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EXPERIMENTAL INVESTIGATIONS ON HIGH-PERFORMANCE JOINTED DUCTILE CONNECTIONS FOR PRECAST FRAMES

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SUMMARY

Recent developments on high performance seismic resisting precast concrete frame systems, based on the use of unbonded post-tensioned tendons with self-centring capabilities in combination, when required, with additional sources of energy dissipation, are herein presented. Alternative arrangements for jointed ductile connections to accommodate different structural or architectural needs have been implemented and validated through quasi-static cyclic tests on a series of exterior beam-column subassemblies under uni- or bi-directional loading regime. The results confirmed the unique flexibility and efficiency of these systems for the development of the next generation of seismic resisting structures, able to undergo high inelastic displacement with limited level of damage and negligible residual displacement when compared to traditional monolithic (cast-in-situ) ductile solutions.

In order to further emphasize the enhanced performance of these systems, a comparison with the experimental response and observed damage of 2-D and 3-D monolithic beam-column benchmark specimens designed according to the NZ3101:1995 seismic code provisions is carried out. The reliability and simplicity of recently implemented special code provisions for the design and analysis of jointed ductile systems is also confirmed by satisfactory results of analytical-experimental comparison. In addition, the practical feasibility and efficiency of simple technical solutions to connect precast floor systems and lateral resisting frame systems, without incurring in damage due to displacement incompatibilities are experimentally demonstrated. The reliability of recently implemented special code provisions for the design and analysis of jointed ductile hybrid systems is also confirmed.

1. INTRODUCTION

Major advances have been observed in the last decade in seismic engineering with further refinements of performance-based seismic design philosophies and definition of the corresponding compliance criteria. Following the worldwide recognized expectation and ideal aim to provide a modern society with high (seismic) performance structures able to sustain a design level earthquake with limited or negligible damage, emerging solutions have been developed for high-performance, still cost-effective, seismic resisting systems, based on a efficient use of traditional materials and more recent technology.

When referring to precast concrete construction, several alternative solutions to provide moment-resisting connections between precast elements for seismic resistance have been studied in the past and developed in literature [Watanabe et al., 2000; Park, 2002; fib, 2003] mostly relying on cast-in-place techniques to provide

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equivalent “monolithic” connections (i.e. equivalent strength and toughness to their cast-in-place counterparts). As implicit in a traditionally accepted seismic design approach, based on the development of a desired inelastic mechanism through the formation of plastic hinge regions in the discrete and controlled locations within the structure (i.e. weak beam, strong column mechanism), different levels of structural damage and, consequently, repair cost, will be expected and, depending on the seismic intensity, typically accepted as unavoidable results of the inelastic behaviour itself.

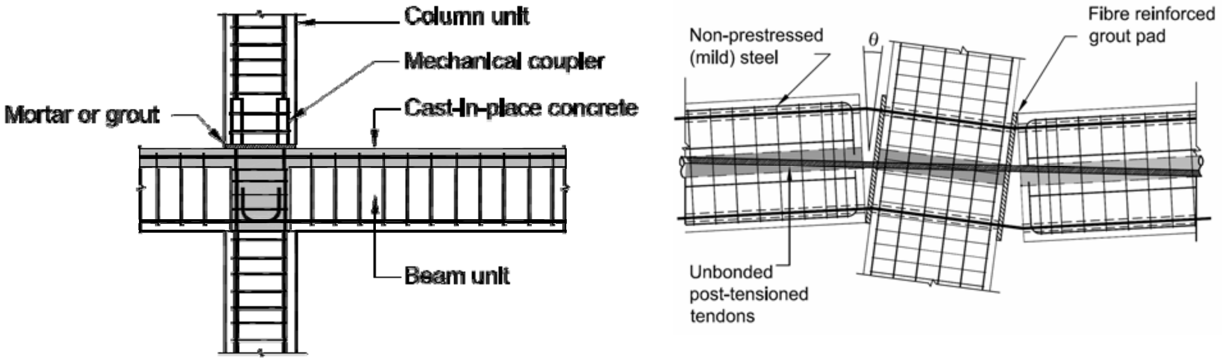

Figure 1: Emulation of Cast-in-place concrete vs. jointed ductile hybrid solutions [Apendix B NZS3101:2006]

In the last decade, a revolutionary alternative approach in seismic design, has been introduced in the solutions developed under the U.S. PRESSS program coordinated by the University of California, San Diego [Priestley, 1991; Priestley, 1996; Priestley et al., 1999] for precast concrete buildings in seismic regions with the introduction of “dry” jointed ductile systems (Figure 1, right side), 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).

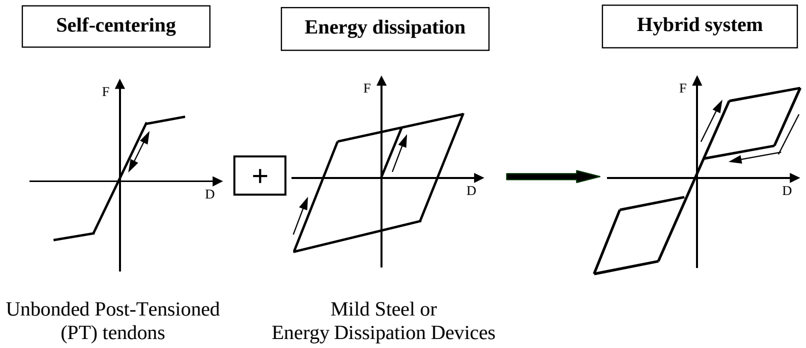

Figure 2: Idealized flag-shape hysteretic rule for a hybrid system (fib, 2003).

