

Numerical study on the seismic interaction between 2D seismic resisting frames and claddings

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Summary

Damage to non-structural components during recent earthquake events, such as the Darfield earthquake in New Zealand (2010) have confirmed the need to better address the interaction between a structure and its non-structural components.

This paper aims to numerically investigate the seismic behaviour of a typical newly designed reinforced concrete multi-storey frame building with and without the interaction due to cladding panels. This interaction is investigated by means of non-linear static and dynamic analyses for common typologies of cladding systems. A seismic risk assessment analysis is also presented in order to develop fragility functions based on damage limit states for cladding connection. These are used for assessing the probability of damage of cladding systems after earthquake events of varying intensity.

Results confirm the high influence of cladding systems upon the seismic performance of multi-storey buildings. Also revealed is the significant variation in possible levels of cladding damage throughout a building. Further investigations are going to be developed, refining the use of fragility functions associated to innovative low damage cladding solutions.

Keywords

numerical modelling, non-structural elements, facades, cladding, seismic design, fragility curves

Theme

buildings – analysis – earthquake – concrete

1. Introduction

Reconnaissance following past earthquakes has shown that damage of non-structural elements during seismic events can cause significant economic losses and disruption to important or critical facilities. Furthermore, failures can result in potential hazards to pedestrians around the building. Recent earthquakes have further highlighted this concept, 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 ongoing research effort with the aim of better understanding the interaction between facade systems and the structure.

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Numerical models of cladding systems have been defined using previous experimental investigations and preliminary data from on-going experimental tests. A parametric analysis of a one-storey single bay frame clad with a precast concrete panel, fully presented in a companion paper (Baird et al. [1]), has been used as the foundation for this study on the seismic behaviour of multi-storey buildings with different cladding distributions. The interaction with the facade has been investigated by use of non-linear analyses utilizing both pushover and time-history analyses of typical cladding-structure systems.

In the first part of this paper, a parametric analysis has been performed, considering different distributions of precast concrete panels and structure heights. In the second part of the paper, a probabilistic risk assessment evaluation due to seismic hazard is presented for some of the claddings which compose the facade. This allows the development of fragility functions based on damage limit states previously identified in order to assess the probability of achieving defined damage levels of the cladding system.

2. Background

Recent studies on the interaction of cladding panels with the primary structure have underlined the need of understanding the influence of facades on the overall system (McMullin et al. [2], Baird et al. [1]). This behaviour is directly dependent on the cladding system analysed, in particular in relation to different connection types, as qualitatively presented in Table 1.

Table 1: Facade connection characteristics (Baird et al. [1])

FACADE	CONNECTION	BEHAVIOUR CHARACTERISTICS	STRENGTH	STIFFNESS	DUCTILITY
Cladding Panel	Tie-Back (Partially Fixed)	Deform easily under lateral forces. Must withstand out-of-plane forces, e.g. wind	Low	Low	High
	Slotted/Sliding/ Rotating	Disconnect the panel by allowing degree of freedom in one or more directions.	NA	NA	Medium
	Dissipative	Dissipate energy in connector body under lateral forces by yielding or friction.	Medium	Medium	Medium
	Fully Fixed (Bearing)	Transfer the self weight of the panel to the structure. No seismic characteristics.	High	High	Low

In order to assess the seismic response of multi-storey buildings with claddings, the solution of precast concrete panels attached to the beams by the use of tie-back (p. fixed) and fully fixed (bearing) connections has been herein adopted. Because of their low strength and stiffness with high ductility, the tie-back connections are the weakest level of the chain in the hierarchy of strength of the overall system. When this is the case it allows greater damping, strength and stiffness over many cycles as opposed to when damage occurs in the panel or frame. However this requires that the tie-back connections are designed to accommodate a large level of displacement demand or ductility (Baird et al. [1]).

Performance-based criteria for cladding connections are presented in this paper in order to determine the likely level of damage from varying earthquake intensity. This is in accordance with the shift towards a performance-based framework for both structural and non-structural system in newly designed buildings (Priestley [3]). The primary function of performance-based seismic design is the ability to achieve, through analytical tools, a building design that will reliably perform in a prescribed manner under one or more seismic hazard conditions (SEAOC [4]).

The in-plane performance of cladding panels are deemed to be sensitive to inter-storey drift (Taghavi and Miranda [5]) therefore the maximum drift of the connections is to be monitored in order to compare damage limit states. It is important to note that the out-of-plane performance (as well as in-plane to some degree) of cladding panels is sensitive to acceleration, for this reason, this paper does not fully encompass all possible

cladding panel damage and failure mechanisms. The probability of damage of the connections has been evaluated following the performance based earthquake engineering (PBEE) methodology used by Cornell et al. [6].

The use of the Incremental Dynamic Analysis (IDA) procedure proposed by Vamvatsikos and Cornell [7] is able to define the probability of different damage limit states being reached. This is done by subjecting a structure to a suite of earthquake accelerograms with the intensity level varied from a low to high level. This study builds on a defined procedure for such an analysis which is being extended and validated for non-structural elements (Stojadinovic and Hunt [8]).

