Finite element analysis of the PreWEC self-centering concrete wall system

R. S. Henry¹, S. Sritharan², and J. M. Ingham³

ABSTRACT: A self-centering concrete wall system has been developed that consists of a Precast Wall with End Columns (PreWEC). A finite element model was developed to investigate the cyclic lateral-load response of the PreWEC system that included allowance for uplift at the wall-to-foundation interface, inclusion of the energy dissipating O-connectors, and inelastic behaviour of the confined concrete in the toe of the wall. The model showed good correlation with the results of a large-scale experimental test of the PreWEC system for both the global and local responses, closely matched the experimental lateral force-displacement response, unbonded tendon stress, neutral axis depth, concrete compressive strains, and connector deformation. Additional analyses were conducted to investigate modifications to the PreWEC design. These modified designs highlighted the influence of the inelastic behaviour of the wall toe and showed that in the PreWEC system the axial load on the wall panel is independent of the number of energy dissipating connectors. Lastly, analyses were conducted to investigate wall-to-floor interaction with the PreWEC system. It was shown that a rigid wall-to-floor connection would result in some damage to the floor diaphragms and an overstrength that should be considered when designing the wall for shear. Alternatively, connectors could be used with the PreWEC system to isolate the floor from the uplift of the wall and eliminate damage to the floor diaphragm.

Keywords: Self-centering; Unbonded Post-Tensioning; Precast Concrete; Shear Walls; Finite element modelling.

¹ Department of Civil and Environmental Engineering, University of Auckland. Email: rs.henry@auckland.ac.nz

² Department of Civil, Construction, and Environmental Engineering, Iowa State University.

³ Department of Civil and Environmental Engineering, University of Auckland.

1 INTRODUCTION

2 Self-centering concrete wall systems that can resist seismic lateral loads without causing
3 significant structural damage have been previously investigated both analytically and
4 experimentally [1-6]. However, the limited implementation of these wall systems led to the
5 development of a new configuration of self-centering walls that consists of a Precast Wall
6 with End Columns (PreWEC) [7]. As shown in Figure 1, the PreWEC system utilises a single
7 precast concrete wall panel with two steel or concrete end columns that are each anchored to
8 the foundation using unbonded post-tensioning. The wall and columns are joined horizontally
9 using special connectors positioned along the vertical joints to provide additional energy
10 dissipation. When subjected to a lateral load, the PreWEC system concentrates inelastic
11 deformation at the wall base using a rocking mechanism. The post-tensioning tendons are
12 unbonded to reduce their strain demand and are designed to remain elastic up to the design
13 level drift, providing a restoring force to self-center the structure. Using this arrangement of
14 components, the PreWEC system maximises the lever arm between the post-tensioning
15 tendons and the compression block in the wall toe and can be designed to have a moment
16 capacity equal to that of a comparable monolithic reinforced concrete wall [7].

17
18 **Figure 1 – PreWEC wall system**

An experimental test of a 6 m high 50% scale wall specimen was previously conducted to verify the PreWEC concept [7] and a simplified design procedure that was developed by Aaleti and Sritharan [8]. The simplified design method was able to capture the monotonic backbone response of the PreWEC wall, but a more detailed numerical model was required to capture the full cyclic lateral-load behaviour. A non-linear finite element model (FEM) was developed to represent all of the main components of the PreWEC system. The FEM explicitly considered the wall and column uplift and contact length, changes in post-tensioning stresses, connector deformation and hysteresis behaviour, and inelastic strains in the confined concrete compression toe. The developed FEM was used to conduct parametric analyses to better understand the cyclic response of the PreWEC system as well as consider system level effects such as the wall-to-floor interaction that must be accounted for during the seismic design of such buildings.

2 DEVELOPMENT OF PREWEC FEM

The FEM developed for the PreWEC system was first validated against the previously tested PreWEC wall [7]. The PreWEC FEM extended on separate models that had been applied to post-tensioned concrete wall panels and the O-shaped energy dissipating connectors [9, 10].

2.1 Test specimen

Drawings of the PreWEC test specimen that was modelled are shown in Figure 2. The precast wall panel was cast with concrete having a measured compressive strength of 63 MPa and post-tensioned with 12×15.2 mm diameter strands. The end columns consisted of concrete filled steel tubes that were post-tensioned with 3×15.2 mm diameter strands, and a total of 10 steel O-connectors were placed between the end columns and the two ends of the wall. The wall and end columns were erected on top of a foundation block that was anchored to the strong floor of the testing laboratory. Lateral loading was applied using a single

1 horizontal actuator that was connected to a T-shaped loading beam on top of the wall. Full
 2 details and results from the PreWEC test have been published elsewhere [7, 11].

(a) Overall dimensions

(b) O-connectors

Figure 2 –PreWEC test specimen

2.2 Model Formulation

A 3D FEM was developed to replicate the PreWEC test specimen as closely as possible, as shown in Figure 3, with the precast concrete wall modelled using solid elements. The wall panel was partitioned to allow a confined concrete material definition to be applied to the confined toe region, and an unconfined concrete definition was applied to the remaining section of the wall. The wall was meshed so that the element size was reduced towards the

corner toes of the wall, both along the wall base and varying with height. This discretisation allowed for a more detailed mesh in the critical toe region, where most of the non-linear behaviour was expected to occur. The element size in the corner toes was approximately 45 mm high $\times$ 50 mm long, and expanded to 100 mm high $\times$ 75 mm long in the body of the wall. The end columns were modelled in a similar way to the wall panel model. The column mesh used six elements along the length of the column to accurately capture the bending and neutral axis depth, and the element height was approximately 120 mm.