A sort of “controlled rocking” motion of the beam (Figure 1, left side) or wall panel occurs, while the relative ratio of moment contribution between post-tensioning and mild steel (typically referred to as λ parameter), [Palermo et al., 2005; NZS3101:2005] governs the so-called “flag-shaped” hysteresis behaviour (Figure 2).

The continuous and rapid development of jointed ductile connections for seismic resisting systems have resulted to the validation of a wide range of alternative arrangements, under the general umbrella of “hybrid” systems, currently available to designers and contractors for practical applications based on a case-by-case (cost-benefit) evaluation. In addition to the relative contribution between recentring and dissipation capacity, main key features differentiating alternative solutions for hybrid systems for seismic resisting frames can be given by: a) the longitudinal profile of post-tensioned tendons: straight, draped tendons/cable profile solutions or combinations of the above depending on the contribution of the gravity and lateral loads effects; b) the type, sources and location of energy dissipation: internal or external supplemental damping device relying on metallic or advanced materials (e.g. shape memory alloys, visco-elastic systems) and implemented following a passive or semi-active control approach; c) the shear transfer mechanism at the critical interface: relying either on friction due to the post-tensioned tendons contribution, or on ad-hoc shear keys or steel corbel. A comprehensive overview of developments on high-performance seismic resisting precast/prestressed systems based on jointed ductile connections has been recently given by Pampanin [2005].

In this contribution, an overview of results obtained as part of an extensive experimental research campaign on going at the University of Canterbury for the refinement and further development of alternative arrangements for hybrid precast/prestressed building systems will be provided. Quasi-static cyclic tests on a series of 2/3 scaled exterior 2-D and 3-D beam-column subassemblies under uni- and bi-directional loading regime are discussed. And briefly compared with the response and damage observed in benchmark specimens representing equivalent monolithic solutions designed according to the NZ3101:1995 code provisions. In addition, the practical feasibility and efficiency of simple technical solutions to connect floor systems, e.g. precast hollowcore units, and lateral resisting systems, e.g. frames, without incurring in damage due to displacement incompatibilities are experimentally demonstrated. The experimental tests of an articulated floor-lateral system solution floor solution

2. RESPONSE OF TRADITIONAL PRESSS-TYPE HYBRID SYSTEM

A first series of tests was carried out to reproduce the basic configuration of a hybrid PRESSS system (modular specimen type 1) as originally proposed by Stanton et al. [1997]. The specimen comprised a) straight profile longitudinal tendons; b) internal mild steel bars as dissipation devices c) friction at the critical beam-column section as the shear transfer mechanism. At the rocking interface, steel plates were embedded on one side of the column and one edge of the beam in order to allow a detailed investigation of the effects of alternative contact surfaces: concrete-concrete, steel-concrete, steel-steel with and without a (fibre reinforced) grout pad to accommodate the construction tolerances.

The typical set-up and imposed displacement regime of the beam-column joint subassemblies under uni-directional testing protocol are shown in Figure 3. Beam and column elements are extended between points of contra flexure, assumed to be at mid-span of the beams and at mid-height of the columns, where pins are introduced. Simple supports at the beam ends were provided by connecting pin-end steel members to the floor.

Quasi-static cyclic tests were carried out under increasing levels of lateral top displacement. The testing protocol complied with the “acceptance criteria” proposed in ACI T1.1-01 & ACI T1.1R-01 2001 and consisted of a series of three cycles of drift, followed by a smaller single cycle.

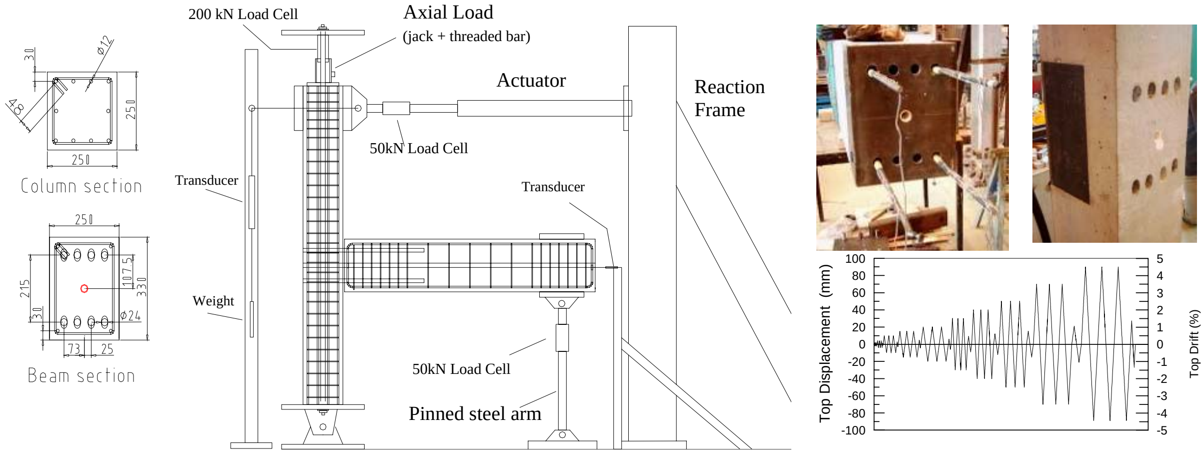

Figure 3: General uni-directional test set-up, loading protocol and construction details of traditional hybrid solution (with internal grouted mild steel).

