

## EXPERIMENTAL PERFORMANCE OF HYBRID FRAMES SYSTEMS WITH NON-TEARING FLOOR CONNECTIONS

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### ABSTRACT :

The effects of beam elongation in precast frame systems have been demonstrated to be a potential source of un-expected damage to precast floor systems, unless adequate detailing is provided to account for displacement incompatibilities between the lateral resisting systems and the floor. The Precast Concrete Seismic Structural Systems (PRESSS) research program carried out in the 1990s has shown the efficiency of dry-jointed ductile connections for moment resisting frames in order to reduce damage while sustaining high lateral loads. However, damage to precast floor systems, resulting from the geometric elongation of the beam, has yet to be addressed in detail. In this contribution, an overview of alternative solutions developed to control and reduce the damage to the floor system is given. The concept of an articulated jointed floor connection for a gapping frame system is first presented. Special attention is then given to an innovative frame connection, referred to as “non-tearing floor”, designed to minimize the problems associated with beam elongation effects, regardless of the floor system adopted, while maintaining low-damage and re-centering characteristics of the hybrid connections. The concept is presented along with the experimental response and validation with a two storeys precast concrete frame system, 2/3 scaled. The experimental and analytical results confirm the unique flexibility of both proposed solutions highlighting their superior seismic performance when compared to traditional systems: minor damage to the frame, wall and floor systems is achieved even under major earthquake loading.

**KEYWORDS:** Precast, Hybrid, Post-Tensioning, Unbonded, Floor, Damage.

### 1. INTRODUCTION

The use of beam sideway mechanism of plastic deformation is a common design practice for multi-storey. This mechanism distributes the plasticity throughout the height of the building by ensuring the formation of plastic hinges in most beams and at the column base (Figure 1-Left).

Experimental and numerical studies [Douglas, 1992; Fenwick and Megget, 1993; fib Bulletin No. 27, 2003] have shown that plastic beam hinges cause growth in the beam length depending on the beam depth, expected position of the neutral axis and rotation (drift) demand. Typical magnitudes for the elongation have been observed to be 2-5% of the beam depth per plastic hinge. Beam growth induces axial load within the beams and shear forces within the columns. This effect increases the moment capacity of the beam and moment demand within the column, both of which can cause undesirable hinging in the columns (weak-column, strong-beam mechanism). Furthermore, the use of special compatibility considerations is required when designing the connections between the floor system and the lateral resisting system. The forces required to restrain this elongation are higher than the capacity of the diaphragm reinforcement and the magnitude of restraining forces depends on the flexural reinforcement within the beams. It is therefore not feasible to prevent beam elongation altogether, rather a more appropriate solution is to accommodate beam elongation by providing support of the floor in the form of special floor connection detailing and/or appropriate seating.

Experimental research on the three-dimensional behaviour of precast super-assemblages consisting of precast moment-resisting frames and precast hollow-core floor units [Matthews, et al., 2003] had reported total collapse of the floor due to the loss of the precast hollowcore floor seating (Figure 1-Centre). Similarly, another research on a two-story precast concrete dual system and using double tees floor system had reported excessive cracks on the topping in the slab-wall joint [Rodriguez and Blandon, 2005]. Recent research on the three-dimensional sub-assemblage of a two-bay moment resisting frame with transverse beams framing into each column with deep

precast prestressed rib floor units had reported that slab can increase the strength of reinforced concrete moment resisting frames by acting as deep beams to restrain elongation in plastic hinges in perimeter frames as shown in Figure 1-Right where the diagonal cracks inclined towards the central column restrain the elongation and the longitudinal cracks parallel to frame, near the end slab, shows the deep beam actions of the floor [Peng, et al., 2008].



Figure 1 Desirable weak beam strong column mechanism (Left); Floor collapse due to lost of floor support (Centre) [Matthews et al., 2003]; Floor slab crack patterns (Right). [Peng et al., 2008].

