Understanding cladding damage: A numerical investigation into a Christchurch earthquake case study

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ABSTRACT: In order to better understand the damage sustained by cladding systems in the earthquake that struck Christchurch on the 22nd of February 2011, a numerical investigation of the effects of cladding-structure interaction has been undertaken.

The numerical investigation focuses on a typical reinforced concrete multi-storey building located within the Christchurch CBD and analyses its seismic performance with and without the interaction due to cladding panels. The cladding panels are precast concrete panels that are attached to the frame using two fixed connections at the base and two flexible tie-back connections at the top. Static push-over analyses are used to determine the change in strength and stiffness of the system. Results show that when cladding interaction is taken into account, the frame has a higher stiffness, strength and earlier onset of collapse.

Dynamic analyses are performed which involve subjecting each system to fifteen earthquake records, each scaled to both design basis and maximum credible earthquake intensities. The maximum inter-storey drift and subsequent cladding connection damage is inferred. Results confirm the high influence of cladding systems upon the seismic behaviour of multi-storey buildings. Also revealed is the significant variation in possible levels of cladding damage throughout a building.

1 INTRODUCTION

Reconnaissance following past earthquakes has shown that damage to non-structural components during seismic events can cause significant economic losses and disruption due to building repair downtime. 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, as shown by photographs in Figure 1, necessitating a detailed study in order to propose and develop innovative solutions able to reduce the risk of damage to non-structural elements. This work is part of an on-going research effort with the aim of investigating the interaction between facade systems and the structure.

Figure 1: Cladding damage to multi-storey buildings in Christchurch CBD

Numerical models of cladding systems have been defined using previous experimental investigations and preliminary data from recent experimental tests being undertaken at the University of Canterbury. A one-storey, single bay frame clad with a precast concrete panel is being tested as the foundation for this study. The present work focusses on precast concrete claddings because of the high risk they pose to pedestrians and because of concerns over their performance in recent seismic events.

The interaction of the cladding has been investigated by use of non-linear pushover and time-history analyses. The numerical model used is that of a typical Christchurch building with and without the presence of cladding panels. In the first part of this paper, quasi-static push-over analyses are presented considering different distributions of precast concrete panels compared with the bare frame. In the second part of the paper, time-history analyses are undertaken with varying earthquake intensity to assess the level of damage to the cladding as well as the inter-storey drift of the structure.

2 BACKGROUND

Recent studies on the interaction of cladding panels with the primary structure have outlined how cladding panels can influence structure's behaviour (Hunt & Stojadinovic, 2010, McMullin et al., 2004, Baird et al., 2011). These studies revealed that it is the characteristics of the cladding connections that are most influential in determining how a cladding interacts with the structure. Typically a precast concrete panel is connected to a structure with two different connections types; one is called a bearing connection and it is intended to carry the self-weight of the panel, the other is intended to accommodate differential movement between the structure and the panel as well as provide out-of plane restraint. The type of connection used to accommodate movement can be varied summary of the parameters associated with typical connections is shown in Table 1.

Table 1: Facade connection characteristics

CONNECTION	FUNCTION	CHARACTERISTICS	STRENGTH	STIFFNESS	DUCTILITY
Fully Fixed/ Bearing	Carry self-weight of the cladding	Rigid link between cladding and structure	High	High	Low
Tie-Back	Out-of-plane restraint, movement allowance	Deform easily under lateral forces	Low	Low	High
Slotted/Sliding/ Rotating	Out-of-plane restraint, movement allowance	Disconnect the panel by allowing degree of freedom in one or more directions	NA	NA	Medium
Dissipative	Out-of-plane restraint, movement allowance	Dissipate energy in connector body under lateral forces	Medium	Medium	Medium

In order to assess the seismic response of multi-storey buildings with claddings, a facade system comprising of precast concrete panels attached to the beams by the use of tie-back and fully fixed (bearing) connections has been used. If the cladding system comprises of the following parts; panel, panel anchorage, connector body and beam anchorage then because of their low strength and stiffness, the tie-back connections are the weakest element in the system and hence govern the behaviour. When this is the case it allows greater damping, strength and stiffness over many cycles as opposed to when damage occurs in the anchor or panel, which risks the panel falling.

The performance of the cladding is therefore directly dependent on the performance of the tie-back connections. This can be quantified using a performance based criteria based on experimental results. This is in keeping with the shift towards a performance-based framework for both structural and non-structural system in newly designed buildings (Priestley, 2000). Cladding panels are 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.

