PERFORMANCE OF PRECAST CLADDING PANELS WITH ROCKING CONNECTIONS UNDER LATERAL CYCLIC LOADING

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Abstract

Several concrete cladding panels were damaged during the 2011 Christchurch Earthquakes in New Zealand. Damage included partial collapse of panels, rupture of joint sealants, cracking and corner crushing. Installation errors, faulty connections and inadequate detailing were also contributing factors to the damage. In New Zealand, two main issues are considered in order to accommodate story drifts in the design of precast cladding panels: 1) drift compatibility of tieback or push-pull connections and 2) drift compatibility of corner joints. Tieback connections restrain the panels in the out-of-plane direction while allowing in-plane translation with respect to the building frame. Tieback connections are either in the form of slots or oversized holes or ductile rods usually located at the top of the panels. Bearing connections are also provided at the bottom of panels to transfer gravity loads. At the corners of a building, a vertical joint gap, usually filled with sealants, is provided between the two panels on the two orthogonal sides to accommodate the relative movement. In cases where the joint gap is not sufficient to accommodate the relative movements, panels can collide, generating large forces and the likely failure of the connections. On the other hand, large gaps are aesthetically unpleasing. The current design standards appear to recognize these issues but then leave most of the design and detailing to the discretion of the designers. In the installation phase, the alignment of panels is one of the main challenges faced by installers (and/or contractors). Many prefer temporary props to guide, adjust and hold the panels in place whilst the bearing connections are welded. Moreover, heat generated from extensive welding can twist the steel components inducing undesirable local stresses in the panels. Therefore, the installation phase itself is time-consuming, costly and prone to errors.

This paper investigates the performance of a novel panel system that is designed to accommodate lateral inter-story drift through a 'rocking' motion. In order to gauge the feasibility of the system, six 2m high precast concrete panels within a single-story steel frame structure have been tested under increasing levels of lateral cyclic drift at the University of Canterbury, New Zealand. Three different panel configurations are tested: 1) a panel with return cover and a flat panel at a corner under unidirectional loading, 2) Two adjacent flat panels under unidirectional loading, and 3) Two flat panels at another oblique corner under bidirectional loading. A vertical seismic joint of 25 mm, filled with one-stage joint sealant, is provided between two of the panels. The test results show the ability of the panels with 'rocking' connection details to accommodate larger lateral drifts whilst allowing for smaller vertical joints between panels at corners, quick alignment and easy placement of panels without involving extensive welding on site.

Keywords: precast structures; architectural cladding; rocking motion; connections; sealants

1. Introduction

Most modern building structures can be assumed to satisfy the 'life-safety' performance criteria defined in building codes. However, damage to 'non-structural' elements has been recurring in moderate earthquakes [1,2,3]. Among the 'non-structural' elements, cladding or façade systems aim to satisfactorily provide a comfortable interior environment for living and day-to-day functioning of a building by sheathing the indoors from external environmental factors such as rain, wind, snow, dust, smoke, noise and excessive heat and cold. Precast concrete cladding panels (or simply 'panels' for brevity) deliver construction speed, low life-cycle cost, durability, flexibility, sustainability and aesthetics [4]. They are, however, one of the most expensive non-structural elements installed in a building [5,6]. Any repairs to panels following an earthquake can compromise their intended purpose and dramatically increase the actual cost of the cladding system. Therefore, the main goals of an engineer are to design panel systems to satisfactorily : (i) resist their own weights; (ii) accommodate structural deformations and thermal movements;(iii) provide necessary clearances or tolerances; and (iv) allow access to components for maintenance and repair [7, 8]. Since panels are sensitive to both acceleration and drift, the connections between the flexible structure and rigid panels are designed and detailed to allow relative movement between the two and to resist the out-of-plane loads during seismic events [4, 9].

In New Zealand and the United States, design traditionally aims for 'swaying' or 'sliding' motion to be induced in the panels, relative to the structure, through tie-back/push-pull connections (which may be in the form of horizontal slots or oversized holes or ductile rods) at the top and bearing connections at the bottom of the panel. The panels only move with the bottom floor or beam and are isolated at the top; preventing any diagonal compressive forces to be developed in the panels for design drift levels. Horizontal slots are recommended for use in stiffer structures and panels, with low aspect ratios [10], installed close to the structure. Long ductile flexing rods are generally found to seismically perform better than the horizontal slots [10, 11, 12]. A large vertical gap is provided between panels meeting at a corner to accommodate large differential movements. Otherwise, the panels collide with each other generating large forces and possible failure of the connections [12, 13].

Several failures of panels and their fastenings have been observed following the recent earthquakes worldwide [13, 14, 15, 16, 17] primarily due to inadequate strength and displacement capacity of the fastenings (the two most important characteristics of cladding connections [18]) and construction/installation errors [1, 14]. Economic losses, repair downtime and fatality [7, 19, 20] are major ramifications of panel failures. Therefore, satisfactory design and detailing of the panel-to-structure connections for the expected earthquake actions and movements is of paramount importance.