Figure 3 –PreWEC FEM assembly

The foundation block and grout pocket directly below the wall were also included in the model. The foundation block had a mesh size of approximately 150 mm, but the mesh was reduced in the grout pocket to approximately match the mesh size of the wall. The horizontal

joint between the wall and foundation (and column and foundation) was modelled using a contact interaction that provided hard contact under compressive stresses while allowing unrestricted gap opening. Additionally, the friction mechanism for transferring shear forces was sufficient to prevent any slip occurring during experimental testing, so a no-slip condition was used to prevent horizontal wall sliding. Because the top of the wall was not critical to the PreWEC behaviour, the T-shaped loading beam was modelled using a discretely rigid member.

The unbonded tendons were modelled as truss elements with the top tendon node coupled to the center of the loading beam or column and the bottom tendon node restraining in both the horizontal and vertical degrees of freedom to represent the anchorage in the foundation. The prestressing strand wedges were power seated and so wedge slip was not observed to be significant during the test. As such, potential wedge slip was not modelled in the FEM.

The CFT columns were modelled in a similar way to the wall panel model. However, the column section was partitioned to allow the shell to be assigned the steel material definition, while an elastic grout definition was used for the centre elements. This partitioning meant that the surfaces between the steel shell and grout fill were restrained together with no debonding or slip accounted for in the FEM.

The dimensions of the O-connectors placed between the wall and end columns are shown in Figure 2b. The O-connectors were modelled as solid elements with a fine mesh and used a combined isotropic/kinematic hardening plasticity model. Additional details of the connector FEM are described by Henry et al. [10]. The welded connection to secure the O-connector was modelled by restraining the weld surface to the column face on one side, and to the

embedded steel plate on the other side. The steel plate was embedded into the concrete wall with an ideal bond assumed between the concrete and steel elements.

Steel channel sections (152.4 mm wide $\times$ 75 mm high $\times$ 610 mm long, and 6.35 mm thick plate) were positioned at the wall base in the corner toes during the test to provide additional protection to the cover concrete. The bottom web of the steel channels was embedded into the concrete wall elements and a contact interaction was applied to the surface between the side of the wall and the channel side flanges. This contact interaction allowed the channel to protect the wall toe from excessive lateral expansion and damage, while also allowing the channel flange to detach from the wall face when it was subjected to excessive bending, as was observed during the test. The inclusion of vertical and horizontal reinforcement in the wall panel was found to have no significant influence on the FEM results with respect to the typically PreWEC response [12] and so was omitted in order to reduce computational time.

2.3 Material Properties

Samples taken from the materials used to construct the PreWEC specimen were tested to verify their actual properties for use in computational models. Full details of these material tests and both measured and modelled properties for each material have been reported previously [11, 12].

The concrete materials in the PreWEC FEM used the ABAQUS concrete damaged plasticity model [13]. The stress-strain definition used for both the unconfined and confined sections of the precast wall was derived from the measured compressive strength and confinement reinforcement details using a stress-strain relationship proposed by Chang and Mander [14]. The slope of the compression strain softening section of the unconfined concrete material model was modified to prevent convergence issues, ending at $0.1f_c'$ at a compressive strain of

0.01. Several techniques were trialled for modelling the confined toe region of the wall that expended 368 mm along the length of the wall and 1600 mm high. As summarised by Henry [12], the most suitable technique adopted in the FEM was to directly assign the confined concrete compressive stress-strain response to the concrete elements in the wall toe. The unconfined and confined stress-strain definitions used in the PreWEC FEM are both shown in Figure 4. In addition to the stress-strain backbone, the unloading stiffness of the compressive and tensile response was included in the concrete damaged plasticity model by defining damage indices based on equations for the cyclic behaviour of confined concrete presented by Chang and Mander [14].

(a) Concrete

(b) PT tendons and steel components

Figure 4 – Material stress-strain definitions used in the FEM

High strength steel fibre reinforced grout was used as a bedding layer at the wall base. The grout layer did not experience significant non-linear behaviour during the test and so was modelled using an elastic stress-strain definition, with an assumed modulus of elasticity of 40 GPa based on the manufacturer's specification. Additionally, the high strength grout that

was used to fill the steel tube end columns was also modelled using the same elastic material definition as the bedding layer grout.

The PreWEC FEM utilised three different steel material definitions for various elements in the structure, including PT tendons, O-connectors, and steel plate used to manufacture the end columns, embedded plates, and toe channels. All of the steel materials were defined using a standard plasticity model with combined kinematic/isotropic hardening and a stress-strain response for each material based on the measured response from the tensile coupon tests. The FEM stress-strain definitions used for the PT tendons, O-connector, and end columns are plotted in Figure 4b.

