LATERAL SEISMIC PERFOMANCE OF MULT-PANEL PRECAST HOLLOW CORE WALLS SUBJECTED TO QUASI-STATIC CYCLIC LOADING

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ABSTRACT

The seismic resistance of a superassemblage of precast hollow core wall units 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.

KEYWORD: Superassemblage, Unbonded Prestressing Tendons, Multi-panel wall, Fuse-bars, Hysteretic Energy Absorption

1. INTRODUCTION

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. Hamid (2006) 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.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 (Hamid, 2006). The main reason for choosing fuse-bars as the only means of energy dissipation is because they are easy to restress or replace 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 hollow core wall units is not a major problem when using hollow core 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).

2. PROTOTYPE DESIGN OF MULTI-PANEL WALLS

Figure 1 shows a warehouse type industrial building that consists of a series of multi-panel precast concrete hollow core 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). 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

$$\gamma = \frac{\delta_h}{t_{gap}} = \theta \frac{B}{t_{gap}} \tag{2.1}$$

in which $\delta_h =$ uplift displacement; $t_{gap} =$ the thickness gap between wall panels; $B =$ panel width; and θ = the target design drift.There are four principal components of forces that contribute to the overall resistance of a multi-panel wall system, as shown in Figure 2, these are given by

$$F_H = F_{SW} + F_{NS} + F_V + F_{CH} \tag{2.2}$$

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

$$F_{SW} = \frac{B}{2H}\left(W_r + W_W + T_1 + T_2\right) + \frac{e_p}{H}\left(T_1 - T_2\right) \tag{2.3}$$

in which $B =$ 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 it self-weight

$$F_{NS} = \frac{B}{2H} W_W \tag{2.4}$$

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

$$F_v = \frac{B}{H} V_r \tag{2.5}$$

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

$$F_{CH} = \frac{2M_p}{H}\left(1 + \frac{1}{n_{ns}}\right) \tag{2.6}$$

where $n_{ns} =$ number of non-seismic walls placed in between the seismic walls; and $M_p =$ plastic capacity of the reduced channel section. By substituting equations (2.3) to (2.6) into (2.2) and normalizing with respect to the total seismic weight (W_T) gives the base shear capacity.

$$C_c = \frac{F_h}{W_T} = \frac{B}{2H}\left(\frac{W_r + W_W + T_1 + T_2}{W_T}\right) + \frac{e_p}{H}\left(\frac{T_1 - T_2}{W_T}\right) + n_{ns}\frac{B}{H}\frac{W_W}{W_T} + \frac{B}{H}\frac{Vr}{W_T}(n_s + 1) + \frac{2M_p}{HW_T}\left(1 + \frac{1}{n_{ns}}\right) \tag{2.7}$$

where the total seismic weight is given by $W_T = W_r + (n_{ns} + 1)W_W$, $n_s =$ number of seismic wall and $e_p =$ distance between tendon and centre of wall.

3. DESIGN AND CONSTRUCTION OF THE MULTI-PANEL WALL SUPERASSEMBLAGE

Figure 3 shows the reinforcement details and the experimental setup of the super-assemblage. 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).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.

4. 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.

4.1. Hysteretic Performance of Phases 1, 2 and 3

Figure 4 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 4(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. Figure 4(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-centering provided by the unbonded fuse-bars and cap beam on top of the walls.

Figure 4(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 hollow core 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.

5. 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 5 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.

