Seismic Analysis Of A Low-Damage Precast Wall With End Columns (PREWEC) Including Interaction With Floor Diaphragms

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SEISMIC ANALYSIS OF A LOW-DAMAGE PRECAST WALL WITH END COLUMNS (PREWEC) INCLUDING INTERACTION WITH FLOOR DIAPHRAGMS

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Abstract

The 2010/2011 Canterbury earthquakes have demonstrated that low-damage structural systems should be adopted to improve the seismic performance of concrete buildings and to reduce the economic and social impact of building damage in future earthquakes. One such low-damage structural system is self-centering precast concrete walls that utilise jointed construction and unbonded post-tensioning. A new low-damage concrete wall system that uses this self-centering design is PreWEC, which consists of a precast wall with end columns. The PreWEC system was designed to overcome the deficiencies of previous low-damage wall systems by increasing the moment capacity in a cost effective manner, so that the PreWEC system is comparable to traditional reinforced concrete construction in addition to providing superior seismic resilience.

It is important that when a building is constructed using a low-damage wall system, its seismic performance, including the self-centering capability, is not compromised by damage caused to other structural elements when the building is subjected to an earthquake input motion. Similarly, it is important to avoid damage occurring during excitation that arises from interaction between the wall system and other structural elements. Experimental and analytical validation of the PreWEC system is summarised in addition to details of analytical studies that were performed to examine the interaction between the PreWEC wall, the floor diaphragms and gravity columns, as well as the seismic response of a prototype building that included PreWEC walls. Recommendations are presented to improve the seismic design practice for low-damage concrete buildings with specific consideration to the wall-to-floor interaction.

1 INTRODUCTION

Concrete walls are frequently used in buildings to provide an efficient and economical structural system to resist the lateral forces generated from earthquakes. In accordance with the prevailing ductile design philosophy, a reinforced concrete (RC) wall should resist lateral forces generated in a building during an earthquake through the formation of a flexural plastic hinge near the wall base. A well detailed plastic hinge is intended to provide sufficient ductility without compromising the lateral strength of the wall. However, cracking and crushing of the concrete in the plastic hinge region, and yielding of the longitudinal reinforcing steel, results in significant and often irreparable damage to the wall during a moderate to large earthquake. Following the 2010/2011 Canterbury earthquakes, both structural engineers and the general public have an increased awareness of this often irreparable structural damage, with a large number of Christchurch multi-storey buildings with RC walls requiring demolition or experiencing significant downtime as costly repairs are made [1, 2].

To minimise the economic and social impact resulting from future earthquakes, structural engineers should adopt design solutions for buildings that reduce structural damage and thus reduce the extent of building

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reparis and reconstruction. One such low-damage structural system is self-centering precast concrete elements that utilise jointed construction and unbonded post-tensioning. The simplest form of a self-centering or rocking wall consists of a precast concrete panel with unbonded post-tensioned tendons that are anchored between the top of the wall and the foundation. When subjected to a lateral force, the behaviour of a self-centering wall is dominated by a single flexural crack at the wall base, as shown in Figure 1, with the lateral strength principally provided by the prestressing tendons. As uplift and rocking occur at the wall base, the unbonded prestressing tendons are elongated, with the strain evenly distributed along the height of the wall. This evenly distributed elongation allows the tendons to be easily designed to remain in their elastic state to provide a restoring force that can re-center the wall back to its original position when the lateral force is removed. A self-centering precast concrete wall can thus safely resist the lateral forces that are generated during an earthquake with minimal structural damage occurring, resulting in superior seismic performance when compared to the extensive damage that would be expected for a comparable monolithic RC wall.

![Figure 1 - Lateral rocking behaviour of an unbonded post-tensioned concrete wall](image)

Low-damage concrete building systems using jointed precast concrete elements and unbonded post-tensioning were originally developed during the joint US-JAPAN PRESSS research program [3-5].

