SOLUTIONS TO CONTROL AND MINIMIZE FLOOR DAMAGE IN PRECAST CONCRETE BUILDINGS UNDER SEVERE EARTHQUAKE LOADING

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SUMMARY

The effects of beam elongation in precast frame systems have demonstrated a potential source of unexpected damage to precast floor systems. The PRESSS research program has shown the efficiency of dry-jointed ductile connections for moment resistant systems, based on unbonded post-tensioning techniques, to achieve a minimum damage while sustaining high lateral loads. However, damage to precast floor systems, resulting from a 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 systems is presented. The concept of articulated jointed floor connections or non-tearing floor solutions are presented along with experimental validations on a series of 2/3 scaled, precast beam-column joints tests with articulated floor systems and on a two storey, single bay, precast concrete frame system with an innovative “non-tearing floor” connection. Both numerical and experimental results confirm the unique flexibility of the proposed solution to reduce damage control in the floor and highlight the superior performance of the overall system under seismic loading when compared to more traditional solutions.

PAST EARTHQUAKE EXPERIENCE IN PRECAST CONCRETE FRAMES BUILDINGS

Floor systems play an important role in the lateral resistance of building structures by providing diaphragm action. In addition to distributing lateral forces such as wind, or earthquake forces to the structural elements as well as gravity loads, the diaphragm must tie the whole structural system together.

Several precast concrete parking structures performed poorly in the 1994 Northridge, California, earthquake [1]. Investigations proved that the damage of the structures was not due to the use of precast concrete itself, but better related to inadequate design/construction detailing as explained below.

Firstly, in many buildings, excessive deformation compatibility between gravity-load frames and earthquake-force-resisting frames were observed. The gravity framing was not capable of maintaining its gravity-load-carrying capacity while deforming with the lateral-load-resisting system under the earthquake motion (Fig. 1).

Secondly, the diaphragm (floor) action was inadequate due to the higher-than expected floor acceleration. Therefore, the lateral-force-resisting system was subjected to minor-moderate damage while the precast floor units collapsed, demonstrating that the diaphragm was the weakest element in the structural system.

Fig. 1 – Collapse of CSUN Parking Structure Northridge Earthquake. U.S. Geological Survey [2].

Additionally, these precast concrete parking structures were with cast-in-place topping slabs, but the diaphragm reinforcement was placed within the topping slabs. Cracking of the topping slabs due to temperature effects and shrinkage led to the development of concentrated cracks in the diaphragm even before the earthquake occurred. The distributed reinforcement commonly used in the topping slabs did not have the strain capacity to bridge these cracks. Fracture of the slab reinforcement led to significant decreases in the calculated shear strength of the diaphragm. The outcome of this poorly precast concrete behaviour under seismic event had led to a series of investigations that have been included in the most important seismic guidelines around the world [3], [4].

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PROBLEMS ASSOCIATED WITH BEAM ELONGATION IN SEISMIC DESIGN OF CONCRETE BUILDINGS

Most of the standards and codes around the world allow the use of design forces that are generally smaller than those required for elastic response, providing that the critical regions of the structure have adequate ductility and energy dissipation capacity. Such approaches are fundamentally based on a casualty-prevention principle, where structural damage is accepted providing that collapse is avoided. Designers must select a proper mechanism of plastic deformation and using capacity design principles they have to ensure that the chosen mechanism can be developed. Normally, the use of beam sidesway mechanism of plastic deformation (Fig. 2) is a common design practice for multi-storey buildings particularly because ductility can be more easily provided by reinforcing details in beams than in columns. This mechanism distributes the plasticity throughout the height of the building by ensuring the formation of plastic hinges in most beams. These effects could be relevant in the response of the first-story columns since the building foundation in this level may try to restrain elongation of the beams of the first story at the formation of their plastic hinges. This restriction may cause shear force in the columns not considered in the conventional analysis, with the possibility of also creating additional plastic hinges in the columns (weak-column, strong-beam mechanism) and possible collapse of the structure or in loss of support for precast floor systems due to an elongation of the beam elements.

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 typically recognized: a) the material contribution due to the cumulative residual strain within the steel, and b) the geometrical contribution due to the presence of a neutral axis and actual depth of the beam.

Experimental and numerical studies 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 [5], [6], [7]. Typical magnitudes for the elongation have been observed to be 2-5% of the beam depth per plastic hinge. Experimental research on the three-dimensional behaviour of precast super-assemblages consisting of precast moment-resisting frames and precast hollow-core floor units [8] had reported total collapse of the floor units due to lost of seating (Fig. 3).

Similarly, another recent research on a two-story precast concrete dual system, using double tees floor units [9] reported excessive cracks on the topping in the slab-wall joint (Fig. 4).