3 NUMERICAL MODEL

The numerical model proposed 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). Figure 2 (left) illustrates the plan view of the structure, with the seismic frame analysed highlighted. The analyses have neglected the beam extensions that form the corner of the building since in 2D these should have minimal effect upon the frame behaviour. 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).

Three possible architectural cladding configurations have been considered for the static analyses; Full Cladding (FC), Pilotis (PI) and Bare Frame (BF). Full Cladding consists of cladding panels in every bay in every storey of the frame where Pilotis consists of panels in every bay and storey except the first storey, as shown in Figure 2.

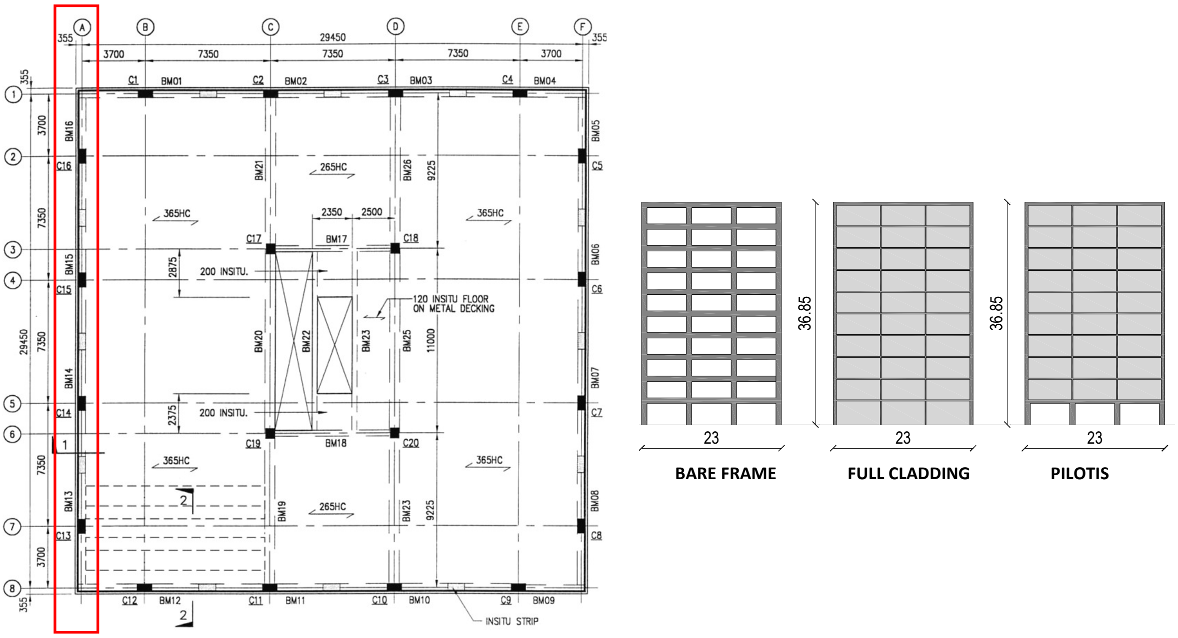

Figure 2: Plan view of the Red Book building (left) and claddings distribution (right)

3.1 *Cladding characteristics*

The cladding system is represented by precast concrete panels of 0.2 m thickness attached to the primary structure on the beam with tie-back connections at the top and with bearing connections at the bottom, as shown in Figure 3 (left). A single panel is located in every bay and spans one full floor height. The panels have been treated as not having any window openings for simplicity. However it can safely be assumed that correctly detailed panels with openings would behave nearly identically to the panels modelled since both provide large in-plane stiffness.

The cladding system has been designed considering a serviceability limit state inter-storey drift of 0.3%. The ultimate limit state requirement that there is necessary strength to prevent the panel detaching has also been met (NZS 1170.5, 2004). Damage limit states have been defined for the tie-back connections based on principles developed for assessing the possible occupancy or the reparability of a building by Dhakal & Mander, 2006. Damage State 1 (DS1) represents elastic behaviour, it therefore concludes at the onset of damage which is best defined by the yield drift of the connection. Damage State 4 (DS4) is defined from the onset of collapse. The other damage stages (DS2, DS3) are more subjective in their definitions. It is suggested that Damage State 3 (DS3) be defined as a level of damage which would cause loss of functionality and repairs are needed to restore the full functionality of the structure. At drifts below this boundary, damage is considered to be slight and only minor repair may be needed. This damage is deemed Damage State 2 (DS2). A summary of the damage states is shown in a table in Figure 3. Also shown is the elasto-plastic behaviour used to represent the cladding connections based on experimental data (McMullin et al., 2004).