These researches had further underlined issues related to the displacement incompatibility between precast floors and the lateral resisting system which included beam-elongation effects. Furthermore, capacity design principles require to determine the strength of plastic regions and calculate the theoretical over-strength of the beams. Appropriate design criteria and detailed technical solutions should thus be adopted to overcome such issues.

In the 1990's, a revolutionary alternative approach in seismic design has been developed under the PREcast Seismic Structural System (PRESSS) Research Program coordinated by the University of California, San Diego [Priestley, 1991; Priestley et al., 1999]. The major characteristics of these connections were the use of rocking connections based on unbonded post-tensioning techniques and grouted internal mild-steel reinforcement.

As part of the project, a number of beam-to-column connections and special floor-frame mechanical connectors, were developed with the intent to reduce the damage in the structural component of the lateral-resisting systems, while also addressing some issues related to beam elongation between the precast floor units and the lateral resistant elements [Priestley, 1996 and Nakaki, et al., 1999]. During construction, the beams and columns are separated by a small gap which is subsequently grouted (Hybrid connection) or partially grouted (TCY-GAP connection). In particular, the later solution (TCY-GAP connection) was developed to avoid the primary elongation effects associated with both geometric and material beam elongation. Under lateral drift, no growth of the centre-to-centre distance between columns is observed. However, a "tearing floor" actions occurring due to the gap-opening at the top of the beam would still occur.

It is worth emphasizing that beam elongation effects are typical of both cast-in-situ concrete and precast concrete frames. Two contributions to beam elongation are commonly recognized: a) a material contribution due to the cumulative residual strain within the steel, and b) a geometrical contribution due to the rotation of the beam about the neutral axis position, thus also function of the depth of the beam. With regards to jointed ductile connections with re-centering characteristics, the extent of beam elongation is significantly reduced, being limited to the geometrical contribution alone. Furthermore, such effects could be minimized when a reduced depth of the beam is adopted due to the use of internal prestressing or external post-tensioning.

## 2. ALTERNATIVE SOLUTIONS FOR HYBRID FRAME SYSTEMS WITH NON-TEARING FLOOR CONNECTIONS

In addition to controlling the damage to the structural frame and walls via the use of a rocking/dissipating system, alternative innovative solutions have been developed and are available to reduce the damage to the floor. The first approach consists of using standard precast rocking/dissipative frame connections (herein referred to as "gapping") in combination with an articulated or "jointed" floor. The second approach would rely upon a newly developed "non-gapping" system in combination with a standard floor solution (i.e. topping and continuous starter bars).

### **2.1. Gapping frame systems and articulated floor**

A number of mechanical connections between double-tee floor units and the lateral load resisting system were developed under the PRESSS research program [Priestley, 1996, Priestley et al., 1999]. A welded X-plate mechanical connector (Figure 2-Left) between the double-tee floor members and the frame beams was implemented for the PRESSS five-storey precast concrete test building with satisfactory results. Based on this previous work and on recent research carried out at the University of Canterbury, an articulated (jointed) floor-frame connection has been developed for traditional hybrid technology [Pampanin et al., 2006].

According to this proposed solution, the hollow-core units are connected to the lateral beams by special sliding/shear mechanical connectors, acting as shear keys when the floor moves (relatively) in the direction orthogonal to the beam and as sliders when the floor moves in the direction parallel to the beam (Figure 2-Right). As a result, the system is able to accommodate, with no damage to the floor, the displacement incompatibility between floor and frame by creating an articulated or jointed mechanism effectively decoupled in the two directions. Results of this research showed that the beam elongation effects, in this case limited to the geometrical contribution, can be avoided by using a traditional gapping solution and a “smarter” floor-frame connection. No damage in the floor system due to the gap opening mechanism are thus expected. Also, due to the low flexural stiffness of the -connectors in the out of plane directions, torsion of the beam elements due to pull out of the floor or relative rotation of floor and edge support, can be limited.