7. CONCLUSIONS AND RECOMMENDATIONS

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

1. The experimental work on a superassemblage of multi-panel precast concrete hollow core 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. 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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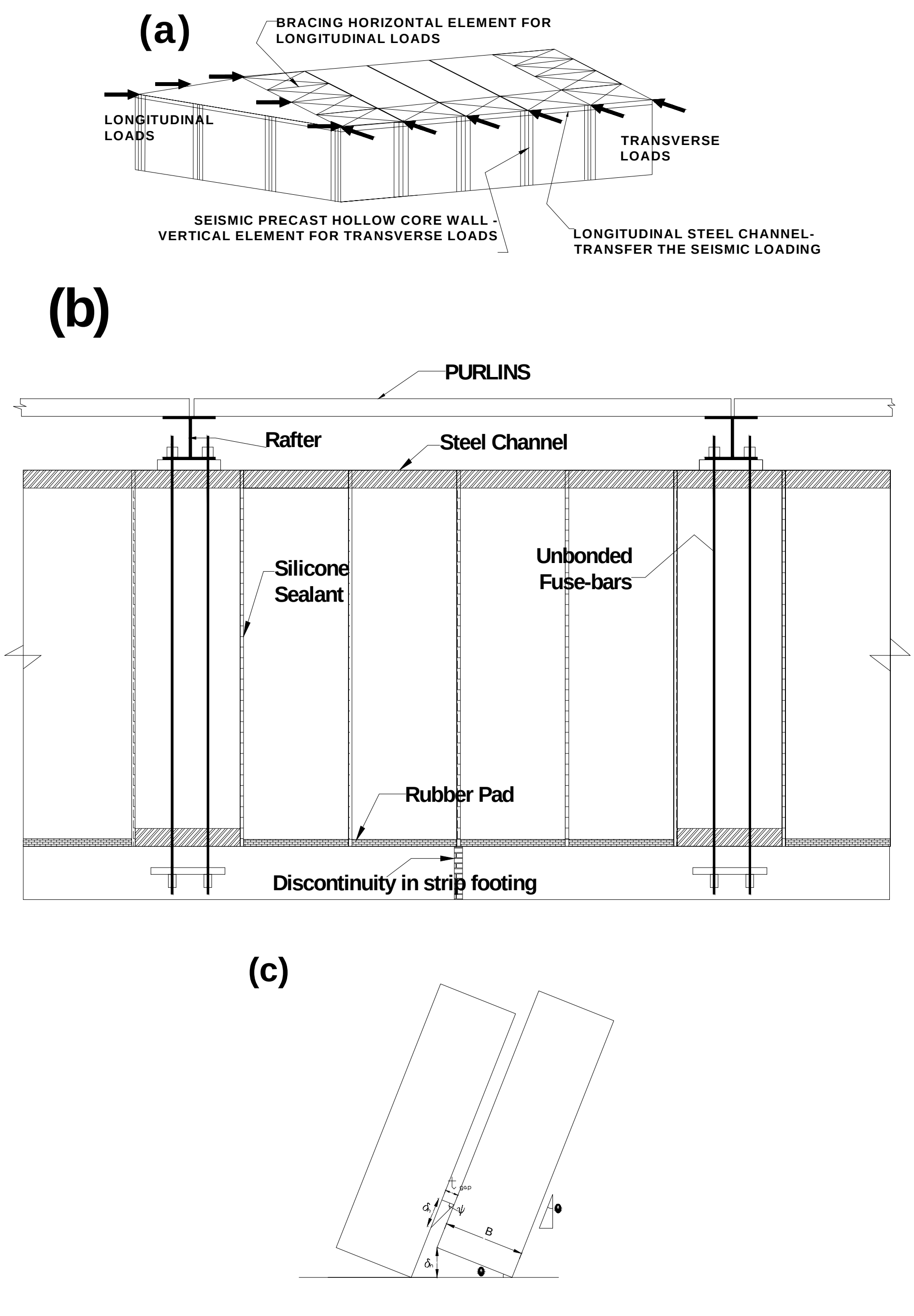


Figure 1: The superassemblage multi-panel wall is representing as part of a prototype warehous

