Experimental and numerical validation of seismic interaction between cladding systems and moment resisting frames

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SUMMARY:
Cladding systems are typically made of stiff, brittle materials, making them particularly vulnerable to earthquake damage. In order to reduce the damage to claddings, an experimental and numerical investigation has been conducted which models typical cladding-structure systems. The experimental investigation consisted of a full-scale precast concrete subassembly which represents a portion of moment resisting frame. The system was also modelled numerically using a lumped plasticity model which was validated using the results of the experimental investigation. The numerical model allowed further local variations of the system to be analysed as well as the expansion of the results to analyse the global structural response when cladding-structure interaction is taken into account. Results showed that cladding can affect the overall strength and stiffness of a structure, altering its bare frame characteristics. The degree to which this effect occurs is highly dependent upon the cladding connection used.

Keywords: cladding interaction, non-structural elements, quasi-static testing, response-history analyses

1. INTRODUCTION

Damage to non-structural components during seismic events can cause significant economic losses and disruption due to building repair downtime. For a typical 5–10 story office building, the exterior enclosure (cladding system) accounts for approximately 14% of the total building costs. This is only slightly less than the cost for the structural elements which account for 18% of the total building costs (Taghavi & Miranda, 2003). Consequently, cladding system damage contribute a significant portion of overall earthquake damage. Furthermore, failures can result in potential hazards to pedestrians around the building. The earthquake that struck Christchurch on the 22nd of February 2011 further highlighted this problem (Baird et al., 2012) as shown by photographs taken of damage in Fig. 1.1.

Figure 1.1. Cladding damage to multi-storey buildings in Christchurch CBD (Baird et al., 2012)

This paper presents the results from numerical models of cladding systems have been defined from recent experimental tests undertaken at the University of Canterbury. A one-storey, single bay frame clad with a precast concrete panel has been tested in order to ascertain the influence the cladding has on the frame as well as to define the cladding performance under increasing inter-storey drift. The numerical models expand the experimental investigation to determine the effect the cladding has upon a ten storey moment resisting frame building as well as the likely cladding damage. This is achieved through both pushover and response-history analyses of typical cladding-structure systems.

2. BACKGROUND OF CLADDING SEISMIC PERFORMANCE

Recent studies on the interaction of cladding panels with the primary structure have outlined how cladding panels can influence a structure's behaviour (Hunt & Stojadinovic, 2010, McMullin et al., 2004, Baird et al., 2011). The seismic performance of a cladding system is commonly determined using the inter-storey deflection (or drift) of the structure. The link between quantitative and qualitative seismic performance is achieved through the definition of the following performance levels: Operational, Immediate Occupancy, Life Safety and High Hazard (FEMA 356, 2000). Graphic illustrations of these performance levels can be seen in Fig. 2.1. Following the magnitude 6.3 earthquake that struck Christchurch on 22 February 2011, a damage assessment survey of facade systems was conducted (Baird et al., 2012). The survey included buildings within the Christchurch Central Business District greater than three stories in height, comprising a total of 371 facade systems. The survey was based on what is visible from outside the building, making it equivalent to a Level 1, or rapid safety assessment (ATC-20, 1989). The survey rated the performance of the facade systems using the performance levels shown in Fig. 2.1. The facade performance composition from the survey is also shown in Fig. 2.1.