Tests were first carried out on unbonded post-tensioned solutions only, varying the level of initial post-tensioning as well as the contact surface at the beam-column interface. In general the behaviour of the different arrangements was satisfactory, with a stable non linear elastic hysteresis without remarkable losses of stiffness at any reloading stage, except for the solution using a 50mm interface grout pad (reinforced with a fiber mesh), which tended to become the most vulnerable element of the connection.

A seven wire strand (A^{pt} = 99mm^2) was used with an initial post-tensioning at 60% of the ultimate stress f_{ptu} (1860MPa), thus equal to an initial post-tensioning force of approximately 110kN. In the hybrid system, additional strength and energy dissipation capacity was given provided by four longitudinal mild steel reinforcing bars (10mm diameter), inserted in embedded metallic corrugated ducts and successively grouted (Figure 4). In order to prevent premature fracturing of the steel, a small unbonded length of 60mm was adopted by wrapping plastic tape around the bars in the proximity of the critical section.

Both the unbonded post-tensioned only and the hybrid system configuration showed very stable hysteresis loops (Non Linear Elastic or Flag-Shape, respectively) with full-recentering capability and no evident damage in the structural members. In the hybrid system, additional strength and dissipation was provided by the internal mild steel. The onset of stiffness degradation due to the bond deterioration between the longitudinal mild steel bars and the injected grout became more evident at the first cycle at 4.5% of drift.

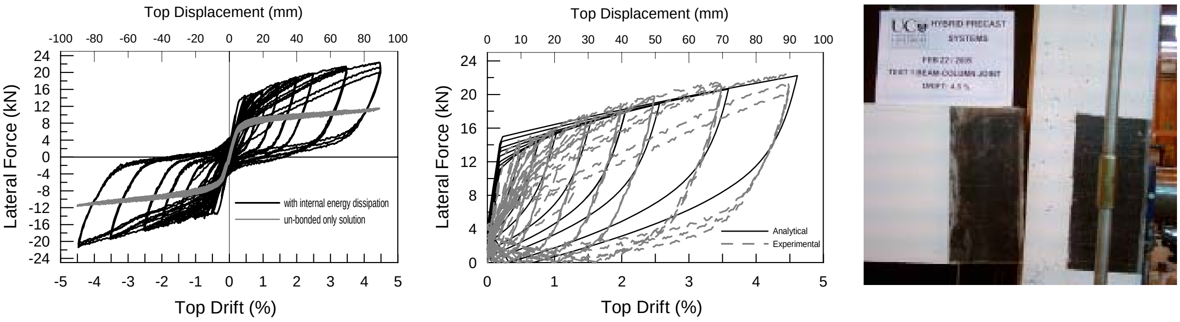

Figure 4: "Traditional" PRESSS-hybrid solutions a) Hysteresis response with and without internal dissipaters b) Analytical-experimental validation using lumped plasticity model c) gap opening at 4.5% drift (steel plate-to-concrete contact at the interface).

2.1 Simplified Modelling approach

It is worth noting that simple analytical model presented in literature and recently implemented in design code provisions [NZS3101:2006] can successfully support the design phase, 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 two rotational springs in parallel as proposed by Pampanin et al,.[2001] and Palermo et al. [2005] is shown in Figure 4. (centre). The moment rotation contribution for the unbonded post tensioned tendon was modelled using a Non Linear Elastic hysteresis rule while a Ramberg-Osgood hysteresis rule was adopted to model the moment-rotation contribution of the dissipaters. It can be noted that the model is, in general, able to satisfactory reproduce the experimental results either in terms of monotonic and cyclic behaviour, while still not fully capturing stiffness degradation effects due to bond deterioration.

2.2 Comparative behaviour of an 2-D equivalent monolithic solution

A further confirmation of the higher performance of jointed ductile connections when compared to an equivalent monolithic solution, provided by the experimental hysteresis loop and observed damage (Figure 5) of a 2-D exterior beam-column joint reinforced concrete specimen, designed according to NZS3101: 1995 to have approximately the same flexural capacity of the hybrid solution. As expected by the adoption of capacity design considerations targeting the development of a weak beam strong column mechanism, the damage is concentrated in the plastic hinge region leading to progressing flexural cracking and spalling of the concrete. In general, a very stable hysteresis loop with higher energy dissipation capacity, when compared with a flag-shape system and limited stiffness degradation or pinching effects due to bond deterioration was observed. In spite of this very satisfactory behaviour when referring to traditional systems, the differences in terms of level of damage and residual deformations between the monolithic and jointed ductile system are evident.

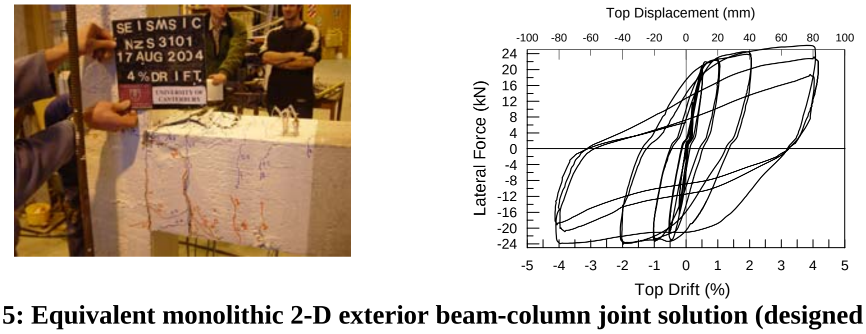

Figure 5: Equivalent monolithic 2-D exterior beam-column joint solution (designed according NZS3101:1995): observed damage at 4% drift and hysteresis response.