3. Case study: multi-storey frame systems

The case study proposed is based on the Red Book building (New Zealand Concrete Society [9]) which acts as a design example of the New Zealand Concrete Code (NZS 3101 [10]). The building was originally designed for the city of Christchurch, but for this study the building has been assumed to be located in the higher seismicity site of Wellington in order to achieve larger damage. Figure 1 (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 do not affect the bare frame behaviour. The bottom floor has a storey height of 4m while the upper floors have a storey height of 3.6 m for a total of 10 levels. Design loads, forces and seismic masses have been calculated according to NZS1170:1 [11] and NZS1170:5 [12] following Force Based Design (FBD) methodology.

In the first part of this paper (SECTION A) different distributions of precast concrete panels were considered as well as various heights of the frame (5-10-15 storeys). 5 and 15 storey buildings have been adapted using the same section properties of the 10 storey building. Two possible architectural cladding configurations have been considered: Full Cladding (FC) and Pilotis (PI), compared to the Bare Frame (BF), (Figure 1-right). Full cladding consists of cladding panels in every bay in every storey of the frame and pilotis is the same without panels at the first floor.

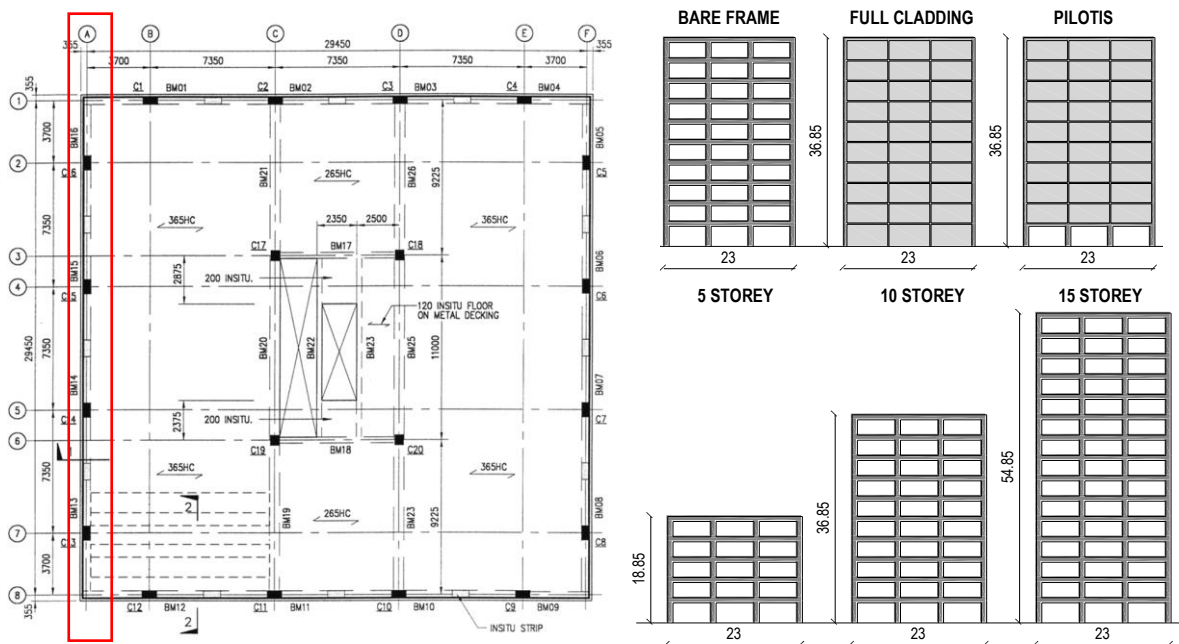


Figure 1: Plan view of the building (left, [9]) and claddings distribution/frames configurations (right)

In the second part of the paper (SECTION B), a probabilistic risk assessment evaluation due to seismic hazard is presented for facade panels basing on the 10 storey cases (FC and PI) in relation to three cladding connections at different floor level.

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. 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 system has been designed considering a suggested drift of 0.3% (Baird et al. [1]). For this study, damage limit states are related to the behaviour of the tie-back connections based on various experimental behaviour (McMullin et al. [2], Stojadinovic and Hunt. [8]) and first outcomes from the experimental test which is currently taking place in the laboratory of Civil Engineering of University of Canterbury (New Zealand). The damage limit states for the connections have been presented as the drift of the connection or 'connection drift'. The connection drift is defined as the relative displacement of the connection divided by inter-storey height. Four damage limit states are herein proposed and shown in Figure 2. 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 the boundary separating DS2 and DS3 should be defined as a level of damage which would cause loss of function and repairs are needed to restore the full functionality of the structure. Below this boundary, damage (categorised as DS2) is considered to be slight and tolerable. Whereas damage suffered in DS3 is significant such that the elements are not likely to perform their function as evidenced by:

- excessive permanent drift at the end of the earthquake;
- excessive damage to connection due to effects such as local buckling

