Numerical modelling of local cladding-structure interaction

Andrew Baird [1]; Riccardo Diaferia [2]; Alessandro Palermo [3]; Stefano Pampanin [4]

Summary

As highlighted by many recent earthquakes, including the Darfield earthquake in New Zealand (2010), damage to non-structural components contributes significantly to the overall earthquake damage and costs. Understanding the interaction between a structure and its non-structural components is critical in order to reduce the damage to non-structural components during an earthquake event.

This paper presents a numerical investigation into the local interaction between cladding systems and moment resisting frames utilizing lumped plasticity models of the cladding connections based on a two-dimensional finite element model. The research is part of a larger coordinated research programme which aims to reduce the damage to all non-structural components during earthquake events.

The modelling exemplifies the different failure mechanisms that can result due to cladding-structure interaction. Results confirm that common design methods which neglect cladding interaction are inaccurate. The authors intend to continue the research to successively develop improved and innovative low damage cladding-moment resisting frame systems. They also aim to produce simple design tools that provide easy inclusion of the effects of cladding-frame interaction to the seismic response.

Keywords

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

Theme

buildings - analysis – earthquake – concrete

1. Introduction

Non-structural elements in most buildings represent a major portion of the total investment cost of the building. The non-structural investment costs (including contents) for a typical office are 82% and for hospitals up to 92% (Taghavi et Miranda [1]), as shown in Figure 1. As well as this, non-structural elements are typically more vulnerable to seismic damage than structural elements. Of the 66,000 buildings damaged by the 1994 Northridge earthquake, approximately three quarters of the buildings suffered damage to only non-structural elements alone (Charleson [2]). The result of this is that the direct and indirect costs associated with damage of non-structural components can be significantly more than the costs associated with the damage to the structure itself after an earthquake. Furthermore, damage to non-structural components, such as that shown in Figure 1, can pose a serious risk to the safety of people inside and outside the building during an earthquake. In order to develop and propose practical and efficient solutions that reduce the risk of damage to non-structural components it is necessary to understand how they interact with a structure. In addition, determining which parameters are most influential in this interaction is essential so that all possible damage and/or failure mechanisms are identified.

This paper presents the results of a numerical study on the interaction between precast concrete cladding panels attached to a reinforced concrete moment resisting frame. This is achieved using static push-over and cyclic push-pull analyses of a lumped plasticity model representing an interior single-storey, single bay of a

[1] PhD Student, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, andrew.baird@pg.canterbury.ac.nz

[2] Master Student, Department of Building Environment and Science Technology, Politecnico di Milano, Milan, Italy, riccardo.diaferia@mail.polimi.it

[3] Senior Lecturer, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, alessandro.palermo@canterbury.ac.nz

[4] Associate Professor (Reader), Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, stefano.pampanin@canterbury.ac.nz

multi-storey building. In order to better understand the different possible damage and failure mechanisms several parameters of the systems are varied. This paper is accompanied by a companion paper which further explores the global effects of the cladding-structure interaction to different multi-storey buildings (Diaferia et al.[3]).

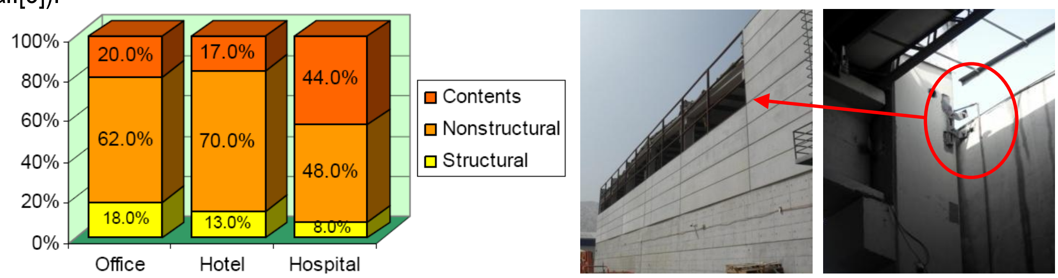

Figure 1: Investment costs for different building typologies (Taghavi et Miranda [1]), example of cladding failure

2. Background

Precast concrete panels are widely used around the world as an exterior cladding for multi-storey buildings. Such cladding can be considered as non load bearing wall systems which are designed primarily to transfer their self-weight and out-of-plane (wind and earthquake) lateral loads to the supporting building structure. The contribution of the cladding system to the lateral stiffness of the building is often ignored in the structural design. However, experimental investigations on newly designed buildings have shown that claddings can contribute significantly to the lateral stiffness of the structure and that the panels can be subjected to significant in-plane forces (Goodno et al. [4]) which might cause unexpected structural failure. In order to avoid this unintended interaction, it is possible to isolate cladding panels, as shown by research using autoclaved lightweight aerated concrete (ALC) panels typically used in Japan (Okazaki et al. [5]). ALC panels can be connected using sliding and rotating connections, as shown in Figure 2, such that they contribute very little to the stiffness and strength of the overall structure, even under very large inter-storey drifts of 0.04 radians.