3. USE OF DRAPED TENDON PROFILE AND SHEAR KEYS

Based on similar concepts, a peculiar connection solution and construction system, named the "Brooklyn" system, has been studied and developed in Italy for gravity-load-dominated frame buildings with the intent of da. combining the structural concept and efficiency of cable-stayed or suspended bridges within a typical multi-storey building system [Pagani, 2001; Pampanin et al., 2004].

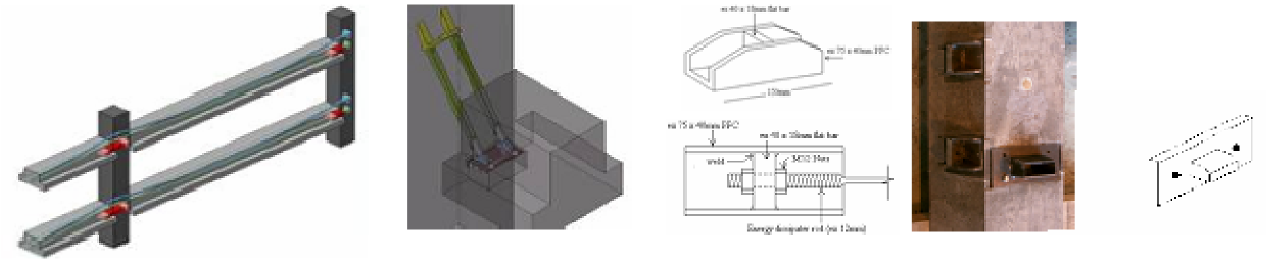

Figure 6: a, b) Use of draped tendon and cable-stayed hidden steel bracket (Brooklyn system, proprietary of B.S. Italia, Italy, c) Details of arrangement for external dissipators and simplified shear key/corbel. [Pampanin et al., 2004; Pampanin, 2005].

Key peculiarities of the system are:

a) the use of a draped tendon profile ("suspended" solution, Figure 6 left side) anchored at the exterior columns of the frame in order to supply an adequate moment resistance at the critical sections under combined gravity and low-to-moderate lateral loads;

b) the use of alternative solutions for steel shear bracket/corbel (Figure 6 right side), able to fully counteract the shear force transmitted at the beam-column interface. In this way the prestressing tendons have only to balance flexural stresses and a large floor slab span (e.g. 10 x 12 m grid) can be achieved. Undesirable consequences related to the yielding or failure of the tendons, or in general, due to the loss of the shear friction transfer mechanism, are thus overcome, in line with recent requirements in code provisions (e.g. NZS3101:2006). Also, by "hiding" the corbel in the depth of the beam, architectural and aesthetic requirements (in addition to fire resistance) can be met.

An overview of the conceptual definition, development and experimental validation (under either gravity or seismic loads only) of the Brooklyn system solution, including a description of practical applications on a series of buildings in regions of low-moderate seismicity can be found in [Pampanin et al., 2004, 2006]. In the following paragraph, the evolution towards a PRESSS-type solution for high-seismicity regions is described.

3.1 Implementation of a double hinge shear key solution

Given the previous considerations, the concept of a "double hinge" shear key was developed and successfully implemented with the intent to provide adequate bilateral (i.e. either upwards or downwards direction) shear transfer mechanism at the critical section.

As shown in Figure 7, two shear key "hinges" consisting of two half cylinders with convex and concave edge were respectively welded to a steel plate at the end of the beam and column faces at the level of the external dissipation devices. As a result, the controlled rocking motion occurs about two pivot points and is thus significantly simplified. This merged "PRESSS-Brooklyn" solution, comprising of draped ("parabolic") tendon profile, a hidden corbel for gravity loads only with an additional double hinge solution for the seismic shear keys was tested either in an unbonded-post-tensioned-only configuration or in an hybrid configuration with the addition of external dissipators. Different levels of initial prestressing were also considered.

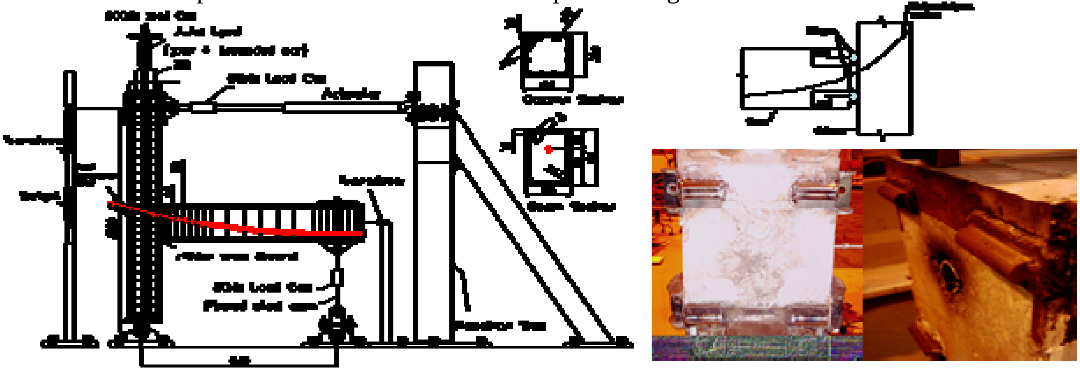

Figure 7: Concept and implementation of a double hinge shear key solution: test set-up, front view (column face) and lateral view (beam face).