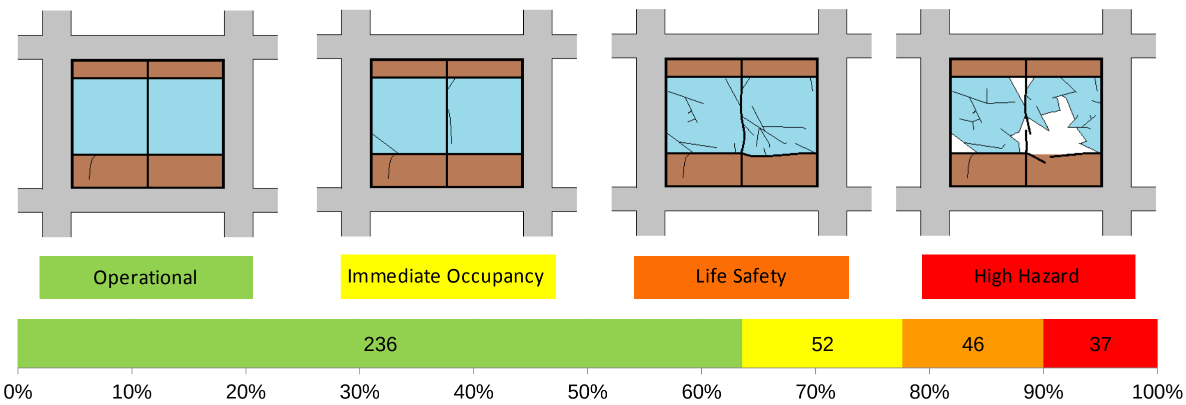


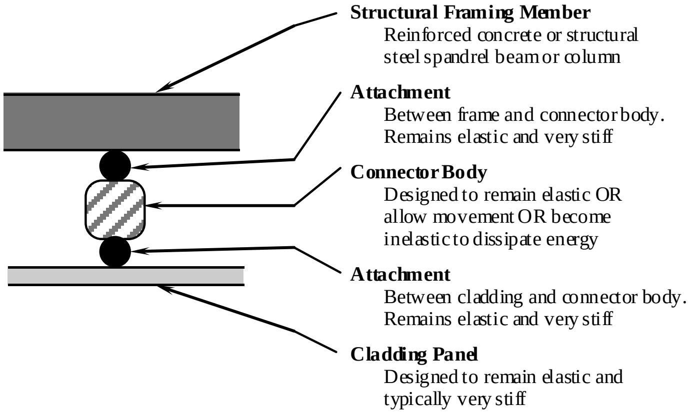
Figure 2.1. Graphical representation of cladding performance levels (top) and facade performance composition following Christchurch 22/02/2011 earthquake

The survey classified the facade systems by eleven individual typologies based on those used in the Post-earthquake Building Performance Assessment Form (ATC-38, 2000). One of those categories used was heavy cladding (precast concrete or stone panels). Although the majority of heavy claddings were deemed to be either 'Operational' or 'Immediate Occupancy (minor cracking and damage to panels), cases of complete disconnection of heavy claddings raised serious concern. The disconnection of several precast concrete spandrels resulted in the death of a woman sitting in her car on the street below (CERC, 2012). Because of the high risk that falling heavy claddings presents, further attention to these systems is required.

2.1 Determining Performance

Capacity design (hierarchy of strength) principles can be used to assess the seismic performance of most cladding system. Assuming that the system is comprised of a structural frame member, a connector body and cladding panel, linked together with strong, stiff attachments, as shown in Fig. 2.2, then the system can be simplified by focussing on the weakest in the system. For most cladding panel systems the weakest (and least stiff) element in the system is the connector body. The connector body is usually required to accommodate relative movement between the cladding panel and the frame as well as provide out of plane restraint. For systems that incorporate glazing, the connector body is typically strong and rigid and relative interstorey movement has to be accommodated within the cladding itself. This is usually achieved by use of gaps around the glass and within the glazing frame (called seismic frame).

It is usually assumed that the attachment of the connector body is stronger than both the cladding and the connector body itself. This is typically the case designed for; however, as observed during the Christchurch earthquakes, mistakes where the attachment ends up being the weakest link are possible. When this is the case the risk of complete detachment of the cladding is very high. For this investigation, precast concrete panels are tested and as such the performance and failure mechanism is expected to be governed by the connector body. Therefore, each performance level can be related to the performance of the connection alone.



Connector Body Type	Characteristics	Strength	Stiffness
Tie-Back	Deform easily under lateral forces	Medium	Medium
Slotted/ Sliding/ Rotating	Disconnect the panel by allowing degree of freedom in one or more directions	Low	Low
Dissipative	Dissipate energy in connector body under lateral forces	High	High

Figure 2.2. Cladding system composition (left) (Pinelli et al., 1993), typical connector body types (right)

3. EXPERIMENTAL DEFINITION OF DAMAGE STATES

In order to assess the seismic response of multi-storey buildings with claddings, a full-scale, single-bay, single storey frame subassembly has been constructed. The frame represents a portion of a reinforced concrete moment resisting frame, as shown in Fig. 3.1.