2.4 Analysis control

Loading was applied using a series of analysis steps. First, the tendons were prestressed by specifying an initial stress in the truss elements and the tops of the tendons were initially restrained to prevent any displacement due to the initial stress condition. The tendon restraints were removed during the first load step, which allowed the prestress force to be transferred into the wall as a precompression. Following the prestressing step, the wall self-weight was applied as a gravity load. Lastly, the reference point at the end of the loading beam that was positioned at the top of the wall was subjected to a displacement controlled lateral load history.

The cyclic FEM analysis was conducted using the explicit solver in ABAQUS. Although the explicit solver is typically used for dynamic analysis problems, it is also suitable to perform pseudo-static analyses with complicated contact conditions. A series of trials that were conducted to determine an appropriate loading speed are described in detail by Henry [12]. It was concluded that a maximum velocity of 9 mm/s and a maximum acceleration of 1 mm/s² could be used without introducing any significant inertia effect that would violate pseudo-

static loading conditions. After the solution control parameters were appropriately defined the explicit solver produced a monotonic lateral load response that was comparable to that obtained from the implicit solver.

3 PREWEC CYCLIC RESPONSE

The PreWEC FEM was subjected to the full reverse cyclic lateral load history that was applied during the PreWEC test and the calculated response was compared with the measured experimental results.

3.1 Global behaviour

In general, the PreWEC FEM was observed to behave in a similar manner to the PreWEC test specimen. The displaced shape and stress contours of the PreWEC FEM at 3% lateral drift are shown in Figure 5. The uplift at the base of the wall and column elements was clearly visible, as well as the plastic deformation of the O-connectors. High stresses and strains were generated in the wall toe, but the confined concrete region was sufficient to prevent significant crushing, which is comparable to the behaviour that was observed during the test.

Figure 5 – PreWEC FEM at 3% lateral drift (displacements magnified 2×)

The lateral force-displacement response for the cyclic FEM analysis is shown alongside the measured experimental results in Figure 6. Overall, the FEM showed good correlation with the experimental lateral force-displacement response, closely matching the initial stiffness, strength, and shape of the hysteresis loops. It was found that the inclusion of the damage indices to control the unloading stiffness of the concrete compressive stress-strain response was critical to accurately capture the unloading behaviour of the PreWEC hysteresis response. However, the FEM response showed some pinching of the hysteresis loops which resulted in an imperfect calculation of the reloading path of the response just after the lateral force crossed the zero axis.

Figure 6 – Comparison of measured and calculated lateral force-displacement responses for the PreWEC test wall

To further examine the global hysteresis behaviour, the energy dissipation and residual drift calculated from the FEM analysis were compared with the experimental results. The equivalent viscous damping, ξ_{eq} , and residual displacements from the cyclic hysteresis response for each cycle are plotted in Figure 7. The calculated energy dissipation from the FEM response closely matched that of the experimental results. However, the pinching behaviour observed in the hysteresis caused the FEM to underestimate the cyclic residual displacement by up to 50%, possibly due to the inability to model concrete debris that potentially became trapped under the wall during the test. However, underestimation of residual drift was not of concern as both the experimental and numerical residual drifts were small and well within acceptable limits for the PreWEC design.

(a) Energy dissipation

(b) Residual drift

Figure 7 – Comparison of measured and calculated hysteresis loop parameters for the PreWEC test wall

3.2 Local response parameters

Local response parameters were investigated to further assess the accuracy of the FEM to calculate the cyclic response of the PreWEC test, including unbonded tendon stress in the wall PT, neutral axis depth, concrete compressive strains, and connector displacements. A selection of comparisons between the FEM calculated and experimental results for these four local response parameters are plotted in Figure 8. The neutral axis depth was calculated from the displaced profile along the wall base and the concrete strains were measured using strain gauges that were positioned vertically in the wall toe, 100 mm in from the wall end face and 150 mm above the wall base. To allow for a visible comparison of the results, only the backbone responses that were extracted from the cycle peaks are shown in Figure 8.

The FEM accurately captured the vertical displacement that the O-connectors were subjected to as well as the small horizontal deformations that occur when the gap between the wall and column reduces as the elements rotate. The O-connector is relatively flexible in the horizontal

- 1 direction and recent tests have shown that the O-connectors can accommodate these minor
- 2 horizontal demands without any significant effect on the connector response [15].

(a) Unbonded tendon stress

(b) Neutral axis (NA) depth

(c) Concrete strain at North wall toe

(d) Vertical displacement at top connector on the North edge of the wall

Figure 8 – Comparison of measured and calculated local response backbones for the PreWEC test wall

- 3 In general, the FEM results showed good correlation with the experimental results for all four
- 4 local response parameters. The accurately calculated local response characteristics confirmed
- 5 the accuracy observed from the global lateral force-displacement response and provided
- 6
- 7

increased confidence in the technique used to model the confined toe region. The overestimation of tendon stress during large drift cycles may have been contributed to by minor seating losses at the post-tensioning anchor during the test, which was not modelled in the PreWEC FEM.