Previous research into low-damage concrete building systems has resulted in the development of several different rocking wall systems, including individual post-tensioned walls [6-8], hybrid walls [9-11], and jointed walls [4, 12]. Additionally, the use of jointed precast concrete construction has been introduced into the New Zealand Concrete Structures standard [13] and is also covered in the Foundation PT tendon Wall handbook published by the New Zealand Concrete Society based on the PRESSS research [14].

Specific guidelines for the design of unbonded post-tensioned concrete walls have also been published by the American Concrete Institute [15, 16].

Two significant buildings that utilise low-damage concrete construction with unbonded post-tensioning have been constructed in New Zealand, being the Alan MacDiarmid building at Victoria University in Wellington [17] and the Southern Cross Hospital’s Endoscopy Consulting building in Christchurch.

The Southern Cross building performed exceptionally well during the recent Canterbury earthquakes, with no significant damage reported [18].

2 PREWEC SYSTEM

The main purpose of self-centering structural components is to offer an alternative building solution that is more resilient to damage caused by earthquakes. However, minimising structural damage alone is not sufficient for a self-centering wall system to be considered as a viable alternative to traditional RC walls. As well as providing superior seismic performance, self-centering concrete wall systems must be economically and architecturally comparable to conventional reinforced concrete construction techniques. The main difficulty in achieving seismic resilience without compromising economic or architectural factors is associated with the placement of energy dissipating devices. In the jointed wall system, a wall is divided into two or more panels to accommodate the placement of energy dissipating shear connectors along the vertical joints, as shown in Figure 2a. This division of the wall into several panels that are typically of equal size reduces the length of the lever arm between the post-tensioning tendons and the compression block of each wall toe. This reduced lever arm was found to be the main cause for the reduced moment capacity of the jointed wall system when compared to a monolithic RC wall with similar dimensions, ultimately reducing the cost-effectiveness of the jointed wall system [19].

Alternatively, the hybrid wall system relies on mild steel reinforcing bars placed across the wall-to-foundation interface to provide energy dissipation, as shown in Figure 2b. With this arrangement of prestressed tendons and mild steel reinforcing bars, a hybrid wall may possibly be designed to match the moment capacity of a traditional reinforced concrete wall. However, due to their placement within the wall, the mild steel reinforcing bars cannot be easily replaced after they are subjected to large inelastic strains and possible fatigue fracture during an earthquake. Therefore, despite improving the seismic performance, a fully resilient building cannot be designed with a hybrid self-centering precast concrete wall. To overcome the deficiencies of jointed and hybrid walls, a new system that consists of a Precast Wall with End Columns (or PreWEC) was developed [19, 20].

The PreWEC system, shown in Figure 2c, consists of a single precast concrete wall panel with two steel
or concrete end columns that are each anchored to the foundation using unbonded posttensioning. Only minimum reinforcing steel is required within the wall panel, except for additional confinement requirements in the corner toes where large compressive strains are expected. The wall and columns are joined horizontally using special connectors along the vertical joints to provide additional energy dissipation. As with previous self-centering technology, when subjected to a lateral load the PreWEC system concentrates inelastic deformation at a single crack that opens up at the base of the wall and columns. The post-tensioning tendons are unbonded to reduce the strain demand and are designed to remain elastic up to the design level drift, providing a restoring force to self-center the structure. Using this arrangement of components, the PreWEC system maximises the lever arm between the post-tensioning tendons and the compression block in the wall toe and can be designed to have a moment capacity equal to that of a comparable monolithic reinforced concrete wall. Additionally, only minor structural damage is expected when the wall is deformed to large lateral drifts, and the energy dissipating shear connectors can be easily replaced, so that performance based targets for a seismic resilient structure can be achieved.

2.1 ADVANTAGES OF THE PREWEC SYSTEM

In addition to the advantages shared by all low-damage self-centering concrete wall systems, the PreWEC system has the following specific advantages:

1. Maximised moment capacity when compared to other jointed wall systems.
2. The energy dissipating connectors can easily be inspected and/or replaced following an earthquake, as discussed further in section 2.2 below.
3. Placement of connectors up the height of the wall panel reduces the congestion of reinforcing bars at the wall base, making construction easier than for a hybrid wall.
4. At large lateral drifts the connectors yield at each end of the wall and the vertical connector forces are approximately equal and opposite. Therefore, the axial load on the wall is only attributed to the post-tensioning and additional gravity loads and energy dissipating connectors can be added as required without compromising the wall performance.
5. The end column arrangement offers additional options for wall-to-floor connections that minimise structural damage and improve the seismic resilience of the building, as discussed in section 5.2.