Figures 8 and 9 report some experimental results in terms of the global force-displacement hysteresis behaviour, the variation of the tendon force versus the drift level as well as the axial stress-strain or force-displacement behaviour of the dissipators. In all cases, no damage occurred up to design drift in the beam or column structural elements, as typical peculiar characteristic of a well-designed ductile jointed connection. Stable hysteresis loops with non-linear elastic or flag-shape behaviour, were observed with full re-centring capability and alternative level of dissipation as designed. No evident loss of stiffness occurred thanks to the protection of the concrete edge corners. The observed asymmetric behaviour in terms of strength was due, as expected, to the non-central position of the cable within the section. Moreover, the different level of displacement achieved was in general not due to material failure: the experimental tests were in fact typically interrupted at a level corresponding to levels of stress in the tendons conservatively below the yielding value, in line with the aforementioned special provisions given by the Appendix B of the NZS3101:2006. In the hybrid solution (figure 9), two types of external dissipators were adopted, consisting of 7mm or 6mm diameter Grade 300 steel fuses with an unbonded length of 150mm and 120mm respectively, grouted into a steel cylinder acting as anti-buckling restrainers.

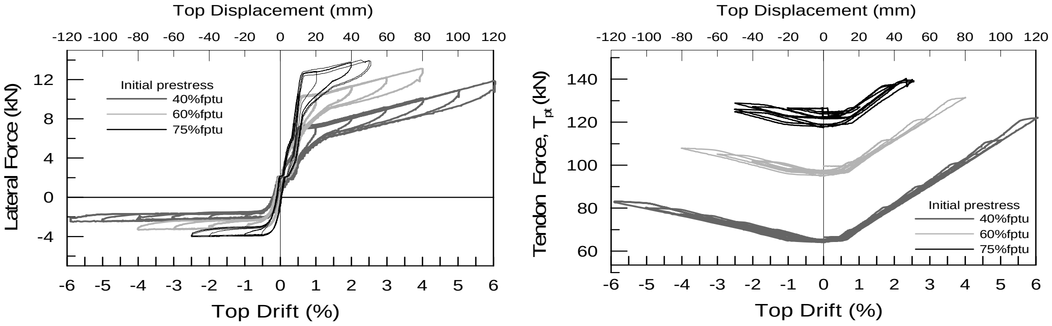

Figure 8: Unbonded post-tensioned solutions: global force-displacement and variation of tendon post-tensioning force.

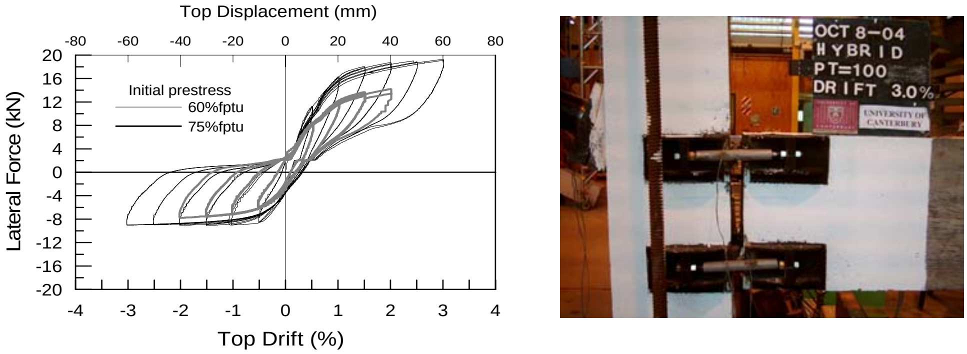

Figure 9: Hybrid solution with draped tendon profile: flag-shape hysteresis loop varying the level of prestress and gap opening at 3.0% drift.

4. USE OF EXTERNAL REPLACEABLE DISSIPERS AND EFFECTS OF BI-DIRECTIONAL CYCLING LOADING

Previous tests described in literature on jointed ductile precast hybrid systems have typically referred to 2-D beam-column subassemblies belonging to plane frame systems. As part of the experimental research investigation herein reported, a 3-D exterior (corner) beam-column joint subassembly, part of a space frame, was prepared with a modular configuration, such that several alternative arrangements of hybrid systems could be tested, after replacing the dissipating devices.

A flexible face plate, acting as a sort of “mask”, was located at both the beam and column faces with different possible locations of the mechanical hinges acting as shear key solutions. As shown in Figure 10, five different positions of hinges and six different locations of the unbonded tendon profiles could be tested. The location of the dissipators could also be either within the beam rectangular lateral profile (thus “hidden” for architectural requirements) or external to it. The 3D specimen was subject to a combined bi-directional “four cloves” loading protocol shown in Figure 11 and the results compared with the uni-directional response, in order to investigate the effects, if any, of bidirectional demand on the response in terms of the hysteretic behaviour (loss of strength and stiffness) as well as, more generally, of the observed damage.