4 MODIFIED PREWEC DESIGNS

Following the successful validation described above, the PreWEC FEM was used to study the influence of the major design parameters in the wall system. Based on design calculations the connectors accounted for only 35% of the total moment capacity of the PreWEC test wall, but perfect self-centering behaviour was not achieved due to inelastic strains in the wall toe. Although the residual drifts measured during the PreWEC test were still within acceptable design limits, a series of modified PreWEC designs were modelled by changing the PT configuration, number of connectors, and wall thickness. A summary of the design changes for the five modified PreWEC designs are given in Table 1, where the FEM of the PreWEC test specimen is referred to as PreWEC-t and the modified designs are labelled A-E.

Table 1 – Summary of changes to the PreWEC design

Label	PT	Connectors	$M_{PT} : M_{con}$	$f_{c, PT}/f'_c$	Wall Thickness
PreWEC-t	$A_{ps} = 1680 \text{ mm}^2$ and $f_{se} = 1200 \text{ MPa}$	20	0.65 : 0.35	0.115	152.4 mm
PreWEC-A	$A_{ps} = 1350 \text{ mm}^2$ and $f_{se} = 1200 \text{ MPa}$	28	0.50 : 0.50	0.092	152.4 mm
PreWEC-B	$A_{ps} = 1900 \text{ mm}^2$ and $f_{se} = 1200 \text{ MPa}$	16	0.72 : 0.28	0.130	152.4 mm
PreWEC-C	$A_{ps} = 1680 \text{ mm}^2$ and $f_{se} = 1200 \text{ MPa}$	28	0.57 : 0.43	0.115	152.4 mm
PreWEC-D	$A_{ps} = 1680 \text{ mm}^2$ and $f_{se} = 1200 \text{ MPa}$	16	0.70 : 0.30	0.115	152.4 mm
PreWEC-E	$A_{ps} = 1680 \text{ mm}^2$ and $f_{se} = 1200 \text{ MPa}$	20	0.65 : 0.35	0.087	200.0 mm

Where:

$M_{PT} : M_{con}$ is the ratio of the base moment provided by the PT to the base moment provided by the connectors at 2% lateral drift

A_{ps} = prestressing steel area

f_{se} = initial stress in prestressing steel after losses

$f_{c, PT}$ = initial concrete compressive stress

A_g = wall gross section area

f'_c = concrete compressive strength

4.1 PT to connector ratio

The first two modified analyses reported in Table 1 were conducted to investigate changing the ratio between the moment provided by the PT and the moment provided by the connectors while maintaining the same lateral strength to that of the PreWEC test. This aim required that as the connector moment contribution was increased the PT moment contribution was reduced, and vice versa. PreWEC-A used an increased number of connectors, from 20 to 28 (14 on each side/end). To maintain the original lateral strength, the PT area in PreWEC-A was reduced from 1680 mm^2 to 1350 mm^2 , which reduced the axial stress ratio ($f_c/A_g f'_c$) from 11.5% to 9.2%. As expected, the resulting cyclic lateral force-displacement response, shown in Figure 9a, indicated increased hysteretic energy dissipation, when compared to PreWEC-t. Interestingly, even with the fatter hysteresis loops, the PreWEC-A response did not result in a significant increase in the cyclic residual displacements. This observation was attributed to the fact that the compression force on the wall toe was less for PreWEC-A, which resulted in less damage to the wall when compared to PreWEC-t. The PreWEC system consists of connectors at each end of the wall, and as the wall rocks the connectors on one end of the wall are displaced in the positive vertical direction, while the connectors at the other end of the wall are displaced in the negative vertical direction. For large lateral drifts these vertical displacements cause the connectors at both ends of the wall to yield, and in doing so the vertical connector forces that are imparted at each end of the wall are approximately equal and opposite. Therefore, the axial force acting at the base of the wall is primarily a result of the PT and gravity loads. As the number of connectors was increased and the PT was reduced, the axial load on the wall was reduced which resulted in less damage to the wall toe. This reduction in axial load is confirmed in Figure 10, which shows plots of the calculated neutral axis depth and critical concrete strain at the extreme compression fibre of the wall for

PreWEC-t and PreWEC-A. When compared to PreWEC-t, both the neutral axis depth and the concrete strains were reduced for PreWEC-A, confirming that there was a reduced axial compression force in the wall toe. This reduced axial compression force resulted in lower inelastic compressive strains in the wall toe of PreWEC-A, which reduced the component of the residual displacement that was attributed to the wall hysteresis behaviour.

(a) PreWEC-A

(b) PreWEC-B

Figure 9 – Lateral force-displacement response calculated for PreWEC-A & B

(a) Neutral axis (NA) depth

(b) Critical concrete strain

Figure 10 – Local response backbones calculated for PreWEC-A & B

PreWEC-B used a decreased number of connectors, from 20 to 16 (8 on each side/end). To maintain the same lateral strength as PreWEC-t, the PT area in PreWEC-B was increased from 1680 mm² to 1900 mm², which increased the axial stress ratio ($f_c/A_g f'_c$) from 11.5% to 13.0%. As shown in Figure 9b, the resulting cyclic lateral force-displacement response indicated reduced hysteretic energy dissipation, but only during low lateral drift cycles. During the larger drift cycles increased damage occurred to the PreWEC-B wall toe as a result of the increased axial load from the PT. This behaviour was opposite to that observed for PreWEC-A, and the increased wall damage resulted in poor performance. The increased axial compression force in the wall toe is highlighted in Figure 10, which shows an increase in the neutral axis depth and concrete compressive strains in the wall toe for PreWEC-B, when compared to PreWEC-t.