2.2 O-CONNECTORS

To cost effectively increase the energy dissipation characteristics associated with the PreWEC system, while not compromising seismic resilience, a new steel connector was designed to be placed between the wall and column elements. Extensive finite element modelling and experimental testing was conducted to investigate a suitable connector [21]. The O-Connector, shown in Figure 3, was selected as the most suitable option for the PreWEC system for the following reasons:

1. Extensive energy dissipation was achieved through flexural yielding of the O-Connector’s legs.
2. A large vertical displacement capacity could be achieved due to the indirect load path.
3. The O-Connectors were welded to the outside of the column and wall, making them easy to install, inspect, and replace.
4. The O-Connectors were cut from thin mild steel plate, making them an economical option with predictable behaviour, as opposed to previously used U-shaped flexural plates [4, 5].
As can be seen from the force-displacement plot in Figure 3c, cyclic deformation of the O-Connector exhibited broad stable hysteresis loops during experimental testing and the response was also accurately calculated using a detailed finite element model (FEM).

3 EXPERIMENTAL VALIDATION

Experimental validation of the PreWEC concept was undertaken in collaboration between Iowa State University, the National Centre for Research on Earthquake Engineering (NCREE) in Taiwan, and the University of Auckland [20]. The PreWEC test specimen, shown in Figure 4, consisted of a 6.100 m high, 1.830 m long, and 0.152 m thick precast concrete panel with two concrete filled steel tube end columns. The wall and end columns were post-tensioned to a concrete foundation block with twelve 15.2 mm diameter strands in the center of the wall, and three 15.2 mm strands in each of the columns. A total of 20 O-Connectors were included in the PreWEC test specimen, with one leg of the O-Connector welded to the steel end column and the other leg welded to a steel plate that was embedded into the concrete wall panel. The PreWEC test specimen was designed to have dimensions, material properties, and lateral strength that was comparable to a monolithic RC wall that had previously been tested [24], allowing for direct comparison between the two wall systems. A single hydraulic actuator was used to apply a lateral load at the top of the wall to simulate the behaviour of the system during an earthquake. An increasing pseudo-static cyclic displacement history was applied and the wall behaviour was monitored with an extensive array of instrumentation. The experimental validation is reported in detail by Aaleti [23] and Srinivasan et al. [20].

2.3 DESIGN PROCEDURE

Conventional sectional analysis that is used to calculate the strength of reinforced concrete members with bonded reinforcement cannot be used for systems with unbonded post-tensioning due to the lack of strain compatibility between the prestressing tendons and the surrounding concrete. Instead, the change in tendon strain is related to the extent of wall deformation that occurs between the two tendon anchorages. However, the behaviour of post-tensioned deformations can be approximated as rigid rocking blocks and thus the analysis becomes a simple case of rigid body mechanics. For individual post-tensioned concrete walls suitable for low-seismic applications, semi-empirical equations can be used to directly calculate the stress in the unbonded tendons [22]. The PreWEC system can be designed using an iterative procedure based on force equilibrium, geometric compatibility, and material properties. A simplified analysis and design procedure for the PreWEC system is discussed in more detail by Aaleti [23].
3.1 TEST RESULTS