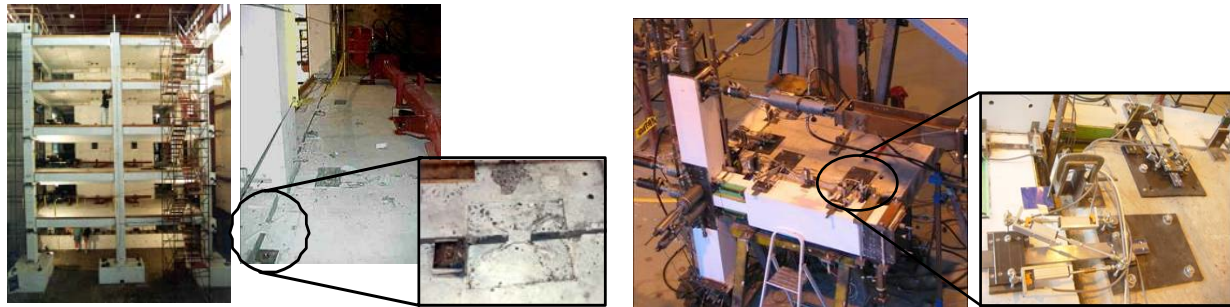


Figure 2. PRESSS X-plate mechanical connector type (left) [Priestley et al., 1999]; Beam-column joint with articulated floor connection (right) [Pampanin et al., 2006].

### **2.2. Non-gapping frame systems and standard floor**

Previous research [Palmieri et al., 1997] indicated that the concept of the TCY-GAP connection could be improved by “inverting” upside-down the system so that the hinge connection is located at the top with the dissipation mechanism at the bottom. Therefore, this would have the advantage of avoiding large inelastic displacements thus reducing damage in the floor [Priestley, 1996]. Experimental research on “emulative” cast-in-situ concrete beam column joints have shown that the damage to the floor using a “slotted beam” (Figure 3-Left), constructed with a narrow vertical slot at the beam ends that runs from the bottom of the beam up to the bottom of the floor [Ohkubo and Hamamoto, 2004]. The flexural strength of the system is governed by the bottom beam reinforcement which is continuous through the slot; meanwhile yielding of the top reinforcement is avoided. Experimental results showed that beam elongation effects were minimized and minor floor cracks were observed comparing with conventional solutions. However, the special detailing required to guarantee an efficient and reliable mechanism and structural performance are yet to be properly developed and translated in simple design recommendations, particularly when using precast concrete solutions.

Towards to a more practical precast beam-column connection system in trying to eliminate beam elongation effects, the concept of a “non-tearing-floor” seismic resisting system have been introduced [Amaris et. al., 2007] using the advantages of the PRESSS technology, while still relying on more traditional floor-to-frame connections (i.e. topping and continuous starter bars). The evolution of the connection started from an inverted TCY-Gap solution based on a single top hinge (top pad or similar contact thick element) with a gap and grouted internal mild steel bars in the bottom part of the beam. This modification prevents both elongation (non-gapping connection) and direct tearing effects in the floor. However, as per the original TCY-gap solution, a re-centering behaviour cannot be developed due to the use of a straight tendon configuration. In order to obtain a full re-centering, while allowing for longer span construction (large open space) typical of prestressed or

Figure 1 consists of two diagrams, (a) and (b), illustrating the reinforcement details of the test beams.

Diagram (a) is a side view of a beam. It shows a central rectangular section labeled "SLAB" with "TOP REINFORCEMENT" and "BOTTOM REINFORCEMENT". The beam is supported by "STEEL TUBE" columns at both ends, which have "SLOT" openings. "SHEAR REINFORCEMENT" is shown as diagonal lines within the slab. The bottom reinforcement is connected to the steel tubes.

Diagram (b) shows two cross-sectional views of the beam. The left view shows the "TENDON 1" and "TENDON 2" anchorage. It features a "METALLIC TOP HINGE CYLINDER SITTED IN A CORBEL" and "UNBONDED POST-TENSIONING STRANDS DRAPPED". The right view shows the "TENDON 1" and "TENDON 2" passing through the beam. Both views show "EXTERNAL MILD REINFORCING INSIDE OF A PACKED" concrete section.