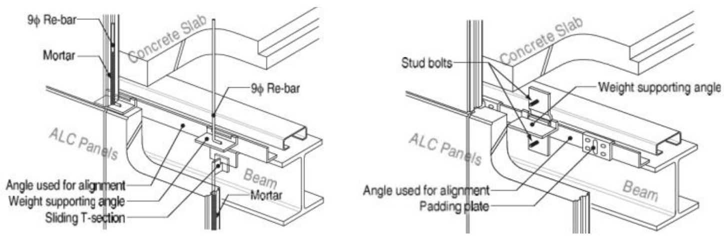

Figure 2: Sliding panel (left) and rotating panel (right), (Okazaki et al. [5])

Complete isolation from the structural system does however mean that the cladding is simply a dead weight. Consequently, investigations have been carried out into ways in which the structure can profit from having cladding panels attached. If the additional stiffness and strength that cladding panels provide is utilized during design then a saving of up to 25% in the volume of steel used in the structure can be achieved (De Matteis [6]). Cladding panels can also be used to provide passive control for the seismic behaviour of buildings with the use of energy dissipative connections. Results show that energy dissipative cladding connections like that shown in Figure 3 could provide the total hysteretic energy required of the structural system (Pinelli et al. [7]).

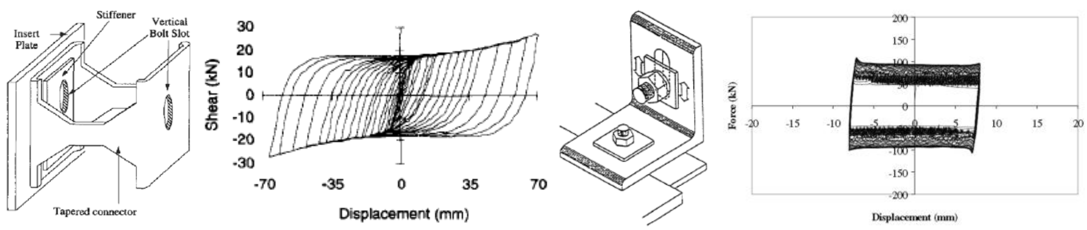

Figure 3: Flexural (left) and frictional (right) dissipative connection with hysteretic loops (Okazaki et al. [7], Lukkunaprasit et al. [8])

The main emphasis of research on cladding systems to date is on detailed research into isolated facade technology. This therefore leaves open the question of whether benefits found can be applied if some parameter is slightly different. This paper aims to help in providing a way to broaden the results of such research so it can be applied to a range of similar facade technology.

2.1 Assessing Seismic Performance of Facade Systems

Assessing facade performance is a complex and broad exercise. One particular performance aspect cannot be examined without taking into account the numerous other functions of a building's facade at the same time. Therefore, while trying to define and ultimately improve the seismic performance of facade systems, it is very important not to neglect the other aspects of facade performance whilst doing so (Palermo et al. [9]).

In assessing the overall performance of a building, traditional design methods tend to consider the structural and the non-structural systems separately. When the behaviour of the bare structure is affected by the interaction with the facade, e.g. additional stiffening and hysteretic damping or more importantly change of collapse failure mechanism, then it becomes more appropriate to assess the performance of the combined system rather than the systems separately. Both of these approaches are shown in Figure 4a using a simplified chain diagram.

Assuming that the cladding system is comprised of a structural frame member, a connector body and cladding panel, linked together with strong and stiff attachments, as shown in Figure 4b, then the problem can be simplified in order to determine where failure is most likely to occur. An advanced performance-based design would thus target the most desirable hierarchy of strength and sequence of events of the overall system.

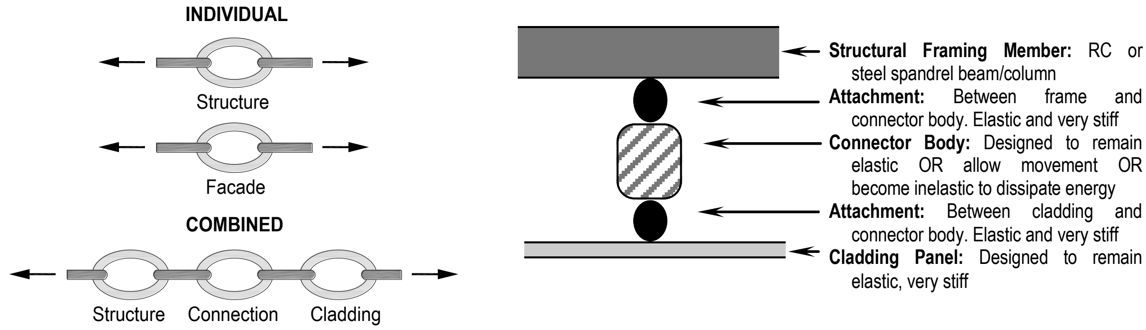

Figure 4: (a) Simplified chain diagram for structure-facade system of a building;
(b) Structure – cladding system (Cohen [10]).

