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EDITORIAL

HYBRID POST-TENSIONED ROCKING (HPR) FRAME BUILDINGS: LOW-DAMAGE VS. LOW-LOSS PARADOX

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The 2010-11 Canterbury Earthquake Sequence inflicted seismic losses worth more than $40B, which is about 25% of the GDP of New Zealand (as per 2011 data). More than 80% of these losses were insured, which comprised of more than $10B covered by the *Earthquake Commission* (a New Zealand crown entity providing insurance to residential property owners) and in excess of $22B by private insurers (comprising of a roughly equal split between domestic and commercial claims) [1]. The scale of financial impact has been perceived to be disproportionately large considering the building regulatory regime in New Zealand is relatively stringent, and the earthquakes and aftershocks were of moderate magnitude. As it is well known that some of the major faults spread in the Wellington region and the subduction boundary passing through the centre of New Zealand can generate much bigger earthquakes (upwards of magnitude 8), people are left pondering whether New Zealand will be able to cope with the financial impact of larger earthquakes. This fearful realisation gradually led to people being dissatisfied with merely life-safe buildings, and to demand more resilient buildings that meet the objectives of performance-based design, i.e. suffer less damage, incur less loss, and remain functional after earthquakes.

In light of the extensive building damage resulting in high financial loss in recent earthquakes, practicing engineers and researchers in New Zealand have been advocating a revision of the current design approach to improve the performance of new structures in future earthquakes [2-5]. As a result, large proportion of buildings constructed in the last decade (including those built to replace earthquake-damaged buildings) have shied away from the traditional damage-friendly ductile structural systems and, instead, adopted one of the new and emerging structural systems claimed to be "low-damage". In many cases, the adopted structural systems are not covered by existing design standards and are approved as alternate solutions through expert peer-review. The "low-damage" attribute of most structural systems has been validated by component (or sub-assembly) level experimental tests, but their interactions with other building components and implications of their use in buildings have not been rigorously scrutinised. Hence, the rushed adoption of some of these systems in buildings could surprise the engineering community in future earthquakes with mismatch between the expected and real performances of the buildings; akin to what New Zealand engineering fraternity is currently going through due to realisation of poor seismic performance of precast hollow-core flooring system that has been widely used in New Zealand buildings without rigorous scrutiny.

One such "low-damage" structural system is precast post-tensioned rocking frames with supplemental energy dissipaters. This paper summarises the development of this structural system, critically reviews the literature reporting the seismic performance of this system, and qualitatively evaluates system-level implications of its use in buildings. The aim of this paper is to better inform engineers about the likely seismic performance of buildings employing this structural system, and to enable them to optimise its benefits by giving due consideration to its effect on other building components.

EVOLUTION OF HYBRID POST-TENSIONED ROCKING FRAMES AS STRUCTURAL SYSTEMS

Realisation of Rocking as a Viable Lateral Load Resisting Mechanism

The prospect of a rigid body rocking mechanism potentially leading to satisfactory earthquake response was first reported in the 1960s by Housner [6]. This was followed by a few more studies exploring the use of rocking between precast members to accommodate seismic demand on building frames [7]. Precast jointed and rocking frames accommodate large drifts by permitting beam-column connections to open, instead of forming traditional energy-dissipating plastic hinges. As a result, they have markedly lesser inherent energy dissipation than ductile monolithic systems [8,9]. It was, therefore, deemed desirable to provide supplemental energy dissipaters to these connections, in order to reduce their deformations under earthquake ground motions.

The first significant study investigating the cyclic behaviour of a precast concrete beam-column subassembly, with a combination of mild steel bars (for energy dissipation) and posttensioning (PT) tendons (for strength) was conducted by Stone et al. [10] and Stanton et al. [11]. These studies were arguably the first to use the term "hybrid" to identify such connections. Soon after, the superior earthquake response of post-tensioned concrete bridge piers designed to rock in the pier-foundation interface was validated by Mander and Cheng [12]. They named the concept *Damage Avoidance Design*, or DAD in short. This was soon followed by a seminal experimental study by Priestley et al [13] on a large-scale 3D precast concrete frame with PT tendons and mild steel bars. The system was named *Precast Seismic Structural System*; or PRESSS in short. In the literature, different names (e.g. hybrid, post-tensioned, DAD, PRESSS) have been interchangeably used for post-tensioned rocking frames with supplemental energy dissipaters. For consistency and brevity, they are henceforth referred as hybrid post-tensioned rocking (HPR in short) systems.