Figure 10 consists of a main elevation view of the experimental setup and three cross-sectional details labeled SECTION A-A, SECTION B-B, and SECTION C-C.

The main elevation view shows a frame with two columns and a beam. Key components and dimensions include:

- Top Section:** JACK, ANCHORAGE SYSTEM, LOAD CELLS/TENDON ANCHORAGE SYSTEM, STUB, TOP ACTUATOR, REACTION FRAME.
- Dimensions:** 6.86, 0.47, 0.25, 0.50, 2.10, 2.06, 0.47, 0.50, 0.24, 0.15, 0.50.
- Other Labels:** ROTARY POT ATTACHED TO INDEPENDENT FRAME, MIDDLE ACTUATOR, LOAD CELL.

**SECTION A-A:** Shows a tendon profile with dimensions 0.47, 0.16, 0.16, and 0.30. Labels include: THREAD BAR D20, TENDON PROFILE, 8-HD16, STIRRUP R8, DISSIPATOR, 8-HD16.

**SECTION B-B:** Shows a tendon profile with dimensions 0.47, 0.30, and 0.50. Labels include: TENDON PROFILE, 4-HD16, stirrup R8, 4-HD16.

**SECTION C-C:** Shows a tendon profile with dimensions 0.50, 0.50, and 0.50. Labels include: TENDON PROFILE, STIRRUP R10, 12-D25, TENDON PROFILE.

The right side of the figure shows a detailed view of the tendon profile and its connection to the column and beam. Key components and labels include:

- Top Section:** ANCHORAGE SYSTEM, TENDON PROFILE, TORSION AND UPLIFT STEEL PLATE, MONOHINGE, BEAM.
- Bottom Section:** CORBEL, THREADED BAR D20 GRADE 8.8, DISSIPATOR STEEL PLATE, DISSIPATOR, TENDON PROFILE, COLUMN, STUB.

The connection design followed the Appendix B of the Concrete Standard NZ3101:2006 for jointed ductile connections. A total of eight 7-wire prestressing strands (0.5 in diameter,  $A_{\text{nominal}} = 99\text{mm}^2$ ) were used for each



beam, with two ducts of four tendons each. This resulted in the required design beam moment of 180kNm at 2% of lateral drift. An initial post-tensioning of 50% the ultimate stress  $f_{pu}$  (1860MPa) was applied, resulting to a total initial post-tensioning force per beam-column connection of approximately 744kN. An axial force of 162kN, representing 22% the ultimate stress  $f_{pu}$ , was applied to each column via four unbonded post-tensioned tendons ( $A_{pt} = 396\text{mm}^2$ ) and illustrated in Figure 4.

### **3.2. Testing Program**

A series of quasi-static cyclic displacement control tests were carried out under increasing levels of lateral top displacement, with force control on the first floor to ensure a correct lateral load distribution. The testing protocol complied with the “acceptance criteria” proposed by the ACI Innovation Task Group 1 and Collaborators and consisted of a series of three cycles of drift, followed by a smaller single cycle.

During the imposed displacement protocol, a rocking mechanism was developed at the column base. Two types of tests were considered, 1) maintaining a constant column axial load (PT), or 2) varying the column axial load ( $PT_{NC}$ ). In the first case, which represents a solution without vertical post-tensioning in the column, a constant load of 162kN (representing the gravity load only) in each column was applied (Figure 4-Left).

In the second case, which represents a solution with vertical tendons in the columns, the post-tensioned tendons were locked off resulting in an increase to the column axial load due to the elongation of the post-tensioned tendons as the columns rocked upon their foundation. In normal construction practice, when vertical tendons are adopted, they would likely be initially post-tensioned. However, in this experimental programme the initial post-tensioned forces were assumed to equal the gravity load of 162kN, which would represent the case of slack (no initial prestressing) vertical tendons. This test parameter was chosen to limit the demands imposed to the strong floor within the Civil Engineering Laboratory. Furthermore, three energy dissipaters using 7, 10 and 13mm diameter mild steel bars were considered for each axial load regime above to demonstrate the re-centring capabilities of the frame.

