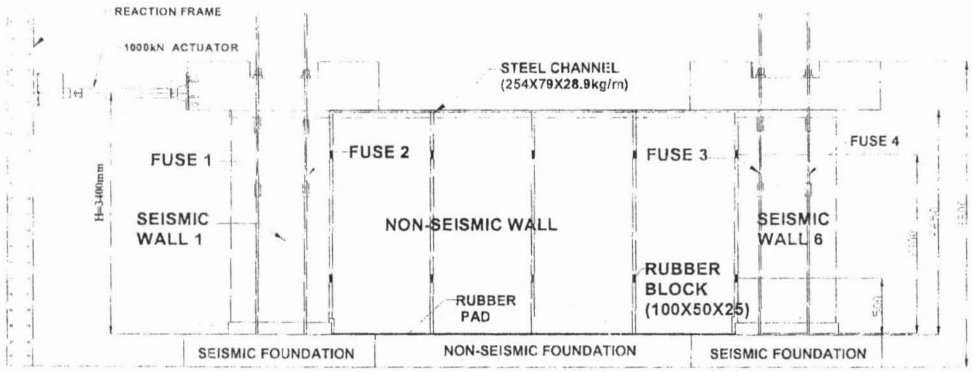


(b)

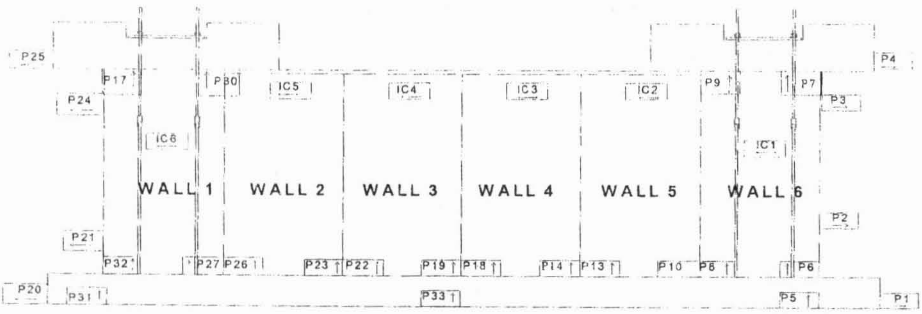
Fig. 3: Construction and Reinforcement Detail of Multi-Panel Precast Hollow Core Walls; (a) Details of Reinforcement and Front Elevation of the Schematic Arrangement Seismic Wall and Non-Seismic Wall; and (b) Multi-Panel Super-Assemblage Representing Part of the PHCW System in a Warehouse Building.

Experimental Setup, Instrumentation and Testing Procedures

Figure 4 depicts the experimental set-up and instruments and of the multi-panel walls as tested on the laboratory strong floor. Figure 4(a) shows the schematic arrangement of seismic wall, non-seismic wall, location of in-series unbonded fuse-bars, steel channel cap beam, foundation block and in-plane actuator attached to reaction frame. Lateral load was provided by a 1000kN actuator with force being measured by an in-series load cell. The experiments were conducted in "drift" control where drift was defined as the angle difference between upper and lower displacement transducers mounted on the wall panel adjacent to the hydraulic actuator (P4). A quasi-static cyclic reversed lateral force regime was applied at the center of the mass which was located at 3400mm height from strong floor. Figure 4(b) shows the instrumentation used during the experiments. Twenty seven linear potentiometers that were used to monitor uplift and sliding of the wall units, the foundation beam, top concrete blocks and rocking toe of each wall were employed. Six rotary potentiometers were attached on both sides of the seismic walls to trace any rotation of the seismic wall panels and sliding of top mass concrete blocks. Six inclinometers were used to measure the inclination angles of each panel during the rocking process. They were positioned at the mid-width of each wall and at a 2450mm height above the foundation beam. "Demec" points (demountable mechanical strain gauges) were placed on a 250mm grid on Walls 1 and 2 to infer concrete strains at different levels of drift and determine the stress contour distribution under the reverse seismic loading. Demec points were also used to measure gap movement between wall units. Strain gauges were affixed to each of the unbonded fuse-bars and calibrated to measure prestress levels and force changes under uplift of the panel units during lateral loading. Prior to testing, the fuse-bars were prestressed individually up to 50% of their yield capacity.