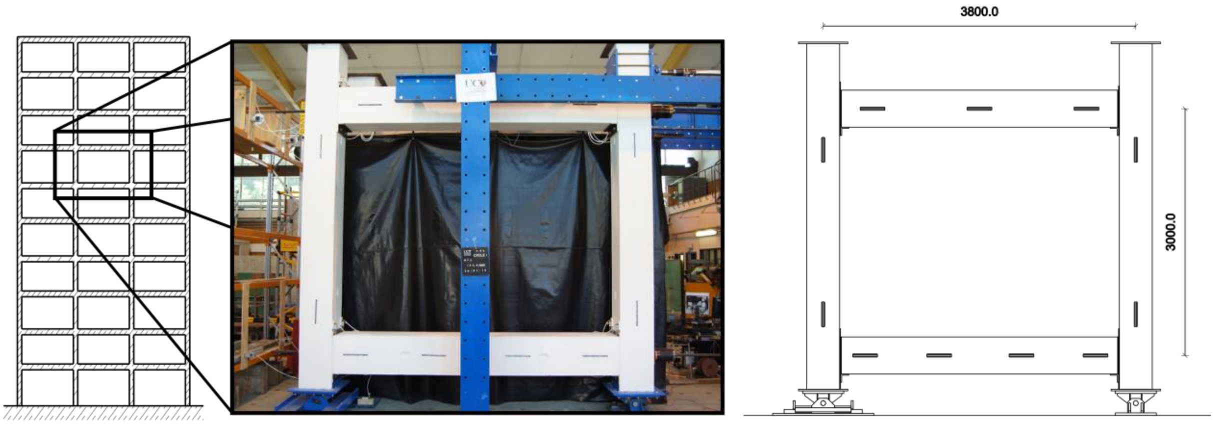


Figure 3.1. Experimental test frame

The frame is subjected to increasing levels of drift using a quasi-static cyclic loading protocol in order to assess its seismic response (ACI 374.1R-05, 2005). The beam-column connections utilise Precast Seismic Structural System (PRESSS) technology which allow the frame to be tested repeatedly to high drift levels with different claddings without sustaining significant structural damage (Priestley et al., 1999).

In order to assess the effect cladding has upon the frame, a single precast concrete panel is attached to the beams by the use of tie-back and bearing connections, as shown in Fig. 3.2. The bearing connections transfer the gravity load of the panel back to the frame and the tie-back connections resist out-of-plane forces due to wind and earthquake loading. The tie-back connections must also be able to accommodate any in-plane relative movement between the frame and the cladding panel during earthquake induced movement.

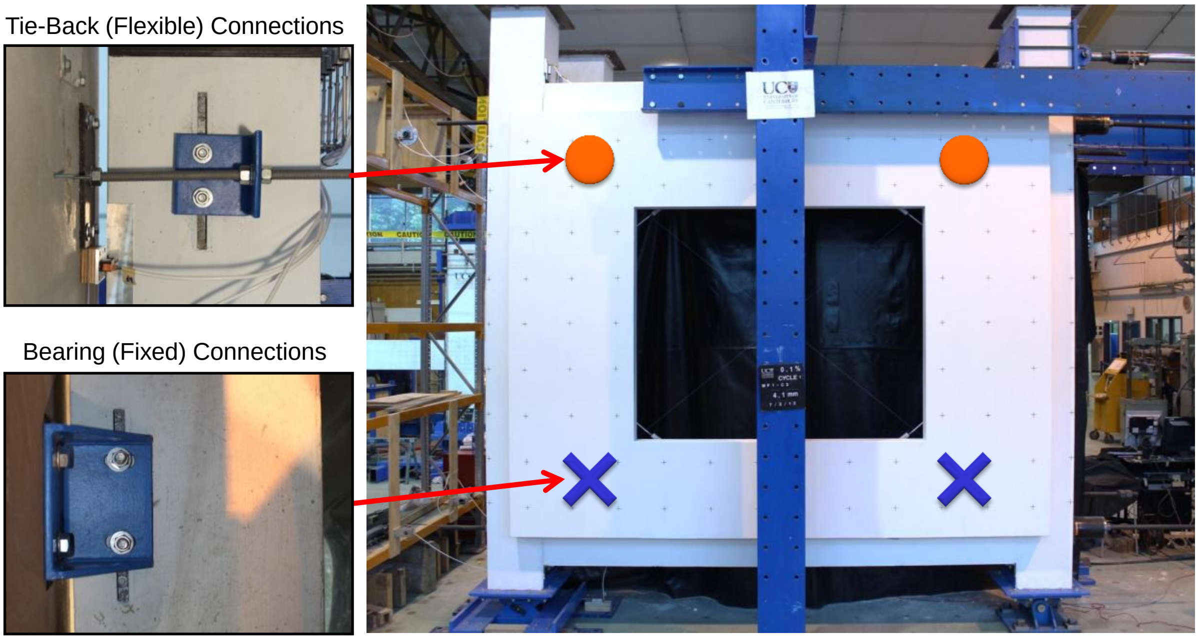


Figure 3.2. Experimental test frame with cladding showing connection assemblies

Shown in Table 1 is a summary of the tests conducted with the single precast concrete panel cladding and the different tie-back connections used. Fig 3.3 (left) shows the force-displacement behaviour of the frame with and without the influence of the cladding for one of the tests (MP-TR1). The corresponding maximum drift shown is 1.5%. The difference between the two gives the strength and stiffness that the cladding provides and is shown in Fig 3.3 (right) for one test (MP-TR1).