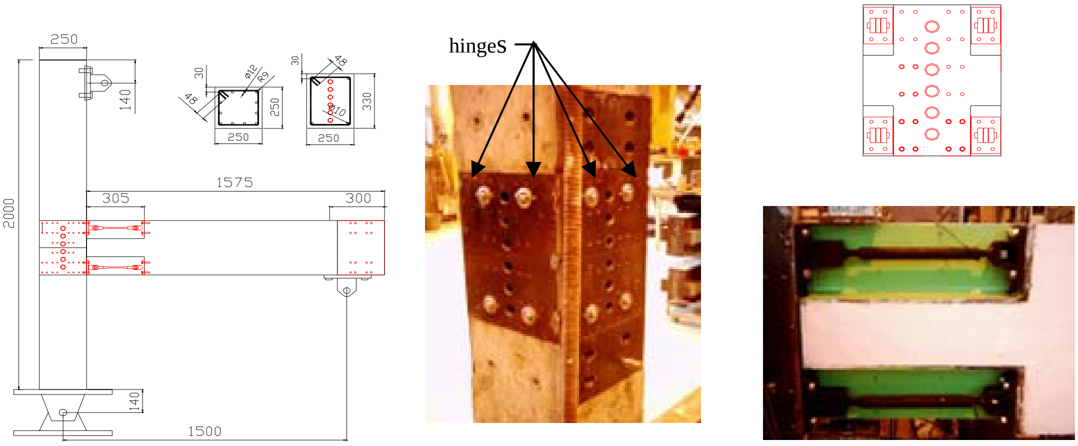

Figure 10: 3-D modular configuration of the Hybrid beam-column joint: Location of the “double hinge” and dissipators.

Three cycles per combined drift level, plus one smaller amplitude cycle, were undertaken in each quadrant, with a similar conceptual protocol to that adopted for the uni-directional testing [ACI T1.1-01 & ACI T1.1R-01 2001]. As a result, it is worth noting that the specimen is actually subjected to a more demanding protocol, with a cumulative number of six cycles in each direction per drift level, instead of the three cycles in the uni-directional testing protocol.

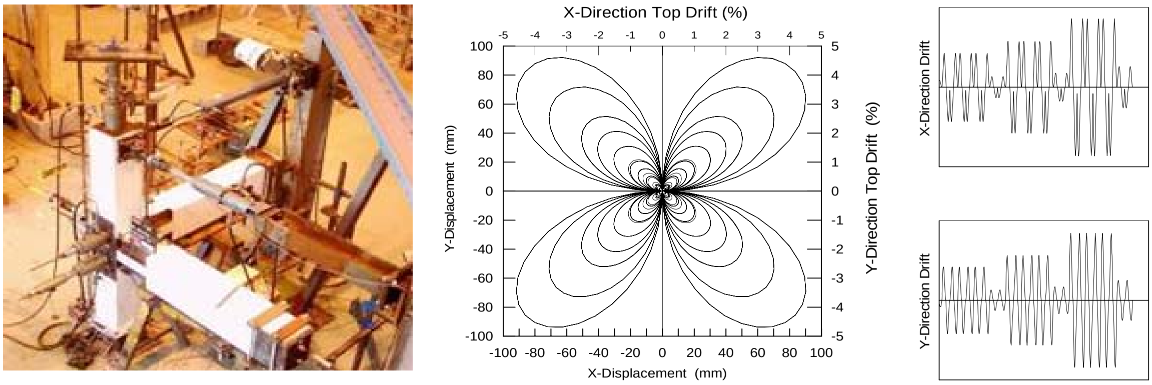

Figure 11: Test set-up and “four clove” bi-directional displacement regimes.

4.1 Response of Unbonded Post-Tensioned-only Solution

In the unbonded post-tensioned only solution, the initial prestress forces were designed in order to obtain a similar target moment capacity in both directions at 4.5% drift. The different location of the tendons in the two beams (i.e. alternated in order to avoid clashing in the column region, see Figure 10), had to be accounted for. As a result, the initial post-tensioning forces were 15% (27.5kN) and 27% (49.5kN) of ultimate stress f_{ptu} (1860MPa) in the X and Y directions respectively. In this 3-D configuration, the double hinge shear key, consisted of small metallic spheres (Figure 10 centre).

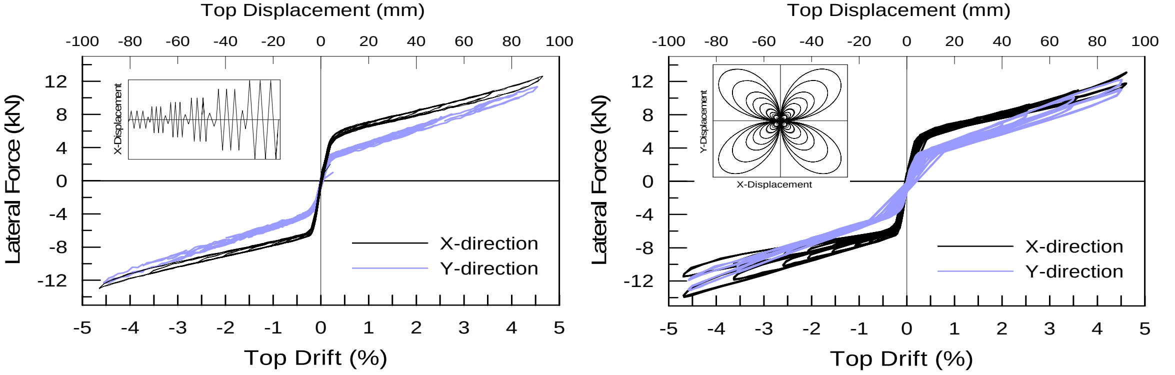

Figure 12: Force-displacement response of 3-D post-tensioned only solution under independent uni-directional (left) or combined “four cloves” (right) testing regime.

As shown in Figure 12, the response of the system was extremely satisfactory in both directions. The effects of bi-axial cyclic loading were almost negligible when comparing the response to that of the same specimen under independent uni-directional loading. An increasing level of damage or reduction of strength/stiffness was not observed, as would be expected in a monolithic configuration. Nevertheless, the onset of beam torsion due to minor constraints in the test set-up (movement of the beam pinned arm in the out-of plane direction) occurred at a high drift level (4.5%) leading to minor losses of prestress in the tendons.

