Performance of exterior facade on reinforced concrete frame under cyclic loading

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Abstract. A growing preference for glass curtain walls (GCW) over traditional infill walls is evident in various multi-story commercial buildings, including hotels, offices, malls, and public structures such as hospitals and government offices. GCW, comprising glass panels supported by a framed structural system, serves as the building's exterior. However, instances of GCW failures during seismic events pose risks to occupants. While much research has been carried out solely on GCW components, research considering its interaction with the supporting reinforced concrete (RC) structure is still unexplored. Therefore, an experimental study is carried out aimed at assessing the seismic vulnerability of GCW systems in multi-story buildings. A 2-story RC frame designed as per Indian seismic code IS 1893 (Part 1)-2016 is mounted with stick GCW as the façade system utilizing HILTI post-installed anchors. The frame is tested under ACI 374.1-05 cyclic loading protocol. The objectives of this study is to identify damage states of GCW correlated with damage in the RC portal frame. By examining the interaction between GCW and the RC structure, this study provides insights into the seismic behaviour of modern architectural assemblies. Furthermore, the experimental approach offers a comprehensive framework for evaluating GCW system performance within the context of the entire structural assembly, developing understanding of their behaviour under dynamic loading conditions.
Keywords: Non-structural element; Post-installed anchor; Reinforced concrete; Quasi-static testing.

1 Introduction

Glass curtain walls (GCWs) are increasingly preferred over infill walls in multistory commercial buildings like hotels, offices, and malls. Many public buildings, such as hospitals and government offices, also use GCWs as their exterior facade. A GCW consists of glass panels, supported by a framed structural arrangement attached to the main structural system, like beams and slabs. However, GCWs have shown vulnerability to seismic activities, posing risks to occupants and pedestrians if not properly designed. Figure 1 shows damages to GCW during 2010 Chile and 2011 Christchurch earthquake. In a survey of 173 RC buildings in Christchurch CBD [1], damage to façade systems, including lightweight (e.g., Curtain Wall (CW)) and heavy (precast panels) facades is documented and characterized as operational, immediate occupancy, life safety, and high hazard.

(a) (b)

Figure 1. Glazing damage observed in past earthquakes (a) 2010 Chile earthquake (b) 2011 Christchurch earthquake.

Due to significant economic and life hazards, researchers have conducted seismic evaluations of GCWs using full-scale testing. In a study [2], 30 window glass panels have been tested under various conditions, including static and dynamic loading and an expression for maximum adjustable drift has been developed. AAMA [3] provides guidelines for in-plane racking tests, performed at specific frequencies and displacement intervals. It is also seen previously that glass lamination and silicone glazing thickness affect the seismic resistance among architectural glass types [4] In some studies, researchers have experimentally tested frame insulated CWs panels and developed non-linear FEM models [5–7]. Predictive models for cyclic racking tests have also been developed [8, 9] to determine acceptable stress levels for sealants in seismic conditions. FEMA356 [10]classifies CW performance as immediate occupancy and life safety. Memari and Shirazi [11] have developed seismic rating techniques for CWs, predicting seismic risks based on glass, frame, boundary conditions, and building drift parameters. Most studies have focused on GCW components, with less emphasis on interaction with primary structures. The GCWs are typically connected to Reinforced Cement Concrete (RCC) structures using post-installed anchors. Such anchors attract cracks during seismic events thus reducing their capacity. This study aims to understand the interaction between GCWs and RCC frames under lateral loading.

2 Experimental study

2.1 Test specimen

A two-story RC portal frame with a stick GCW façade system, as shown in Figure 2, representative of a commercial G+3 building is designed for a high seismic zone like Delhi, with a double-height ground floor commonly found in commercial buildings for a spacious feel. The design follows the latest Indian seismic guidelines [12] and Indian ductile detailing criteria [13]. The concrete grade for beams and columns is M25, with a 28-day compressive strength of 25 MPa. Fe500D grade steel reinforcement, with nominal yield strength of 500 MPa and ultimate strength of 565 MPa, is used for longitudinal steel, while Fe415 steel, with nominal yield strength of 415 MPa and ultimate strength of 485 MPa, is used for transverse reinforcement. Both types of reinforcement have a minimum nominal elongation of 14.5%. The material properties for reinforcement and concrete are given in Table 1.