(a)



(b)

Fig. 4: The Experimental Set-up of Multi-panel Superassemblage of Precast Hollowcore Wall Units; (a) the Loading Frame Including the Arrangement of Seismic, Non-Seismic Wall and Fuse-Bars for Multi-panel Wall System; and (b) the Schematic Arrangement of Potentiometers with their Direction of Measurements.

The super-assemblage was tested under completely reversed cyclic lateral load in three phases as follows:

Phase 1: Rubber blocks spacers between the gap with 20mm diameter and 500mm length of fuse-bars were tested at ± 0.1%, ± 0.5% and ±1.0% for 2 cycles at each level drift. The positive semi-cycle drift were imposed by loading the double-actuator ramp from west to east and conversely, the negative semi-cycles drift from east to west direction. Any uplift of the foundation block was recorded by two linear potentiometers located at both ends of the seismic foundation block.

Phase 2: To mitigate the potential for foundation uplift, smaller 13mm diameter of unbonded fuse-bar was used. The new fuses were also strain gauged and the superassemblage was re-tested under two cycles at each drift amplitude of ± 0.1%, ± 0.5%, ±1.0%, ±1.5% and ± 2.0%.

Phase 3: A silicone sealant was installed on both faces on the walls. After a two week curing period the specimen (with 13mm diameter fuse-bars, sealant and rubber blocks) was tested under two reversed cycles at drift amplitude of ± 0.1%, ± 0.5%, ±1.0%, ±2.0%, ± 3.0% and ± 4.0%.

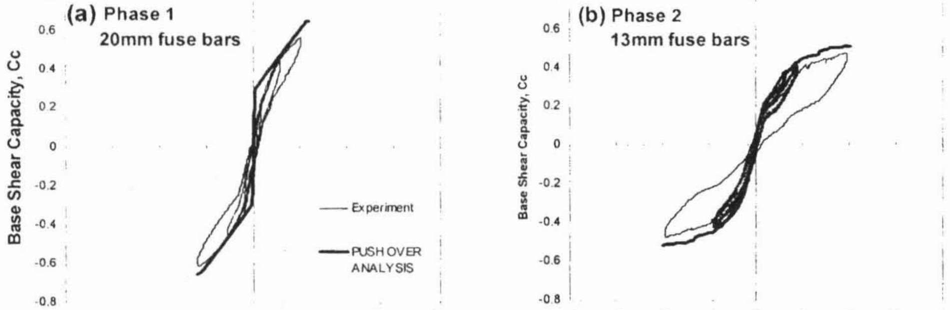
Based on the foregoing criteria a joint width of 40mm between wall panels was adopted along with 100x50x40mm rubber block spacers. A propriety silicone sealant (Silaflex MS) was then applied to fill the remaining gaps between wall panels. It should also be noted that beside these structural requirements, the gaps must also fulfill the usual non-structural requirements such as durability, sound insulation, fire resistance, thermal insulation, watertightness, appearance and accessibility for inspection, maintenance and replacement.

Experimental Results and Observations

The overall and individual seismic performance of multi-panel walls on each phase as described above is presented in this section. The experimental results are classified according to their overall hysteretic performance, visual observation deformation of rubber block, sealant and damage on sealant. An important comparison is the potential for uplifting of the foundation block when using the 20mm and 13mm fuse-bars. The seismic performance of Phase 3 which represented the final construction state of a multi-panel precast hollow core wall system at 2.0% and 4.0% drift is also presented in this section.