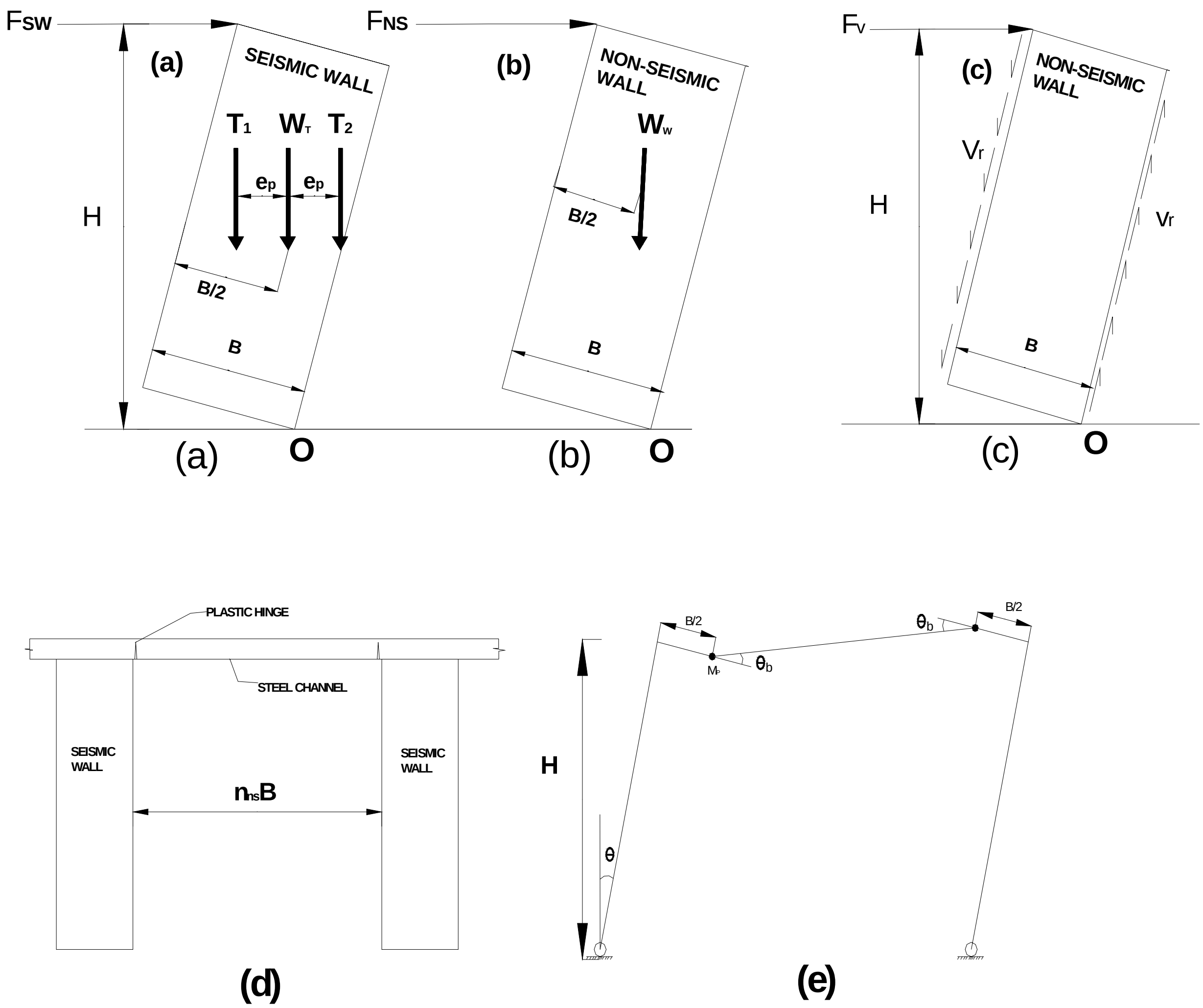
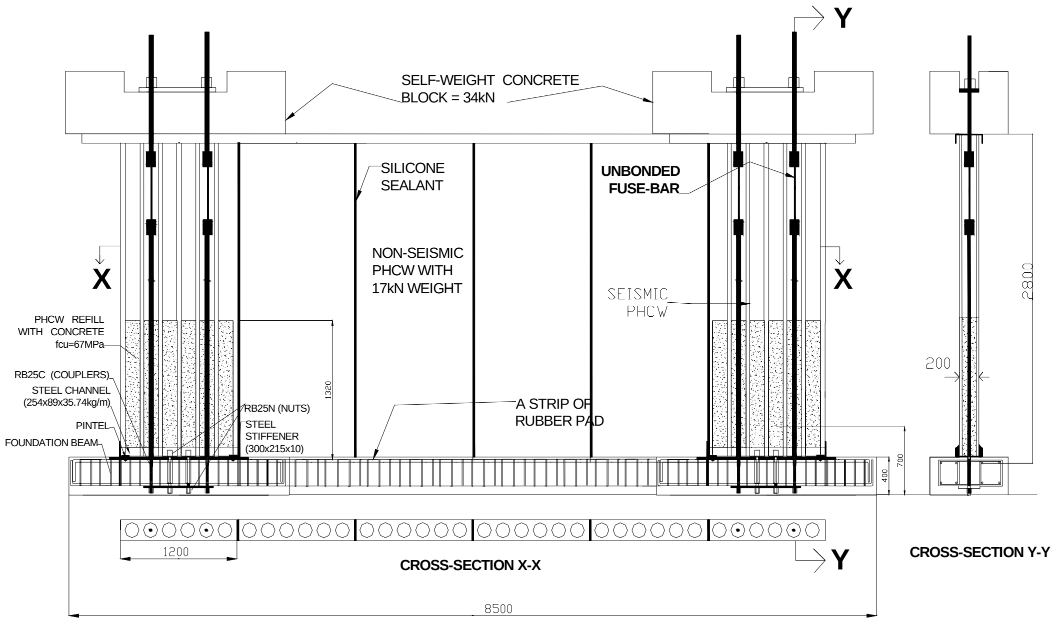