2.2 Hierarchy of Strength

When a combined system approach is taken, capacity design principles can be used to define a number of different possible failure scenarios, as shown in Figure 5. These depend upon the hierarchy of strength of the elements comprising the facade-structure system, easily defined by the in-plane yield strength of each component.

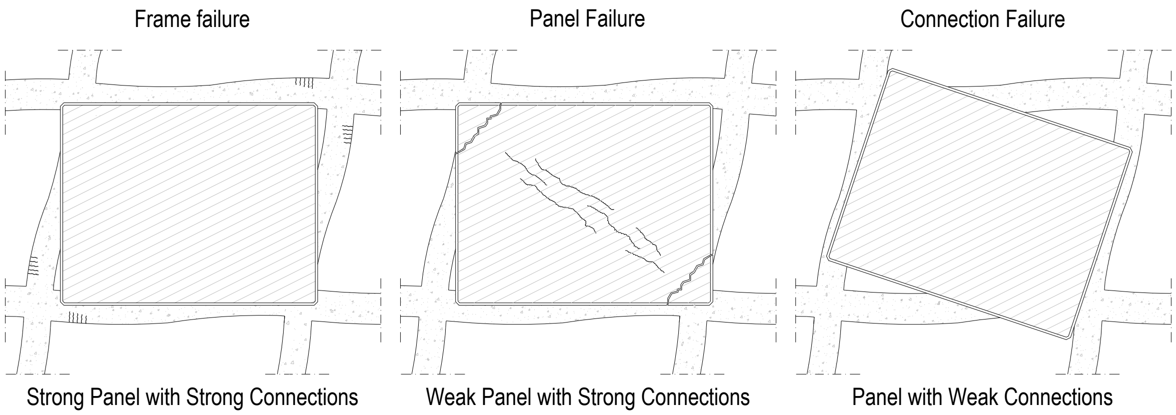

Figure 5: Different failure mechanisms and push-over behaviour of precast panels attached to a frame system

The weakest element in the system (weakest link of the chain) will determine the overall cladding failure mechanism. For example, if the connector body strength is weaker than the frame, attachment and cladding panel, then the system strength is dependent upon the connection strength and the rest of the elements strength are arbitrary. The same is true of stiffness: the lowest stiffness governs the force that can be carried by the cladding at any level or inter-storey drift.

It is usually assumed that the 'attachments' of the connector body are stronger than both the cladding and the connector body itself so it can be simplified to a system that consists of frame – connector body – cladding panel. From herein the connector body is referred to as the 'connection'. Despite the simplicity of these design principles, errors have been made in the past where the attachment ends up being the weakest link in the system. When the attachment governs the failure then the risk of falling panels is very high.

There is also the highly undesirable case where the panel, connector and attachments are stronger than the frame member and failure occurs in structural members due to local interaction. This scenario is unlikely on a local scale; however the chance of global weakness due to cladding interaction is a real possibility. The type of connection used can have a large influence on the type of behaviour observed (Figure 6). This is due to the connection typically being the weakest and least stiff component in the system. This means that the forces that are carried in the cladding are normally restricted by the connection properties. Table 1 shows different facade connection characteristics for cladding panels in terms of their strength, stiffness and ductility.

Table 1: Facade connection characteristics

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

3. Numerical Investigation

The numerical investigation has been considered using a monolithic single bay, singly storey reinforced concrete frame with the presence of a single precast concrete cladding panel. The setup is based on a corresponding experimental program that is currently undergoing considering cladding typologies and various cladding panel/connection configurations. The cladding system is designed to yield at a relative displacement between the cladding and the frame using the suggested drift limit of 0.3%.

The numerical investigation only considers a single panel and all openings in the panels are herein neglected. The model has been implemented using the program RUAUMOKO (Carr [11]), based on a lumped plasticity model, with the beams and columns 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 et Sake [12]).

The aim of the numerical investigation is to understand the different behaviour that can result from cladding-structure interaction as a result of varying the hierarchy of strength. As well as this, a parametric analysis will be explored in order to understand the level of which other parameters affect the behaviour of the facade-structure system.

3.1 Cladding Strut Model

If the cladding connection is assumed to be stronger than the panel or frame, then the failure mechanism is governed by the strength of either the panel or the frame. When this is the case it is essential that the panel is modelled with appropriate non-linear behaviour. Diagonal springs are herein presented to represent the compressive struts of the panels as is typically done for infill panels (Crisafulli [13]). The general model and force displacement behaviour for panel governed failure and frame governed failure is shown in Figure 6.