Hysteretic Performance of Phases 1, 2 And 3

Figure 5 shows the overall hysteretic performance of the multi-panel precast hollow core walls system under Phase 1, 2 and 3. The initial run was conducted at ± 0.1% drift to ensure that all instruments recorded the correct magnitudes and directions of the lateral, uplift and rotation movements. Figure 5(a) shows the overall performance of multi-panel walls system at ±0.1%, ±0.5% and ±1.0% drift tested under Phase 1 using 20mm diameter fuse-bars and rubber block spacers. The yield base shear of multi-panel walls is 100kN with yield drift of 1.0%. As the level of drift increases, more energy was dissipated by engaging the base rubber pad and rubber blocks spacers. Under Phase 1, the superassemblage was only tested up to 1.0% drift because of a 7mm recorded uplift of the foundation block. The uplift caused some cracks within the foundation. Figure 5(b) depicts the overall seismic performance of multi-panel walls under Phase 2 at ± 2.0% drift with 13mm fuse-bars and rubber block spacers between the walls. The analytical results of base shear capacity show acceptable agreement with the experimental results. The overall system produced a reasonably good behaviour with self-centring provided by the unbonded fuse-bars and cap beam on top of the walls. Figure 5(c) shows the overall performance of multi-panel walls under Phase 3 using 13mm fuse-bars, rubber block and sealant tested up to ± 4.0% drift. Similar experimental results were obtained as predicted analytically. The base shear at 2.0% drift was 94kN under Phase 3 is slightly higher than 83kN under Phase 2. The multi-panel precast hollowcore wall system with sealant (Phase 3) dissipated more energy than Phase 2 (without sealant) as indicated by the increased area enclosed by the hysteretic loops.



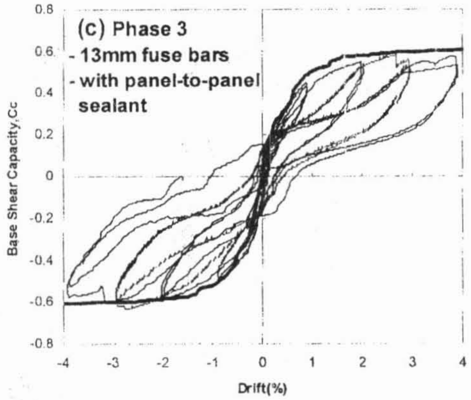


Fig. 5: The Overall Hysteretic Performance of Phases 1, 2 and 3: (a) Experimental Result for Phase 1 up to 1.0% Drift; (b) Experimental Result for Phase 2 up to 2.0% Drift; and (c) Experimental Result for Phase 3 up to 4.0% Drift.

Visual Observations and Damage

Several photographs taken during the course of the experimental study are presented in Figure 6. The in-plane lateral movement of the multi-panel superassemblage at + 2.0% and – 4% drift are shown in Figure 6(a) and (b), respectively. The steel channel effectively transferred the lateral force from first to the second seismic wall. However, due to the compressibility of the rubber block spacers, all non-seismic walls had smaller displacements than the drifts imposed on the seismic walls. Figure 6(c) shows how the shear strain distribution of rubber block spacer varied linearly between Wall 5 and Wall 6. A similar, almost linear, shear strain distribution is evident in the silicone sealant between the panels as shown in Figure 6(d). Based on the overall visual observation, the super-assemblies of multi-panel walls performed very well up to almost 3.0% drift. Although no structural damage was observed in any of the superassemblage specimens, some minor non-structural damage was evident in the silicone sealant which became torn following the 3% drift amplitude. No structural damage occurred to the rocking toe of both seismic walls as expected throughout the entire experiments.Figure 6(e) shows a local tensile bond failure at the sealant of Wall 5 and Wall 6 at -3.0% drift. This failure occurred when cohesive strength of sealant greater than cohesive strength on the edges of the walls. This is also attributed to imperfect preparation of the concrete surfaces. Due to the presence of concrete debris attached to the sealant, tearing away from that surface commerce early. Figure 6(f) illustrates the second example of failure known as folding failure. This failure arises when the silicone sealant experienced an excessive movement in compression resulting in permanent set leading to folding of the sealant. Figure 6(g) shows a general adhesion failure at + 4.0% drift. This failure occurred when the sealant generally lost its adhesive bond with the concrete panel surface. This became more pronounced when the sealant peeled off from the walls and displaced from it original position.

Similar to the conclusions drawn by Holden et al. (2003), this experiment showed that multi-panels precast hollowcore walls are able to perform better than conventional cast-in-situ reinforced wall because of non-existence of a conventional plastic hinge zone (PHZ) at the wall-foundation interfaces. In addition, self-centering rocking connection between wall and foundation block produced a pinching on hysteresis loops during unloading and allows them to rock backward and forward on their bases. The rubber pad, rubber block spacers and silicone sealant together provide a means to cushion and absorb seismic energy during rocking excursions. These materials are therefore recommended as suitable for future construction of industrial type buildings such as warehouses.