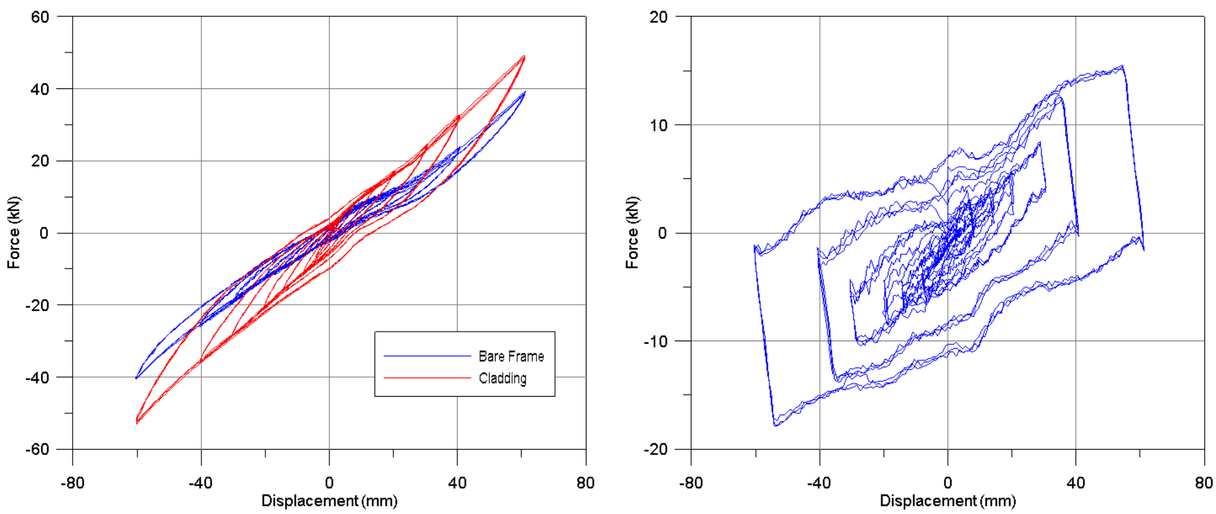


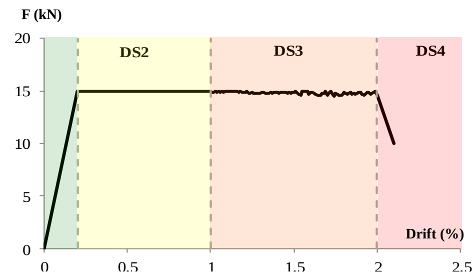
Figure 3.3. Comparison of bare frame and clad frame (left), contribution to frame by cladding (right)

Table 1: Summary of Tests

Test ID	Connection Type	Size	Length	No. of Tests
MP-TR1	Threaded Rod	20 mm x 2	275 mm Rod	3
MP-TR2	Threaded Rod	12 mm x 4	275 mm Rod	3
MP-TR3	Threaded Rod	20 mm x 2	550 mm Rod	3
MP-TR4	Threaded Rod	12 mm x 4	550 mm Rod	3
MP-SL1	Slotted	20 mm	250 mm Slot	3
MP-SL1	Slotted	20 mm	125 mm Slot	3

3.1 Definition of Damage States

In keeping with the shift towards a performance-based framework for both structural and non-structural system in newly designed buildings (Priestley, 2000). Cladding is deemed to be sensitive to inter-storey drift (Taghavi & Miranda, 2003) therefore the maximum differential displacement of the connections is to be monitored in order to compare damage limit states. If the cladding system is compared to that introduced earlier, then the tie-back connections represent the connector body and are the weakest element in the system and hence govern the system behaviour. The performance of the cladding is therefore directly dependent on the performance of the tie-back connections. The idealised behaviour and corresponding damage states are presented below in Fig. 3.4.



	Damage level	Drift limit	Behaviour	Repair required	Outage
DS1	None	< 0.2%	Elastic	None	No
DS2	Minor	0.2% ≤ x <1.0%	Post-yielding	Inspect	< 3 days
DS3	Moderate	1.0% ≤ x <2.0%	Local cracking	Repair elements	< 3 weeks
DS4	Failure	> 2.0%	Failure	Replacement	> 3 weeks

Figure 3.4. Damage states definition

4. NUMERICAL MODEL

Numerical modelling has been implemented using the programme RUAUMOKO (Carr, 2010) and consists of a local 'cladding model' being introduced to a global 'frame model.

4.1 Cladding Model

The precast concrete panel has been modelled as quadrilateral elastic elements, while the connections have been considered as springs attached directly to points along the beams, as shown in Fig. 4.1 (left). The connection springs are characterised by the bi-linear elasto-plastic rule. The top (tie-back) connections have a lower strength and stiffness than the bottom (bearing) connections which results in the bearing connection springs remaining elastic.

The cladding model is first introduced to a model of the test frame in order to verify it. The test frame members have been modelled as elastic elements with a multi-spring element used to represent the PRESSS beam-column connection. As can be seen in Fig. 4.1 (right), the numerical model of the frame-cladding system replicates the physical behaviour of the experiment very well, so it can be assumed that the cladding model is an accurate representation of the cladding system.