Key Features of HPR Frames

Precast concrete buildings (with or without posttensioning) are traditionally constructed with "wet" connections using on-site concreting (or grouting) to connect the precast members.

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Although modular precast buildings with “dry” steel connections have been recently proposed [14-16] to facilitate rapid construction and deconstruction, as of now, HPR connections seem to be the preferred connection type for new precast concrete buildings (at least in New Zealand). The combination of a bilinear elastic response of the PT tendons, and a bilinear elasto-plastic behaviour of the energy dissipaters, results in a flag-shaped hysteretic behaviour of HPR connections, as schematically illustrated in Figure 1. By carefully proportioning the yield strength of the energy dissipation system in relation to the elastic limit of the PT tendons, HPR frames can be designed to self-centre when any static and/or dynamic loads acting on them are released. This ensures that precast frame buildings made of HPR connections return to zero residual drift after severe earthquake shaking. This “self-centring” characteristic of such structures, which has been argued to render them functional after a major earthquake, is a significant improvement over traditional ductile moment resisting frames that often sustain residual tilts after severe shaking and require straightening before they can be reused.

HPR structural systems are designed to accommodate large drifts by rocking at specially detailed joints, and have been proven to provide a level of seismic resistance comparable to current standards while remaining almost damage-free. In the last three decades, the superior cyclic/seismic behaviour of HPR connections have been extensively validated at component and sub-assembly levels. The lessons learnt from these studies have led to the gradual improvement of their design and detailing for better performance.

Gradual Improvement of the Design and Performance of HPR Systems

The first series of tests on HPR connections in the 1990s [8-11,13] were conducted with deformed mild steel bars enclosed inside ducts running across the connection (the gaps were filled by grouting). These bars were intended to dissipate energy through tension-compression yielding (TCY). Although the hysteretic energy dissipation of the tested connections was satisfactory, degradation of stiffness and strength occurred which was attributed to deterioration of the bond between the grout and steel. To avoid this issue, Bradley et al. [17] externally bolted the mild steel TCY bars across the joint region, which resulted in a stable hysteric response with negligible stiffness or strength degradation. Nevertheless, it was found that that the TCY bars were prone to buckling, which reduced their axial load and energy dissipation capacities. Amaris et al. [18] restrained the externally mounted mild steel bars against buckling, which resulted in slightly improved performance.

In some of the above tests, however, the mild steel bars fractured due to low-cycle fatigue, exposing an inherent weakness in any yielding-based steel energy dissipation approach. The low-cycle fatigue damage and residual stress accumulation incurred by these yielding devices would require them to be replaced after each moderate earthquake. In addition, the bars were found to be inefficient in dissipating energy as they would offer minimal or no energy dissipation during smaller cycles which form the bulk of an earthquake input.

A cost-effective and efficient energy dissipation device was developed and used by Rodgers et al. [19], in order to overcome the abovementioned weaknesses. This simple device, named high force-to-volume (HF2V) lead-extrusion damper, consists of a central shaft with a bulge, encased in lead. The movement of the shaft forces the lead to move across the bulge, which enables it to sustain a constant force upon yielding. Unlike mild steel yielding bars, these devices do not need replacement after earthquakes, and can be easily embedded/hidden inside the beams, as shown by Rodgers et al. [20].

In addition to the gradual movement towards more efficient supplemental energy dissipation system, the quest to enhance the performance of HPR frames also led to improved connection detailing. Noteworthy among the improved features are the use of high-strength concrete for the filler pour to reduce cracking, steel-armouring the rocking surfaces to avoid localised crushing damage in the beam edges/corners, smoothing the PT cable duct profile to avoid friction, and efficient profiling of shear keys to resist torsion and enhance construction speed [21,22].