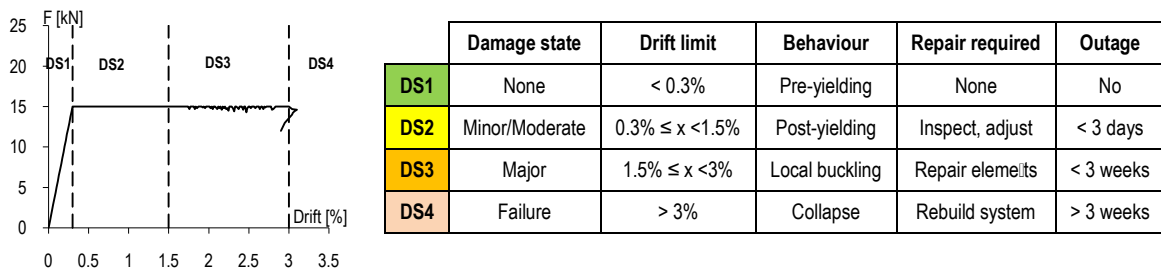


Figure 2: Proposed damage limit states related to tie-back connections

3.2 Modelling issues

The models have been implemented using the programme RUAUMOKO (Carr [13]). Beams and columns have been represented by elastic elements with inelastic behaviour concentrated in plastic hinge regions (Giberson model) and defined by the moment curvature hysteresis rule 'Modified Takeda' (Otani and Saka [14]). Precast concrete panels have been modelled as quadrilateral elastic elements, while the connections have been considered as springs associated to a non-linear rule and attached directly to the beams in two points, as shown in Figure 3. All the connections are characterised by a bi-linear elasto-plastic rule. The top (tie-back) connections have high ductility without strain hardening and the bottom (bearing) connections have high strength and high stiffness with the intention of them to remain elastic.

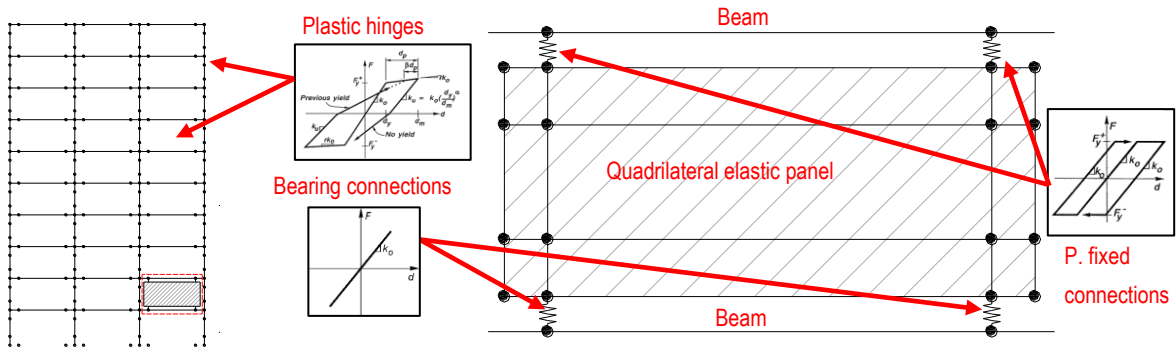


Figure 3: Modelling of the frame (10 storey case) and cladding panel with hysteretic rules used

4. SECTION A: numerical investigation

In the following paragraphs, the results from the non-linear pushover and time-history analyses are presented for the different configurations described in section 3.

4.1 Pushover analyses

Static non-linear pushover analyses of the models were performed to investigate the lateral base shear and top displacement relationship of the building. The analyses compare the behaviour of the systems under a triangular distribution of the forces acting up the height of the building meant to represent earthquake demand. In Figure 4(left) the monotonic response of the 10 storey building case (base shear vs. top drift) is shown, representing the three different configurations analysed: Bare Frame, Full Cladding and Pilotis. As expected, an increase in stiffness and strength is observed for FC and PI cases compared with BF due to the presence of the elastic cladding panels.