In general, the PreWEC test unit behaved as was expected and displayed superior seismic performance when compared to the equivalent RC wall. Inelastic deformation was concentrated at a single crack that opened up at the wall base and minimal damage resulted. This superior performance is highlighted in Figure 5, which shows a comparison at similar lateral drift levels between the condition of the PreWEC wall and the extensively damaged RC wall specimen. Some minor spalling was observed in the PreWEC wall due to the development of high inelastic strains in the wall toe, but the confinement reinforcement was sufficient to prevent any significant loss of strength or stability. The measured lateral force-displacement response of the PreWEC wall, shown in Figure 6, indicated a stable response with energy dissipation occurring due to deformation of the O-Connectors and with self-centering occurring as a result of using unbonded post-tensioning. Some of the O-Connectors began to fracture during the cycles to 3% lateral drift, corresponding to an O-Connector vertical displacement of approximately 45 mm. However, it should be noted that in their fractured state the connectors still transferred compression forces, and only minor loss in strength and energy dissipation was observed during the final cycles of the force-displacement response. This behaviour is considered a desirable failure mechanism because the connectors can be easily replaced following a large earthquake and the observed failure occurred at a lateral displacement that was in excess of the 2% design level lateral drift. The post-tensioning tendons in both the wall and the end columns remained essentially elastic and provided sufficient restoring force to self-center the wall, with only a small residual displacement observed at the conclusion of the test [20].

(a) PreWEC at 3% lateral drift
(b) RC wall after 2.5% lateral drift [24]

Figure 5 – Comparison between the PreWEC and the RC wall specimens

Figure 6 – Measured lateral force-displacement responses from the PreWEC test
3.2 SELF-CENTERING AND RESIDUAL DRIFT

The main objective when designing seismic resilient building systems is to safely dissipate the energy imparted to a structure during an earthquake without causing significant structural damage. Given the difficulty in re-centering a building which is left with a significant residual drift following an earthquake, self-centering behaviour is a critical aspect of seismic resilient design. As shown in Figure 6, despite the O-Connectors contributing to only 35% of the lateral load capacity of the PreWEC test unit, a small residual drift was observed in the cyclic hysteresis response. However, dynamic analysis of a single degree of freedom system that represented the hysteresis behaviour of the PreWEC test unit indicated that these small residual drifts would be reduced to well below acceptable residual drift limits for selfcentering structures due to shake-down expected to take place due to the tail end of the earthquake ground motion [25].

Despite the experimentally observed behaviour of post-tensioned concrete members, current design procedures that are used to ensure that self-centering is achieved are predominantly based upon an ideal flag-shaped hysteresis response. The procedures included in Appendix B of NZS 3101:2006 [13] are intended to ensure that the moment contribution from energy-dissipating reinforcement equates to less than 50% of the probable flexural strength of the member. However, in addition to ignoring dynamic shake-down effect previously highlighted by MacRae and Kawashima [26], analysis has shown that the NZS 3101 procedure does not adequately ensure that self-centering is achieved when an idealised system is subjected to static loads and that the predicted structural behaviour can be either conservative or non-conservative [27]. A simple design check that can be performed to accurately estimate the upper bound residual drift that could be expected for a self-centering building following an earthquake is discussed by Henry [25].

4 ANALYTICAL MODELLING

Following successful experimental validation of the PreWEC wall system, a finite element model (FEM) of the test specimen was developed using ABAQUS [28]. The FEM of the PreWEC test specimen extended upon previously developed modelling techniques, including a 3D FEM for post-tensioned concrete wall panels [7] and the O-Connector FEM [21]. The behaviour of the confined concrete in the wall toe was a critical component in the PreWEC model, and an extensive investigation that was conducted into the procedures for modelling confined concrete is described by Henry [25]. The concrete damaged plasticity model available in ABAQUS was used to represent the concrete behaviour in the FEM, with appropriate stress-strain responses defined for both the confined and unconfined sections of the wall panel.

4.1 DESCRIPTION OF THE FEM

The precast concrete wall was modelled using solid brick 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 concrete filled steel tube columns were modelled in a similar manner to the wall panel and partitioned to allow the shell to be assigned steel material properties and the core assigned the properties of the high strength grout used to fill the steel section. The foundation block that was used during the experiment was also included in the model and the horizontal joint between the wall and foundation was modelled using a contact interaction that provided hard contact under compressive stresses while allowing unrestricted gap opening. 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 bottom tendon node restrained in all directions and the top tendon node coupled to the top of the end column and the loading beam elements. The welds of the O-Connectors were modelled by tying the weld surface to the column face and the steel plate that was embedded in the concrete wall. All of the material definitions were assigned using the measured concrete and steel properties. A more detailed description of the development of the PreWEC FEM is published by Henry [25].