Panels with rocking connection details, prevalent in Japan, have been experimentally proven to perform satisfactorily without any visible damages to the panels and their connections under seismic actions [12, 21]. However, their practice is sparse outside Japan, limited to long and narrow slender panels such as column covers in the US [11], possibly because of their complex details, susceptibility to installation errors and high associated costs [12]. This stimulated the development of novel rocking connection details in New Zealand, as shown in Fig.1.

The novel connections comprise of four steel-embeds with vertical slots and two weld-plates cast into the panels. The panel and the structure are connected through the bolts placed inside the vertical slots. The vertical slots in steel embeds allow panels to efficiently rock upwards (Fig.1) to accommodate the desired inter-story drift while resisting the out-of-plane loads in the panels. The weld-plates seat on top of the bearing connections and prevent the local spalling and chipping of concrete during rocking of the panels. Further details regarding the concept, design, fabrication, installation and experiments of the panel system comprising the novel rocking connections under lateral cyclic loadings can be found in [22]. The experimental results showed greater seismic resilience in the precast cladding panel system with the novel rocking connections as compared to the traditional tie-back/push-pull connection systems.

This paper further investigates the seismic resilience in three different sub-assemblies of panel system incorporating the novel rocking connections under lateral cyclic loadings.

2. Test setup and panel details

A one-story 3D steel frame structure (Fig. 2) is designed and erected to test three different sub-assemblies of panel pairs: inclined-corner assembly, return-cover corner assembly and in-line assembly (Fig.2 and Fig. 3). The individual panel dimensions and their calculated/designed in-plane inter-story drift capacities are shown in Table 1. The connections at the base of the column and between columns and beams are hinged to allow a shear mode of deformation in the structure when the top slab is subjected to lateral quasi-static cyclic loading up to maximum inter-story drift of 4.18 % following FEMA 461 [23] loading protocol (Fig. 4). The bottom slab and cross-bracing provide torsional resistance to the frame. The main components of the structure associated with the panel attachments are horizontal slotted steel plates and rectangular hollow sections for bearing as shown in Fig.5. The bolts from the steel embed pass through the horizontal slots (provided for tolerance). Once the panels are in position, the washers are tack-welded to lock the panels in place. The nut is loosened half-a-turn to allow the panels to rock under the lateral loads. The vertical joint width between adjacent panels is 25 mm. One layer of Sika AT-Façade sealant is interposed into the joints to depths of about 15 mm. The sealant was fully cured as it was left for 33 days before the testing and 7-14 days of curing of sealant is recommended by PCI [4].