4.2 Frame Model

The frame model that the cladding model is introduced to is based on the Red Book building (Bull &

Brook, 2008) which acts as a design example of the New Zealand Concrete Code (NZS 3101, 2006). The building is designed for Christchurch prior to the increase in seismic hazard factor from 0.22 to 0.3 (DBH, 2011). Fig. 5.1 (left) illustrates the plan view of the structure, with the seismic frame analysed highlighted. Beams and columns have been represented by elastic elements with inelastic behaviour concentrated in plastic hinge regions (Giberson model). The inelastic behaviour has been defined by the moment curvature hysteresis rule 'Modified Takeda' (Otani & Sake, 1974). The bottom floor has a storey height of 4m while the upper floors have a storey height of 3.6 m. Design loads, forces and seismic masses have been calculated according to New Zealand Design Standards (NZS1170:1, 2002 and NZS1170:5, 2004).

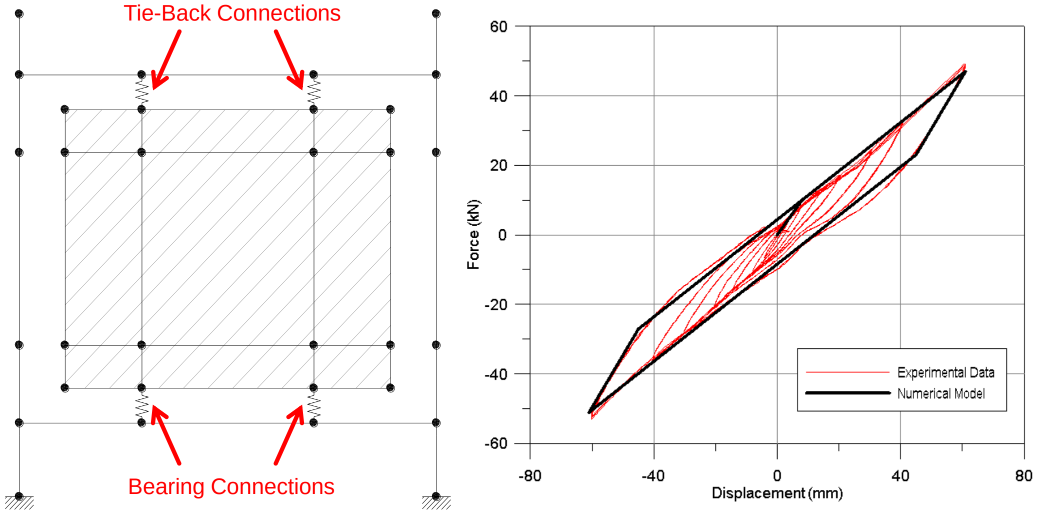


Figure 4.1. Cladding model in test frame model (left), numerical validation of model (right)

Three possible architectural cladding configurations have been considered for the static analyses; Full Cladding, Piloti and Bare Frame. Full Cladding consists of cladding panels in every bay in every storey of the frame where Piloti consists of panels in every bay and storey except the first storey, as shown in Fig. 5.1 (centre).

5. NUMERICAL ANALYSES

Non-linear static pushover and response-history analyses are conducted for the different building configurations as presented in the previous section.

5.1 Static Analyses

Static non-linear pushover analyses of the models were performed to investigate the lateral base shear and roof displacement relationship of the building. The analyses compare the behaviour of the systems under a distribution of the forces acting up the height of the building meant to represent earthquake demand as prescribed under NZS1170.5, 2004. In Fig. 5.1 (right) the monotonic response of the three different configurations is shown. As expected, an increase in stiffness and strength is observed for Full Cladding and Piloti cases compared with the bare frame due to the presence of the cladding panels.

Tracking the activation of the plastic hinges showed extensive formation of plastic hinges at the second and third floor levels in the full cladding case, while the absence of claddings at the ground floor in the piloti case resulted in higher demands at the ground floor level which lead to the formation of a soft-storey mechanism at that level. Collapse of the frame occurred earliest in the piloti case, followed by the full cladding case and finally the bare frame.