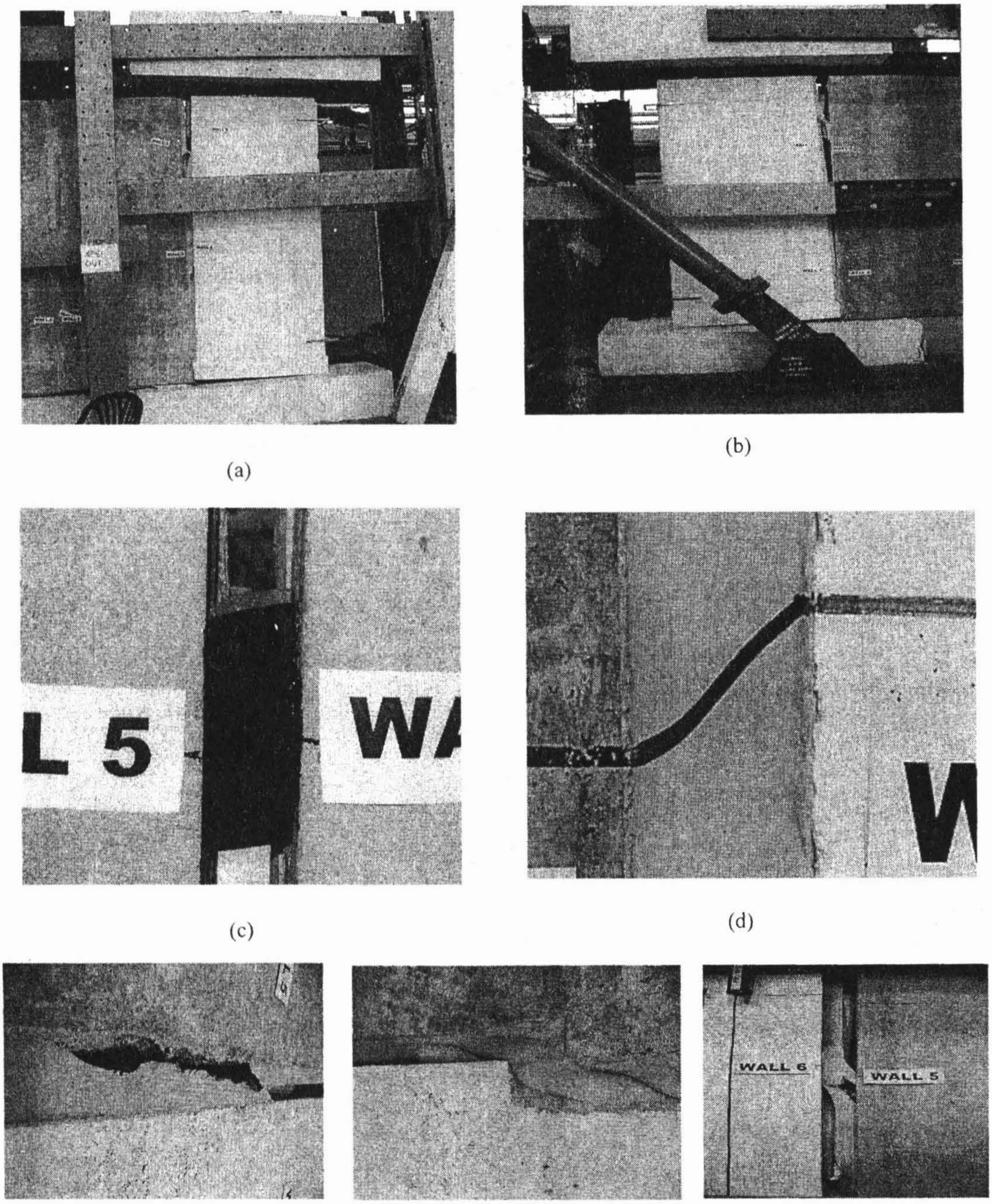


Fig. 6: Visual Observation and Damage; (a) The Walls Rocking at +2.0% Drift at E-W Direction; and (b) The Seismic Wall Rocking at -4.0% Drift at W-E Direction; (c) Deformation of Rubber Block at 2.0% Drift; (d) Deformation of Rubber Block and Sealant between the Gap; (e) Tensile Bond Failure when Sealant under Tension; (f) Folding Failure when Sealant under Excessive Compression; and (g) Adhesion Failure when Sealant under Excessive Tension.