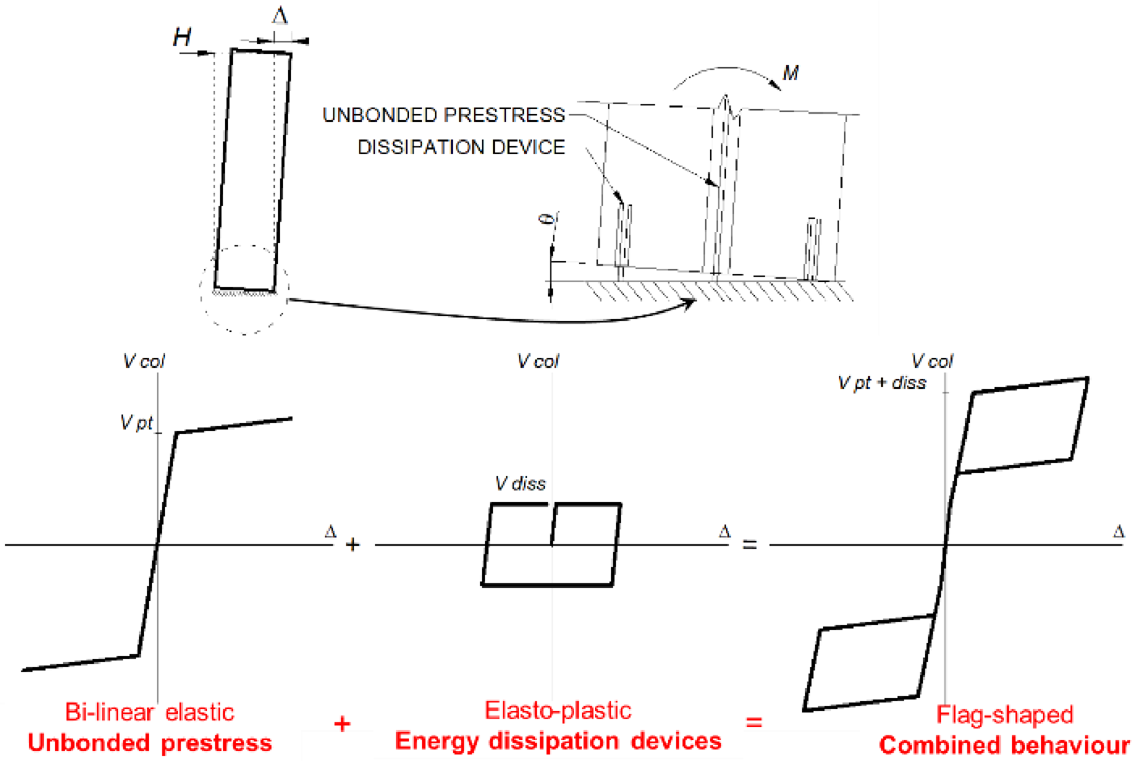

Figure 1: Components of a post-tensioned hybrid connection and their contributions to the system hysteretic response.

Wider Application of HPR Connections

In addition to frames, HPR systems have also been implemented in bridge piers to achieve low-damage seismic performance and accelerated construction [12,23-24]. With wider recognition of the advantages of HPR connections, researchers began exploring its use in other structural systems (e.g. RC rocking walls [25]). The concept of assembling prefabricated members via post-tensioning and using supplemental devices to dissipate seismic energy has also been extended to timber structures [26,27].

Research Gaps and Adoption Barriers

A number of experimental studies have been conducted to develop and refine this structural system, and research to better understand and improve various aspects of rocking systems is currently underway [28,29]. Nevertheless, one glaring omission in the literature is the consideration of the interaction between HPR frames/walls and other building components. In the author's observation, almost all the experimental studies reported on this system, except Priestley et al [13], have been conducted on HPR frames without any direct/indirect consideration of the effects of floor slabs. Even the few tests involving 3D frame subassemblies reported in the literature [17,21] have been conducted on specimens without slabs. In the PRESSS system test [13], possibly the only test on multi-storey HPR frames including floor slabs, minor to moderate damage to connections between the precast floors and the HPR frame was reported. Hence, despite a large number of experimental tests proving that such HPR frames incur distinctly lesser damage than their monolithic and ductile counterparts, lack of evidence of their satisfactory (if not superior) interaction with other primary/secondary building components has left a big question mark in terms of this system's effectiveness in buildings. This has been identified as one of the key reasons behind the relatively slow practical adoption of this structural system in precast concrete buildings [30].