4.2 FEM RESULTS

Loading was applied using a series of analysis steps. First, the tendons were prestressed by specifying an initial stress in the truss elements. Following the prestressing step, the wall self-weight was applied as a gravity load during the second load step. Lastly, a reference point at the end of the loading beam at the top of the wall was subjected to a displacement controlled lateral load history that was derived from that measured during the PreWEC experimental test.

In general, the PreWEC FEM was observed to behave in a similar manner to the PreWEC test unit. The displaced shape and stress contours of the PreWEC FEM at 3% lateral drift are shown in Figure 7. 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 compressive strains of up to 0.0150 m/m at 3% lateral drift were generated in the wall toe, but the extent of the confinement was sufficient to prevent significant concrete crushing, which was comparable to the behaviour that was observed during the experimental test.
The lateral force-displacement response for the cyclic FEM analysis is shown in Figure 8. Overall, the FEM exhibited satisfactory correlation with the experimental response, closely matching the initial stiffness, strength, and shape of the hysteresis loops. The FEM calculated response was slightly pinched, particularly during the reloading section just after the lateral force crossed the zero axis. This deviation from the experimental response was attributed to the complex wall-to-foundation contact that is idealised in the FEM. During the experimental test, some debris from the spalled cover concrete was trapped under the wall and this would have influenced the contact and subsequent reloading stiffness in the measured hysteresis response.

In addition to closely calculating the global force-displacement response of the PreWEC test, the FEM also accurately captured the measured local response parameters. The measured tendon stress, connector displacement, wall uplift, neutral axis depth and concrete strains in the wall toe were all closely matched by the FEM, as described in more detail by Henry [25].

### 5 WALL-TO-FLOOR INTERACTION

The behaviour of lateral load resisting shear walls is often investigated in isolation from the other structural and non-structural elements in a building. However, the entire structure needs to be analysed to fully appreciate the expected seismic behaviour of a building that utilises a self-centering system. Although concrete walls are often designed as the primary lateral load resisting system in a building, the wall interacts with the surrounding structure, including other gravity load resisting elements. When a low-damage structural building system is being designed, such as a rocking or self-centering wall, accounting for interaction between the wall and surrounding structural elements is critical in order to ensure that the desired performance targets of the structure can be achieved during an earthquake.

There is no advantage in designing a seismic resilient system if extensive damage is likely to occur in other parts of the structure, which is likely to compromise the self-centering ability of the building during an earthquake. A critical consideration when examining the seismic response of a self-centering building 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. To ensure that a seismic resilient building is achieved, the vertical displacement incompatibility at the wall-to-floor...
connections needs to be accounted for when designing both the floor diaphragms and the wall system.

The recent Canterbury earthquakes have highlighted that the interaction between the wall and the surrounding structural elements is also a significant issue for all buildings, including those that are designed with traditional cast-in-place RC walls. The poor performance and/or partial collapse of several RC walls during the Canterbury earthquakes has been partially attributed to the vertical elongation of the wall resulting from the formation of flexural cracks and subsequent development of a plastic hinge, which when restrained by the floor diaphragms, results in an increase in axial and shear forces in the wall [1, 29].

5.1 PROTOTYPE BUILDING ANALYSIS

To demonstrate the influence of wall-to-floor interaction on the seismic response of a building that included a self-centering wall system, a prototype structure was designed and analysed. The layout for the four storey prototype building is shown in Figure 9a, with an assumed inter-storey height of 3.66 m. Lateral resistance in the N-S direction relied exclusively on two PreWEC walls. The dimensions of the PreWEC walls were equivalent to the full-scale version of the test unit described in Sections 3 and 4. Although the analysis focused on the response of the building in the N-S direction, lateral resistance in the E-W direction of the prototype building was provided by two moment resisting frames. 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 N-S and precast hollowcore units spanning E-W.