Recent experimental research at the University of Canterbury on the effect of beam elongation on the response of a three-dimensional sub-assembly consisting or a two-bay moment resisting frame with deep precast prestressed rib floor units have shown that the presence of the slab can significantly increase the strength of the beam plastic hinges, by acting as deep beams to restrain their elongation. This effect is shown in
Fig. 5 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 [10].

All the above mentioned researches further underlined issues related to the displacement incompatibility between precast floors and the lateral resisting system, including the effects of beam-elongation. Appropriate design criteria and detailed technical solutions are thus required to overcome such issues.

**PRECAST CONCRETE IN NEW ZEALAND**

**Floor Unit support and Continuity**

Most of floors in New Zealand buildings are constructed of precast concrete units made of either prestressed concrete or reinforced concrete. Hollow-core floor units are the most common floor type used in the country and typically rely upon cast-in-place concrete topping with at least 50mm thickness and minimum mesh reinforcement to guaranty a transfer mechanism of the in-plane diaphragm forces. Un-topped solutions (i.e. precast concrete floors without cast-in-place toppings and with adequate shear connection between the elements) have been more developed in the U.S. and European practice [11], [12], but have not yet received a wide acceptance and use in New Zealand.

The support for precast concrete floor units may be simple (special for long spans or heavily loaded structures) or continuous. Three types of support for precast hollow-core concrete floor units are identified depending on the depth of the supporting beam prior to placement of the cast-in-place concrete.

Adequate support of precast concrete floor unit is required under the imposed displacement caused by earthquakes or the increase due to plastic hinges in the seismic resisting system. Additional allowances are made for: manufacturing tolerances, long-term effects of volume changes due to concrete shrinkage, creep and temperature effects.

The New Zealand Concrete Standard NZS3101:2006 [13] recommends for precast concrete floor with or without the presence of a cast-in-place concrete topping slab and/or continuity reinforcement that the distance from the edge of the support to the end of the precast member in the direction of its span is at least 1/180 of the clear span but no less than 50 mm for solid or hollow-core slabs or 75 mm for beams or ribbed members.

A typical construction detail for precast floor units is shown in Fig 6. Inertia forces are transferred to the perimeter frame via shear friction (between the cast in-situ topping and the perimeter beam) and/or strut and tie mechanisms. Reinforcing starter bars (from the perimeter beam) are cast within the in-situ topping. This reinforcement can be in the form of hanger, saddle, horizontal or draped reinforcement depending of the type of connection.

When the seating is lost, the reinforcement within the cast-in-place topping alone cannot be relied upon to support the floor as the topping slab may separate away from the precast concrete units.

**Emulation Cast-in-place for moment resisting frames with precast elements**

Moment resisting frames in New Zealand are widely used mostly relying on cast-in-place techniques to provide equivalent “monolithic” connections (i.e. equivalent strength and toughness to their cast-in-place counterparts). The precast elements can be either prestressed or consist of ordinary reinforced concrete.

When referring to precast concrete construction, several alternative solutions (Fig. 7) to provide moment-resisting connections between precast elements for seismic resistance have been studied in the past and developed in literature [12], [14], [15].

As implied in a traditionally accepted seismic design approach, based on the development of a
desired inelastic mechanism through the formation of plastic hinge regions in discrete and controlled locations within the structure (i.e. weak-beam strong-column mechanism according to capacity design principle [16], different levels of structural damage and, consequently, repair cost, will be expected and, depending on the seismic intensity, typically accepted as the unavoidable results of the inelastic behaviour itself.

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.

A number of beam-to-column connections and mechanical connectors, limiting or totally cancelling the effects of beam elongation between the precast floor units and the lateral resistant elements have been developed [18], [19]. 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.

One of the main connection system, referred to as Hybrid frame connection (Fig. 8) uses a combination of unbonded post-tensioning through the centre of the joint, acting as a self-centring, clamping force with mild-steel reinforcement placed inside ducts and grouted for bond conditions as energy dissipation. In this case, thanks to the re-centering contribution, the beam elongation effects, typically given by a geometrical and a material contribution, are limited to the geometric component.

Jointed ductile hybrid systems

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 [17] for precast concrete buildings in seismic regions with the introduction of “dry” jointed ductile systems as an alternative to the traditional emulation of cast-in-place solutions and based on the use of unbonded post-tensioning techniques. As a result, high seismic performance structural systems can be obtained, with the unique potentiality to undergo inelastic displacement similar to their traditional monolithic counterparts, while limiting the damage to the structural system and assuring full re-centring capabilities (negligible residual or permanent deformations).