Because of the increased energy dissipation and reduced damage to the wall toe, PreWEC-A exhibited improved seismic performance when compared to PreWEC-t, whereas PreWEC-B exhibited poorer seismic performance. Because of the increased inelastic strains and damage to the wall toe when a higher level of PT is used, increasing the number of connectors will not necessarily result in increased cyclic residual displacements, as has been traditionally assumed for such self-centering systems.

4.2 Number of connectors

Two further analyses were conducted to examine the effect of varying the number of energy dissipating connectors while keeping the initial PT force unchanged. PreWEC-C used an increased number of connectors, from 20 to 28 (14 on each end), and PreWEC-D used a decreased number of connectors, from 20 to 16 (8 on each end). In Figure 11 the lateral force-displacement responses of the PreWEC-C and D FEMs are compared with that of the PreWEC-t FEM. As expected, when more connectors were added to the system there was a

1 noticeable increase in lateral strength and hysteretic energy dissipation. The opposite was
 2 observed when the number of connectors was reduced, with decreased lateral strength,
 3 hysteresis area, and residual drift.

Figure 11 – Lateral force-displacement response calculated for PreWEC-C & D

5 As explained previously, despite the displacement of the connectors at each end of the wall
 6 not being equal, once the connectors had yielded the vertical connector forces at each end of
 7 the wall were approximately equal and opposite. As a result, the connectors do not introduce
 8 any significant axial force acting at the wall base. Because the initial PT force was unchanged
 9 between the PreWEC-t, C and D analyses, the axial load on the wall was not significantly
 10 affected by the number of connectors. The consistency in the wall behaviour is highlighted in
 11 Figure 12, which shows plots of the calculated neutral axis depth and critical concrete
 12 compressive strain in the wall toe for the PreWEC-C and D analyses. Both the neutral axis
 13 depth and the concrete compressive strain showed no significant change between the three
 14 different analyses, which confirmed that the wall axial load and forces in the compression toe
 15 were independent of the number of connectors. As a result of this observation, connectors

can be used to alter the PreWEC strength and energy dissipation capacity without compromising the wall behaviour, and in particular the highly stressed compression toe.

Figure 12 – Local response backbones calculated for PreWEC-C & D

4.3 Wall Thickness

Another method to reduce the axial compression force in the toe of a PT wall is to modify the properties of the wall. Increasing the concrete strength was considered to not be a practical option because the concrete strength of 63 MPa was already considered an upper bound value for standard precast concrete components. Instead, the wall thickness in PreWEC-E was increased from 152.4 mm to 200 mm and the resulting lateral force-displacement response is shown in Figure 13. The increased thickness resulted in a small increase in strength due to the increased moment lever arm associated with a reduced neutral axis depth. However, a more noticeable difference between the responses of PreWEC-E and PreWEC-t was the change in the unloading path, which was a result of significantly reduced inelastic compressive strains in the wall toe during large lateral drift cycles. As a result of a more elastic behaviour in the wall toe, the PreWEC-E response approached the ideal flag-shape

hysteresis behaviour and the residual displacements were almost zero. The behaviour of the PreWEC-E wall toe was confirmed by the values of neutral axis depth and concrete strain at the extreme compression fibre, which are shown in Figure 14. A significant reduction in the neutral axis depth was observed due to the increased contact width, resulting in a large reduction in concrete compressive strain. This reduced compressive strain demand prevented any significant inelastic damage in the wall toe, which ultimately led to a much more desirable seismic response.

Figure 13 – Lateral force-displacement response calculated for PreWEC-E

Figure 14 – Localised response backbones calculated for PreWEC-E

4.4 Initial axial stress ratio

From the FEM analyses that were conducted it appears that the inelastic compressive strains in the wall toe of the PreWEC test specimen contributed to the residual drift observed in the cyclic hysteresis response. The initial axial stress ratio ($f_c/A_g f'_c$) can be used as an indication of the expected demand on the wall toe and in the PreWEC test the initial axial stress ratio on the wall was 0.115 (or 11.5% of the wall axial capacity). When the initial PT force was increased for the PreWEC-B analysis to an initial axial stress ratio of 0.13 (or 13% of the wall axial capacity) the resulting response was significantly worse, with increased damage to the wall toe. Conversely, when the initial PT force was reduced for PreWEC-A and PreWEC-E to initial axial stress ratios of approximately 0.09 (or 9% of the wall axial capacity), the response was improved with less damage in the wall toe. Additionally, for PreWEC-E the residual drift observed in the cyclic hysteresis response was reduced to an insignificant amount.