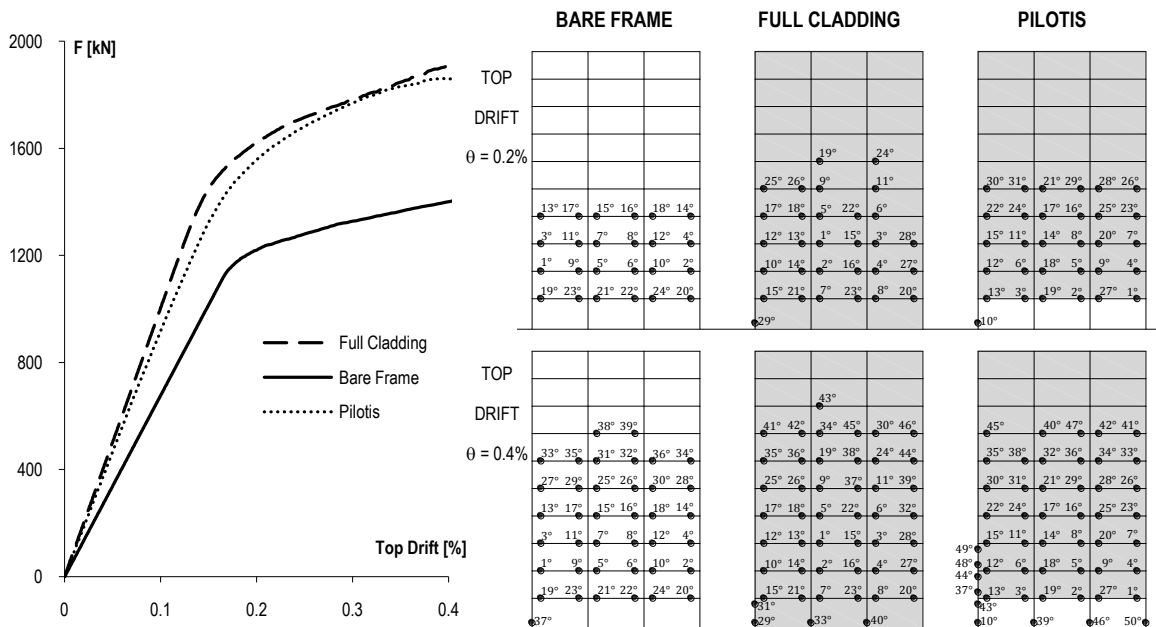


Figure 4: Pushover analysis (10 storey case) - monotonic response (left); comparison of the damage mechanism with activated plastic hinges at 0.2% and 0.4% top drift (right)

In Figure 4(right) the activation of the plastic hinges is shown for the three configurations at two different values of top drift; 0.2% and 0.4% respectively. FC presents extensive formation of plastic hinges at the second/third floor levels, while the absence of claddings at the ground floor in the PI case results in a higher shear demand at the ground floor level which can lead, as expected, to a soft-storey mechanism at that level.

For all the multi-storey frames studied (5-10-15 storeys), the effects of claddings for both the FC and PI cases are investigated and summarised in Table 2 comparing the initial stiffness increment with the BF stiffness.

Table 2: Pushover analysis - stiffness increment compared with Bare Frame case

Building configuration	5 STOREYS		10 STOREYS		15 STOREYS	
	Full Cladding	Pilotis	Full Cladding	Pilotis	Full Cladding	Pilotis
Increase in initial stiffness respect to the bare frame $\frac{(K_x - K_{BF})}{K_{BF}}$	+702 %	+217 %	+49 %	+38 %	+34 %	+23 %

The initial stiffness ratio between the system with claddings (K_x) and the bare frame (K_{BF}) is higher, especially as the height of the frame decreases, underlining the positive contribution of the claddings. In particular, the 5 storey case presents a large increase compared with the other configurations, especially for the FC configuration. This can likely be attributed to the fact that the 5 storey frame has not been designed as a 5 storey frame but has been derived from the 10 storey frame. The consequence of this is that low frame height compared to the frame width (length of the base) means the frame is may behave like a squat frame.

4.2 Time-history analysis

Time-history analyses have been performed investigating how the variables considered (panel distribution, building height) can affect the response of the building. A suite of fifteen recorded and scaled natural accelerograms have been used (Pampanin et al. [15]). The records have been scaled according to NZS1170:0 [16] and NZS1170:5 [12], considering $S_a=0.4g$ (Wellington, soil type C) as shown in Figure 5. A period range of interest has been defined between T_{min} and T_{max} where $T_{min}=0.4T_1$ and $T_{max}=1.3T_1$. T_1 is the fundamental period of the structure equal to 1.665s (mean of the fundamental periods of every system studied in the parametric analysis).

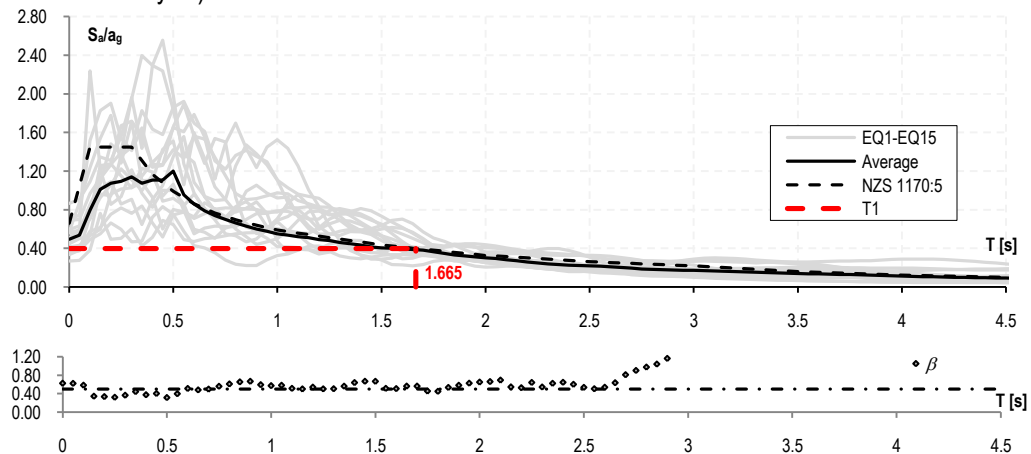


Figure 5: Scaled spectra to $S_a=0.4g$ at the fundamental period T_1 and lognormal coefficient of variation β

The lognormal coefficient of variation is plotted in order to show the dispersion across the spectrum. Because its value is around 0.45 provided that the period is less than about 2.6s, the scaled suite is satisfactory (Figure 5). According to FEMA-302 [17], 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 [4]), 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.