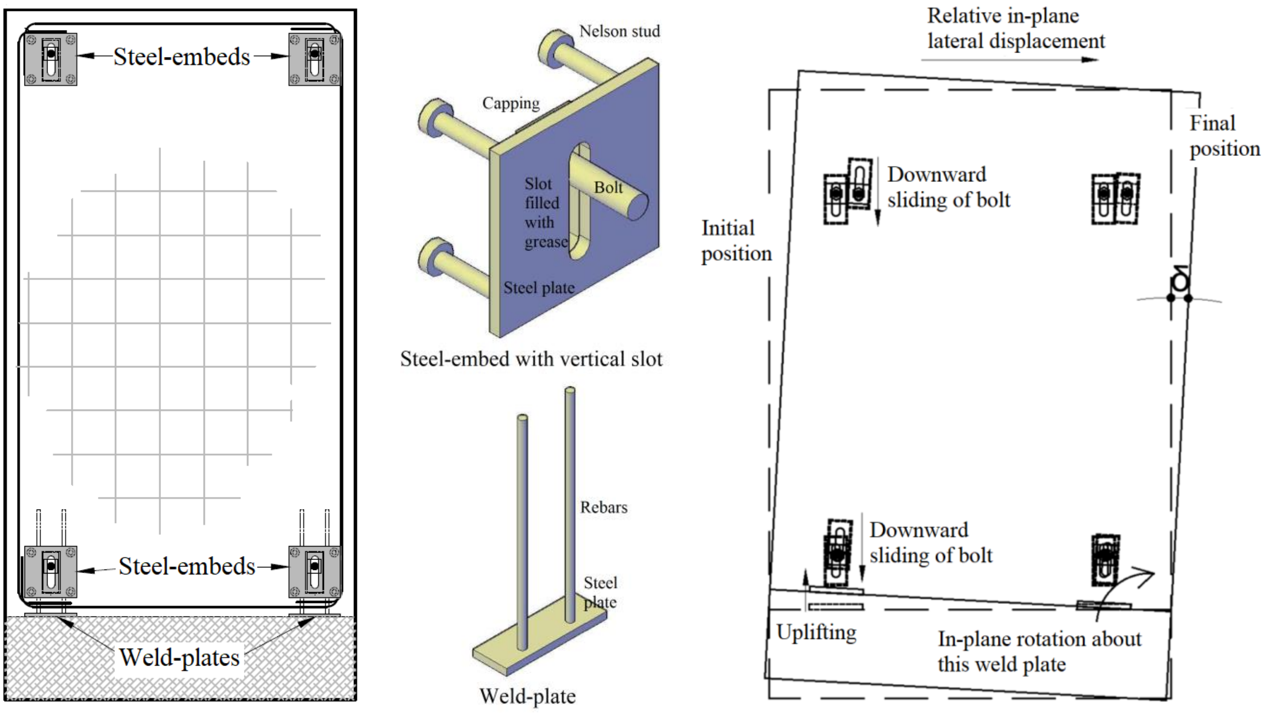


Fig. 1 – Panel with novel rocking connections (left), Details of novel rocking connections (center), and rocking mechanism of panel under top lateral displacement (right)

A 300KN actuator with stroke length of ±200 mm is attached to the center of the top slab. Three string-pots (string-pot-1, 2 and 3) are used to record the top frame displacements (Fig. 2). They also detect any sloppiness and torsion in the frame. Each sub-assembly is instrumented with potentiometers (Fig. 6) to gauge the uplift at edges of panels, changes in the vertical joint width and out-of-plane movements of panels, where necessary. Paper masking tapes with measurements outlined up to 100mm, to scale, are adhered near the vertical slot to measure approximate sliding of the bolt in the slot. Two horizontal lines are drawn across the sealant at each vertical joint to measure the shear displacements in the sealant at peak inter-story drifts in each cycle.

3. Experimental results and discussions

The structure was subjected to lateral cyclic displacements at the center of the top slab. The top displacements in Frame A and Frame B were recorded by string-pot-3 and string-pot-1, respectively, (Fig.2). The force-drift hysteresis loops for Frames A and B are shown in Fig. 7. Note that the corresponding forces shown in Fig.7 are of the entire structure, not for a particular frame. These recordings show that the structural connections in the frame have some slackness which is clearly distinct while pulling the structure (positive drift cycles). The loading first starts (in the negative direction) with some initial stiffness due to shearing resistance provided by the interposed sealant. Under small lateral forces applied to the top connections of the panel, the panels are expected to behave as integral elements with the structure as they are equilibrated by the panel's weight until the top lateral forces reach Wb/2h; where, 'W', 'b' and 'h' are the weight, width and height of the panel, respectively.

Fig. 2–Test setup, instrumentation, panel sub-assemblies

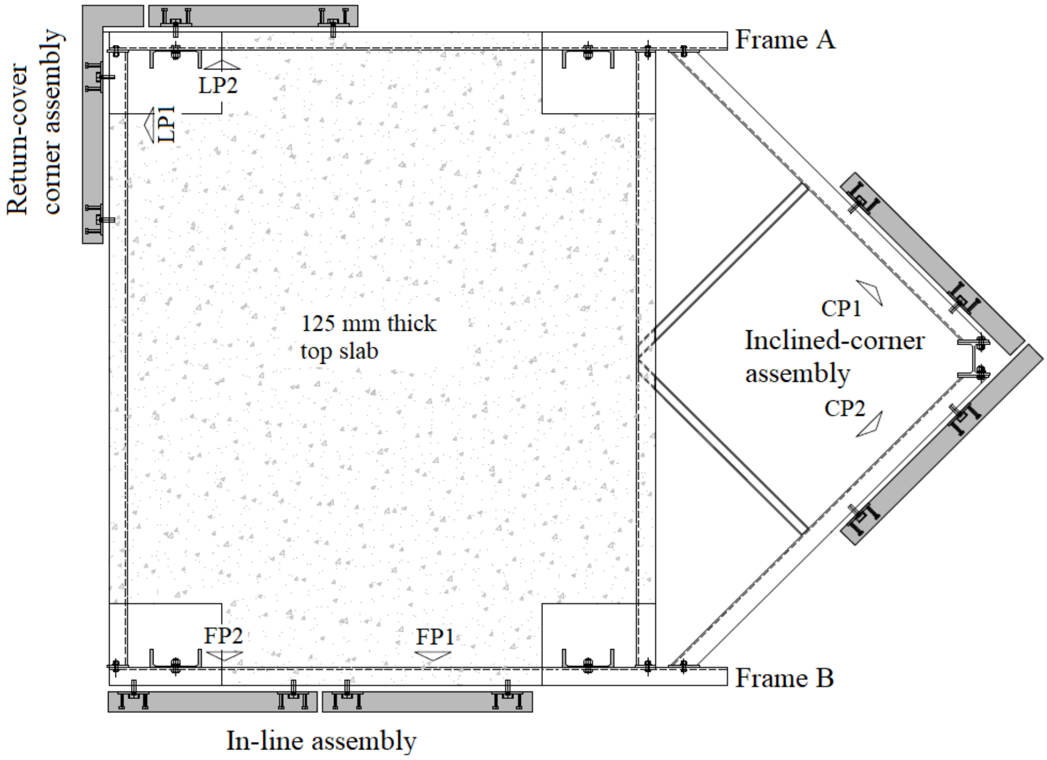


Fig. 3 – Plan showing panel sub-assemblies

Table 1 – Panel specimen details and their in-plane drift capacities

Panel sub-assembly*	Panel Specimen	Height (mm) x Width (mm) x Thickness mm)	In-plane drift capacity (%) from [22]
In-line	FP1	2000x1015x120	8.34%
	FP2	2000x1015x120	8.50%
Return-cover corner	LP1	2000x1165x120x295 (return cover)	9.32%
	LP2	2000x1015x120	8.50%
Inclined-corner	CP1	2000x1165x120	9.37%
	CP2	2000x1290x120	9.37%