COMPARISON OF THE SEISMIC RESPONSE OF HPR FRAMES TO OTHER STRUCTURAL SYSTEMS

Most of the studies on HPR systems have conducted quasi-static tests on connections and subassemblies to validate the ability of the hybrid connections to accommodate large drifts without undergoing significant damage. Such studies do not shed much light on the likely response of buildings with HPR frames to ground motions of different intensities. While dynamic tests on 3D multi-storey HPR frames (which are scarce [13]) can help quantify the response of such building systems to different levels of ground motions, they cannot be put in perspective without comparison to the response of traditional monolithic ductile frames under similar levels of shakings. For this purpose, numerical studies investigating the dynamic response of different structural systems (including the HPR system) are sourced from the literature [9,29,31].

Inter-Storey Drifts

Although HPR frame buildings experience significantly lower residual drifts, they have been found to incur larger maximum drifts compared to traditional monolithic moment resisting frames [9,31]. This result, however, is expected since HPR connections are typically more flexible than monolithic connections.

Priestley and Tao [9] conducted a numerical study comparing the seismic responses of bilinear plastic hysteretic models (typical of monolithic ductile frames) to flag-shaped bilinear degrading models (typical of HPR frames). This study showed that the bilinear degrading hysteretic response consistently led to higher drift demands than the bilinear plastic model under multiple earthquake records. Table 1 lists the ductility demands computed in their study [9], for structures with four different hysteretic characteristics, subjected to a number of earthquake records scaled to 0.4g peak ground acceleration. The bilinear degrading model can be seen to consistently experience greater ductility demands than the bilinear elasto-plastic (and linear) model, but the difference reduces as the structural period increases. The table indicates that the drift/ductility demands experienced by low-to-medium rise HPR buildings (represented by low periods) could easily exceed the demands of their ductile monolithic counterparts by more than 50%.

Another piece of valuable information that can be deduced from Table 1 is the relative difference between the demands of bilinear elastic system and the bilinear degrading (i.e. flag-shaped) system. These two hysteretic models typically represent the post-tensioned rocking frames without and with the supplemental energy dissipaters, respectively. Although the addition of energy dissipaters is intended to reduce the drift demands on the post-tensioned frames, the results in Table 1 indicate that this is not true for many cases. For several combinations of structural period and ground motions, Priestley and Tao [9] found that the maximum ductility is higher for the bilinear elastic system than the flag-shaped system.

Floor Accelerations

Information on the floor acceleration response of HPR frames could not be found in published literature. No dynamic (i.e. shaking table) tests of multi-storey HPR frames have been published, to the best of the author's knowledge. Moreover, the few numerical studies available in literature, that conduct time-history analyses of HPR frames [9,29,31], have focussed only on comparing their peak and residual drifts with those of conventional frames, without examining their acceleration responses. Hence, only a qualitative comparison of peak floor acceleration responses of HPR vs traditional monolithic frames is made here based on indirect evidence and inferences.

Basic principles of statics suggest that softer systems require lesser force to produce the same deformation as stiffer systems. Since base shear forces are assumed to be proportional to spectral acceleration in the equivalent static force method of seismic design, the HPR frames, being softer than monolithic frames, can be argued to experience smaller peak floor accelerations. The same argument can also be made by referring to the elastic response spectrum, which gives lesser spectral acceleration for more flexible structures with longer natural periods (especially, beyond the constant acceleration region of the response spectrum).

Nevertheless, the theory of structural dynamics implies that the acceleration response of a structure to a ground motion depends on the dynamic properties of the structure and characteristics of the motion. Being more specific, peak acceleration response of a structure depends mainly on whether a modal frequency of the structure is within or very close to the dominant frequency range of the ground shaking. The effect of higher order modes, although not significant in terms of drifts, can be significant in terms of acceleration and shear forces. Hence, it is prudent not to generalize the acceleration responses based on theory of statics only.

In the only experimental study conducted on a multi-storey 3D HPR frame [13] (to the best of the author's knowledge), the acceleration responses of different floors were not measured and reported. This is understandable as the tests conducted were pseudo-dynamic (as opposed to shaking table). However, the lateral forces at different floors of the tested multi-storey HPR frame were reported to have exceeded the anticipated levels. Analytical studies confirmed that the storey forces were significantly higher than those considered in design, which was

Table 1: Displacement ductility demands for different force-deformation characteristics (Priestley and Tao [9]).