Foundation Uplift

Figure 7 shows the uplift of the foundation block that occurred during the Phase 1 experiment as measured by potentiometers labeled as P31 and P5 (see Figure 4(b) for location). It is evident from these results that if the fuse-bars are permitted to transmit large forces, then foundation uplift, rather than wall rocking, will occur. To inhibit foundation uplift from occurring, the foundation block should either be made heavier or tension piles provided. Both solutions may be unduely expensive. Therefore, an alternative (counter-intuitive) solution is to provide a smaller prestress force through using smaller diameter fuses. Thus in Phases 2 and 3 of the experiments 13mm diameter fuses were chosen to replace the 20mm diameter fuses used initially in Phase 1.

Figure 7 also shows the comparison of uplifting two bottom corners of foundation block under Phase 1 and Phase 2 with same length of 500mm fuse-bars and rubber block spacers between infill walls. Potentiometers labeled as P31 and P5 were located on left and right hand side far end bottom corner of foundation block. This graph shows that there is no uplift of foundation block when using the 13mm fuse-bars with 50% initial prestress. With the bottom corners of foundation block being uplifted by 7mm, the bearing pressures beneath the remaining contact area increased. Although this was not a problem in the laboratory, in prototype field conditions resisting these increased bearing pressures could be unduely expensive.

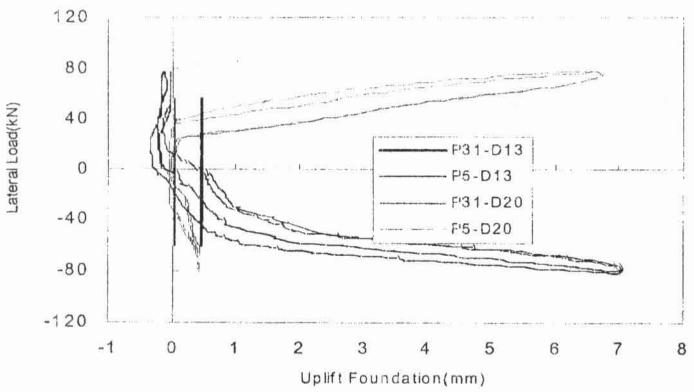


Fig. 7: A Comparison of Uplift Foundation Block between 20mm Fuse-bar and 13mm Fuse-bar in the Superassemblage of Precast Hollowcore Wall System.

Infill Wall Displacements

While Walls 1 and 6 showed similar displacement performance to the overall behaviour under Phase 1 as shown in Figure 8(a), the non-seismic panels, Walls 2 to 5 displaced somewhat less as shown in Figures 8(b) to (e), respectively. Figure 9 presents the comparative displacement results of the performance of each of the individual wall units in the superassemblage during Phase 3 of the experiment. The thin and thick lines represent the performance when the overall specimen was cycled through the 2% and 4% drift amplitudes, respectively. It will be noted that the seismic wall units (Walls 1 and 6) both were forced to experience the full displacement imposed, whilst the non-seismic wall units (Walls 2-5) experienced a decreasing amount of the imposed displacement as this was transmitted through the series of compressible rubber spacer blocks. Thus, at the 4% drift amplitude between Walls 5 and 6 there was a drift deficiency of 2.2%. This translates into a 65mm widening of the gap between Walls 5 and 6. This tearing displacement contributed to the deterioration of the sealant between the units.