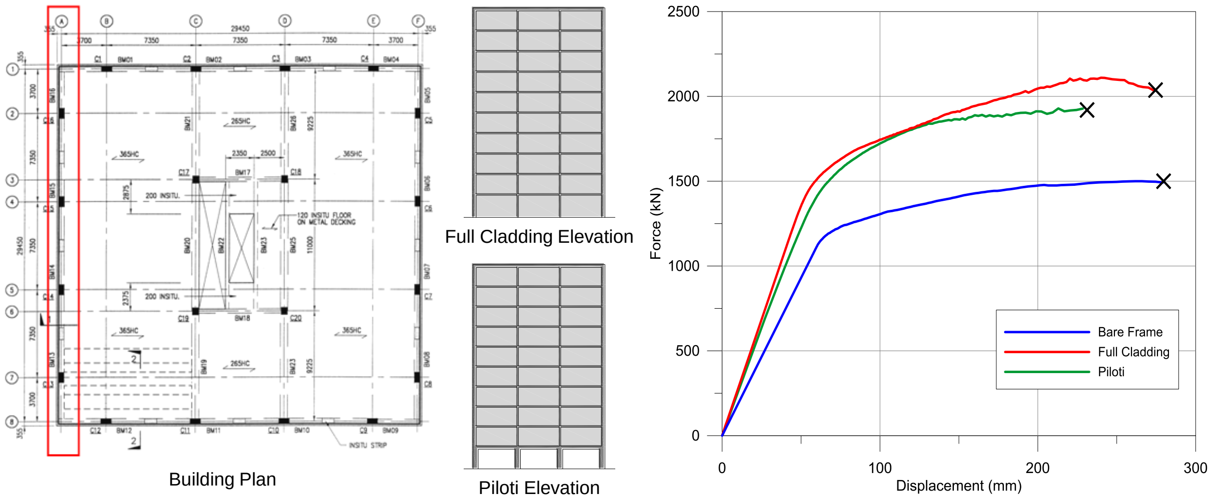


Figure 5.1. Pushover analysis response of cladding systems compared to bare frame

It is evident that the inclusion of the effects of cladding causes increased stiffness, strength and earlier collapse initiation. The effects of claddings for both the Full Cladding and Piloti cases are summarised in Table 2 in comparison with the bare frame.

Table 2. Pushover analysis – change in respect to Bare Frame case

Building configuration	Initial Stiffness	Maximum Base Shear	Drift at Collapse
Full Cladding	+47%	+41%	-7%
Piloti	+37%	+29%	-22%

5.2 Dynamic analyses

Response-history analyses have been performed investigating how the panel distribution can affect the response of the building. A suite of fifteen recorded and properly scaled natural accelerograms have been used (Pampanin et al., 2002). The records have been scaled according to NZS1170:0, 2002 and NZS1170:5, 2004, considering a seismic hazard factor of 0.3, soil type C, annual probability of exceedance of 1/1000 (R_s = 1.3) and a fundamental period of the structure equal to T_1=2.02 seconds (Bull & Brunsdon, 1998). Shown in Fig. 5.2 are the 15 scaled records and the average of the scaled records compared to the New Zealand design spectrum.

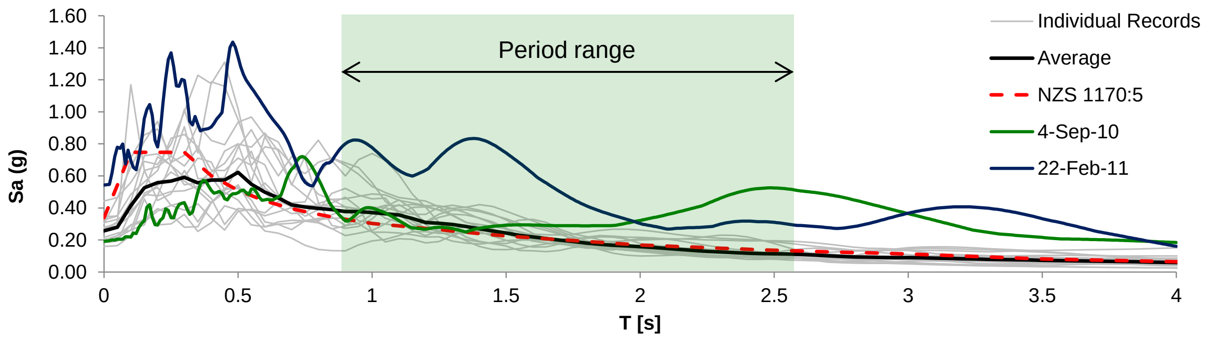


Figure 5.2. Scaled fifteen accelerograms and average compared with NZS 1170.5 design spectrum and Christchurch earthquake records

In accordance with FEMA-302 (NEHRP, 1997) two earthquake intensity levels have been considered in the numerical analyses, subjecting the structure to two corresponding response spectra: the Design

Basis Earthquake (DBE) ground shaking (probability of exceedance of 10% in 50 years) and the Maximum Considered Earthquake (MCE) ground shaking (probability of exceedance of 2% in 50 years). Referring to the performance objectives matrix (SEAOC, 1995), the Basic Safety Objective is attained when a structure achieves both the Life Safety Performance level under the DBE level and the Collapse Prevention Performance level under the MCE level.

Fig 5.3 presents the maximum interstorey drift in each level for the bare frame under both DBE and MCE level excitation. It can be observed that the highest levels of interstorey drift occur in the lower storeys of the building.