4.2 Response of Hybrid Solution with External Dissipators

The hybrid solutions were obtained by adding external dissipators, with the clear aim of demonstrating the flexibility of the design and the possibility of having a reliable control of the flag-shape behaviour. The same moment capacity at target drift (4.5%) and similar energy dissipation were thus aimed for. Given the same tendon layout in the two directions as in the post-tensioned only solution (Figure 11), initial prestress in the X and Y-directions were respectively, 25%fptu (i.e. 46kN) and 27%fptu (49.7kN).

Four external dissipators consisting of either 7 or 8 mm diameter fuses (with 150 mm unbonded length) were installed, in the X and Y directions respectively, and inserted (“hidden”) in existing slots on both sides of the beam (Figure 10). The experimental response under uni-directional testing, (i.e. X and Y-direction independently, showed an extremely efficient and stable hysteresis loop (Figure 13). Valuable confirmations of the reliability of a flexible design approach were obtained, where dissipators, post-tension location and levels can be varied while maintaining the desired level of moment capacity and overall dissipation/recentring properties. The presence of the double-hinge shear key solutions (small metallic balls, Figure 10) guaranteed two fixed pivot points, with no stiffness or strength losses up to a high level of drift (4.5%).

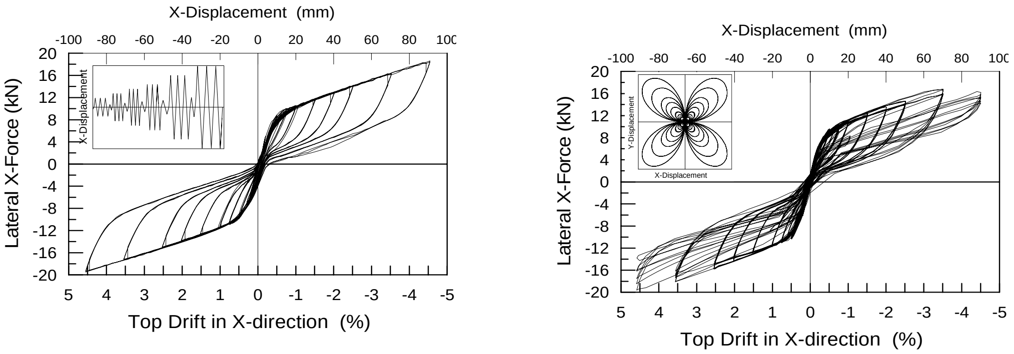

Figure 13. Force-displacement response of 3-D hybrid solution under uni-directional (left) or combined four cloves (right) testing regime.

The response of the hybrid system under the bi-directional (four clove) testing regime was very satisfactory up to 3.5% of drift. Up to this stage, the effects of bi-axial loading seemed to be negligible, when compared with the uni-directional response. At higher level of drifts, however, the torsion effects on the beam, observed during the tests on the post-tensioned solution and mainly due to the test set-up constraints, led to losses of prestress in the tendon as well as to general stiffness degradation. The subsequent increased level of strain demand in the dissipators, combined with the aforementioned highly demanding testing protocol, led to the premature fracture of dissipators when moving to 4.5% drift in the X-direction.

4.3 Comparison with the response of a 3-D equivalent monolithic benchmark

The high-performance of the hybrid system when subjected to bi-directional loading regime can be more properly appreciated when comparing the response to that of an equivalent monolithic 3-D exterior (corner) beam-column joint, designed according to the NZS3101:1995 and representing a spare frame version of the 2-D benchmark specimen described in par. 2.2). Figure 14 shows the observed level of damage at 2.5% drift (left) and 4.5% (centre) and the hysteresis response (in the X-direction). It is interesting to note that, in spite of the adoption of code-design provisions based on capacity design considerations, the bi-directional loading regime proved to be very demanding for the joint region. Extensive cracking of the joint panel zone developed at 2.5% drift level, progressively leading to crushing and spalling of the whole cover concrete in the joint panel zone region at 4.5% drift. Due to the bond deterioration under the combined bi-directional loading, flexural cracking in the beam concentrated at the interface with the column through opening of a single crack, instead of developing within a traditional plastic hinge region. As a result, a marked pinching behaviour was observed in the hysteresis response at earlier level of drift, in addition to increasing level of stiffness and strength degradation, also underlining a significant reduction in performance when compared to the 2-D equivalent monolithic benchmark specimen. The superior performance of the hybrid solution, which showed (Figure 13) a remarkably consistent satisfactory behaviour, regardless of the loading regime, is again evident.

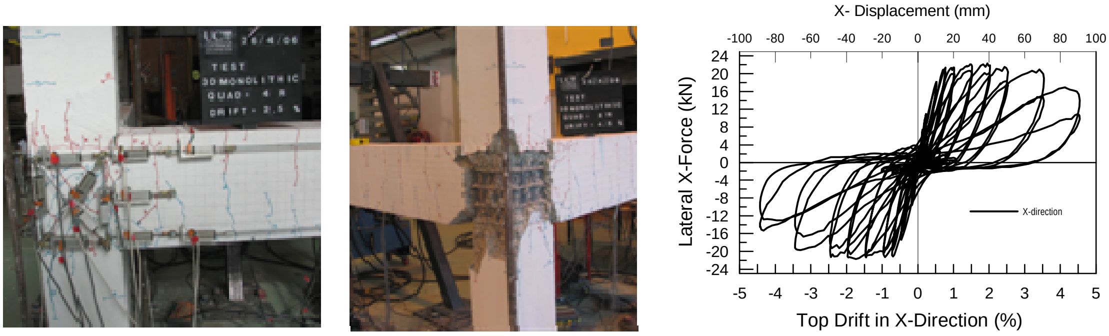

Figure 14: Equivalent monolithic 3-D exterior beam-column joint solution (Designed according to NZS3101:1995): observed damage at 2.5% drift (left), 4.5% drift (centre) and hysteresis loop (right).