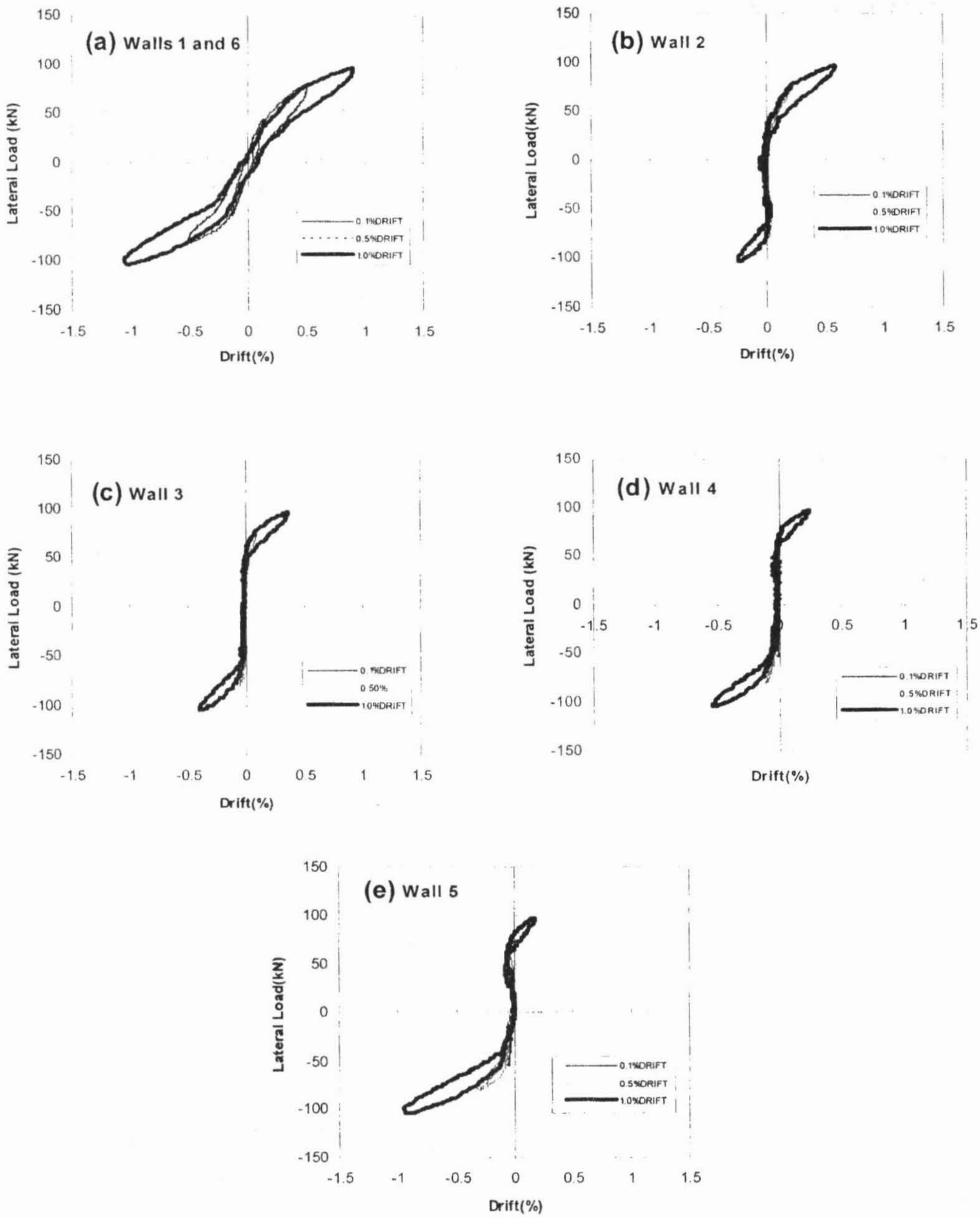


Fig. 8: Phase 1- Experimental results of The Multi-panel Precast Hollow Core Wall Superassemblage where 20mm diameter Fuse-bars and Rubber Block Spacers were used. Results are presented for up to 1.0% drift amplitude showing; (a) the overall Superassemblage behaviour similar to Seismic Wall 1 and (f) seismic Wall 6; (b) Non-seismic Wall 2; (c) Non-seismic Wall 3; (d) Non-seismic Wall 4; and (e) Non-seismic Wall 5.