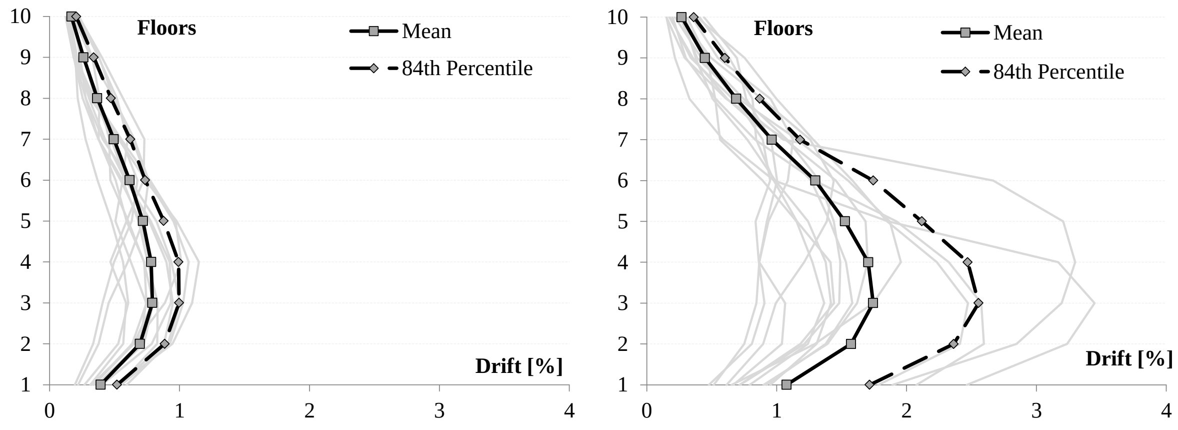


Figure 5.3. Interstorey drift of the bare frame building for DBE (left) and MCE (right)

Presented in Fig. 5.4 is the mean interstorey drift of the three different building configurations for DBE and MCE. As expected, by including the stiffening effects of the cladding, the mean interstorey drift is reduced for both DBE and MCE.

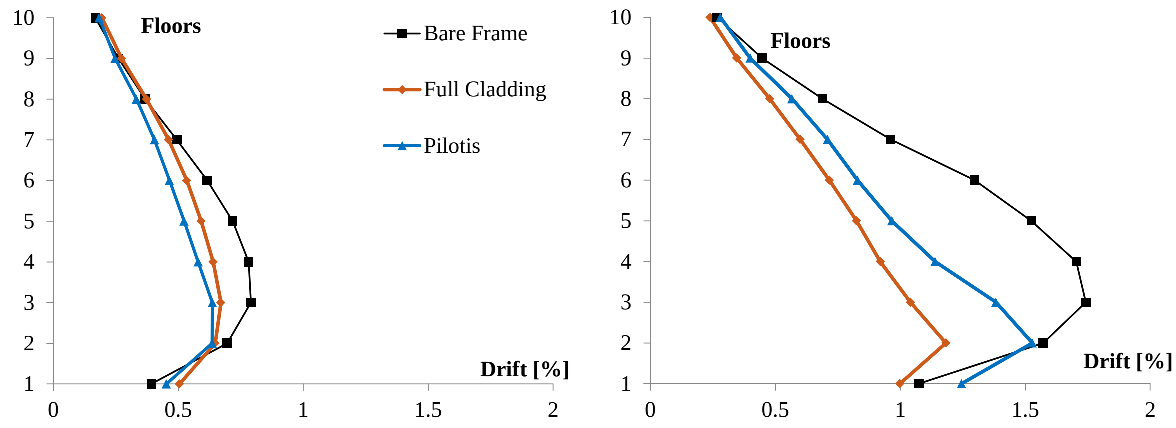


Figure 5.4. Mean interstorey drift of different building configurations for DBE (left) and MCE (right)

The maximum differential displacement between the cladding and the frame can also be found for each panel. The damage state of each connection can then be inferred using the damage limit states presented in Fig. 3.4. The connection damage state is presented for each floor level in Fig. 5.5, with the mean and 84th percentile of the 90 connections shown (fifteen earthquakes and six connections per floor).

As the connection damage is dependent on the maximum interstorey drift reached, the relationship between Fig. 5.4 and Fig 5.5 is evident. Under DBE it can be seen that most connections are within DS2 which means that they have yielded but the damage is minor. Under MCE it can be seen that there is the risk of connection failure since the maximum displacement falls within DS4.

A summary of the distribution of connection damage states is also shown in Fig 5.5 (bottom right). This is a count of the maximum damage state that each connection reaches. It can be seen that the difference in distribution of damage between Full Cladding and Piloti is relatively small for both DBE and MCE events. In a DBE event it is expected that 85% of connections will suffer at least minor damage. This increases to 95% in an MCE event. The average connection ‘drift’ and corresponding damage state composition is also shown for the two major Christchurch earthquakes. It can be seen that more damage is expected from the 22nd February earthquake than what the design code suggests we should expect from a MCE event.