The implementation of this rule requires several parameters which are based largely on experimental data. The most critical parameter dictating the behaviour of the strut is the compressive strength. This does not represent the standard compressive stress of the concrete but rather of the strut. Hence it should take into account the inclination of the strut and expected mode of failure (Crisafulli [13]). Accordingly, without experimental data to verify the accuracy of a panel of particular geometry and materials, the precision of this parameter is, at this stage, likely to be limited. However, the strut strength has been estimated assuming that panel failure consists of crushing of the concrete where the strut bottlenecks into the panel connection locations. Values which have been used refer to Brown et al. [14]. The strut's compressive strength is varied as part of the investigation in order to appreciate the transition between panel governed failure and frame governed failure, as shown in Figure 6. The equivalent panel strength is varied from a low strength, typical of aerated concrete, up to a high strength (from 4 MPa to 30 MPa). The 'medium' strength panel represents the failure transition point from panel failure to frame failure.

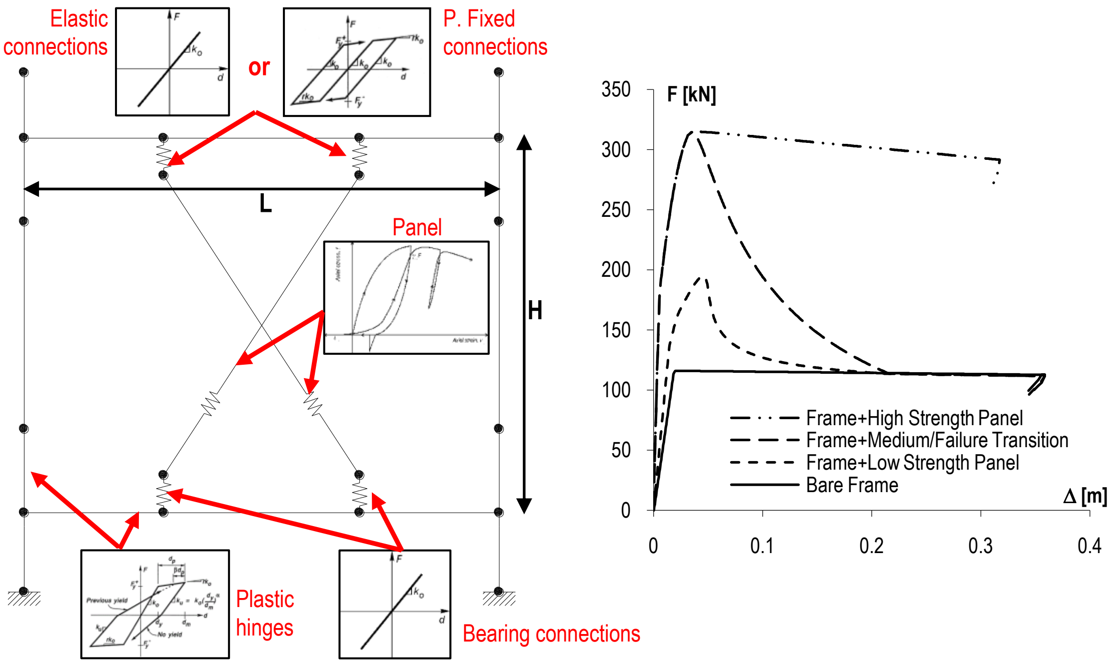

Figure 6: Panel/frame governing model (beam connection case) with force-displacement behaviour

The panel element has four connections; two bearing connections and two seismic connections. The bearing connections (henceforth fully fixed connections) are located on the lower half of the panel and are intended to transfer the self weight of the panel to the structure. The seismic connections (henceforth tie-back-partially fixed or slotted connections) are located on the top half and are intended to deform under in-plane loading whilst providing out-of-plane support. Each connection is modelled by a spring member with horizontal and vertical stiffness. For the bearing connections, the connector elements in both horizontal and vertical directions are assumed to be very stiff and very strong. The stiffness and yield strength of the seismic connections are varied in order represent different possible setups.

Initially the seismic connections are assumed to be infinitely stiff and to remain elastic. This assumption is valid for panels of low strength, where strength hierarchy dictates that panel failure governs; however, for higher strength panels, the force transmitted through the connections becomes very large and would be unrealistic if a typical cladding connection is used. As such, the yield strength of commercially available connections is then introduced. The behaviour of the connections is based on experimental testing of several connections used to support precast concrete cladding (McMullin et al. [15]). The connections are treated as bi-linear, either having post-yielding stiffness or not to represent strain hardening effects. The model does not take into account the possibility of a shear failure mechanism in the joint induced by the panel.

3.2 Model Results

The cyclic behaviour of the cladding-structure systems is shown in Figure 7. The push-pull analyses presented are performed at repeating cycles of 4% drift. The graphs present the combined behaviour of the frame and the cladding as well as the bare-frame result. Where there is non-linear behaviour in the panel this is also presented, otherwise it is omitted for clarity. When the panel is weak and the connections are assumed to be infinitely stiff and elastic, it can be seen in Figure 7a that after one cycle in either direction the behaviour returns to that of the bare frame. This is due to the compressive strut representing the panel becoming effectively destroyed, evident by significant strength degradation.