Table 1. RCC frame material properties

Material type		Strength	Elongation (%)
Reinforcement	Longitudinal	f_y: 500 MPa; f_u: 565 MPa	16
	Transverse	f_y: 415 MPa; f_u: 485 MPa	14.5
Concrete	28-day cube test	f_{ck}: 25 MPa	-

Member cross-section details are as shown in Table 2. The columns have a cross-section of 300 mm x 300 mm with 1.79% longitudinal reinforcement. Beams have cross-sections of 230 mm x 450 mm, with two 16 mm diameter bars at the top and bottom. Concrete cover is kept as 20 mm for beams and 40 mm for columns. The foundation beam of size 450 mm x 450 mm is provided at the bottom to anchor the frame to the strong floor.

Table 2. RCC frame member cross-sections

Column		Beam		Foundation Beam	
Size (mm x mm)	Rebar	Size (mm x mm)	Rebar	Size (mm x mm)	Rebar
300x 300	8-16 φ (1.79%)	230 x 450	2-16 φ (top) 2-16 φ(bottom)	450 x 450	5- 25φ(top) 5- 25φ(bottom)

In India, the standard for tall buildings [14] provides seismic design guidelines for non-structural elements like façades, specifying that façades must accommodate building drift during seismic events. Similar guidelines exist in the American code [15]. However, in India, it is common to neglect seismic forces when designing façade elements, even in high seismic regions. Thus, GCWs are typically designed for wind load and self-weight.

For the Delhi region, wind load is calculated per Indian code [16], considering a risk coefficient of 1, a design life of 50 years, and a basic wind speed of 47 m/s. For a 12m high building, with a 16 m x 9.6 m plan size. The terrain and height factor (k_2) is 0.97, and the topography factor (k_3) is 1. The calculated wind pressure of 1.62 kN/m² is applied to the façade glass panels to determine the bending moment and shear force in the mullion and transom. The dead load generates axial and flexural stress in the mullion and flexural stress in the transoms. Utilization ratios for aluminum sections under various loads are found to be less than 10%.

The mullion and transoms, along with their connections, are designed per Indian code [17]. The curtain wall details are as in Figure 2. Glass panels utilized in the experiment are annealed glass of size 2250 mm x 840 mm and 6 mm thickness. Aluminum alloy sections are selected from the Jindal catalog: section 22737 for mullion and transom, and section 22841 for the glass frame. Connections are made using an 8 mm diameter, 38 mm long screw, spaced 500 mm apart. ASTM [18] is followed to assess tensile strength of aluminum alloy and the coupon test results are given in Table 2. To ensure the façade is airtight and watertight, rubber gaskets are provided between sections to prevent abrasion.

Table 3. Engineering properties based on the tensile test of coupons.

Specimen	Yield Strength (fy)	Ultimate Strength (fu)	Elastic modulus (E)	Elongation (%)
Sample1	65.2	91.6	11070	9.4
Sample2	58.7	88.9	10947	10.6
Sample3	64.7	94.8	9977.2	11.4

Glass panels for façades are tested for load resistance per ASTM [19]. Finite element analysis software ensures deflection and stress due to wind pressure are within permissible limits. For this study, 6 mm thick, 840 mm x 2250 mm annealed glass panels are used, attached to the frame with structural sealant consisting of double adhesive tape. Setting blocks support the glass at the corners as shown in Figure 2 (b) and (c).

Mullions and transoms are connected with T and + joints using Jindal 16143 aluminum angles and 8 mm diameter, 19 mm screws. These angles are connected to the RCC frame with 200 mm x 75 mm x 6 mm mild steel brackets and Hilti M12 mechanical anchors, as shown in Figure 2 (b).

Table 4. Specifications of GCW

Particulars	Details
Type of Curtain Wall System	Stick System
Type of Glass	Annealed
Glass Panel Size	840 mm x 2250 mm
Thickness of Glass Panel	6 mm
Effective Length of Mullion per panel	2250 mm
Effective Length of Transom	840 mm
Wind Pressure on Panels	1.62 kN/m^2
Bracket Connection	M. S. bracket with Hilti M12 anchors