In an alternative solution referred to as the Tension-Compression Yield–GAP solution (TCY GAP) (Fig. 9) beams and columns are separated by a small “gap” for most of the depth of the beam. A grout pad is acting as pivot point for the rotation of the beam relative to the column. So doing the primary elongation effects associated with both geometric and material beam elongation are completely avoided. The centre-to-centre distance between columns remains constant during the earthquake response. However, such a solution would not account for the tearing floor actions occurring due to the gap-opening at the top of the beam.

Furthermore, top mild-steel bars were inserted into grouted sleeves for energy dissipation while unbonded post-tension tendons were used at the bottom of the beam as a clamping force.
Thus, no re-centering contribution can be provided in such a system, since the tendons located with a straight profile in the centre of the compression grout would not elongate.

REDUCING DAMAGE IN THE FLOOR

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).

Solution 1: gapping frame systems and articulated floor.

A number of mechanical connections between (double-tee) floor units to perform under in-plane seismic forces were developed under the PRESSS research program [20]. A welded X-plate mechanical connector (Fig. 10) between the double-tee floor members and the frame beams was for example implemented in the PRESSS Five-Storey precast concrete building tested at the University of California, San Diego, with satisfactory results.

Recent investigation on different alternative behaviour of double-tee flange connectors subjected to in-plane monotonic and reverber cyclic loads has been performed [21]. Experimental evidence indicates that strength, stiffness, and deformation capacity are highly dependant on the constraint and load condition.

Further developments of this work, have resulted to the development of an articulated “jointed” floor system [22] to be combined with a traditional jointed (“gapping”) hybrid connection or, in principle, with any other standard moment resisting connection.

According to this proposed solution, the hollow-core unit is connected to the lateral beams by on special sliding/shear mechanical connectors which act 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 (Fig. 11, 12). 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 shear keys-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.
Solution 2: Non-gapping frame systems and standard floor

Previous research [23] indicated that the concept of the TCY-GAP connection could be inverted so that the hinge connection was 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 [18].

Experimental research on “emulative” cast-in-situ concrete beam column joints have shown that the damage to the floor using a “slotted beam” (Fig. 13), 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 [24].

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, a “non-tearing-floor” seismic resisting system concept have been introduced [25] using the advantages of the PRESSS technology while still relying on more traditional floor-to-frame connections (i.e. topping and continuous starter bars) herein called non-gapping frame systems.

The evolution of the connection started from an inverted TCY-Gap solution as proposed [23] 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 and tearing effects in the floor whilst no-recentering capacities can be provided due to the location and straight profile of the tendons.

Very satisfactory results were obtained through experimental quasi-static cyclic testing on 2/3 scaled beam-column joint sub-assemblies, implementing the proposed non-tearing floor solution [25]. However, as per the original TCY-gap solution, a full re-centering behaviour cannot be developed due to the use of a straight tendon configuration.

In order to obtain re-centering, while allowing for longer span construction (large open space) typical of prestressed or post-tensioned solution, a further conceptual evolution of the system has been proposed. A single top-hinge connection guarantees no beam elongation while an antisymmetric profile of the unbonded post-tensioned tendons provide the required re-centering capacity, in addition to additional gravity-load carrying capacity.

External replaceable mild-steel dissipation devices can be used in addition to provide the required supplemental damping and connection strength (Fig. 14).
University of Canterbury are presented along with analytical-experimental comparisons.

**EXPERIMENTAL TESTING OF A TWO-STORY FRAME WITH AN ALTERNATIVE NON-TEARING CONNECTION**

A precast concrete prototype building consisting of a six storey, five bays by four bays was designed following a direct displacement based design (DDBD) procedure [26]. After distributing the base shear, the internal actions were scaled to respect similitude requirements for the test specimen. However, due to the space and capacity limitations within the structural laboratory, a two-storey frame, 2/3 scaled, was tested in the laboratory, taken as a super assemblage of the prototype for a proof of concept of the “non-tearing-floor” system.

**Test set-up and specimen description**

The test set-up is shown in Fig. 15. The precast frame consists of two storeys of 2.06m and 2.10m inter-storey height respectively and one bay of 6.86m long. The precast beams are composed of a rectangular section of 470x300mm. An I-shape section was designed at the beam ends to accommodate replaceable external energy dissipaters, while limiting the architecture invasiveness. A 500mm square column cross section was adopted. External column end stubs of 600x250mm at the beam column joint were used on both sides of the joint to accommodate the post-tensioning anchorages with a more efficient distribution of compression stresses within the joint and to reduce reinforcement congestion in the joint region.

![Fig. 15 - Test set up and specimen description.](image)

The connection design was according to Appendix B of the Concrete Standard NZS3101:2006 [14] 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 $f_{\text{ptu}}$ (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 $f_{\text{ptu}}$ was applied to each column via four unbonded post-tensioned tendons ($A_{\text{pt}} = 396\text{mm}^2$) and illustrated in Fig. 15.