*Joints in the sub-assemblies possessed 25 mm thick sealant to depths of about 15 mm

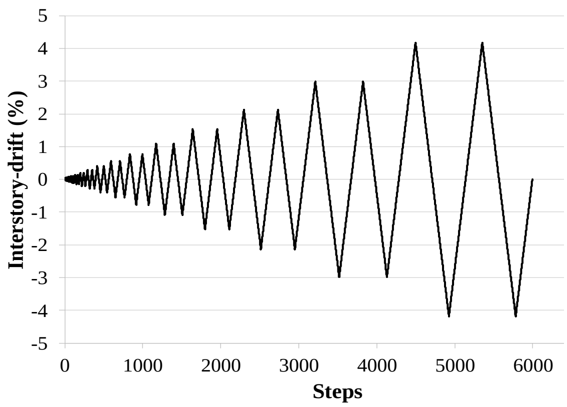


Fig. 4 – Loading protocol following FEMA-461[23]

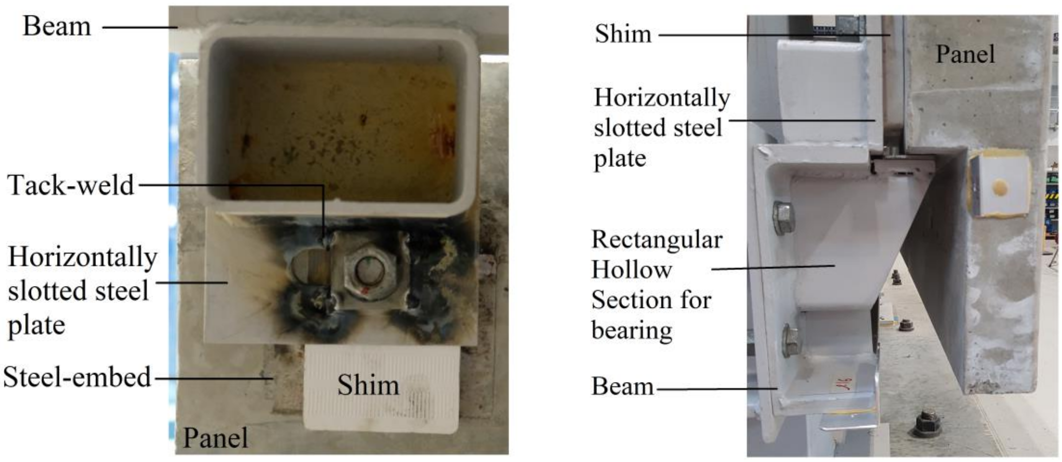


Fig. 5 – Main components from structure associated with panel connections: Top connections (left) and Bottom connections (right)

After this limit is overcome, the panels are expected to be behave as isolated from the structure, allowing lateral floor movements without imparting significant stiffness to the structure. In any rotated position, a constant horizontal stabilizing force is provided by the bearing connection on which the panel is seated. However, additional stiffness is observed in the hysteresis loops (Fig.7) as the bolts in the bottom steel-embeds gradually get engaged with the sides of the slot generating frictional resistance [22]. This also can lead to slight torsion in the structure (a maximum twisting angle of about 0.015 radians between Frames A and B was observed in the test). Strength and stiffness degradation can be observed in the hysteresis loops of the same drift cycle and consecutive drift cycles, respectively, due to relaxation and/or tearing of the sealant.