Period	**Model ⬇ / Earthquake Record ➔**	**El Centro 1940 NS***	**Taft 1952***	**Corroletis 1989***	**James Rd 1979***	**Orion Blvd. 1971***	**Hachinohe 1968***	**Average**
T = 0.4 s	Linear+	5.5	6.1	5.4	7.7	8.1	8.0	6.8
	Bilinear Elastic	19.2	12.7	19.1	11.4	16.4	26.2	17.5
	Bilinear Plastic	7.8	7.1	7.8	5.3	12.4	10.0	8.4
	Bilinear Degraded	16.7	14.3	13.1	9.4	20.0	20.6	15.7
T = 0.8 s	Linear+	7.9	7.8	13.7	4.6	8.6	8.5	8.5
	Bilinear Elastic	7.5	4.3	10.4	8.3	19.3	16.7	11.1
	Bilinear Plastic	5.8	5.6	6.3	5.2	10.1	8.2	6.9
	Bilinear Degraded	10.2	5.6	7.1	8.1	20.0	15.8	11.1
T = 1.2 s	Linear+	6.1	2.6	5.7	4.4	9.6	6.6	5.8
	Bilinear Elastic	4.9	4.6	4.7	7.9	10.9	15.4	8.1
	Bilinear Plastic	5.5	2.7	5.8	4.3	8.1	6.9	5.6
	Bilinear Degraded	6.0	4.8	4.0	7.7	15.8	15.5	9.0
T = 1.6 s	Linear+	4.4	2.8	4.4	3.9	10.5	4.2	5.0
	Bilinear Elastic	6.5	2.8	3.6	8.0	7.5	11.6	6.7
	Bilinear Plastic	4.1	2.8	2.6	5.9	8.1	5.6	4.9
	Bilinear Degraded	4.6	4.5	2.4	6.9	11.1	10.6	6.7
T = 2.0 s	Linear+	4.6	3.1	2.3	4.0	7.8	7.3	4.9
	Bilinear Elastic	6.8	6.2	2.1	6.5	5.9	7.5	5.8
	Bilinear Plastic	3.4	3.5	1.7	5.2	7.0	4.9	4.3
	Bilinear Degraded	5.0	4.6	2.4	5.9	7.4	7.9	5.5

* Scaled to 0.4g peak ground acceleration

+Related to yield displacement of other loops

attributed to higher mode effects. If these higher floor forces are argued to represent diaphragm forces (induced by inertial effects during the dynamic shaking), it suggests that the peak floor accelerations in HPR frames could also be higher than (or at least, comparable to) those anticipated in conventional frame systems, for which current design procedures have been tailored. It should be noted that only a preliminary argument based on indirect qualitative interpretation of the limited available information has been presented here. Further studies are needed (and highly recommended) to compare the peak floor acceleration profiles of multi-storey HPR frames and monolithic ductile frames.

LIKELY SEISMIC LOSSES IN HPR FRAME BUILDINGS

Building seismic losses are traditionally classified into three categories, known as the three Ds: *Dollars*, *Downtime* and *Deaths*. The first, *Dollars*, represents the direct losses incurred in the form of repair or replacement costs of damaged components (or the whole building if the cumulative damage to components renders the building irreparable). The second, *Downtime*, represents the indirect losses due to business interruption and/or loss of income until the full functionality of a damaged building is restored. The third category represents the loss of human productivity through fatality (arguably intended to include injury as well) resulting from building damage/failure. A qualitative comparison of these three categories of seismic losses between HPR frame buildings and traditional monolithic ductile moment resisting frame buildings is presented below.

Direct Losses: Repair/Replacement Costs

The direct losses are typically correlated to the extent of damage to different building components. The extent of damage to building components, in turn, depends on the peak demands they experience under the earthquake ground motion. Although the majority of the building elements participating in the transfer of gravity and seismic loads to the foundation (commonly known as *structural components*) incur damage due to excessive deformations; some (such as floors and their connections to frames/walls) can be argued to incur damage due to excessive diaphragm inertial forces. Similarly, building components other than the structural frames, walls, floors and foundations (commonly known as *non-structural elements*, NSEs in short) can incur damage either due to excessive deformations (e.g. internal walls, doors/windows, claddings, glazing, etc.) or excessive inertial forces (e.g. contents, ceilings, chimneys, plumbing/mechanical/electrical service equipment, parapets, etc). Hence, building components are commonly divided into three categories to estimate direct seismic losses: structural components, drift-sensitive NSEs, and acceleration-sensitive NSEs. The relative proportion of each of the three categories of building components depends on the building's primary function, but as shown in Figure 2, the structural components contribute to only 20% or less of the total building value for a majority of building usage categories [32].