**Loading history**

A series of quasi-static cyclic displacement control tests were carried out under increasing levels of lateral top displacement, where the structure was displacement controlled for the top floor and force controlled for the first floor: this ensured the correct lateral load distribution between the two levels. The testing protocol complied with the “acceptance criteria” proposed by the ACI [27], [28] and consisted of a series of three cycles of drift, followed by a smaller single cycle.

**Testing program**

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 (PTNC). In the first case, representing a solution without vertical post-tensioning in the column a constant load representing the gravity load of 162kN in each column was applied and maintained constant during testing using a release valve within the hydraulic jack system (Fig. 15).

In the second case, representing 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.
Connection details

The connection detail (Fig. 16) 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 unbounded post-tensioned tendon profile is adopted and combined with external energy dissipaters for supplementary damping.

Mono-hinge: 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.

Fig. 16 - Connection detail.

Corbel: 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.

Torsion, uplift and construction tolerances: The T-shape steel section (located directly above the steel cylinder) was located at both ends of the beam (Fig. 16) to prevent torsion, beam uplift and tolerance issues. Uplifting occurs due to the vertical component of the post-tensioned forces (static case) and due to the laterally induced beam shear. Torsion could be occur (as typical of any other system) when the precast floor units sit on the beam with an eccentricity. Typical construction tolerances expected on real application were represented and addressed in the following manner:

- Product tolerances were considered for the precast beam by making them 20mm shorter in length than required. Steel shims were used to make up these tolerances and to transmit the axial compression forces from the beam to the column.
- Erection tolerances were expected to be, in general terms, not dissimilar from what would typically occur in normal construction process.

Mild steel dissipater fuse and steel plate: An important component for hybrid solutions are the external energy dissipaters used as supplemental damping devices. The objective of adding external energy dissipation is to dissipate the earthquake-induced energy via sacrificial elements that can easily replaced after a strong earthquake. This minimises costs associated with repair and downtime when compared to conventional buildings.

The mild steel dissipater (Fig. 17a) is 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 unbounded length.

Fig. 17a - Dissipaters rods detail.

Fig. 17b - Dissipaters rods construction.

The unbounded 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 (Fig. 17b) 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 (Df) were considered: 7mm, 10mm and 13mm with an unbounded 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.

**TEST FRAME EXPERIMENTAL RESPONSE**

In the following paragraphs, an up-to-date summary of recent results is outlined. An extensive experimental research programme is ongoing at the University of Canterbury on the development of an innovative floor-to-lateral-load-resisting, “non-tearing floor” connection. Fig. 18 shows the test specimen tested under different axial load configurations and replaceable energy dissipaters.

### 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>N</sub>C) or b) having a constant axial load (test PT). Fig. 19 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 PT<sub>N</sub>C. However, a larger bi-linear stiffness is observed for test PT<sub>N</sub>C when compared to test PT due to the increment in axial load increasing the column moment and thus the lateral capacity.

### Additional Energy Dissipation Capability

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 Fig. 20 correspond to the force displacement hysteretic behaviour of test PT<sub>7</sub> (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.

### Behaviour of the Hybrid Solution

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.

**ANALYTICAL-EXPERIMENTAL COMPARISON**

A simple analytical model can successfully reproduce the experimental results, providing there is 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 Fig 21.

Appropriate hysteretic rules are assigned to each spring property to correctly represent the inelastic behaviour at the beam-column joint which is evaluated based on a global member compatibility condition using the monolithic beam analogy principles [29]. The non-linear finite element program Ruaumoko2D [30] was used to model the series of experimental tests adopting each of the moment-rotation springs above (Fig. 22). The recorded lateral force time history during testing was used as the input loading history for the model at each floor.

Elastic elements are used to represent the precast structural members as proposed [29]. 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 [31]. 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 19 and 20 illustrates how the global hysteresis response is accurately represented for tests PT and PT7. It can be seen the model can reproduce both experimental results with reasonable accuracy. The analytical validation of test PT was simpler than adding energy dissipation solution (test PT7) due to the complexity of the dissipation behaviour in compression.
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.

It was demonstrated that a global re-centring/dissipating response (typically referred to as flag-shape behaviour) can be controlled either by only relying upon the contribution of the self weight of the structure and/or by adding unbonded post-tensioned tendons such that full re-centring can be achieved by the ratio between axial force and dissipation.

Validation of the results using simple moment rotational springs has provided good agreement with the experimental results. 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.

A series of numerical investigations are undergoing to provide further confirmation of the enhanced global seismic behaviour for practical implementation within design guidelines.

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