5.1.1 FINITE ELEMENT MODEL

A finite element model (FEM) was developed to investigate the lateral load response of one half of the four storey prototype building, with both a rigid and an isolated wall-to-floor connection modelled. The PreWEC wall was modelled in the same way as that described in Section 4.1. 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 9b, and more detailed information on the finite element model has been published by Henry [25].

5.1.2 FEM RESULTS FOR CAST-IN-PLACE FLOOR

The first analysis was conducted to simulate the behaviour of a cast-in-place floor slab with a rigid wall-to-floor connection. The shell element representing the floor diaphragm extended all the way through the concrete wall, and the cast-in-place connection was modelled using an embedded constraint to couple the floor nodes within the connection region to adjacent nodes of the wall.

The deformed shape of the building FEM at 3% lateral wall drift is shown in Figure 10a. The PreWEC wall behaved in the same manner as the FEM of the individual PreWEC unit [25], 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 conducted in the N-S direction, 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 11a, 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.

![Figure 9 - Four-storey prototype building](image-url)
However, strains of a large magnitude are confined to a small region located directly adjacent to the end of the wall. To determine how wide the cracks in the floor slabs would open, strain magnitudes in the steel reinforcement were investigated. The FEM calculated maximum strains in the top layer of reinforcing steel in the level 4 floor slab are plotted in Figure 11b, 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 end of the wall were expected to yield, which indicated that most of the concrete cracks should close up when the load is 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. [30]. Consequently, the damage to the floor slab would result in residual crack widths being less than 1.0 mm, which can be easily repaired. However, yielding of the reinforcement results in hysteretic energy dissipation, which combined with the residual crack width, may result in larger residual displacements and a loss of the wall system’s self-centering feature.

Figure 10 – Elevation view showing the displaced shape of the prototype building FEM (displacements magnified 3x)

Figure 11 – 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 12, 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 drift 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 could lead to undesirable failure mechanisms such as shear failure or base sliding of the wall panel. Clearly the influence of the floor diaphragm should be considered and quantified during the design process and it is evident that analysing or designing a wall by itself without accounting for interaction between the wall and floor is not sufficient, even when the wall is the primary lateral load resisting structural element.

![Figure 13 - Displaced shape of the floor section FEM at 3% wall drift (displacements magnified 10x)](image)

Figure 14 – Monotonic and cyclic moment-drift response of the cast-in-place floor section

5.1.3 FEM RESULTS FOR ISOLATED FLOOR

In addition to the rigid cast-in-place 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 the vertical uplift of the wall while still providing a load path for the horizontal inertia forces. To achieve the isolated connection in the FEM, a cut-out gap was provided in the floor diaphragm around the entire PreWEC wall and the horizontal displacements of floor nodes were coupled 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 10b 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.
As well as preventing damage to the floor diaphragms, the isolated wall-to-floor connection resulted in a more predictable and dependable lateral strength. Figure 15 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.

5.2 WALL-TO-FLOOR CONNECTIONS FOR PREWEC

As shown above, the deformation of the floor diagrams can be significant when the floor is rigidly connected to a post-tensioned wall. However, for a jointed wall system with two closely spaced wall panels of equal size, the wall-to-floor interaction could be significantly more damaging than that predicted by the FEM analysis described above. If the floor is rigidly connected to both coupled wall panels, then a large displacement incompatibility would exist in the floor diaphragm at the location adjacent to the joint between the wall panels. Careful detailing would be required at the wall-to-floor connection to avoid causing damage to the floors and allow the two walls to rock independently. For example, the jointed wall tested in the 5-storey PRESSS building used a special pivoting connector to connect the floor diaphragms to the center of each of the two wall panels [4], which is a detail that is considered impractical for real construction.

When compared to a single concrete wall or a jointed wall, the arrangement of the PreWEC system can offer unique solutions for wall-to-floor connections. 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. One possible isolating detail for a wall-to-floor connection that could be used in conjunction with the PreWEC wall system is shown in Figure 15. The precast floor units could be seated on a steel angle that is only attached at each end to the PreWEC end columns. The gravity load from the floor area would be transferred to the column elements, and the seismic inertia forces could be transferred to the wall by placing pinned struts between the columns and the wall or by using slotted connectors located between the floors and the wall.