5 INTERACTION WITH FLOOR DIAPHRAGMS

A critical consideration when examining the seismic response of a building with self-centering walls is how the wall system interacts with the floor diaphragms. The uplift at the base of the wall causes a vertical displacement and rotation at the location of the wall-to-floor connections. Despite also occurring in reinforced concrete wall buildings, wall-to-floor interaction is a particularly important consideration for the post-tensioned wall systems due to the low-damage design philosophy. Few previous studies have considered the effect of wall-to-floor interaction in post-tensioned walls. In previous projects that involved experimental testing of a building system that utilised self-centering precast concrete members, such as the PRESSS five storey building [1] and the precast building tested on the UCSD shake table [16], the wall-to-floor connections were isolated in order to minimise the uncertainty in the

behaviour due to wall uplift. It should be noted that interaction between post-tensioned frames and floor slabs also requires attention during design. Floors that are connected to both the columns will restrain the intended beam-column gap opening and innovative solutions have been proposed to minimise this issue [17-19].

If both the wall and end columns in the PreWEC system were rigidly connected to the floor diaphragm, then the relative vertical displacement between the wall and end columns would likely result in tearing of the floor. However, the PreWEC system can also offer unique solutions for wall-to-floor connections that can minimise the potential of damaging the floor diaphragm. In the PreWEC system the column uplift is negligible when compared to the wall uplift, and so the floor diaphragms can be attached to the columns to isolate the floor from the vertical displacement of the wall. The gravity load from the floor area would be transferred to the column elements, and the seismic inertia forces would need to be transferred directly from the floor to the wall by using a connection that provided a horizontal load path while avoided transferring gravity actions. A slotted connector can achieve this type of isolated floor connector with the PreWEC system, as shown in Figure 15 and tested by Watkins et al [20].

Figure 15 – Proposed isolated floor connection for PreWEC system

5.1 Prototype building model

A prototype structure was designed and analysed to demonstrate the influence of wall-to-floor interaction in a building that included the PreWEC wall system. The layout for the four storey prototype building is shown in Figure 16a, with an assumed inter-storey height of 3.66 m. Lateral resistance in one direction relied exclusively on two PreWEC walls with dimensions equivalent to the full-scale version of the 50% scale test specimen described earlier. Additionally, the prototype building was designed with three possible options for the floor diaphragms, including a fully cast-in-place floor slab, precast double-T units spanning parallel to the wall and precast hollow-core units spanning perpendicular to the wall. The prototype building is described in more detail by Henry [12].

Figure 16 – Four-storey prototype building

The PreWEC FEM developed earlier was extended to investigate the lateral load response of one half of the four storey prototype building, with two different wall-to-floor connections representing both the cast-in-place and double-T floor options. The PreWEC wall was modelled in the same way as that described earlier, the floor diaphragms were modelled using plane-stress shell elements, and the beams and columns were modelled as wire beam and truss elements respectively. The meshed assembly is shown in Figure 16b and more detailed information on the finite element model has been published by Henry [12].

5.2 Cast-in-place floor connection

An analysis was first conducted to simulate the behaviour of a cast-in-place floor slab with a rigid wall-to-floor connection. The floor slab was assumed to be 250 mm thick with D20 reinforcement at 300 mm centers top and bottom parallel to the wall and D10 reinforcement at 300 mm centers top and bottom perpendicular to the wall. The floor elements were constrained to the wall panels and penetrations were placed in the floor to prevent any connection between the floor and end column elements.

The deformed shape of the building FEM at 3% lateral wall drift is shown in Figure 17a. The PreWEC wall behaved in the same manner as the FEM of the individual PreWEC unit, with deformation primarily concentrated at a single crack at the wall base. The rigid behaviour of the cast-in-place wall-to-floor connections is visible, with the floors constrained to uplift and rotate with the wall. Because the edges of the floor were constrained to the exterior columns, which did not experience significant vertical displacement during the analysis, the uplift and rotation at the wall-to-floor connection caused bending of the floor in the out-of-plane direction.

The principal concrete strains in the top surface of the level 4 floor are plotted in Figure 18a, where tensile strains that exceed the concrete cracking strain are represented by the white shaded region. At 2% lateral wall drift, which corresponds to the design level displacement, the area of the cracked concrete is significant and covers in excess of 30% of the floor area. The FEM calculated maximum strains in the top layer of reinforcing steel in the level 4 floor slab are plotted in Figure 18b, with strains exceeding the yield strain of the reinforcing steel represented by the white shaded region. At 2% lateral drift only the reinforcing bars located directly adjacent to the ends of the wall were expected to yield, both in the top of the slab at the tension edge of the wall and in the bottom of the slab at the compression edge of the wall.

The limited area of reinforcement yielding indicated that most of the concrete cracks would be expected to close up when the load was removed. Additionally, the maximum tensile strain at 2% drift was only 0.0098, which is just below the seismic serviceability limit strain of 0.010, as suggested by Priestley et al. [21]. Consequently, the damage to the floor slab would result in residual crack widths being less than 1.0 mm, which can be easily repaired.

Figure 17 – Elevation view showing the displaced shape of the prototype building FEM (displacements magnified 3x)

Figure 18 – Predicted damage to the level 4 cast-in-place floor slab at 2% lateral drift

The monotonic moment-drift response calculated from the FEM of the prototype building with cast-in-place (CIP) floors is plotted in Figure 19, alongside the FEM response of just the PreWEC wall (ignoring the floors and gravity frame). It can be seen that inclusion of the floors in the FEM analysis significantly altered the predicted response, with the deformation and framing action of the floor diaphragms increasing the moment resistance and secant stiffness of the building by approximately 44% and 50% at 2% and 3% lateral wall drifts respectively. This overstrength that comes from the floor interaction is routinely ignored in design, but would have a significant effect on the seismic response of the wall and building due to higher shear and axial forces than that calculated using capacity design principles. The increased moment resistance resulted in up to a 50% increase in the shear demand on the PreWEC wall, which should be accounted for when designing the wall for shear or to prevent base sliding.