Table 2: Damage limit states of tie-back connections

	Damage state	Drift limit	Behaviour	Repair required	Outage
DS1	None	< 0.2%	Pre-yielding	None	No
DS2	Minor/Moderate	0.2% ≤ x <1.0%	Post-yielding	Inspect / adjust	< 3 days
DS3	Major	1.0% ≤ x <2.0%	Local buckling	Repair elements	< 3 weeks
DS4	Failure	> 2.0%	Failure	Replacement	> 3 weeks

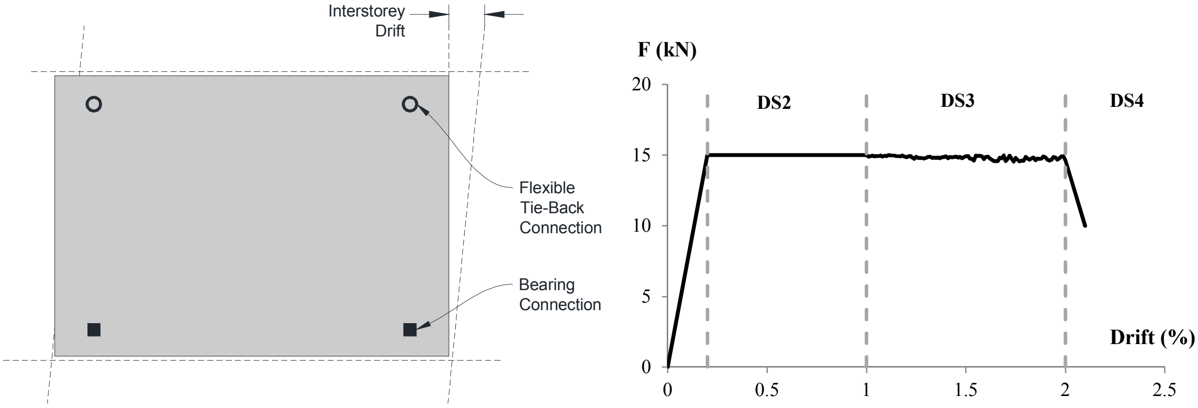

Figure 3: Diagram of typical cladding panel connection configuration (left) (Charleson, 2008), idealised force-drift behaviour of tie-back connections (right)

3.2 *Model characteristics*

The model has been implemented using the programme RUAUMOKO (Carr, 2010). 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). Precast concrete panels have been modelled as quadrilateral elastic elements, while the connections have been considered as springs attached directly to points along the beams, as shown in Figure 4. 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 essentially being elastic.

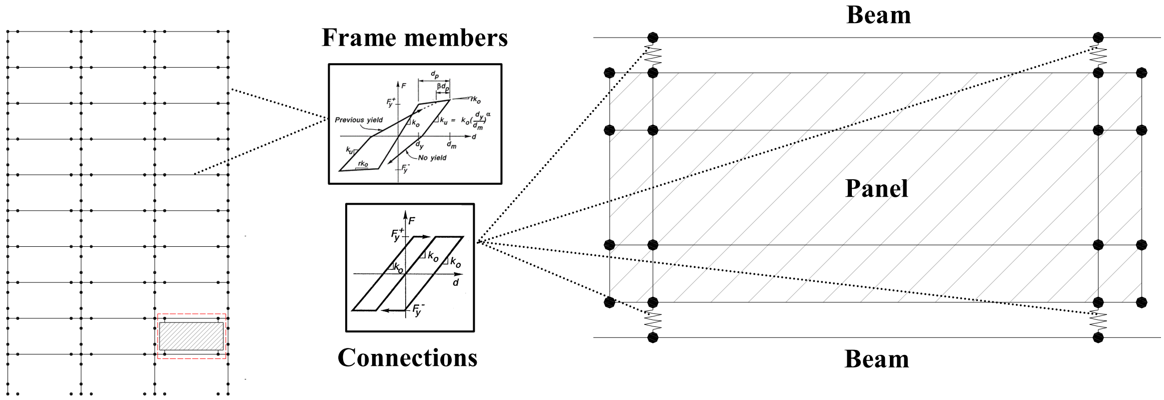

Figure 4: Modelling of the frame and cladding panel with hysteretic rules used

4 NUMERICAL ANALYSES

In the following paragraphs, the results from the non-linear pushover and time-history analyses are presented for the different configurations described in section **Error! Reference source not found.**.