In Figure 6 the results related only to the 10 storey case are shown. Figure 6(left) presents the maximum

interstorey drift for BF, FC and PI. In correspondence of the second floor the highest values of drift are concentrated for all the three configurations analysed. This result is reflected in Figure 6(middle) and Figure 6(right) where the maximum and the median drift are plotted for every floor of the tie-back connections. In relation to the damage limit states presented in Figure 2. The claddings of the PI case are particularly affected by the earthquake, revealing the possibility of reaching at least in one case a maximum drift which can lead to the failure of the system.

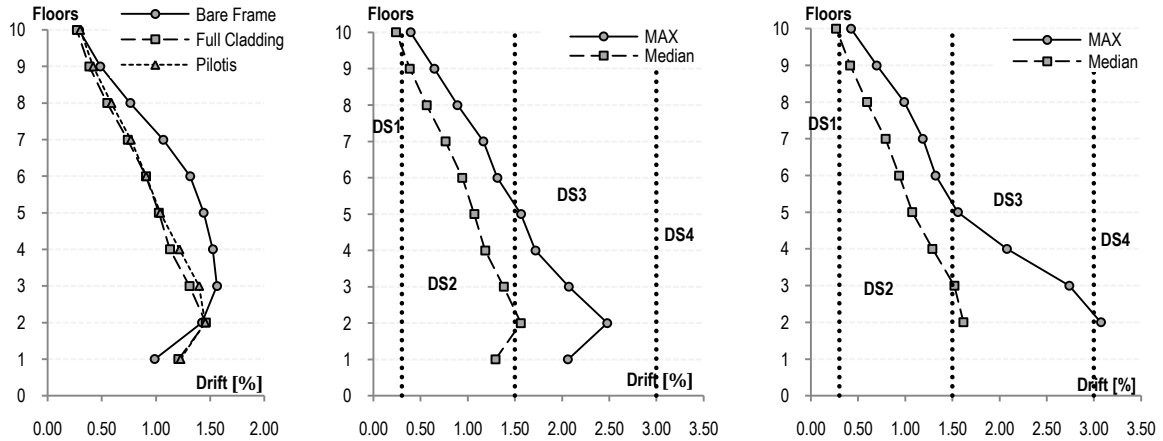


Figure 6: Maximum interstorey drift of the building (left), maximum and median drift of the cladding connections per storey (FC – middle, PI – right) for Design Basis Earthquake (DBE) intensity

Table 3 represents a summary of the results related to the connections obtained from the time-history analyses. Regardless of the number of storeys or the configuration type, the highest connections drifts are always localised at the second floor of the building. These results give a clear indication that the level of damage to claddings can be high, even for a DBE.

Table 3: Median of the maximum drift of the tie-back connection θ_{max} for the FC and PI cases under DBE and MCE intensity

	FULL CLADDING		PILOTIS	
	DBE θ_{max} [%]	MCE θ_{max} [%]	DBE θ_{max} [%]	MCE θ_{max} [%]
5 storeys	1.70 (1 st floor)	2.49 (2 nd floor)	1.67 (2 nd floor)	2.56 (2 nd floor)
10 storeys	1.56 (2 nd floor)	2.40 (2 nd floor)	1.62 (2 nd floor)	2.55 (2 nd floor)
15 storeys	1.26 (2 nd -3 rd floor)	2.02 (2 nd floor)	1.38 (2 nd floor)	2.05 (2 nd floor)

5. SECTION B: risk assessment analysis for cladding systems

In order to understand the probability of damage of some of the cladding connections analyzed in the previous section, a seismic risk assessment is presented. The performance of the cladding system under seismic load can be estimated using the Incremental Dynamic Analysis (IDA) procedure proposed by Vamvatsikos and Cornell [7]. IDA allows the evaluation of various outcomes for given hazard intensity levels, such as the Design Basis Earthquake (DBE) or the Maximum Considered Earthquake (MCE) already described in the last section. The 15 accelerograms presented previously are scaled from 0.1g to 1g with a step of 0.1g in relation to their spectral acceleration (S_a). S_a has been considered as the Intensity Measure (IM) as opposed to Peak Ground Acceleration (PGA) because it has been observed from past researchers to be more appropriate (Shome and Cornell [18]). In total, 150 analyses have been performed for every connection analysed,

considering the maximum relative displacement between the frame and the cladding (cladding connection drift) as the Engineering Demand Parameter (EDP). The damage limit state of the tie-back connections defined in Figure 2 has been defined as the Damage Measure (DM). Three connections are herein considered for the 10 storeys building already presented with two different cladding panels distributions (FC and PI). The choice of the connections is related to the consideration from the analyses in Section A, as described in Figure 7.