Figure 19 – Calculated monotonic pushover response of the prototype building FEM with both cast-in-place and isolated floors

5.3 Isolated floor connection

In addition to the rigid cast-in-place wall-to-floor connection, a fully isolated wall-to-floor connection was also modelled. This isolated connection represented the use of a special

connector that will isolate the floors from vertical uplift of the wall while still providing a load path for the horizontal inertia forces. Instead of explicitly modelling this wall-to-floor connector in the FEM, the connection was instead modelled using constraints that simulated the intended horizontal load path and vertical isolation. The horizontal displacements of floor nodes in the floor were constrained to the corresponding node of the wall while leaving the vertical displacements unconstrained.

The deformed shape of the building model with isolated floors is shown in Figure 17b at 3% lateral wall drift. Unlike the cast-in-place connection, the uplift and rotation of the wall was not transferred to the floors, and as a result the floor diaphragms remained relatively undeformed as they displaced horizontally with the wall. A closer inspection confirmed that no significant damage occurred to the floor diaphragms, with the predicted strains not exceeding the concrete cracking strain at any location.

In addition to preventing damage to the floor diaphragms, the isolated wall-to-floor connection resulted in a more predictable and dependable lateral strength. Figure 19 shows a comparison of the moment-drift response of the building with an isolated wall-to-floor connection, alongside the response of only the PreWEC wall and the building with a cast-in-place (CIP) wall-to-floor connection. Unlike the cast-in-place connection, which showed a significant increase in strength, the response of the building with isolated floor connections is almost identical to that calculated from the analysis of the individual PreWEC wall. This similarity in the response means that when the floors are isolated, the entire lateral resistance is provided by the PreWEC system and the seismic response of the building can be calculated with a high level of confidence.

5.4 Requirements for isolation connector

In the analysis with the isolated wall-to-floor connection, the floor-to-wall connectors were located 540 mm in from the ends of the full scale wall. At this location the connector was required to sustain a maximum vertical displacement of 71.8 mm upwards and 9.4 mm downwards at 3% lateral drift as the wall uplifted independently of the floors. Additionally, as the wall rotated the equivalent horizontal distance between the connectors increased and elongated the floor diaphragm, which could potentially change the force transfer distribution through each connector. For the prototype building analysed this elongation was only 1.3 mm, which is likely to be within the tolerance of the slotted connectors and have minimal effect.

6 CONCLUSIONS

A FEM was developed to model the cyclic lateral-load response of the PreWEC self-centering concrete wall system. The FEM replicated each of the components in the PreWEC system and included allowance for uplift at the wall-to-foundation interface, energy dissipating O-connectors, and inelastic behaviour of the confined concrete in the toe of the wall.

Overall, the FEM showed good correlation with the results from the PreWEC experimental test for both global and local responses. The FEM closely matched the experimental lateral force-displacement hysteresis response, and accurate modelling of inelastic behaviour in the confined wall toe was critical to capturing the degradation in the unloading and reloading paths. Additionally, accurate calculation of the hysteretic energy dissipation, unbonded tendon stress, neutral axis depth, concrete compressive strains, and connector displacement further confirmed the accuracy of the PreWEC FEM.

The FEM was used to investigate several modifications to the PreWEC design, and the following conclusions were drawn from these analyses:

- The connector forces at each end of the wall panel are approximately equal and opposite at large lateral drifts. Although the local connector forces should be considered during the panel design, the axial force acting at the base of the PreWEC wall is independent of the number of connectors and is a function of the wall post-tensioning and gravity load transmitted to the wall panel. This is a significant advantage of the PreWEC system arrangement when compared to other self-centering wall systems.
- When the moment contribution from the connectors was increased, by adding more connectors and reducing the magnitude of the PT force, the PreWEC response was improved with increased energy dissipation and reduced damage to the wall toes. The reduced axial load on the wall reduced the residual drift caused by inelastic strains to the wall toe, which is contrary to what is traditionally assumed. The opposite behaviour was observed when the wall PT force was increased and the number of connectors was decreased.
- When the PreWEC wall is subjected to large axial loads, inelastic strains in the wall toe can affect the unloading path of the cyclic hysteresis behaviour and result in increased residual drifts. The FEM analysis of the PreWEC system indicated that limiting the wall initial axial load ratio to less than 10% of the wall axial capacity will result in a good lateral-load response with no significant damage to the wall toe and minimal residual drifts.

- When rigid wall-to-floor connections are detailed, damage to the floor diaphragms will occur and the increase in lateral resistance of the building results in higher shear forces in the wall. This building overstrength can be significant and was up to 50% for the four storey prototype building analysed.
- Damage to floor diaphragms can be avoided by using suitable connectors in conjunction with the PreWEC system to isolate the floor from uplift of the wall. For the prototype building analysed these isolating wall-to-floor connectors would need to accommodate a total vertical displacement of approximately 80 mm.