### **3.4. Connection Details**

The connection detail (Figure 4-Right) comprises of a top mono-hinge, steel armouring at the beam ends and a hidden corbel (acting as the beam-shear transfer mechanism). A T-shaped steel element is used as a shear key to prevent beam uplift and torsion while accommodating the tolerances in the beam length. An asymmetric unbonded post-tensioned tendon profile is adopted and combined with external energy dissipaters for supplementary damping. A steel armour was cast into the concrete during construction of the beams which consist of three steel plates (one top and two lateral) welded together. A steel cylinder (with one quarter removed) was connected to the top of the steel plate and welded to the steel armouring. The steel hinge element is attached to the beam using four high strength threaded bars bolted to the top steel plate and threaded into the underside of the beam. The corbel consists on a steel angle, stiffened with four welded steel plates which added to the shear and bending capacity of the corbel. The steel corbel was attached to the column using high strength bolts, threaded into cast in-situ inserts. The mild steel dissipaters (Figure 5-Left) are fabricated from round mild steel bar, threaded at each end and machined down to a specific bar diameter over a pre-determined length; defined as the unbonded length. The unbonded length prevents premature fracture of the bar by limiting the strains to allowable limits. A 34mm (outside diameter) steel tube with a wall thickness of 2mm (anti-buckling restraint), is located over the machined area of the steel bar and temporary fixed in place. Epoxy was then injected into a hole (Figure 5-Right) supplied at the bottom of the steel confinement tube to ensure that all the air was expelled out from the top.

Three different fuse diameters ( $D_f$ ) were considered: 7mm, 10mm and 13mm with an unbonded length of 90mm. The device was designed to ensure that all the plastic deformation is confined to the fused region of the steel bar. The dissipaters are expected to be displaced in net tension and compression displacements due to the specific nature of the mono-hinge connection. Cyclic tests reveal that the dissipater has a reduced energy dissipation capacity when displaced into net negative displacements and then was reduced further by buckling at relatively low displacements.

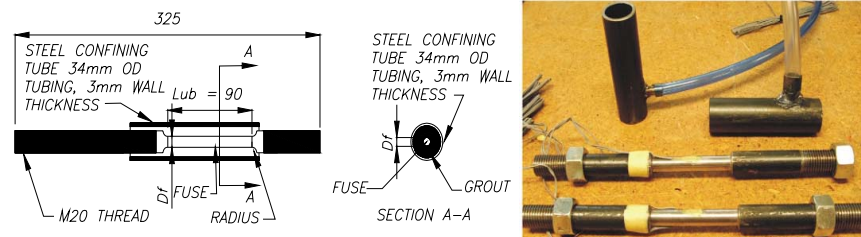


Figure 5. Dissipators rods detail (Left); dissipators rods construction (Right)

#### 4. TEST FRAME EXPERIMENTAL RESPONSE

##### 4.1. Behaviour of Unbonded Post-Tensioned-only Solution

As mentioned, two tests were first carried out on an unbonded post-tensioned only solution: a) varying the column axial load (test PT<sub>NC</sub>) or b) having a constant axial load (test PT). Figure 6 –Left shows the force displacement hysteretic behaviour up to 3.5% drift for the post-tensioned solution having constant axial load. In general the behaviour was a stable non linear elastic response with some dissipation coming from friction within the ducts and within the steel mono-hinge. No significant reduction in stiffness on loading was observed. Similar behaviour was observed for the variation of axial load, test PTNC. However, a larger bi-linear stiffness is observed for test PTNC when compared to test PT due to the increment in axial load increasing the column moment and thus the lateral capacity.

##### 4.2. Behaviour of the Hybrid Solution

Additional energy dissipation capability was added to the unbonded post-tensioned only solution in the form of mild steel external dissipaters. As per testing reported above, the same series of tests and test set-up were carried out for the hybrid solution. The experimental results presented in Figure 6-Centre correspond to the force displacement hysteresis behaviour of test PT7 (7 mm fuse dissipaters, constant axial load). A stable flag shape hysteresis is observed with higher dissipation when compared to the unbonded post-tensioned only solution. Furthermore, re-centring is achieved up to a lateral drift of 3.5%. The concave bilinear slope indicates the onset of stiffness degradation as a result of buckling of the external dissipaters in compression.