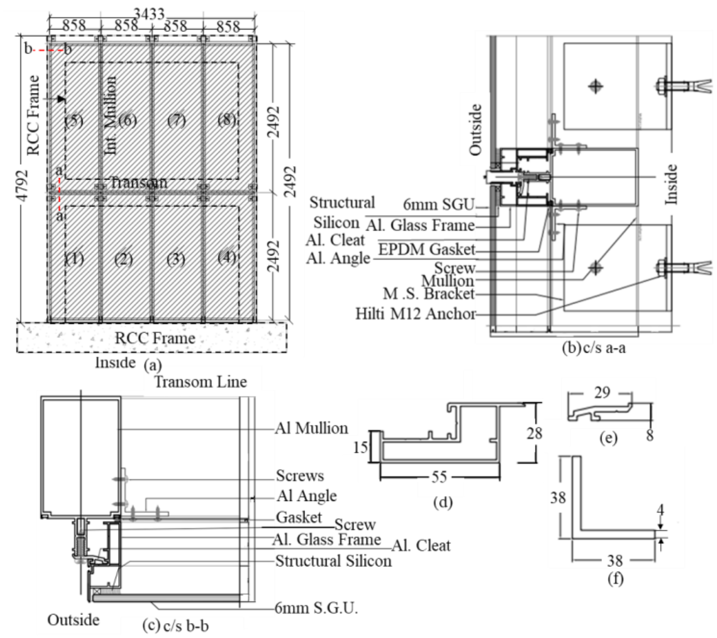


Figure 2. (a) GCW frame details (b) Intermediate mullion/transom joint details (c)Mullion section (d) Glass frame section (e) glass section cleat. (f)Al. angle at the joint.

2.2 Setup and Loading Protocol

The foundation beam is connected to the strong floor with the help of high-strength bolts. The lateral supports at the topmost RC beam have been provided to restrain the out-of-plane movement of the test specimen as shown in Figure 3. A 100 mm clearance has been provided between the roller bearings and bracket connecting the top transom to the RC frame to avoid roller interaction directly with any of the components of the façade frame. A loading beam with three servo-hydraulic actuators is used to simultaneously apply vertical loads and lateral displacement at top beam. Two 250 kN servo-hydraulic actuators initially apply a 40 kN vertical load on each column to simulate the gravity load imposed on the test specimen in a force-controlled mode. A servo-hydraulic actuator having capacity 500 kN and ± 250 mm stroke length is utilized to apply the displacement-controlled loading protocol on the loading beam.

Strain gauges 120 Ω resistance and 5 mm gauge length are attached to the reinforcement within the RCC frame in potential hinge regions. Additionally, they have been placed on the mullions and transoms to measure the level of inelasticity observed in the curtain wall system. Linear variable differential transformers (LVDT) are placed at critical locations on the frame to capture the global deformation of the GCW. The lateral strength of the test specimen has been recorded using the load cell in-built into the actuator.

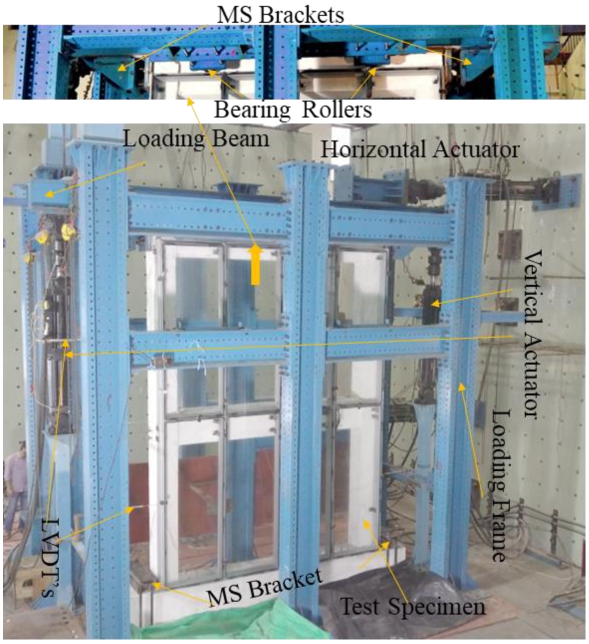


Figure 3 details the experimental setup (a) Loading Frame photograph taken from the front left. (b) Image containing bearing roller and side brackets.

To find the capacity of post-installed mechanical anchors, a torsion-controlled pull-out test has been conducted on Hilti M10 anchors located in cracked concrete. The test consists of a 2 m x 2 m concrete block of concrete grade M25 grade. A pull-out test with the help of a hydraulic torsion-controlled instrument on anchors embedded in uncracked concrete parts has been carried out, followed by generating a minimum 0.3 mm wide crack passing through anchors manually with the help of a hammer, as shown in Figure 4.
As the objective of the study is to understand the RC frames and curtain wall's combined response, the displacement loading protocol has been applied to the RC frame according to ACI [20] as shown in Figure 5.