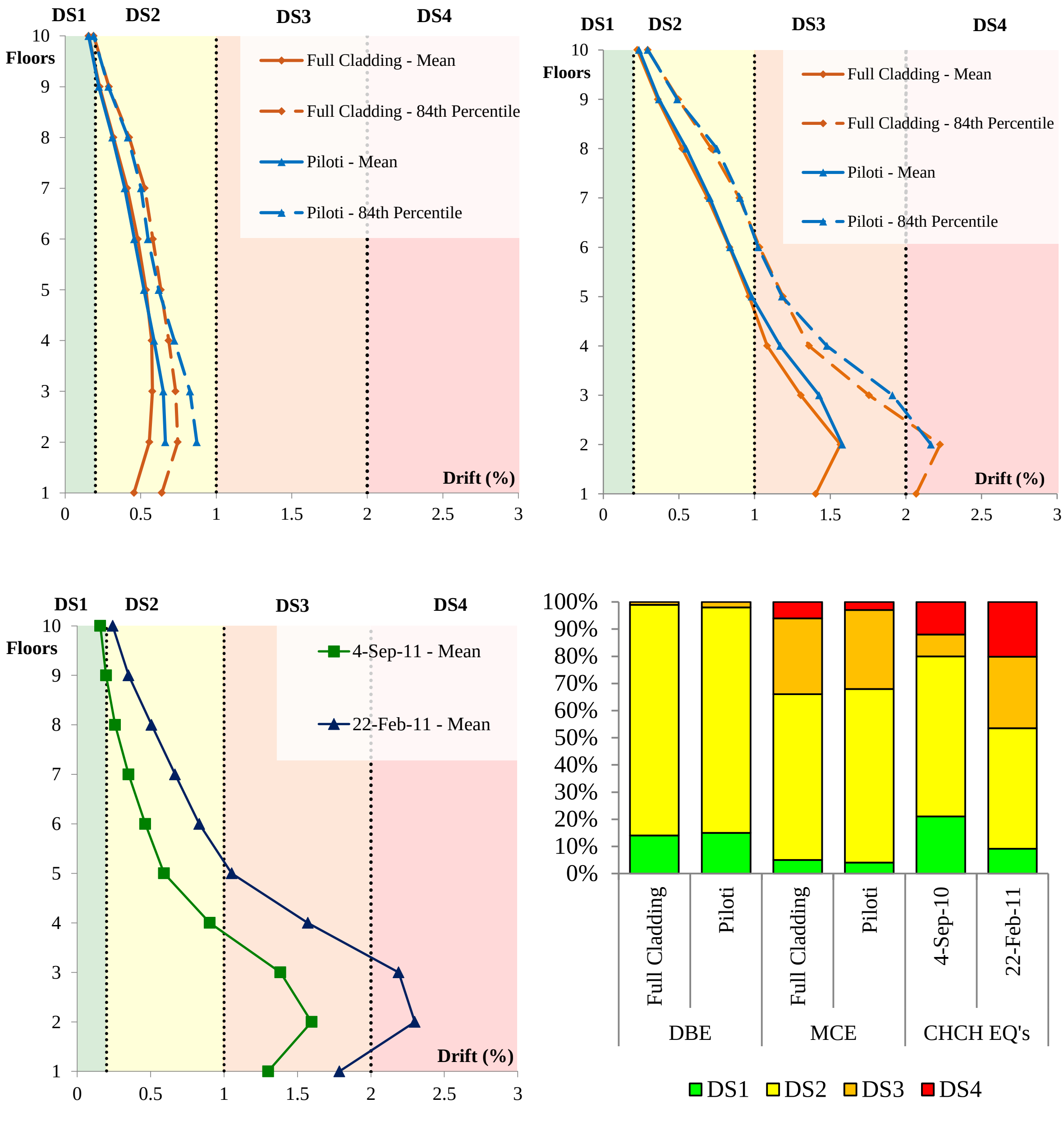


Figure 5.5. Differential displacement between panel and frame and corresponding connection damage state for DBE (top left), MCE (top right) and Christchurch Earthquakes (bottom left) and damage state composition (bottom right)

6. CONCLUSIONS

The seismic behaviour of a typical newly designed reinforced concrete multi-storey frame building has been analysed by means of non-linear static and dynamic analyses with the inclusion of common typologies of cladding systems. Results confirm the high influence of cladding systems upon the seismic performance of multi-storey buildings. An increase of between 30 and 50% in initial stiffness is observed for both cladding configurations compared to the bare-frame. A higher strength is also observed for both cases. The piloti case exhibits a soft-storey mechanism as expected, but in general the maximum inter-storey drifts are concentrated on the first three floors.

The results also show that in a DBE event it is expected that 85% of connections will suffer at least minor damage and in an MCE event it is expected that 3-6% of connections will be at high risk of failure. The authors intend to investigate further in order to estimate potential losses due to cladding damage in order to encourage the use of low damage solutions.

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