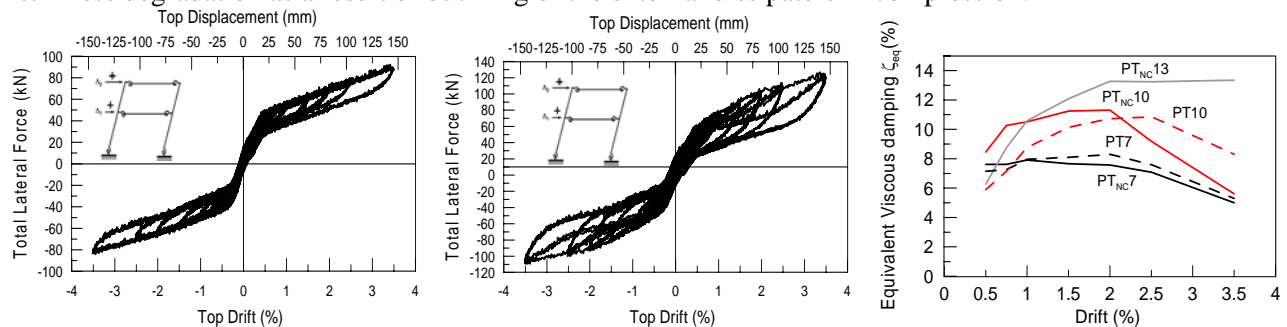


Figure 6. Unbonded post-tensioned only solution test PT (Left); Hybrid system with external dissipaters Test PT7 (Centre); Equivalent viscous damping (Right)

A similar force displacement response was obtained for the hybrid system having no constant axial load. However, a comparison between the two tests (constant or varying the axial load) with the same energy dissipation devices, shows that the test with constant axial load has a reduction in the total strength and lower bilinear stiffness due to the constant moment contribution at the column base and higher energy dissipation being provided by friction from the axial load control system.

Area based equivalent viscous damping was calculated for all the tests and illustrated in Figure 6-Right. A maximum of 14% of equivalent viscous damping is observed for test PT<sub>NC</sub>13. Due to buckling of the dissipaters between a lateral drift of 2% and of 2.5%, a reduction in the damping is observed there after. The dissipaters within test PT<sub>NC</sub>13 have greater anti-buckling resistance and better performance was observed. A comparison between the constant and variable load tests shows no significant differences in terms of hysteretic damping.

## 5. ANALYTICAL-EXPERIMENTAL COMPARISON

A simple analytical model can successfully reproduce the experimental results, providing a reliable control over the expected hysteresis and dynamic behaviour. As an example, an analytical-experimental comparison using a lumped plasticity model based on the combination of different non-linear inelastic rotational springs located in parallel at the rocking interface is shown in Figure 7. The spring properties can be evaluated via a monotonic moment-rotation analysis which is evaluated based on a global member compatibility condition using the monolithic beam analogy principles [Pampanin et al., 2001].