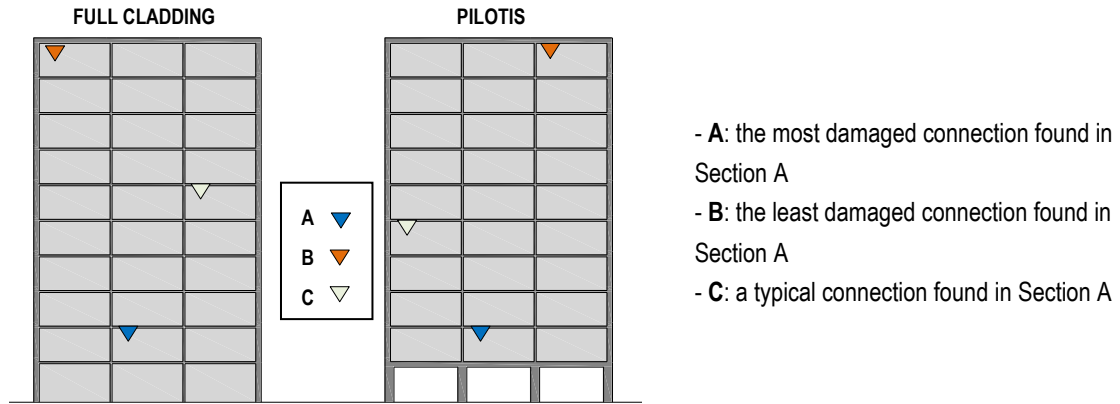


Figure 7: Cladding connections monitored for the risk assessment analysis

5.1 Derivation of fragility curves

The results of the IDA for the cases defined above are used to find the probability of reaching/overcoming each damage limit state as a function of the IM. Because non-structural elements are dependent on not only their own performance but the performance of the primary structure, the fragility curves have been prescribed to take into account of the occurrence global collapse using Eq. 1. This concept has been widely described by Jalayer [19] and is commonly referred to as “total probability”.

$$P[EDP_i > edp|S_{a_i}] = P[EDP_i > edp|NC, S_{a_i}] \cdot P[NC|S_{a_i}] + P[EDP_i > edp|C, S_{a_i}] \cdot P[C|S_{a_i}] \tag{Eq. 1}$$

Where edp and EDP are the Engineering Demand Parameters (damage limit state obtained from the analysis), S_a is the Spectral Acceleration. NC represents ‘No Collapse’ while C is ‘Collapse’. For this study the condition of C (collapse) has been represented by FEMA recommended drift limit for ‘Collapse Prevention’ of a concrete frame of 4% drift (FEMA 356 [20]). This therefore assumes that the structure either collapses or is damaged beyond repair and will need to be demolished if it reaches a drift of 4% or greater. Figure 8 shows the contribution of the irreparability of the building condition.

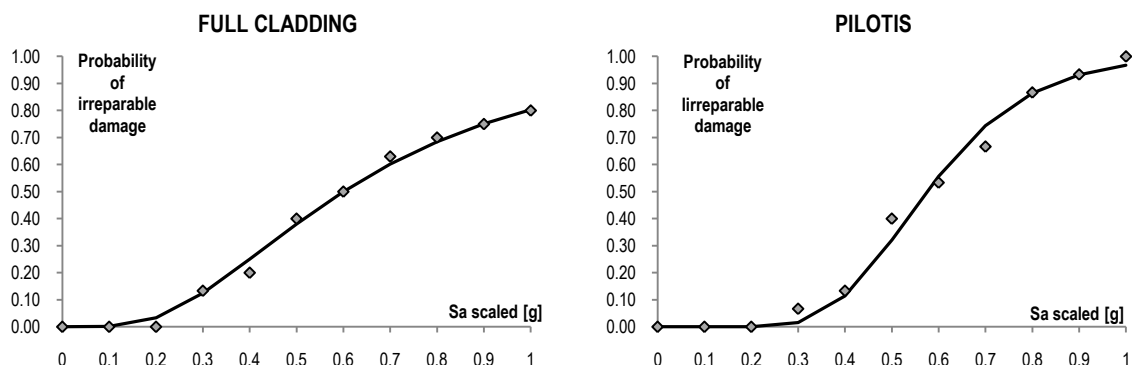


Figure 8: Lognormal probability of occurrence collapse/irreparability of the FC and PI buildings

This condition becomes particularly significant from 0.4g onwards when the probability overcomes 10-15%. At 0.6g half of the models are considered to be irreparably damaged for both building types. At 1.0g, the probability of the buildings overcoming the irreparability limit is around 80% and 95% for FC and PI respectively. Fragility curves for the connections can be derived based on these considerations. The graphs related to the connections A and B of the FC case are shown in Figure 9 using a lognormal cumulative distribution for their representation.