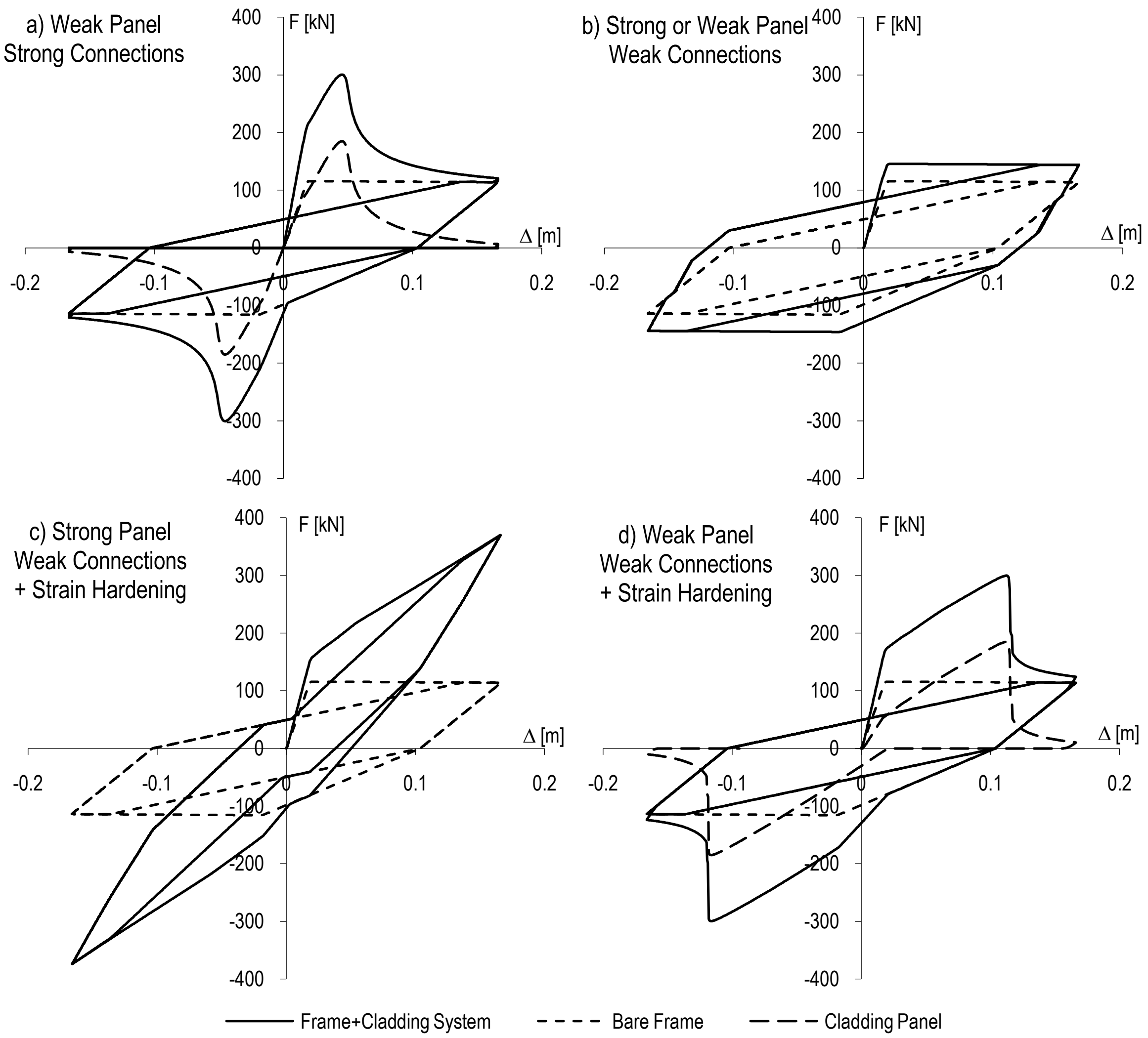

Figure 7: Force-displacement behaviour of panel/frame governing cases

When the connections yield at a force less than that of the panel strength, it can be seen in Figure 7b that the hysteretic cycle remains stable with a larger area than that of the bare frame, likely leading to a higher level of damping, which will be explored later. For this case the strength of the panel does not affect the overall structural behaviour so long as the panel strength is higher than the yielding strength of the connections.

However, when strain hardening is taken into account, the strength of the panel once again becomes important. As can be seen in Figure 7c, the strain hardening in the connections can lead to the panel carrying much larger forces than that when strain hardening is ignored. If the panel is still stronger than the ultimate strength of the connection then the cyclic behaviour will become stable in a hysteretic loop different to that of the bare frame. Conversely, if the panel is not stronger than the ultimate strength of the connection then a sudden failure of the panel can result, as shown in Figure 7d. This result shows a dramatic drop in the strength of the system which is undesirable and it also exemplifies a case that may not be foreseen by a cladding designer.