Despite excessive drifts being responsible for the majority of the damage incurred by traditional buildings, the larger drifts attracted by the HPR frames have not been identified as an issue in past studies. This is because the past experimental tests on HPR connections and subassemblies have proven that large levels of drifts can be accommodated by rocking between the precast members, and the rocking surfaces in HPR frames are carefully detailed and armoured to avoid significant damage despite undergoing large rotations. Hence HPR frames can remain more-or-less damage-free despite large drift responses. Preventing the damage to the structural frame, however, reduces only a small proportion of the overall building damage repair cost, since a large fraction of the direct losses (i.e. damage repair/replacement cost) in buildings is contributed by the NSEs. For example, Figure 3 shows the contributions of different components of a traditional monolithic frame building, on the total damage repair cost, under a design level earthquake [33] based on an inventory of components typical of office buildings in New Zealand.

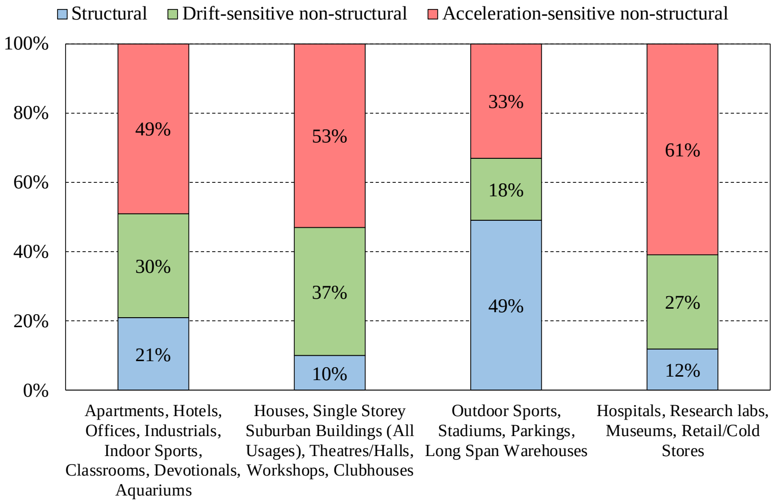

Figure 2: Contributions of structural and non-structural components in total building cost [32].

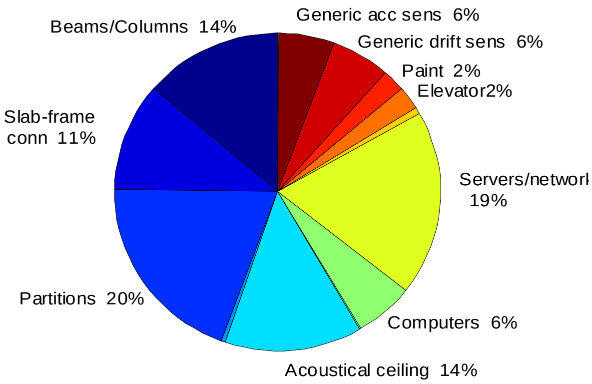

Figure 3: Contribution of different building components to seismic loss [33].

As seen in the figure, minimising structural damage in such buildings can reduce only about one-quarter of the total repair cost. Even among the structural components, damage to slab-frame connections contributes to 44% of the repair cost (i.e. 11% of the 25% total structural contribution), which will not be alleviated by the use of HPR frames. Moreover, it can be argued that the inevitable increase in maximum drift will increase the damage to the drift-sensitive NSEs connected to the frames (such as partitions, paints, and generic drift-sensitive components, which amount to 28% in Figure 3); thereby further shrinking the advantage of using HPR frames in such buildings. Since the floor acceleration demands on such buildings are presumably comparable to traditional monolithic frame buildings, no reduction of potential damage to acceleration-sensitive NSEs (e.g. ceilings, computers, servers, elevators and generic acceleration sensitive components, amounting to 47% in Figure 3). Hence, the reduction of structural repair costs in HPR frame buildings is expected to be offset by the increased repair/replacement costs of other components, diminishing the incentive to use this system in practice.