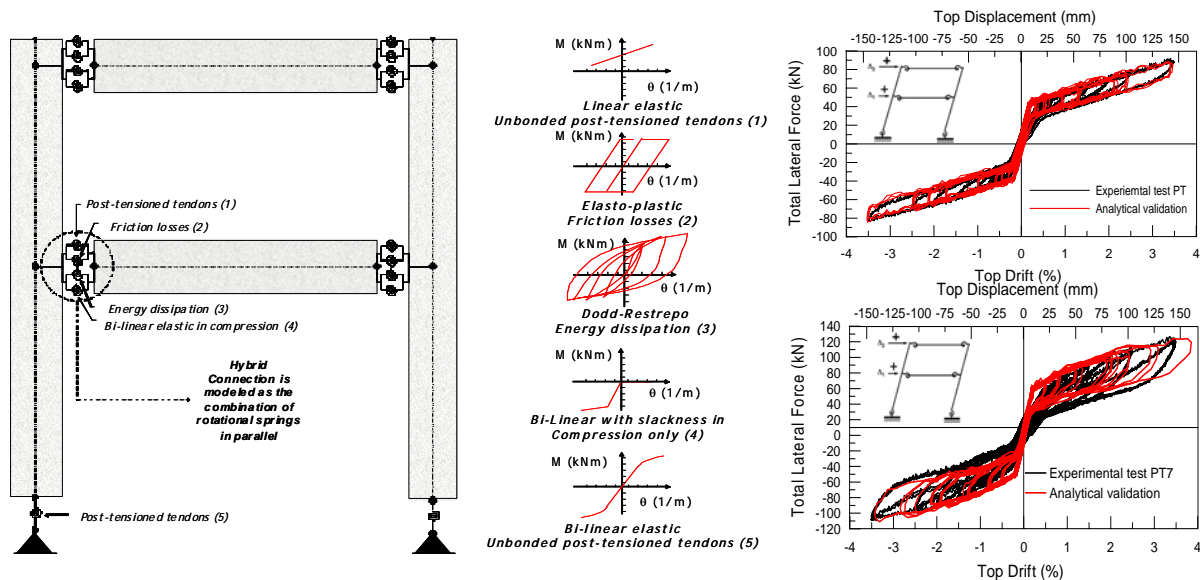


Figure 7. Lumped plasticity model implemented and Hysteretic rules assigned to each rotational spring (left); Force displacement response of test PT (Right-top); Force displacement response of test PT7 (Right-bottom).

The non-linear finite element program Ruaumoko2D [Carr, 2008] was used to model the series of experimental tests adopting each of the moment-rotation springs above. The recorded lateral force time history during testing was used as the input loading history for the model at each floor. The unbonded post-tensioned tendon was model using a linear elastic rotational spring, while a second non-linear inelastic spring was used to represent the mild steel energy dissipation contribution using Dodd-Restrepo hysteresis rule. As the dissipaters are displaced into net negative displacements and buckling is initiated the dissipater does not contribute to the energy dissipated in the system. However, the bearing effect on the epoxy filled material increases the compression stiffness and strength. Therefore an additional compression-only moment-rotational spring is added at the same location as the dissipaters such that the combined stiffness and strength between the two springs in parallel represents the observed behaviour during testing of the dissipater elements. Friction forces occurring between the tendons and the plastic ducts were also considered and implemented with an elasto-plastic hysteresis rule considering a high initial stiffness where the yield moment corresponded to the friction force in each tendon. A simple bi-linear elastic hysteresis rule was adopted at the column base to model the axial force moment contribution due to the post-tensioned tendons. Given the simple hysteresis rules adopted, Figure 7 illustrates how the global hysteretic response is accurately represented with reasonable accuracy for tests PT and PT7.

## 6. CONCLUSIONS

An overview of alternative solutions developed to control and reduce the damage to the floor systems has been presented and discussed. Two solutions had been experimentally implemented with satisfactory results in reducing damage to the floor. The first solution consists of using standard precast rocking/dissipative frame connections in combination with an articulated or “jointed” floor. The second solution would rely upon a newly developed “non-tearing” system in combination with a standard floor solution.

In particular, focus has been given to the second solution, because the use of standard floors seems to be more

practicable. The experimental results on cyclic testing of a two-story, one bay precast frame, implementing the proposed non-tearing floor solution relying on unbonded post-tensioning techniques and the use of external dissipation, provide satisfactory confirmation that the effects associated with beam elongation can be eliminated, thus significantly reducing the expected damage in the floor.

Validation of the results using simple moment rotational springs has provided good agreement with the experimental results. A series of numerical investigations are under-going to provide further confirmation of the enhanced global seismic behaviour for practical implementation within design guidelines.

Following the successful proof of concept of the proposed solutions, further refinements are under going targeting simplified and improved construction detailing for a wider adoption by the construction industry.

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