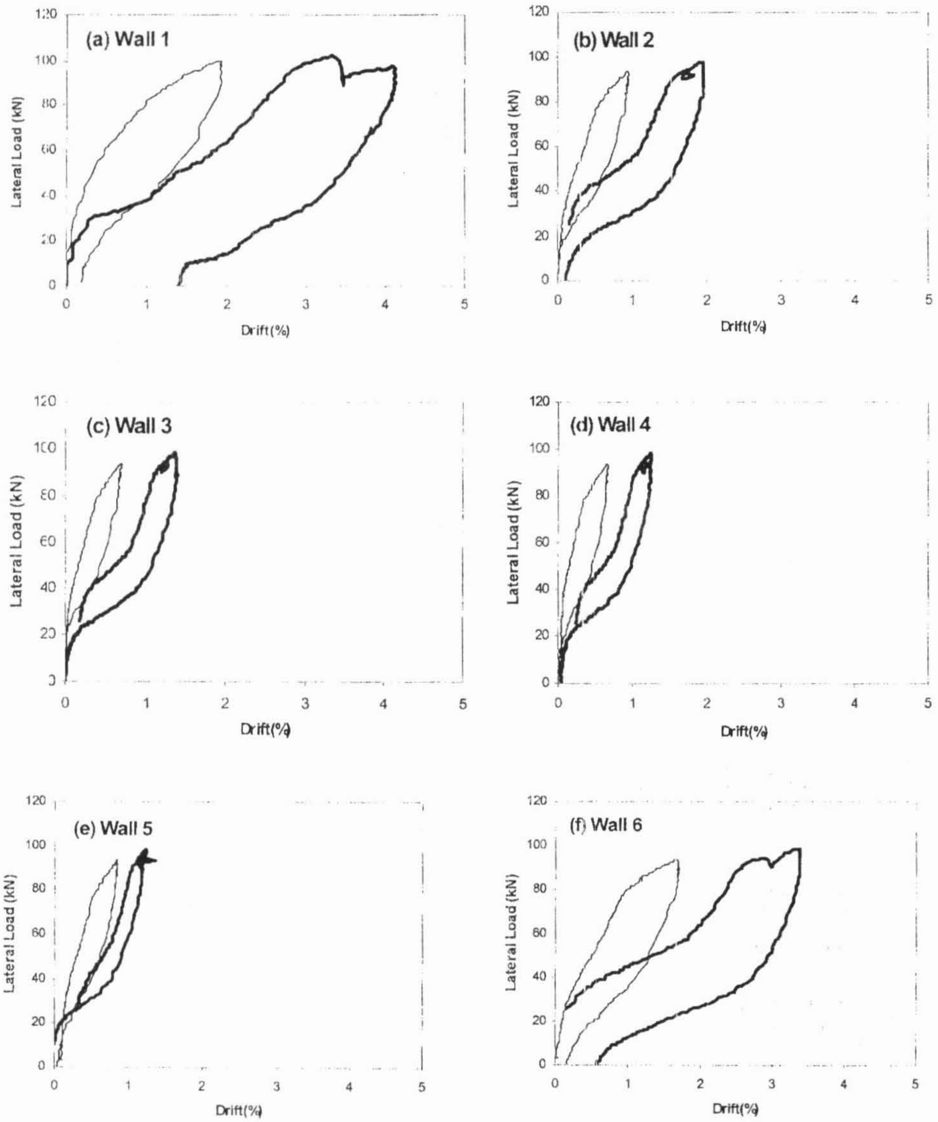


Fig.9: Phase 3 – The individual half-cycle performance of the wall units at 2.0% and 4.0% drift. Walls 1 and 6 are the outer seismic walls in the assemblage, while Walls 2 to 5 are the interior "non-seismic" or cladding wall units.

Equivalent Viscous Damping

Each graph shows experiment results of points plotted for equivalent viscous damping for the energy absorbed over the previous full cycle of lateral loading at that drift amplitude. Experimental results are plotted for the first and second cycles. For the second cycle of equivalent viscous damping is approximately 60% of the first cycle. The reduced energy absorption results from tendon yielding that occurred in the previous (first) cycle and leads to hysteresis loops with a smaller enclosed area on the subsequent (second) cycle. Figure 10 also shows the theoretical equivalent viscous damping (ξ_{eq}) where the analytical hysteresis model is used to result for one equi-amplitude cycle. In the realistic constructed condition (Phase 3) where the panel-to-panel sealant was present, the theoretical prediction is some 10% in excess of the experimental observation. Notwithstanding this outcome, it appears that the equivalent viscous damping is reasonably constant for drifts in excess of 2%. Thus a value of, say, 12% of equivalent viscous damping could be used for seismic design purposes.