(a) (b)

Figure 4. (a) Torsion-controlled hydraulic pullout tester. (b) Crack generation in the line of anchors with the help of steel wedges

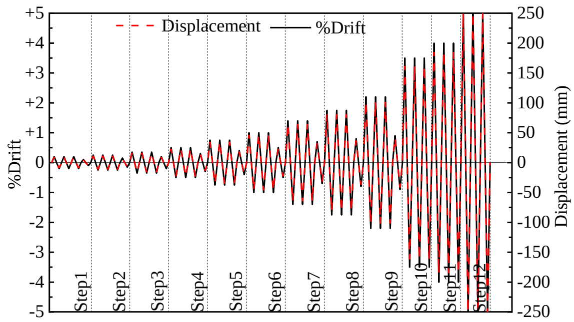


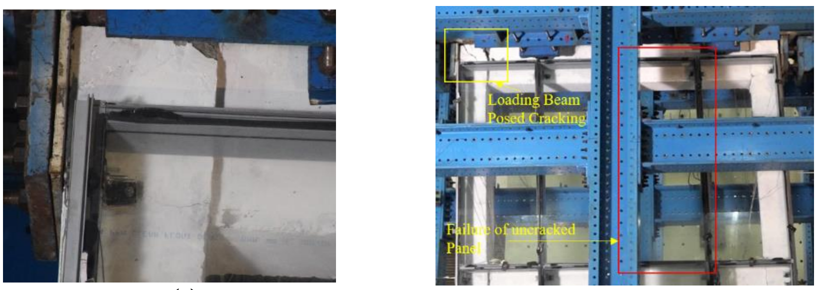
Figure 5. Loading protocol adopted for the test.

3 Experimental Observations

Till 0.5% drift, no cracking in any part of the test specimen, including the RC frame and GCW, is observed. At 1% drift, the structural sealant used to join the glass with the aluminum frame started coming out of the double-sided sealant stip. This observation indicates the interaction of the glass panel and aluminum frame. A vertical crack in the beam near the beam-column joint at the 2nd floor beam is also observed (Figure 6(b)). At 1.4% lateral drift, spalling of concrete is observed (Figure 7(a)) at the top left corner of the RC frame. This may be attributed to the interaction between the loading beam and the second-floor beam. The glass panel failure is observed at 3.5% drift.

(a) (b)

Figure 6. (a) Squeezed silicon fluid, (b) Cracks near the beam-column joint.



(a) (b)

Figure7. (a) loading beam-RC frame interaction (b) falling-out failure of the glass panel

Finally, at 5% drift, fallout-type failure of other glass panel occurred. During the inspection carried out post-completion of all loading steps, it is observed that clips installed to fix the glass panels temporarily had dislocated. The structural sealant double-sided tape used to paste glass panels is observed to be dislodged, as shown in Figure 8. A random clearance has been provided between the face of the aluminum section, and the edges of the glass have varied, and the functionality of GCW has been lost.

(a) (b)

Figure 8. (a)Tilted clip placed on intermediate mullion (b) Squeezed silicon and damaged silicon tape of the concrete frame

Post completion of the test, major cracks are found to be concentrated near the joint. In Figure 9, the top right corner has been shown with and without bracket. Crack measured over all joints varied in size. Most of the anchors in the cracked concrete filed by slipping from the cracks in the pull-out strength with reduced capacity.

(a) (b)

Figure 9. (a) Image of cracked joint with bracket (b) Mapped cracking and cracks passing through anchors

4 Results and Discussion

The hysteresis curve is plotted for RCC frame, as shown in Figure 10. The RCC frame has a maximum capacity of 101 kN at 122.6 mm displacement. The sealant damage is seen at 50 mm displacement. The first glass panel failure is also observed at peak load of the frame.