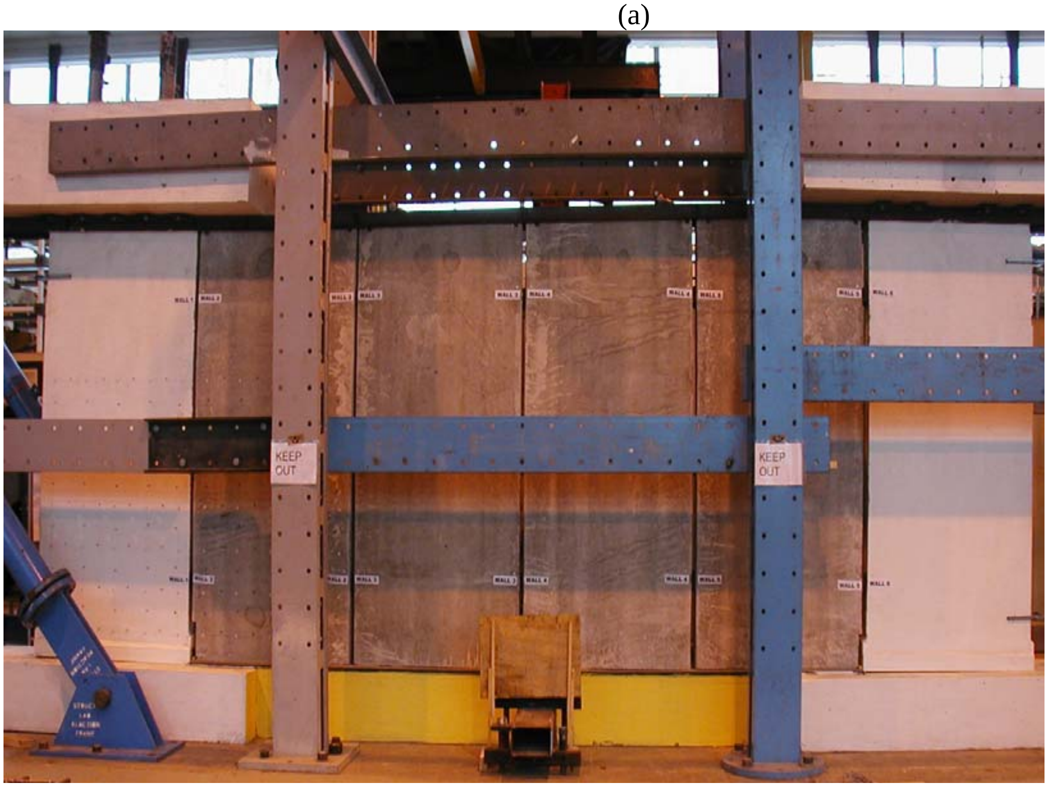