4.1 *Pushover analyses*

Static non-linear pushover analyses of the models were performed to investigate the lateral base shear and roof displacement relationship of the building. A triangular distribution of forces acting up the height of the building is used to represent the equivalent earthquake demand as prescribed under NZS1170.5, 2004. In Figure 5(left) the monotonic response is shown, representing the three different configurations analysed: Bare Frame (BF), Full Cladding (FC) and Pilotis (PI). As expected, an increase in stiffness and strength is observed for FC and PI cases compared with BF due to the presence of the cladding panels.

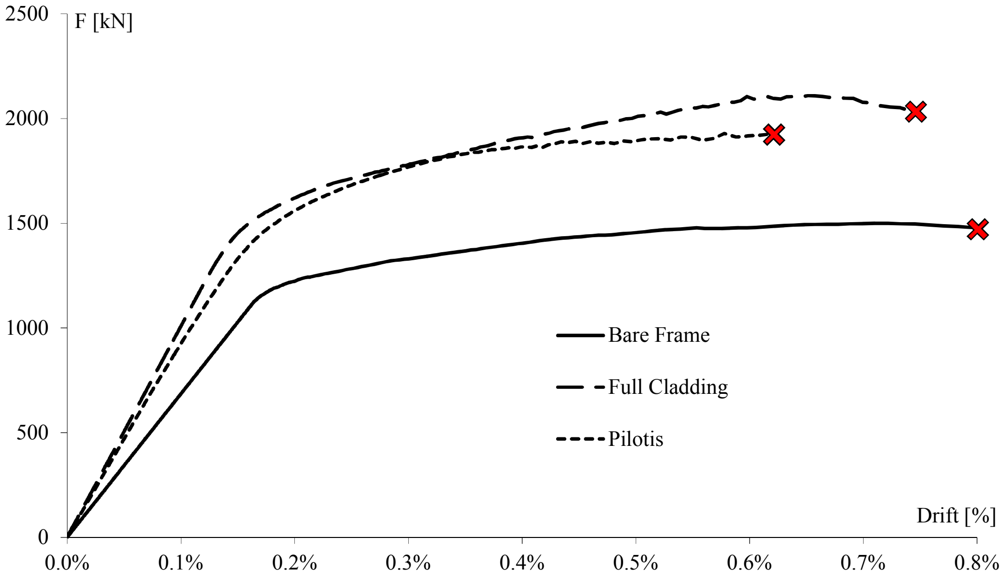

Figure 5: Pushover analysis response of cladding systems compared to bare frame

By tracking the activation of the plastic hinges, the failure mechanism of the structure could be determined. The FC case showed extensive formation of plastic hinges at the second/third floor levels, while the absence of claddings at the ground floor in the PI case resulted in higher demands at the ground floor level which lead to the formation of a soft-storey mechanism at that level. It can be seen that the collapse of the frame occurred earliest in the PI case, followed by the FC case and finally the BF case. It is evident that the inclusion of the effects of cladding causes increased stiffness, strength and initiated a premature collapse. The effects of claddings for both the FC and PI cases are summarised in Table 2 compared with the BF case.

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%
Pilotis	+37%	+29%	-22%

4.2 *Time-history analysis*

Time-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 & Brook, 2008). Shown in Figure 6 are the 15 scaled records and the average of the scaled records compared to the design spectrum.