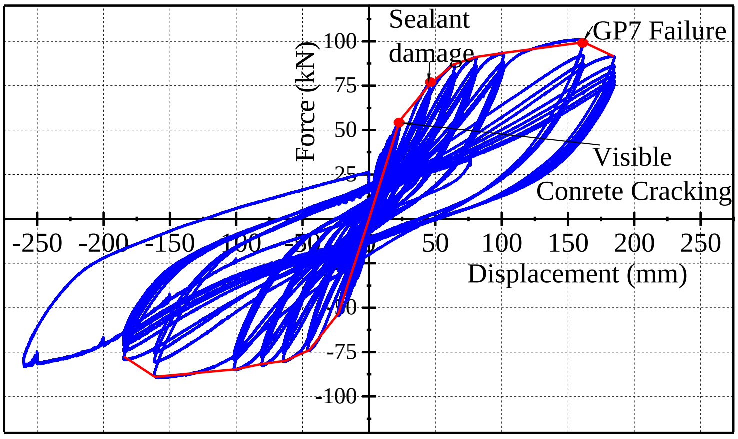


Figure 10. Hysteresis curve of the RCC specimen

The state of strain level at the point located in the abovementioned figure has been determined in all subsequent steps for associated drift from most of the working strain gauges placed on rebars and plotted in Figure 11. The top bars of the upper beam yielded at 0.75% drift followed by left column bars near the top corner at 1.75% drift. The strain data for aluminum sections indicated strain less than yield strain, except SG 25, thus indicating aluminum sections have not been yielded and remained elastic throughout the test. Due to local deformations, an aluminum section at a point located near SG25 has yielded, as shown in Figure 12. Figure 12 (a) is a plot of strain data from all working SG, and Figure 12 (b) is a plot of strain data from the same SGs except SG25. It is an industrial practice to consider the end condition of the mullion/transom as a pin connected, evidenced by feeble elastic strain recorded near the mullion-transom joint.

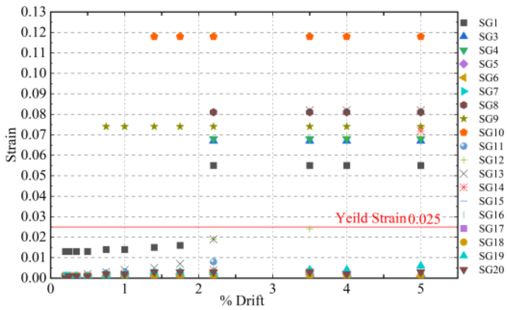


Figure 11. Strain status at various strain gauge locations placed on the rebars.

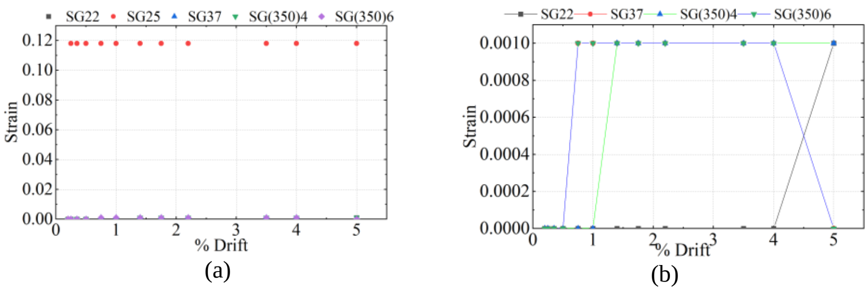


Figure 12. (a) Strain status with the advancement of loading in of all working SG's (b) Feeble strain recorded by SG's placed on aluminum sections except SG25

From visual inspection after testing, it is found that the four-way joint and connection of the GCW frame to the RC frame remained undamaged. No damage is observed in the mild steel bracket and Hilti M10 post-installed anchors connections. Minor cracks have been observed in the RCC frame passing through the anchor location at the connection in the left column adjacent to the bottom beam-column joint. LVDTs that are placed in the out-of-plane direction of the middle point of the mullions recorded an insignificant amount of displacement value. Also, no out-of-pane buckling of aluminum sections is observed.

5 Conclusions

The RCC frame mounted with GCW exterior façade, is tested under quasi-static loading protocol and following conclusion can be drawn:

- The top beam in RCC frame yielded at 0.35% drift followed by column in later drift cycle, but no yielding of aluminum GCW frame is observed.
- The four-way joint and the connection of the GCW frame to the RC frame, including mild steel brackets and Hilti M10 post-installed anchors, remained undamaged.
- The GCW frame's deformation is directly dependent on the RC frame's deformation, highlighting the need for synchronized design considerations.
- The lateral deformation of the primary structural system (RC frame) led to relative displacement of the transoms, resulting in the failure of the glass panels. This underscores the importance of addressing relative movements between structural and non-structural elements to prevent such failures during seismic events.

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