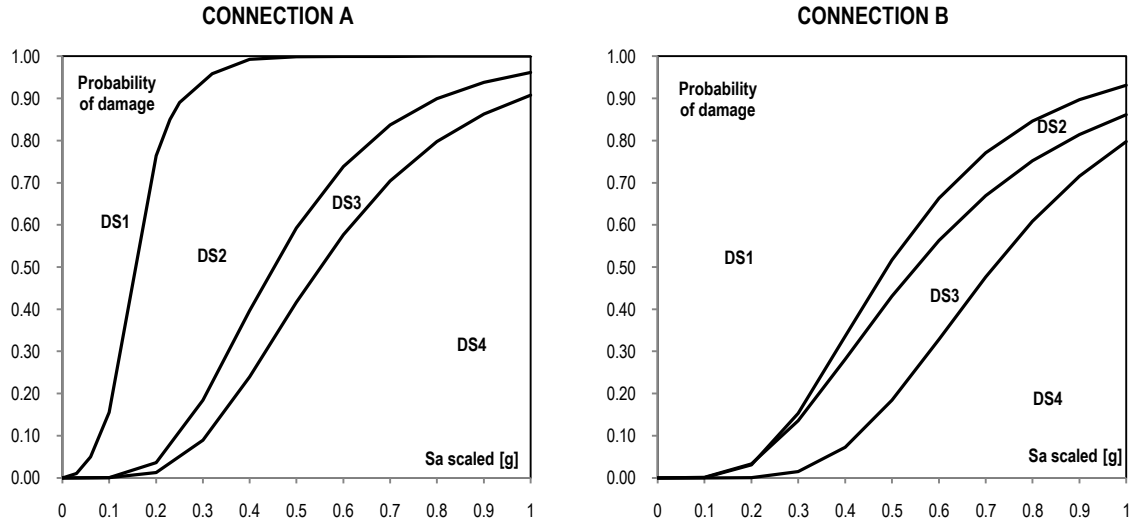


Figure 9: Fragility curves of the connections A and B (FC case)

As expected, Figure 9 shows that the position of the connection up the building has a large influence on the probability of damage. Connection A has a higher probability of damage compared with Connection B. For example at 0.6g, Connection A has a 57% probability of being in DS4, while Connection B this probability is only 32%. The comparison between Connection A and B highlights how the failure of the connections over the 8th floor is largely attributed to the irreparable damage of the building rather than to the failure of the actual connections.

5.2 Quantitative risk assessment

In order to determine damage probabilities of the cladding system in relation to annual frequency, the seismic performance predicted through IDA is required to be combined with the consensus probabilistic seismic hazard map applicable to the location. Fragility curves are herein re-plotted by changing the horizontal axis from the Intensity Measure IM to the annual probability p_a or return period in years, using the following seismic hazard relationship as a function of p_a (Maniyar et al. [21]):

$$p_a = \sqrt[q]{\frac{S_a^{(T=475)}}{S_a^{(T=T_r)}}} \cdot \frac{1}{475} \quad \text{Eq. 2}$$

Where $S_a^{(T=475)}$ and $S_a^{(T=T_r)}$ are the Spectra Acceleration at the natural period of an earthquake with respectively return period of 475 years and with return period T_r (the target S_a). The parameter q is dependent of the local seismicity (0.333 for New Zealand).

Using this relationship it is possible to obtain the probability of the defined damage states not being exceeded for earthquake size of various annual probabilities. Based on the annual probability or return period of the earthquakes (in particular, DBE and MCE), Figure 10 shows the likelihood of the induced damage being within the limits of the four possible damage states for the connections A and B (FC case).

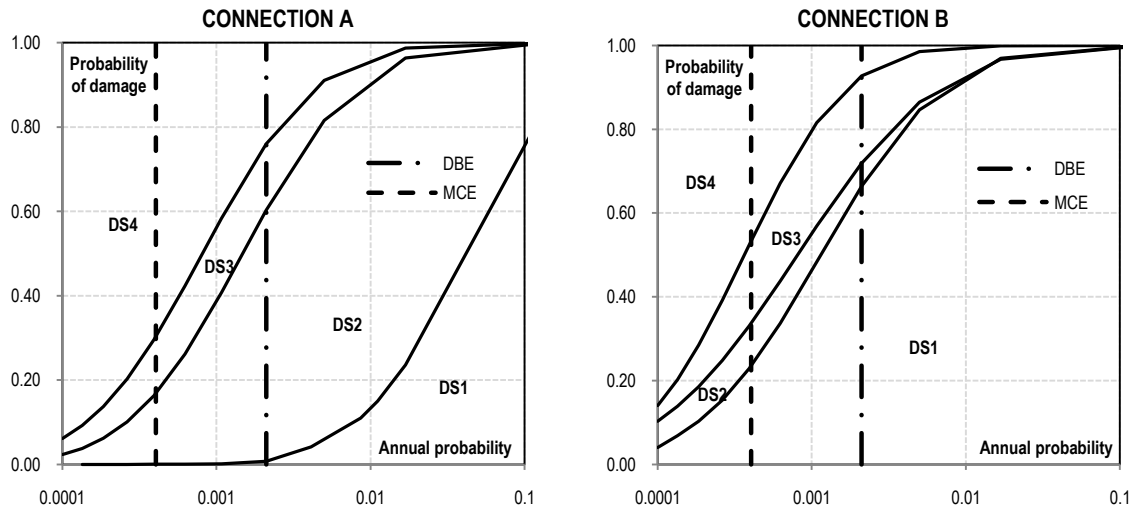


Figure 10: Quantitative risk assessment of connection A and B (FC case)