Indirect Losses: Business Interruption and Loss of Income

Although structural damage is significantly minimised in HPR frame buildings, more pronounced damage to the NSEs is likely to lead to delays in building occupancy after earthquakes. Experience from recent earthquakes suggests that buildings can be rendered unusable for significant lengths of time even due to damage to secondary elements only. The time taken to restore a damaged building to full functionality is influenced by: (i) the time to conduct a building-level damage assessment, and to make decision on the remedial action [34]; (ii) the time to plan and design the repair/replacement strategy; and (iii) the time to repair/replace the damaged components. Since structural damage is more challenging to repair than damage to NSEs, the total repair time can be arguably less in HPR buildings (as they will suffer less structural damage) despite greater NSE damage. Nevertheless, the first two phases including the damage assessment, making repair/demolition decisions (including insurance claim settlement), planning and designing the repair work, usually takes much longer than the actual repair time itself. Hence, the total indirect losses due to business interruption and/or loss of income can be argued to be comparable between the HPR frame buildings and traditional ductile frame buildings.

Human Losses: Injury and Casualty

Casualty is usually caused by local or global structural collapse, and both HPR and monolithic ductile frame buildings fare very well in terms of life-safety and collapse prevention. Although some risk to the lives of occupants is associated with the collapse of precast floors, there is no evidence to suggest that this risk is higher in one system compared to the other. Due to larger peak drift responses and more pronounced opening of beam column connections, the chance of unseating of inadequately seated precast floors can be argued to be higher in HPR frames.

On the other hand, the risk of injury to occupants arises mainly from the localised failure of structural or non-structural components and sliding and/or toppling of building contents [35,36]. Since research reporting injury risks in buildings is scarce [37], and the relative floor acceleration responses (which govern the movements of building contents) of traditional and HPR frames are not known, it is difficult to compare the injury risks in buildings with these two structural systems.

SEISMIC LOSS IN OF HPR FRAME BUILDINGS

Following the discussion presented in the previous section, assuming the buildings are used for the same purpose and contain similar secondary components, a HPR frame building is likely to incur less structural damage than a conventional monolithic frame building, more damage to secondary components (i.e. NSEs, especially drift-sensitive NSEs) and comparable indirect losses. Therefore, despite the proven low-damage (arguably damage-avoidance) characteristic of HPR connections and frames, even under large lateral drifts, not all buildings equipped with these frames are expected to benefit noticeably in terms of loss minimisation under seismic events. The main reason behind this is the increased inter-storey drift demands experienced by HPR frame buildings compared to traditional monolithic frame buildings, which result in increased damage to drift-sensitive building components and the associated repair cost and time. Past studies [33,34] have proven that such drift-sensitive components in buildings usually contribute more to the overall building value as well as to the total direct seismic loss after an earthquake. Hence, the reduction in damage and repair cost of the structural frames only may not be able to fully offset the increased loss from damage to drift-sensitive building components.

Other types of structures can, however, benefit from the use of HPR connections as the primary means of seismic resistance. Bridges, for example, are well suited to use post-tensioned rocking piers, since they do not contain secondary components that can be damaged from the larger displacements produced by this structural system. Past studies [12,24] have proven that bridges with post-tensioned rocking pier base can remain damage-free and upright even after being subjected to a large drifts, greater than those expected from major earthquakes. Hence, the superior seismic performance of HPR bridges in comparison to traditional monolithic bridges (with ductile fixed-base piers) is unquestionable.

In the context of buildings though, most residential and commercial buildings depend heavily on secondary elements, fit-outs, and services for continued functionality and occupancy. Hence, HPR frames may not be the most ideal choice for the structural systems of these types of buildings, unless they are accompanied by "low-damage" and resilient drift-sensitive NSEs. The same applies to health facilities, which need to be operational after major earthquakes, since their ability to remain functional depends, to a great extent, on contents and equipment that are sensitive to accelerations. Since floor acceleration demands in buildings have not been proven to be reduced by using HPR frames, they offer little advantage in such buildings, unless the valuable and clinically-critical contents and equipment are adequately braced/restrained to meet the acceleration demands.