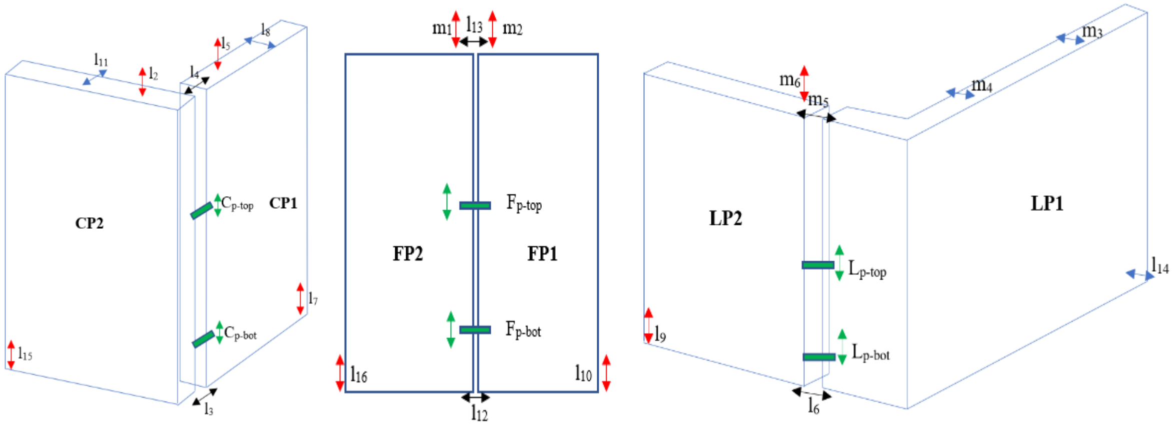
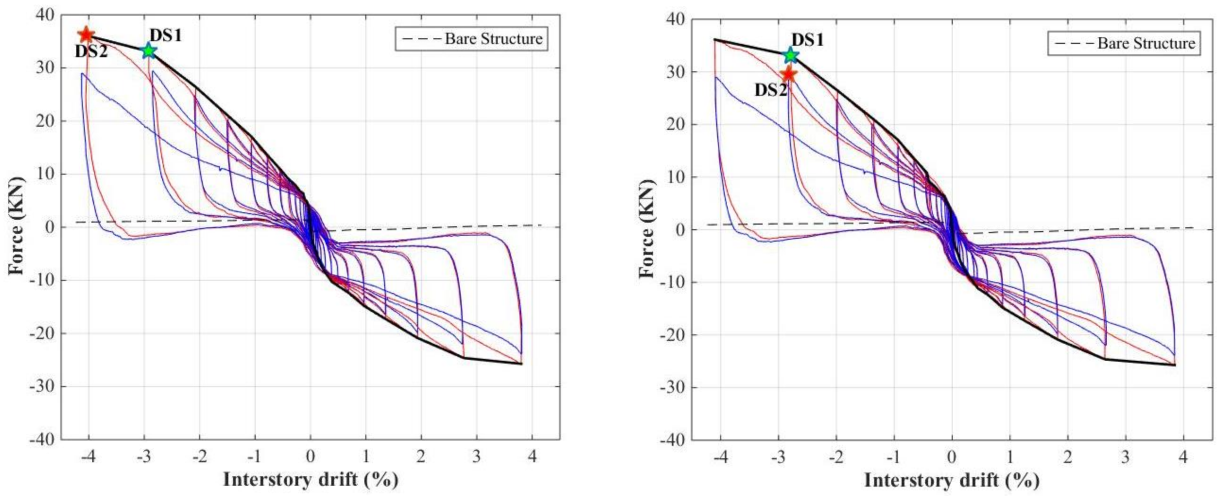


Fig. 6 – Panel instrumentations: potentiometer labels (red: vertical displacement; blue: out-of-plane displacement; black: horizontal displacements in the joints) and horizontal green lines: vertical displacements in the joints



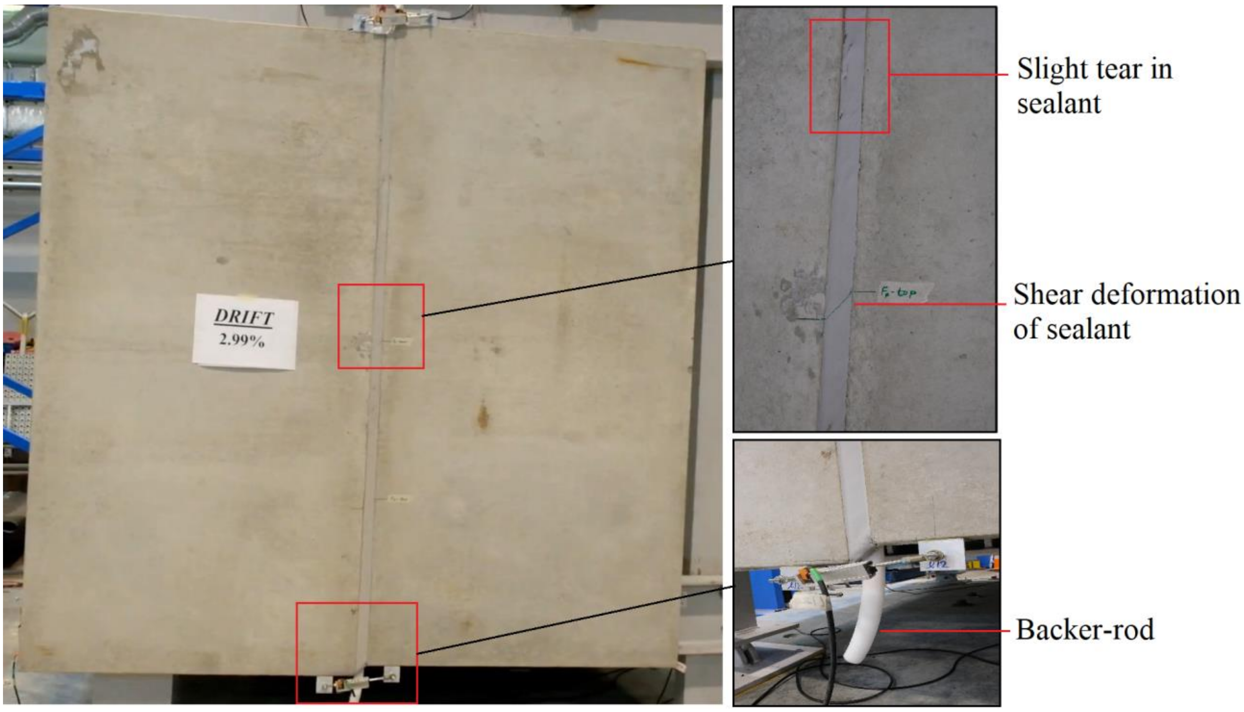
a. Frame A with return-corner sub-assembly