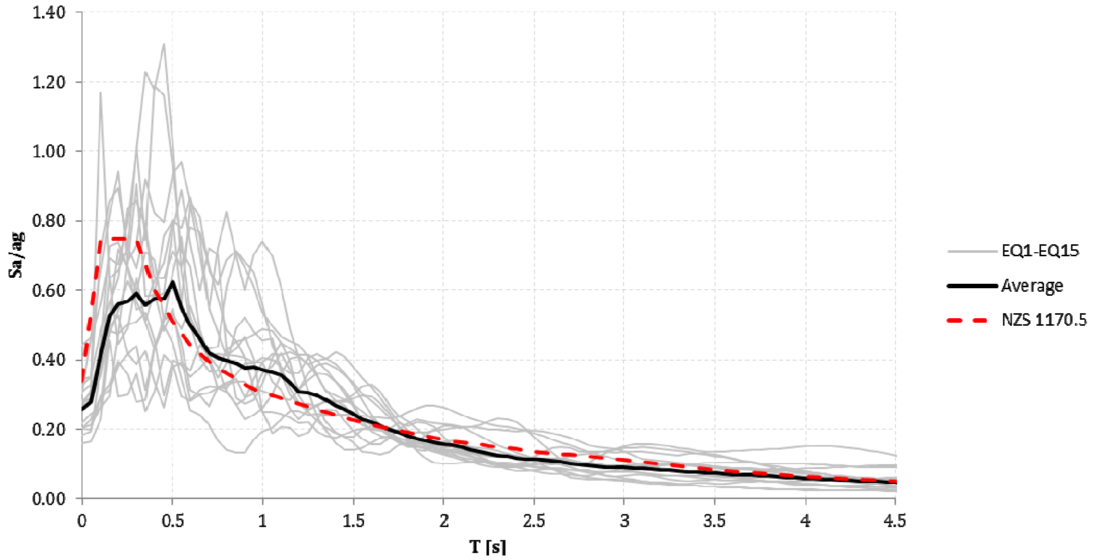

Figure 6: Scaled fifteen accelerograms and average compared with 1170.5 design spectrum

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). Figure 8 below, presents the interstorey drift for the bare frame for DBE and MCE. The grey lines represent the maximum inter-storey drift reached in each of the fifteen earthquakes. The mean and maximum of these is also shown. It can be observed that the highest levels of interstorey drift occur in the lower storeys of the building and that in an MCE event there is a risk the building will be verging on collapse with drifts in excess of 3% observed. Presented in Figure 8 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.

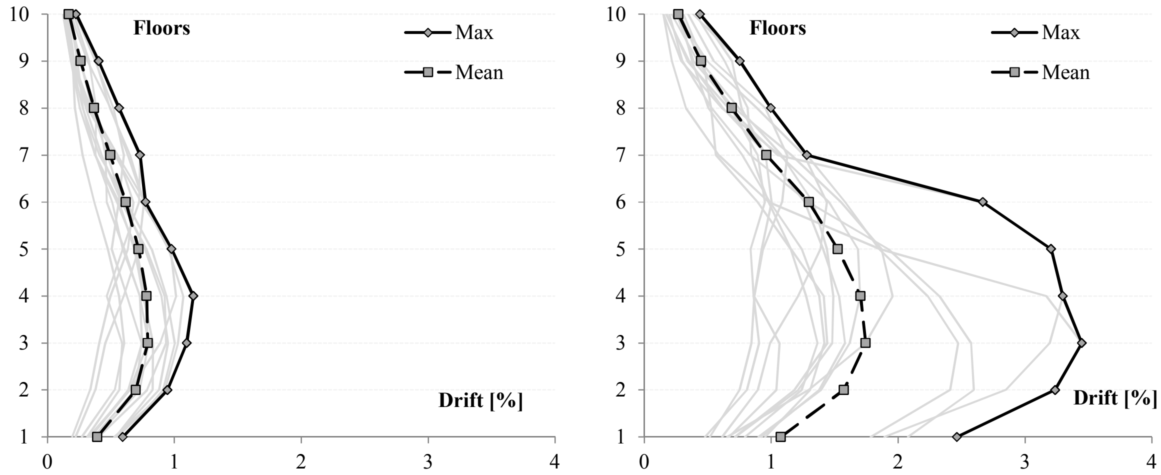

Figure 7: Interstorey drift of the bare frame building for DBE (left) and MCE (right)