The information shown in Figure 10 can be translated into tabular form as shown in Table 4. The table shows the probability of the damage states not being exceeded (*a*) and of being in a given damage state (*b*) for both the connections A and B after DBE and MCE intensity earthquakes. For example, in case (*a*) if an earthquake of annual frequency equal to DBE strikes, the probability of DS1 not being exceeded is 1% for the connection A, while 66% for the connection B. This highlights how connection B has a very high probability to not being damaged (DS1 = 66%) compared with A which will almost certainly be damaged (>DS1 = 99%).

Table 4: Probability of damage not being exceeded and of being in a given damage limit state for connection A and B (FC case) with earthquake of DBE and MCE intensity

CONNECTION	DAMAGE NOT BEING EXCEEDED (<i>a</i>)					BEING IN A GIVEN DAMAGE LIMIT STATE (<i>b</i>)				
	P_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]
A		P[DS1]	P[DS2]	P[DS3]	P[DS4]		P[DS1]	P[DS2]	P[DS3]	P[DS4]
	DBE	1%	60%	76%	100%	DBE	1%	60%	16%	24%
	MCE	0%	17%	30%	100%	MCE	0%	17%	13%	70%
B		P[DS1]	P[DS2]	P[DS3]	P[DS4]		P[DS1]	P[DS2]	P[DS3]	P[DS4]
	DBE	66%	72%	93%	100%	DBE	66%	5%	21%	7%
	MCE	23%	34%	53%	100%	MCE	23%	10%	20%	47%

In case (*b*), if for example an earthquake with annual frequency equal to MCE strikes, there is a 0% probability that the damage state will be in DS1 for the connection A while for the connection B there is a 23% probability. The probability of being in DS2 is respectively 17% and 10% and so on. Thus, if the intensity of the earthquake is MCE there is the certainty that connection A will be at least damaged in minor/moderate way (DS2). Considering the two configurations (FC and PI) and all the connections monitored, it is possible to present a summary table (Table 5) where all the results are interpreted in terms of the proportion of a large number of cladding connections with the same characteristics in similar buildings likely to undergo different levels of damage in a seismic event of given intensity.

Table 5: Probability of experience the same damage limit states in proportion of large number of cladding connections with same characteristics in similar buildings

	FULL CLADDING						PILOTIS					
	A		B		C		A		B		C	
	DBE	MCE	DBE	MCE	DBE	MCE	DBE	MCE	DBE	MCE	DBE	MCE
DS1	1%	0%	66%	23%	0%	0%	0%	0%	64%	16%	0%	0%
DS2	60%	17%	5%	10%	65%	28%	64%	16%	11%	9%	75%	24%
DS3	16%	13%	21%	20%	19%	20%	11%	9%	10%	9%	11%	7%
DS4	24%	70%	7%	47%	15%	52%	25%	76%	15%	66%	14%	69%

Each column adds to unity so each connection must be in a given damage state. If for example it is desired to know the probability of being in DS2 for a population of 1000 similar buildings (FC case) subjected to an earthquake with DBE intensity, about 600, 50 and 650 (connections A, B and C respectively) are likely to be in DS2. The same reasoning can be made for another population of 1000 buildings (PI case) with the same conditions just described. This time 640, 110, 750 (connections A, B and C respectively) are likely to be in DS2.

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 40 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 pilotis case exhibits a soft-storey mechanism as expected, but in general the maximum inter-storey drifts are concentrated on the first three floors. This trend does not appear to be dependent upon the number of floors or the distribution of claddings. The stiffness ratio between the claddings and the structure is higher, especially as the height of the frame decreases, underlining the positive contribution of the claddings.

Based on the results collected in the first section, a seismic risk assessment analysis was also presented in the form of fragility functions. These were based on damage limit states for cladding connections related to the differential displacement between the structure and the cladding. This revealed the significant variation in possible levels of cladding damage throughout a building. The level of damage in the connections is similar for the first 8 and 6 floors for the Full Cladding and the Pilotis case, respectively. Hereafter, the failure of the upper connections is largely attributed to the irreparable damage of the building rather than to the failure of the actual connections. According to the fragility curves presented, the main differences are observed in damage states DS1 and DS2, which is in relation to the differential damage up the height of the building. For example for a DBE level earthquake, the probability of an upper level connection being damaged is 66%, compared with a lower level connection which has only a 1% probability of not being damaged.

The use of a 2D model meant representing out-of-plane failure due to high accelerations was not considered. Therefore the analyses only took into account drift related damage and failure of cladding panels. In order to expand upon these results, analyses which include the sensitivity to both drift and acceleration are suggested.

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