b. Frame B with in-line sub-assembly

Fig. 7 – Force-drift hysteresis loops showing onset drifts for damage states in the interposed sealants

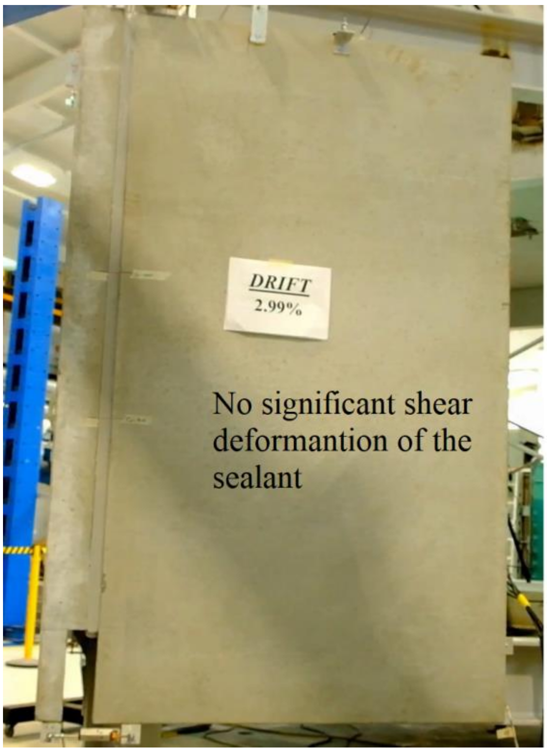
The panels exhibited rocking motion (Fig. 8) as expected under the lateral cyclic loadings. The bolt inside the vertical slot of the steel-embed slid ‘smoothly’, aided by grease, when the panels rocked as shown in Fig. 9. Visual observations during and after the testing showed no noticeable spalling or cracking of panels, no yielding of steel components and no loosening of the steel-embeds. The joint width was found to remain approximately constant throughout the test. However, significant shear deformation in sealants was observed in the joint interfaces (Fig.8.a and Fig.8.b) except in the joint interface of the inclined-corner sub-assembly (Fig.8.c), where panels CP1 and CP2 rocked without inducing significant differential vertical displacements at the joint.

The damages observed in the sealant, in this test, are grouped into the two damage states as shown in Table 2 depending upon their repairability [22]. The pictures corresponding to these damage states are shown in Fig. 10. The inter-story drifts corresponding to the damage states DS1 and DS2 of the sealant interposed in the return-cover corner sub-assembly are 2.92% and 4.06% and in the in-line sub-assembly are 2.78% and 2.82%, respectively. These damage states are also indicated in the hysteresis loops (Fig. 7.a and Fig.7.b) of respective frames in which these sub-assemblies are attached. The ultimate shear strain capacity of the sealant was found to be around 132%. Moreover, due to alternate rise and fall of the panels during their rocking motion, the backer-rod was observed to gradually slide downwards (Fig.8.a and Fig.8.b).