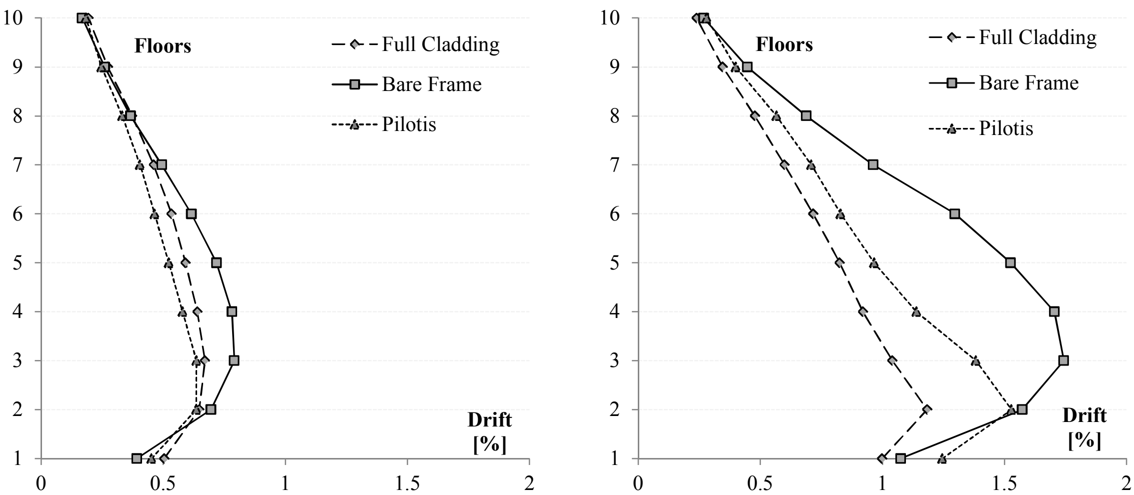

Figure 8: Mean interstorey drift of different building configurations for DBE (left) and MCE (right)

The differential displacement (or drift) between the cladding and the frame can also be found for each panel. This can then be compared to the damage limit states presented in Figure 3. Presented in Figure 9 is the likely damage state of the connections. Both the maximum and mean drift is shown for the FC and PI cases under DBE and MCE level earthquakes. 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.

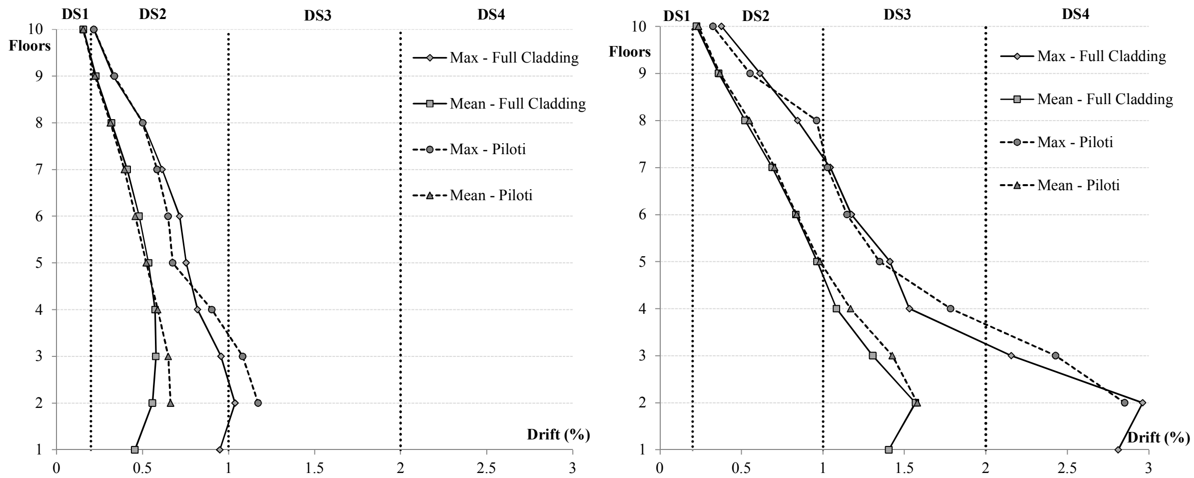

Figure 9: Differential displacement between panel and frame and corresponding connection damage state for DBE (left) and MCE (right)

A summary of the distribution of connection damage states is shown in Table 3. It can be seen that the difference in distribution of damage between FC and PI 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.

Table 3: Connection damage state distribution

Earthquake Intensity	Building configuration	DS1	DS2	DS3	DS4
DBE	Full Cladding	14%	85%	1%	0%
	Pilotis	15%	83%	2%	0%
MCE	Full Cladding	5%	61%	28%	6%
	Pilotis	4%	64%	29%	3%

5 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. This increases to around 95% in an MCE event. In an MCE event it is expected that 3-6% of connections will be at high risk of failure.

It should be noted that the results from this study are for one particular design that utilises tie-back connections to allow for relative in-plane movement between the cladding and the frame. The strength, stiffness and damage states of different tie-back connections, as well as other connection types, will all vary according to their design. Further research is currently being undertaken at the University of Canterbury to ascertain the sensitivity of the results presented here to different connection characteristics.

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