Figure 8 demonstrates how this post-yielding stiffness can influence panel failure. The graph presents four cladding panels of various compressive strut strengths with all other properties the same as the testing arrangement. A relationship can be found relating the panel strength and the connections' post yielding stiffness to the expected drift which panel failure will occur. This could aid in the design requirement of the panel strength. For example, a structure designed to reach an inter-storey drift of 2%, using cladding connections with a post-yielding stiffness of 30% of initial stiffness would require cladding panels with a compressive strut strength of at least 8 MPa. The relationship is dependent on other parameters which are not shown here, including connection strength, connection stiffness, strut area, and panel geometry.

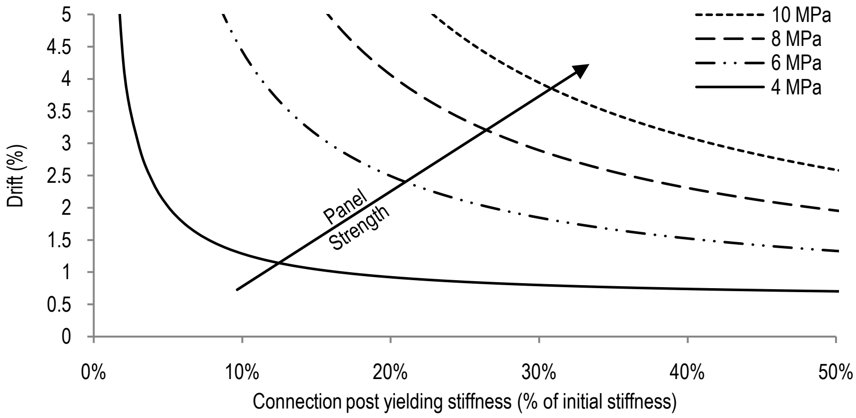

Figure 8: Expected panel failure curves for various panel compressive strut strength

3.3 Cladding Quadrilateral Model

Damage to the cladding panel is not a desirable outcome and as such, cladding connections are typically designed to be the lowest in the hierarchy of strength in order to achieve a behaviour like that shown in Figure 7b. This system is independent on the strength of the panel and the panel can be shown to behave nearly perfectly elastically. For this reason, the panel model has been altered by replacing the diagonal struts with an elastic quadrilateral element, as shown in Figure 9. The quadrilateral has been given material properties of a strong precast concrete panel; however the properties of the panel are somewhat irrelevant so long as it is stiffer than the connections, since the strength and stiffness of the seismic connections control the force that the panel can carry.

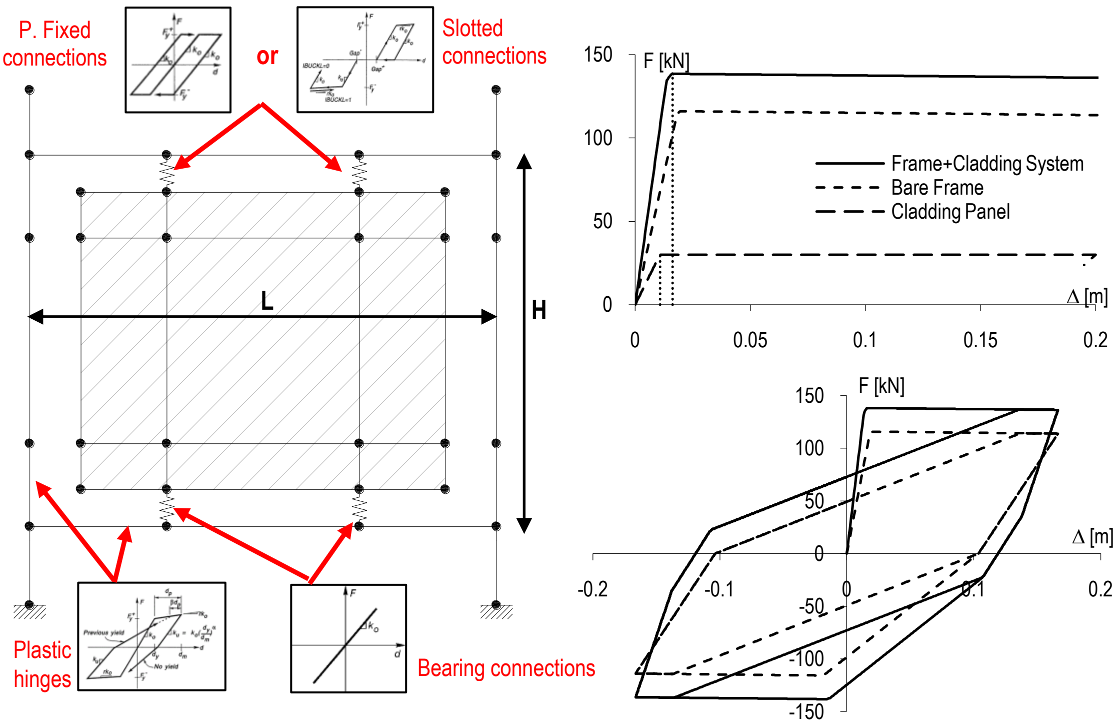

Figure 9: Cladding quadrilateral model (beam connections case) with force displacement behaviour