Figure 2: Resistance mechanism of a multi-panel wall system; (a) lateral resistance due post-tensioned tendons and self-weight of seismic walls; (b) lateral resistance coming from self-weight of non-seismic wall; (c) shear resistance from silicone sealant ; (d) plastic hinge occurred at V-cut shape of steel channel closed to seismic wall; and (e) the plastic mechanism on steel channel cap beam.



(a)



(b)

Figure 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.

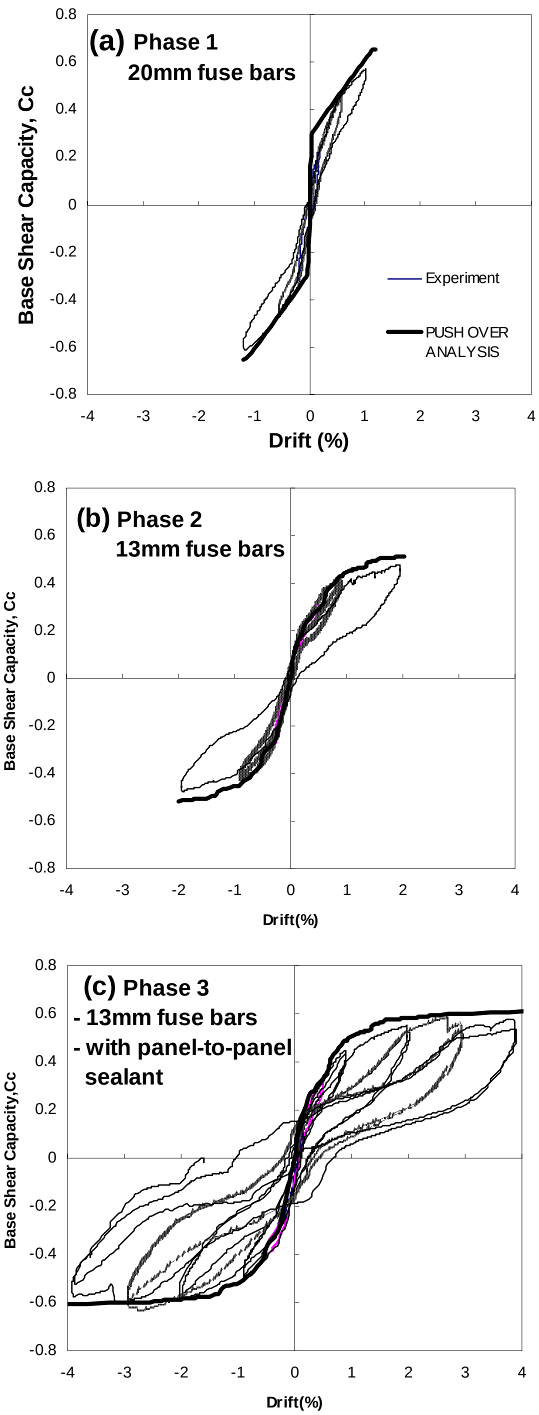


Figure 4: The overall hysteretic performance of Phases 1, 2 and 3: (a) experimental result for Phase 1 up to 1.0% drift; (b) experimental result for Phase 2 up to 2.0% drift; and (c) experimental result for Phase 3 up to 4.0% drift.

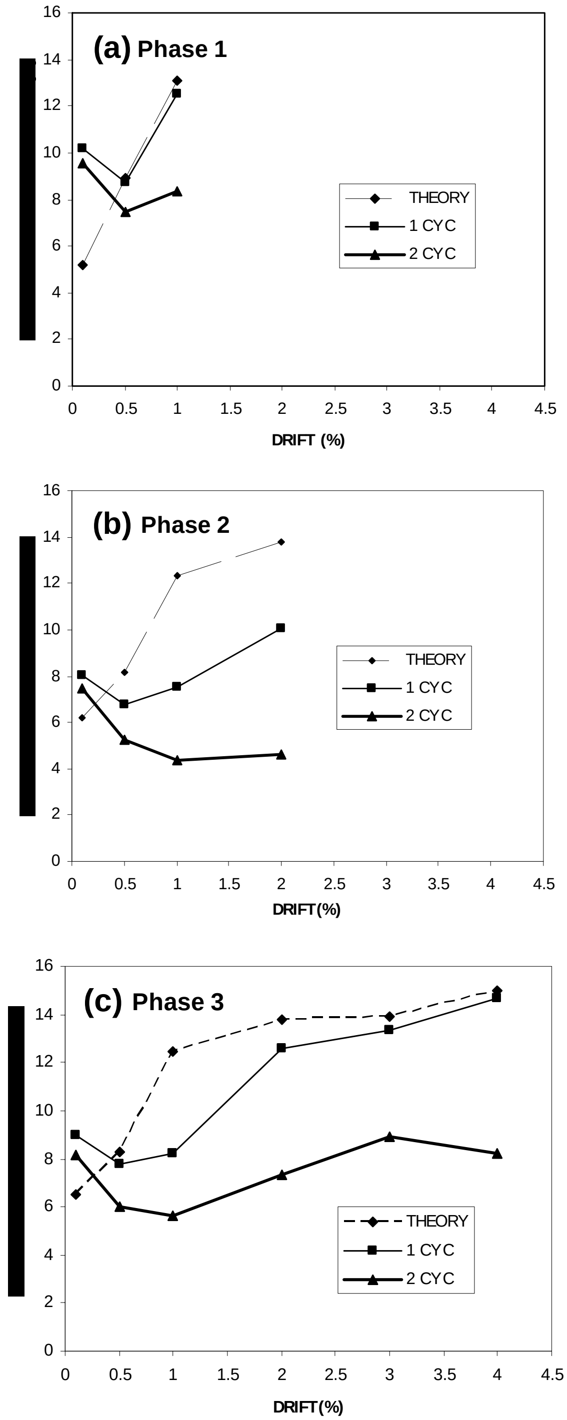


Figure 5: Equivalent viscous damping represents overall multi-wall panels; (a) Phase 1 using 20mm fuse-bars and rubber block spacers; (b) Phase 2 using 13mm fuse-bars with rubber block spacers; and (c) Phase 3 using13mm diameter fuse-bars, rubber block spacers and sealant.