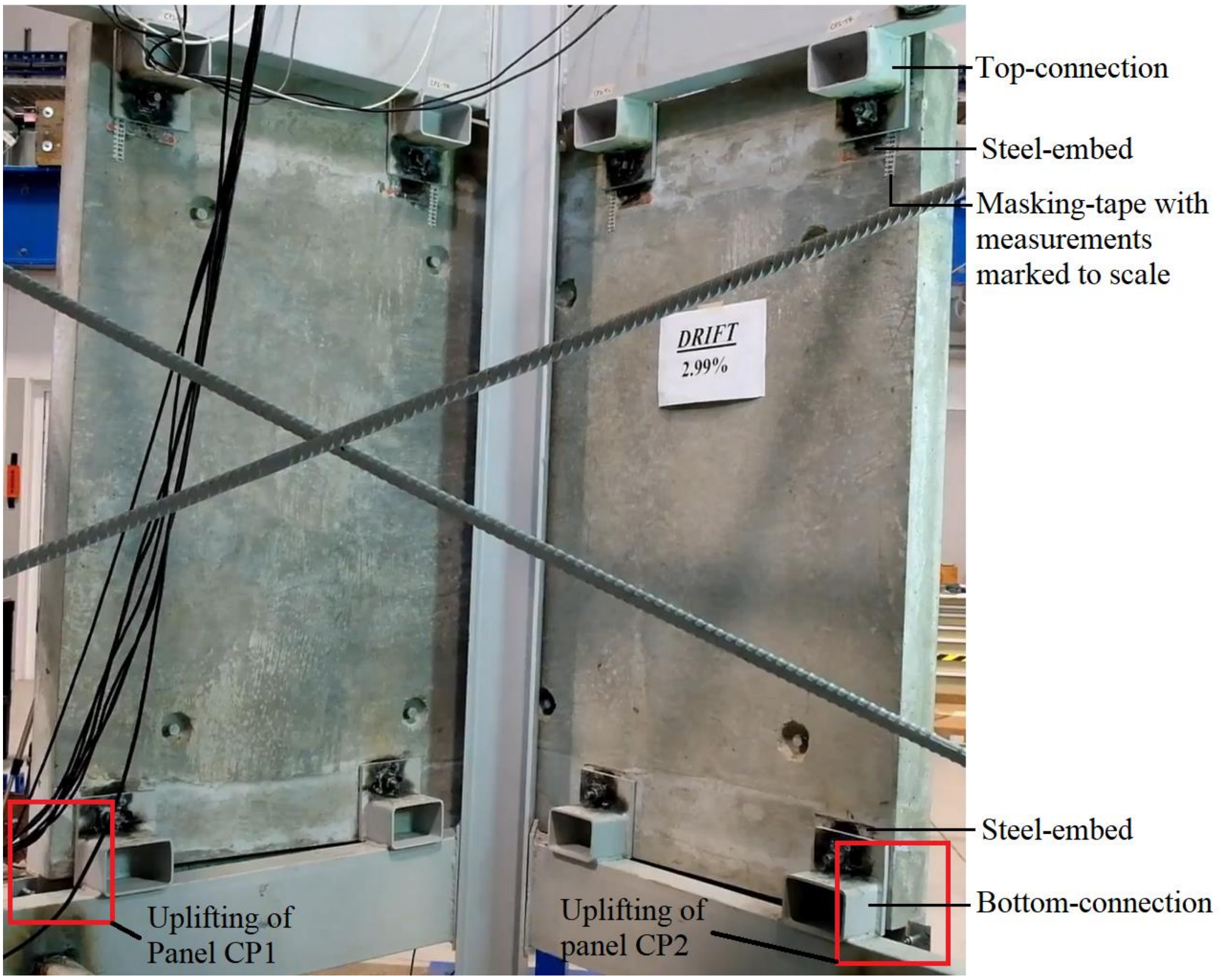


a. In-line sub-assembly

b. Return-cover corner sub-assembly



c. Front-view of inclined-corner sub-assembly



d. Rear-view of inclined-corner sub-assembly

Fig. 8 – Rocking of panels at peak of first -3% inter-story drift cycle

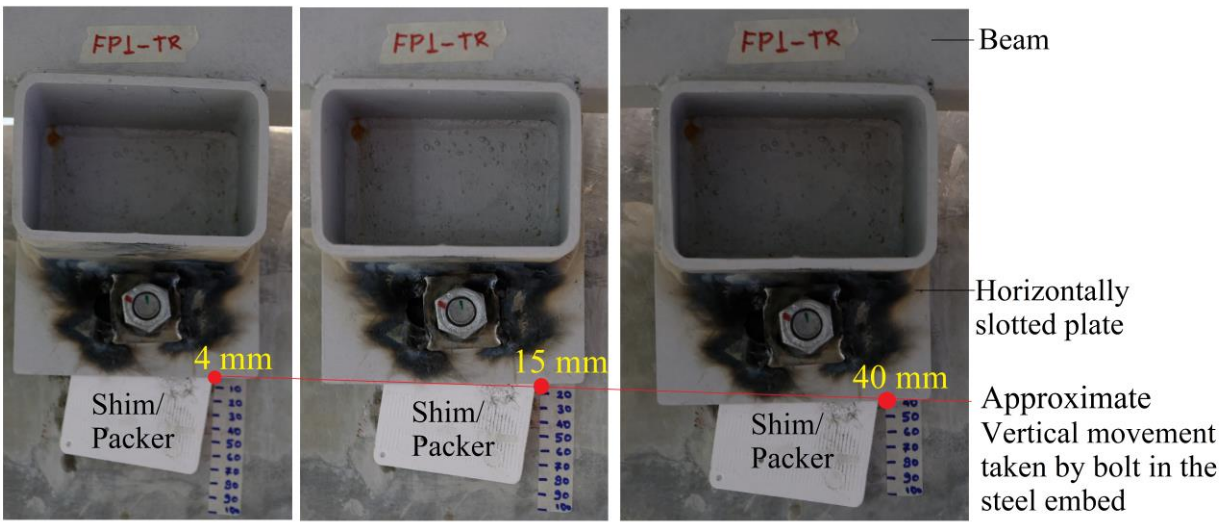


Fig. 9 – Example of gradual sliding of the bolt in the vertical slot of steel-embed located at the top-right corner of Panel FP1 as the right edge of the panels uplifts at: -0.44% (left), -1.35% (center) and - 4% (right) inter-story drifts