7 ACKNOWLEDGEMENTS

Funding for this research was partially provided by NSF grant CMMI 1041650, for which Dr. J. Pauschke served as the Program Director, in addition to Fulbright New Zealand, the New Zealand Ministry of Research, Science and Technology, and the University of Auckland. Opinions, findings, conclusions, and recommendations in this paper are those of the authors, and do not necessarily represent those of the sponsors.

8 REFERENCES

1. Priestley, M.J.N., S. Sritharan, J.R. Conley, and S. Pampanin, *Preliminary results and conclusions from the PRESSS five-story precast concrete test building*. PCI Journal, 1999. **44**(6): p. 42-67.
2. Perez, F.J., R. Sause, and L.W. Lu, *Lateral load tests of unbonded post-tensioned precast concrete walls*. ACI Special Publication, 2003. **211**: p. 161-182.
3. Kurama, Y., R. Sause, S. Pessiki, and L.-W. Lu, *Lateral load behavior and seismic design of unbonded post-tensioned precast concrete walls*. ACI Structural Journal, 1999. **96**(4): p. 622-632.
4. Restrepo, J.I. and A. Rahman, *Seismic performance of self-centering structural walls incorporating energy dissipators*. Journal of Structural Engineering, 2007. **133**(11): p. 1560-1570.

5. Kurama, Y.C., *Hybrid post-tensioned precast concrete walls for use in seismic regions*. PCI Journal, 2002. **47**(5): p. 36-59.
6. Marriott, D., S. Pampanin, D. Bull, and A. Palermo, *Dynamic testing of precast, post-tensioned rocking wall systems with alternative dissipating solutions*. Bulletin of the New Zealand Society for Earthquake Engineering, 2008. **41**(2): p. 90-103.
7. Sritharan, S., S. Aaleti, R.S. Henry, K.C. Tsai, and K.Y. Liu, *Precast concrete wall with end columns (PreWEC) for seismic resistant design*. Earthquake Engineering and Structural Dynamics, 2014(under review).
8. Aaleti, S. and S. Sritharan, *A simplified analysis method for characterizing unbonded post-tensioned precast wall systems*. Engineering Structures, 2009. **31**(12): p. 2966-2975.
9. Henry, R.S., N.J. Brooke, S. Sritharan, and J.M. Ingham, *Defining the concrete compressive strain in unbonded post-tensioned walls*. ACI Structural Journal, 2012. **109**(1): p. 101-111.
10. Henry, R.S., S. Aaleti, S. Sritharan, and J.M. Ingham, *Concept and finite element modeling of new steel shear connectors for self-centering wall systems*. ASCE Journal of Engineering Mechanics, 2010. **136**(2): p. 220-229.
11. Aaleti, S. and S. Sritharan, *Performance verification of the PreWEC concept and development of seismic design guidelines*. 2011, ISU-CCEE Report 02/11, Iowa State University: Ames, IA. p. 97.
12. Henry, R.S., *Self-centering precast concrete walls for buildings in regions with low to high seismicity*, PhD thesis, in *Civil and Environmental Engineering*. 2011, University of Auckland: Auckland. p. 441.
13. ABAQUS user's manual version 6.9. 2009, Dassault Systèmes Simulia Corp.
14. Chang, G.A. and J.B. Mander, *Seismic energy based fatigue damage analysis of bridge columns: Part I - Evaluation of seismic capacity*. 1994, NCEER technical report no. NCEER-94-0006, State University of New York: Buffalo, NY.
15. Twigden, K.M., R.S. Henry, *Experimental response and bilinear approximation of steel plate O-connector dissipaters*. Structures, 2015. **3**: p. 261-271,
16. Schoettler, M. J., Belleri, A., Dichuan, Z., Restrepo, J. I., and Fleischman, R. B. *Preliminary results of the shake-table testing for the development of a diaphragm seismic design methodology*. PCI Journal, 2009. **54**(1), 100-124.
17. Chou, C. C., Weng, C. Y., Chen, J. H. *Seismic design and behavior of post-tensioned connections including effects of a composite slab*. Engineering Structures, 2008. **30**(11): p. 3014–3023.
18. Chou, C.C., Tsai, K.C. and Yang, W.C. *Self-centering steel connections with steel bars and a discontinuous composite slab*. Earthquake Engineering and Structural Dynamics, 2009. **38**: p. 403–422.

- 1 19. Amaris, A., Pampanin, S., Bull, D., and Carr, A. *Experimental performance of hybrid*
2 *frame systems with non-tearing floor connections*. 14th World Conference in Earthquake
3 Engineering (14WCEE), 2008, Beijing, China, October 12-17.
- 4 20. Watkins, J., S. Sritharan, R.S. Henry, *An experimental investigation of a wall-to-floor*
5 *connector for self-centering walls*. Proceedings of the Tenth U.S. National Conference on
6 Earthquake Engineering, 2014, Anchorage, Alaska, July 21-25.
- 7 21. Priestley, M.J.N., G.M. Calvi, and M.J. Kowalsky, *Displacement-based seismic design*
8 *of structures*. 2007: IUSS Press.

9