Figure 15 – A possible detail for an isolating wall-to-floor connection in a building with the PreWEC wall system

6 TIME-HISTORY ANALYSIS OF A PROTOTYPE BUILDING

Time-history analysis of the prototype building shown previously in Figure 9 was conducted to verify the seismic response and design procedures for buildings that incorporate PreWEC walls [25]. A lumped plasticity model was developed to represent the prototype building, including PreWEC walls, beams, columns, and floor elements. The inelastic hysteresis response of the PreWEC test unit was modelled using a series of calibrated rotational springs at the wall base and the wall-to-floor interaction was included using rotational springs placed between the floor and wall elements, with a hysteresis definition derived from the floor section FEM that was described in Section 5.1. The prototype building was designed for use in a region of high seismicity using a direct displacement-based design method [30]. The model was subjected to eight different earthquake ground motion records, scaled to both the design base earthquake (DBE), and maximum considered earthquake (MCE) intensities.

The analyses of the prototype building were repeated with both a rigid cast-in-place and an isolated wall-to-floor connection. The average maximum and residual interstorey drifts from the time-history analyses of the prototype building with both the isolated and cast-in-place floor connections are shown in Figure 16. Both building configurations performed well, with average maximum interstorey drifts being below the 2% and 3% drift limits for the DBE and MCE respectively and average residual interstorey drifts being less than 0.06%. Due to the overstrength provided by the rigid connection, the buildings with cast-in-place wall-to-floor connections had lower maximum and residual interstorey drifts than did the buildings with isolated wall-to-floor connections. However, the buildings with cast-in-place wall-to-floor connections also attracted higher peak floor accelerations and a peak base shear, than for the buildings with isolated wall-to-floor connections. The difference in the dynamic behaviour of the two building configurations highlighted that although the influence of the wall-to-floor interaction results in only a small
change in the seismic response of a building with self-centering walls, the wall-to-floor interaction should be accounted for during seismic design in order to ensure that the building conforms to performance based design objectives. It should be noted that these analysis results are only applicable to the specific prototype building reported here, and that the difference between the seismic response of buildings with isolated and cast-in-place floor connections will be influenced by the building layout and design.

2. Experimental testing verified that the PreWEC system provides excellent seismic resilience by combining a self-centering rocking wall with easily replaceable energy dissipating connectors.

3. The cyclic response of the PreWEC test specimen can be accurately captured using a detailed finite element model.

4. An extension of the finite element model of the PreWEC specimen highlighted the importance of accounting for the wall-to-floor interaction when designing low-damage buildings. When rigid wall-to-floor connections are detailed, damage to the floor diaphragms will occur and the increase in lateral resistance results in higher shear forces in the wall.

5. Time-history analysis has confirmed that a building with PreWEC walls can be designed using displacement based design procedures, but the inclusion of the entire structure in the analytical model is important in order to ensure that the seismic performance based design objectives are achieved.

8 FUTURE RESEARCH DIRECTION

Further research into the topics detailed herein is in progress at both the University of Auckland and at Iowa State University. Current research is focusing on the dynamic response of self-centering concrete walls and the analysis and design of buildings that utilise self-centering walls. Dynamic tests of post-tensioned walls will soon be conducted at the University of Auckland using an eccentric mass shaker. These dynamic tests will complement testing being undertaken by researchers at Iowa State University and the University of Minnesota [31] as part of a large NEESR project, which includes shake table tests on rocking walls incorporating PreWEC systems. Additionally, large-scale subassembly tests of a PreWEC wall and floor diaphragms will be completed as part of the NEESR project, with the preliminary building analysis described herein serving as the premise for the experimental configuration. The results of these experimental and analytical investigations will be used to refine and simplify the design procedures for buildings with self-centering walls.

The authors would greatly appreciate feedback regarding the PreWEC system and topics discussed herein. Furthermore the PreWEC concept is not restricted by patents and a design methodology has been established. The authors would welcome collaboration to implement such a system into new buildings.

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