Table 2 – Preliminary damage states of the sealant in this test [22]

Damage state	Description	Repairability	Method of repair
DS1	The length of tear is small compared to the total length of the joint	Minor	A larger portion, around 200mm in length (100 mm on each end of the tear), in addition to the torn length of sealant is removed (or scraped off), the surface is cleaned, and new sealant is applied
DS2	The length of tear is large compared to the total length of the joint	Severe	Entire length of the sealant is removed (or scraped off), the surface is cleaned, and new sealant is applied

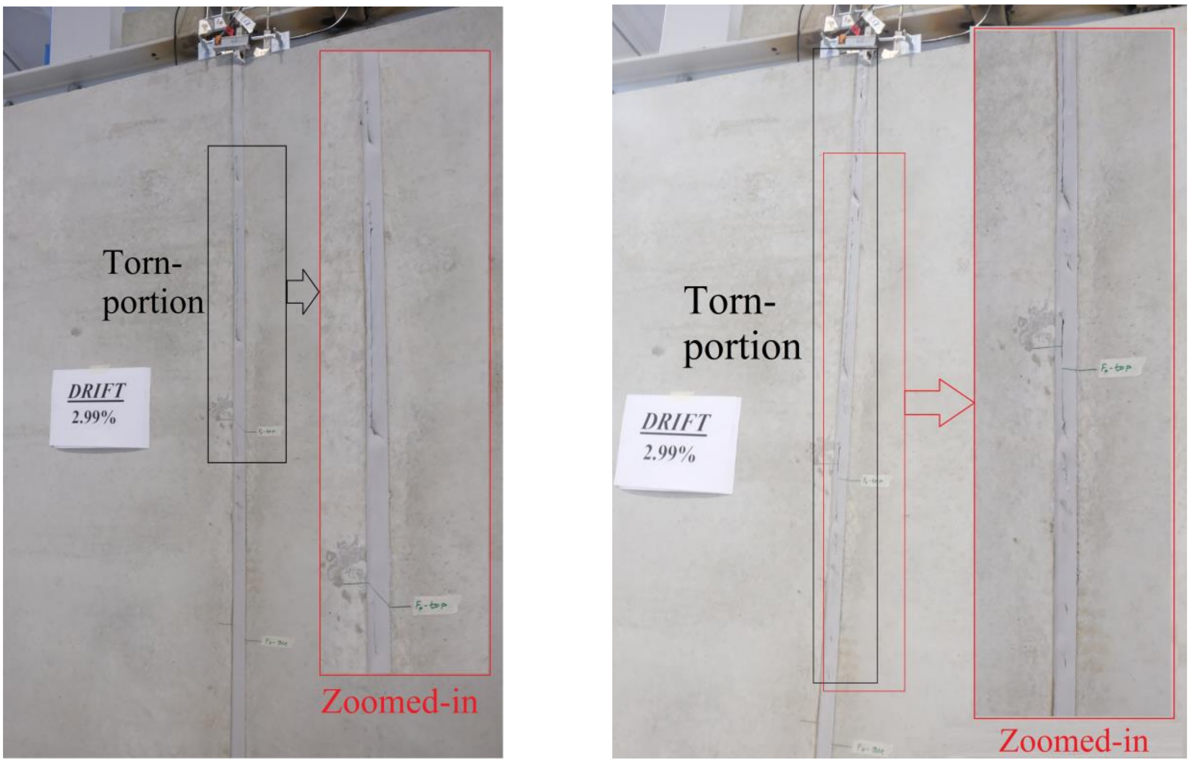


Fig. 10 – Examples of damage states DS1 (left) and DS2 (right) of sealant interposed at the in-line sub-assembly

4. Conclusions

Three sub-assemblies of precast concrete cladding panels (in-line assembly, inclined corner assembly, and return-cover corner assembly), incorporating novel rocking connections (steel-embeds with vertical slots and weld-plates), assembled on a 3D steel structure designed to deform in shear mode were subjected to lateral cyclic drift demands. A 25mm vertical joint gap between the panels in each sub-assembly was filled with one layer (one-stage) of Sika AT-Façade sealant. The panels were able to sustain large drift demand by undergoing rocking motion as expected. From the experimental results, it can be concluded that a precast concrete cladding panel system with these novel rocking connections can accommodate large inter-story drifts ($\approx 4\%$, in this test) without any noticeable damage to the panels and their connections. The only damage observed was the tearing of the sealant at an inter-story drift of 2.78% where the ultimate strain in the sealant reached around 132%.

5. Acknowledgements

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