5. IMPLEMENTATION AND TEST OF AN ARTICULATED (JOINTED) FLOOR CONNECTION

The peculiarity of a jointed ductile connection, consisting of an articulated assembly of precast elements can be exploited and extended when designing solution for floor-to-lateral load resisting system connections. Recent experimental results on the 3-dimensional performance of precast super-assemblages including frames and hollowcore testing units (Matthews, 2003), have further underlined issues related to the inherent displacement incompatibility between precast floor and lateral resisting system, including beam-elongation effects, although not being limited to precast concrete solution. Appropriate design criteria and detailed technical solution should thus be adopted Figures 15 and 16 show the overall view, conceptual solution and experimental response of a floor-frame connection solution capable of accommodating the displacement incompatibility between floor and frame by creating an articulated or jointed mechanism effectively decoupled in the two directions. According to the proposed solution, the hollowcore unit is in fact connected to the lateral beams by shear mechanical connectors acting as shear keys when the floor (relatively) moves in the direction orthogonal to the beam and as sliders when the floor moves in the direction parallel to the beam. So doing, beam elongation effects causing damage in the floor system due to the gap opening mechanism can be avoided. Also, due to the low flexural stiffness of the shear keys-connectors in the 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. As a result, no differences can be noted in the response of the 3-D beam-column joints subassemblies, due to over-strength or, in general, interaction, when compared to the response of the bare beam-column joint without floor (Figure 13).

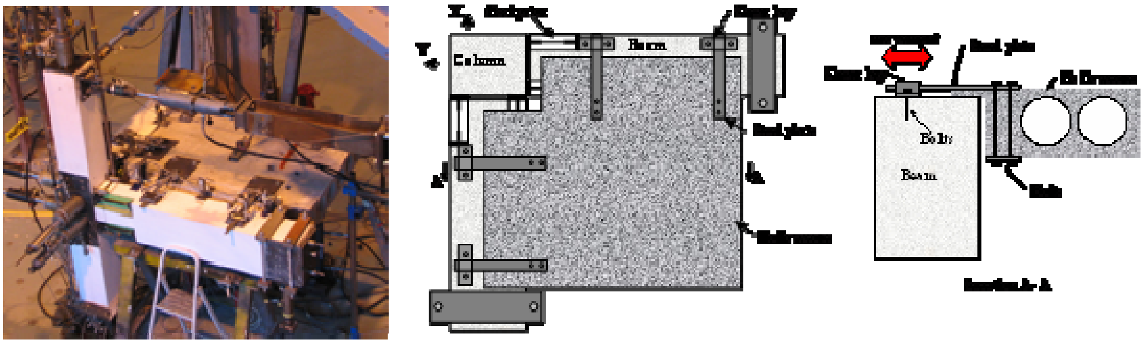

Figure 15: 3-D beam-column joint with articulated floor unit. Overall view, concept and connection details

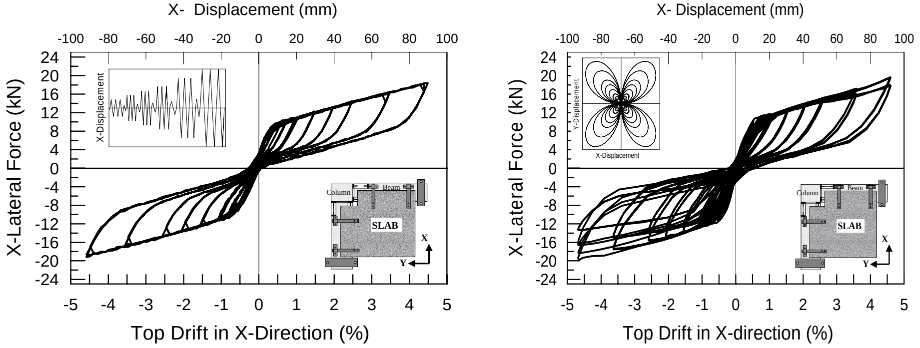

Figure 16 :3-D beam-column joint with articulated (jointed) floor solution. Response in X-direction due to uni- diretional nd bidirectional testing regime.

CONCLUSIONS

The implementation and experimental validation of several arrangements for precast jointed ductile connections, relying on unbonded post-tensioned techniques have been presented. Alternative configurations could be obtained by varying the longitudinal profile of the tendons, the type and location of the energy dissipation devices as well the shear transfer mechanism at the rocking critical section. Quasi static tests on a series of exterior beam-column joint subassemblies have been carried under either uni- or bi-directional loading regime and critically discussed. In general, very satisfactory performance of the several alternative configurations was observed, particularly when compared to equivalent monolithic solutions. The efficiency of a simple and practical floor-to-frame system connection, able to accommodate displacement compatibility due to the effects of the beam elongation (gap opening) has also been tested within a 3-D corner joint subassembly subjected to bi-directional testing regime.

6. ACKNOWLEDGEMENTS

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