On the other hand, the proven low-damage nature of HPR frames can lead to a significant reduction in the overall seismic loss, if the building's structural skeleton dominates its component inventory. For example, multi-storey garage buildings seem to be a good fit for using HPR frames, since these buildings have very few (if any) drift-sensitive NSEs, and the increased drift response will likely not lead to increased damage or downtime. Similarly, outdoor stadia are another class of structures for which the low-damage nature of HPR frames can result in significant advantages in terms of seismic loss reduction and enhanced seismic resilience.

CONCLUSIONS

The following conclusions can be drawn from the discussions presented in this paper.

- Precast frames, identified in the literature as hybrid post-tensioned, DAD, or PRESSS (referred as *hybrid post-tensioned rocking* and abbreviated as HPR in this paper), in which prefabricated members are joined together using unbonded post-tensioned tendons, and connections are provided with supplemental energy dissipaters, can accommodate large drifts via rocking at the connections. There is plenty of evidence in the published literature that demonstrates their superior structural performance in comparison to monolithic frames.
- HPR frame buildings experience larger inter-storey drifts and similar floor accelerations compared to monolithic frame buildings. Hence, while HPR frames themselves suffer little damage to their members and connections despite the larger drifts induced by earthquakes, they cannot minimise damage to the other building components they support. As such, damage to some building components may even be increased if HPR frames are used instead of traditional frames.
- In terms of minimising seismic losses, HPR frames can be very effective for bridges and those buildings that comprise only of a structural skeleton, with very few non-structural components (such as multi-storey garages and stadia). The damage-free HPR frames are likely to enable these buildings to survive major earthquakes with little damage, and to remain functional with little or no downtime.
- If post-tensioned frames are chosen as the structural system for commercial/residential buildings, the drift-sensitive components must be designed and built such that they remain undamaged despite the increased drift demands, in order to reduce the direct and indirect losses under earthquakes. Furthermore, the acceleration-sensitive components must be adequately braced/restrained to withstand design level floor accelerations.

Note: Notwithstanding the conclusions stated above, regardless of a building's structural system and intended use, designers are strongly recommended to adequately design and detail the NSEs (and their connections with the structural components) to meet the design drift and acceleration demands.

ARTICLES IN THIS ISSUE OF NZSEE BULLETIN

In addition to this editorial, this issue includes four other technical articles [38-41]. Among them, the first two relate to non-structural elements (NSEs); i,e, secondary components that do not contribute to the building strength. The first paper by Rashid et al. [38] provides a thorough review of the state-of-practice related to seismic design of acceleration-sensitive NSEs, highlights the (source of) inconsistencies between different NSEs design codes/standards/guidelines and recommends the key issues needing improvement. The second paper by Bhatta et al. [39] assesses seismic performance of timber-framed plasterboard internal partition walls with details typically used in New Zealand. Through experimental tests and theoretical derivations, it highlights the strengths and weaknesses of this common type of internal partitions. Following on from the special issue dedicated to *Seismic Performance on Non-Structural Elements* (SPONSE) published in the Bulletin in 2017 and a stream of other papers published more recently on different aspects/issues of NSEs [42-50], these two papers further underline the emergence of the Bulletin as a prominent outlet of state-of-the-art papers on this important topic.

The third paper by Ahmad et al. [40] presents an experimental and numerical validation of using steel haunches to repair critically damaged and RC frames. The paper also proposes a code-based procedure for seismic analysis and preliminary design of steel haunches for strengthening of seismically deficient RC frames. The findings of the paper is likely to be useful in avoiding brittle joint failure in old RC frame buildings designed and built before the modern ductile design philosophy came into practice.

The final paper by Soleimankhani et al. [41] numerically investigates the responses of structures (idealised as SDOF systems) with different energy dissipation mechanisms (represented by different hysteretic models) to impulse-type and long duration earthquake records. Five commonly used hysteretic models are considered and their seismic responses are compared based on their energy dissipation characteristics. The paper introduces a new measure called *oscillation resistance ratio* (ORR) to quantify the unloading characteristics of different hysteretic models in energy terms. The paper reports that response to impulse type records are similar for all hysteresis models, but for long duration records the models with less ORR (like flag-shaped models) result in greater response than those with higher ORR.

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