4. Parametric Analysis

In order to understand the effect that various parameters have upon the behaviour and performance of cladding panels under in-plane seismic loading, a parametric analysis has been conducted. Three parameters have been varied to identify whether these have influence upon the overall system behaviour. The analyses consist of push-over and push-pull displacement controlled tests where displacement is applied at the top of the column. The push-over analyses aim to reveal the yielding and failure characteristics of both individual components and of the system as a whole. The push-pull analyses are performed at repeating cycles of 4% drift in order to provide additional information on the hysteretic behaviour. Both the panel and the bearing connections are assumed to be stiff and elastic. The following parameters are to be varied: frame height to span ratio (1:1, 1:1.5, 1:2), connection configuration (beam vs. column) and connection behaviour (partially fixed vs. slotted). The beam connection configuration is shown in Figure 9, with the difference for the column connection configuration being the connection springs are located instead on the redundant column nodes.

4.1 Results

The results of the analyses that use the quadrilateral panel model all exhibit similar behaviour since the connection strength governs the behaviour. The force-displacement results of the push-over and push-pull analyses for a frame ratio of 1:1 with fixed beam connections is shown Figure 9 (right). It is evident that the cyclic response shown in Figure 9 (lower right) is the same as that in Figure 7b, as would be expected. In order to compare the results, the frame's base shear and top deflection at yielding has been found for all of the analyses conducted and is presented in Table 2. It is evident there was a larger increase in base shear of the system when the panel is attached to either the beam or the column, however, the increase is more significant when attached to the beam. There was no difference between the base shear when the connection is partially fixed or slotted since both connections yield at the same, however, the slotted connections delay the onset of yield, as seen by the higher yield displacement of the system. The frame ratio (H:L) has is negligible influence upon the base shear, however, a longer span appears to cause the system to yield at a lower displacement compared to the bare frame case.

Table 2: Parametric results for push-over analyses of frame/panel system

		BARE FRAME	CLADDING PANEL – FRAME SYSTEM			
	Frame Ratio (H:L)	No Connections	P. Fixed$_{beam}$	Slotted$_{beam}$	P. Fixed$_{column}$	Slotted$_{column}$
Base Shear at Yield	1:1	115.8 kN	138.3 kN (+19.4%)	138.3 kN (+19.4%)	129.3 kN (+11.7%)	129.2 kN (+11.6%)
	1:1.5	115.8 kN	138.4 kN (+19.5%)	138.3 kN (+19.4%)	129.4 kN (+11.7%)	129.3 kN (+11.7%)
	1:2	115.8 kN	138.4 kN (+19.5%)	138.4 kN (+19.5%)	129.4 kN (+11.7%)	129.4 kN (+11.7%)
Yield Displacement of Frame/Panel System	1:1	0.020 m	0.021 m (+5.0%)	0.033 m (+65.0%)	0.016 m (-20.0%)	0.020 m (+0.0%)
	1:1.5	0.025 m	0.024 m (-4.0%)	0.033 m (+32.0%)	0.017 m (-32.0%)	0.020 m (-20.0%)
	1:2	0.030 m	0.029 m (-3.4%)	0.033 m (+10.0%)	0.019 m (-36.6%)	0.020 m (-33.0%)

The effect of the cladding panels on the individual member demands can be seen in Figure 10. These member demand envelopes are shown for a frame ratio of 1:1 with partially fixed beam connections. Again, the general shape of these plots is similar for all these analyses, where connection failure governs; so the full results for panels connected to the beams have been tabulated in Table 3 for comparison purposes. Due to its symmetry, the table only displays the member demands at points A and B, as indicated by Figure 10.

Varying the connection between partially fixed and slotted has negligible effect upon the member demands. A reduction in moment and shear demand in the middle of the beam (point B) is evident when the cladding panel is present due to the panel distributing load away from the frame. There is also a large increase in shear demand at the beam between the cladding connections and the beam-column joint. This shear force increases from +120% to +261% that of the shear in the bare frame as the frame ratio (H:L) increases. It is evident that in order to ensure all possible failure mechanisms are captured, the model needs to include shear failure of the members and the beam-column joint. It was also found that the column moment demands (not presented in a table) were not affected by the presence of the cladding panel, so the load was being passed down solely through the beams.