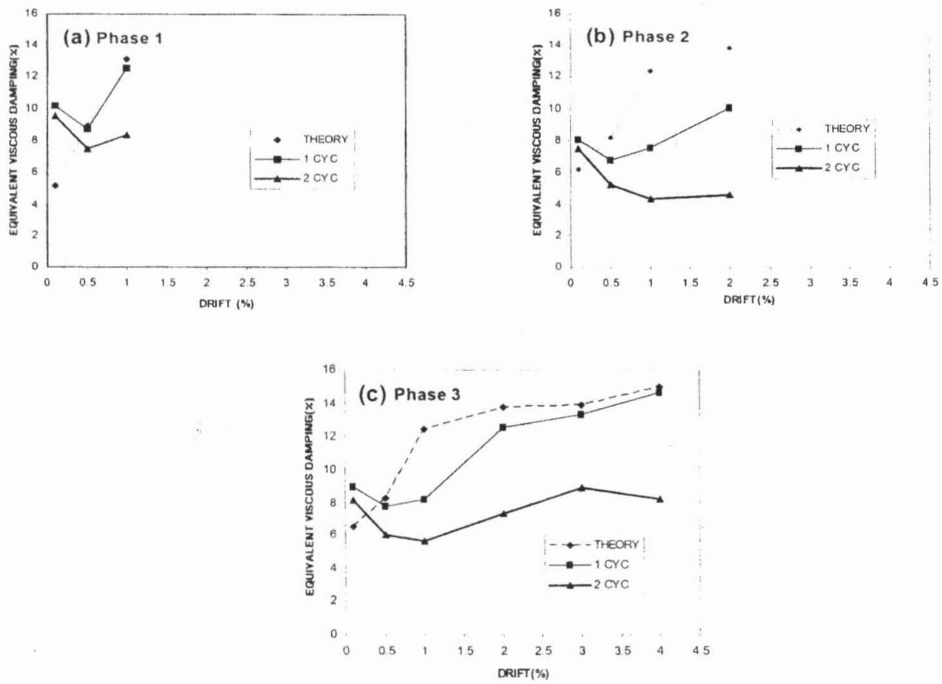


Fig. 10: Equivalent viscous damping represents overall multi-wall panels; (a) Phase 1 using 20mm fuse-bars and rubber block spacers; (b) Phase 2 using 13mm fuse-bars with rubber block spacers; and (c) Phase 3 using13mm diameter fuse-bars, rubber block spacers and sealant.

Conclusions and Recommendation

Based on the experimental findings presented herein the following conclusions are drawn:

1. The experimental work on a superassemblage of multi-panel precast concrete hollowcore wall units has demonstrated that the seismic and non-seismic wall units can be implemented in the construction of single storey warehouses. Under large drifts (>3%) damage is limited to the sealants. Such damage is inexpensive to repair.
2. By steel-armouring seismic wall units at the wall-foundation interface, and seating the non-seismic walls on rubber bearing pads a damage avoidance performance can be achieved. These damage avoidance design (DAD) details accommodate higher displacement and contact pressures at the rocking toe during uplift of precast hollow core walls. The thickness of rubber pad and rocking steel plate can be designed based on maximum base shear imposed on their rocking base to dissipate energy during ground shaking. Shear keys or pintles can be welded beneath the steel seating channel to inhibit sliding.
3. There were no cracks observed either on seismic wall or non-seismic walls up to 4.0% drift.
4. The rubber seating pad, silicone sealant and rubber block spacer are good materials to accommodate differential displacements between units and to absorb some energy. Such materials provide an economical alternative to using vertical shear connectors.
5. It recommended that each seismic wall panel be located at the center of a single precast foundation beam unit. Each foundation beam unit should be discontinuous with neighbouring units in order to reduce soil bearing pressure which could prevent the uplifting of the foundation beam during severe shock. Joints between foundation units should be detailed to transmit some shear force, but no moment.

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