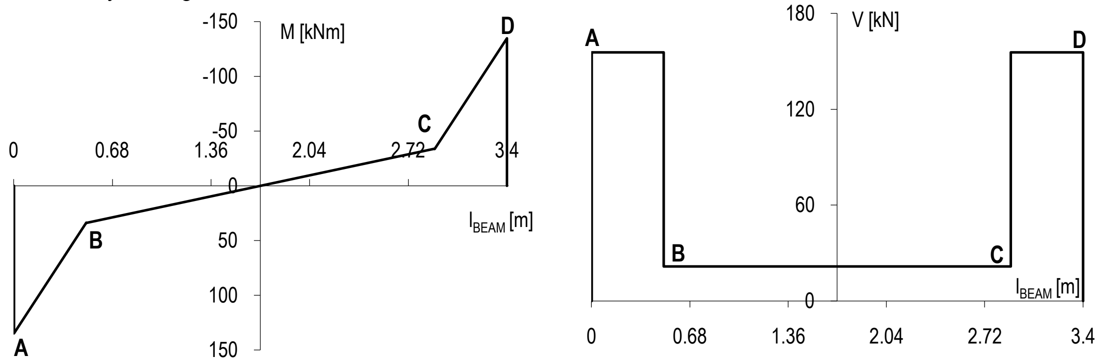

Figure 10: Moment/shear envelope for beam p. fixed connections case where B and C are locations of panel connections

Table 3: Full results for moment/shear envelope when panel attached to beam

		BARE FRAME				**CLADDING PANEL – FRAME SYSTEM**							
		No Panel Connections				Partially Fixed Connections				Slotted Connections			
		M_{beam} (kNm)		V_{beam} (kN)		M_{beam}		V_{beam}		M_{beam}		V_{beam}	
		A	B	A	B	A	B	A	B	A	B	A	B
Frame Ratio (H:L)	1:1	135	85	71	71	0%	-69%	120%	-69%	0%	-69%	118%	-69%
	1:1.5	135	103	51	51	0%	-71%	194%	-73%	0%	-71%	194%	-73%
	1:2	135	112	41	41	0%	-72%	261%	-76%	0%	-71%	261%	-76%

The changing level of equivalent viscous damping of the system as well as individual elements is shown in Figure 11 for increasing levels of displacement ductility. The four lines show the damping of the bare frame, the frame-cladding system and the contribution from each component to the system. There is a clear increase in the level of in the damping for all levels of ductility.

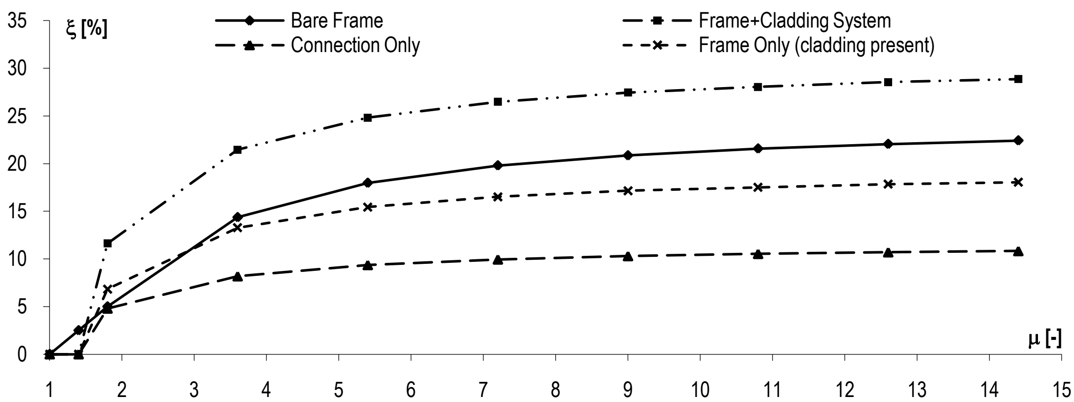

Figure 11: Damping vs. ductility for partially fixed connections case

Conclusions

The results show that the presence of cladding panels increases the system strength by at least 10-20%. This contribution is greater when panels are attached to the beams rather than to the columns. There is also an increase in hysteretic damping for all systems. When both panel and connections are strong the capacity of the system is increased but the ductility is decreased. The model limits the additional hysteretic damping provided by the panel crushing to one cycle, when in reality the panel would still be dissipative, but this dissipation is not reliable.

The slotted connections delay the onset of yield, as seen by the higher yield displacement of the system, however their influence upon other results are minimal. The frame ratio (H:L) also has a minimal influence upon most results; a longer span does cause a larger increase in shear demand in the beam between the cladding connections and the beam-column joint. This showed that to improve the analyses, the consideration of shear failure in the beams, columns and beam-column joint is vital. It is also worth investigating other connection typologies with different behaviour, e.g. non-ductile.

It is apparent that it is more advantageous to have and thus design for a system where the connection is the weakest element in the hierarchy of strength. 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, which only provides such benefits over a single cycle. However this assumes that the tie-back connections are designed to accommodate a large level of ductility. In terms of the cost and ease of repairs; the substitution of failed connections is also seen to be more favourable than having to replace entire damaged panels. This aspect has been numerically investigated on multi-storey buildings with cladding interaction effects and is presented in a companion paper (Diaferia et al. [3]).

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