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Seismic Assessment of Glazed Façade Systems

Sivanerupan S[1], Wilson JL[2], Gad EF[3], Lam NTK[4]

1. Corresponding Author. PhD student, Faculty of Engineering, and Industrial Sciences, Swinburne University of Technology, Hawthorn, VIC 3122, Australia. Email: ssivagnanasundram@groupwise.swin.edu.au
2. Professor of Civil Engineering, Faculty of Engineering and Industrial Sciences, Swinburne University of Technology, Hawthorn, VIC 3122, Australia. Email : jwilson@groupwise.swin.edu.au
3. Associate Professor, Dept. of Civil and Environmental Engineering, The University of Melbourne, Parkville, VIC 3010, Australia. Associate Professor of Civil Engineering, Faculty of Engineering and Industrial Sciences, Swinburne University of Technology, Hawthorn, VIC 3122, Australia Email: egad@groupwise.swin.edu.au
4. Professor, Dept. of Civil and Environmental Engineering, The University of Melbourne, Parkville, VIC 3010, Australia. Email: ntkl@unimelb.edu.au

Abstract

The curtain wall façade system is popular in all types of buildings including commercial, industrial and institutional structures. The structural design of curtain walls normally takes into account in-plane and out-of-plane loading from wind, thermal movement and deflection from supporting structural elements due to gravity loads and creep. Seismic loads on the structure can potentially impose significant in-plane loading on the glazing system and may lead to damage if adequate detailing is not provided. In this paper, codified inter-storey drift limits for buildings are reviewed and seismic drift assessment methods of glazed façades in buildings are suggested with increasing accuracy and complexity. Performance of glass façade systems are then assessed with analysis results and conclusions presented.

Key words: curtain walls; inter-storey drift; seismic performance.

1. INTRODUCTION

Glazed façade systems may be subject to racking action due to the relative lateral movement of building from earthquake excitation. The performance of façade system is dependent on the amount of drift and the interaction of the glass panel with the façade support structures. There are two major concerns related to architectural glazing performance during and immediately after a seismic event (Saflex Solutia Architectural Glazing 2007):

- Hazards to people from falling glass. This may cause injuries at street level from broken storefront and elevated glazed panels.
- Building down time and cost to repair. Bringing a building back to operation can be delayed by a breached building envelope due to glazed façade systems damage.

It is evident that in the past earthquakes, glazed systems with sufficient clearance between edges of the glass panel and the supporting structures have performed well. The performance of fixed windows and storefront glazing systems has been tested in laboratories over the past few decades. Researchers have suggested improvements such as addition of smooth corners around each glass panel and adoption of more robust glass types such as heat strengthened, toughened and laminated glasses (Behr 2006). A limited number of analytical studies related to the simulated seismic performance of glazed façade systems were also developed (Memari et al. 2007).

The Standard for earthquake actions in Australia, AS 1170.4 (2007), limits the inter-storey drift to 1.5% in buildings and states that, the "attachment of cladding and façade panels to the seismic-force-resisting system shall have sufficient deformation and rotational capacity". However the seismic drift performance of glazed façades is generally not considered in the design stage by façade engineers.

Analysis results indicate that the inter-storey drift is much less than 1.5% for most buildings in Australia for the 500 year return period (RP) event except for soft storey structures. However a simplified approach is required to calculate the maximum in-plane drift demand and assess the performance of façades. A detail approach is not considered practical since façade engineers are given limited structural information on the building.

This paper addresses three key issues: (i) a review of codified inter-storey drift limits and industry practice (ii) assessment methods for calculating inter-storey drift demand, and (iii) in-plane drift capacity of glazed façade systems. The outcome of this study will be the development of a simple assessment procedure to ensure a minimum level of protection against seismically induced damage in glazed façades for new buildings as well as for existing buildings for retrofitting, for regions of low to moderate seismicity such as Australia.

2. CODIFIED INTER-STOREY DRIFT LIMITS AND INDUSTRY PRACTICE

Drift provisions in Standards are recommended for serviceability and ultimate limit states. "Structural design actions", AS/NZS 1170.0 (2002) provides out-of-plane and in-plane serviceability limit state criteria for building elements. The Standard recommends an in-plane drift limit of H/600 for the brittle masonry wall (where H is the height of the wall) but no limits are specified for glazed façade systems.

The Australian Standard "Concrete structures", AS 3600 (2001) specifies in clause 2.4.3, "unbraced frames and multi-storey buildings subject to lateral loading shall be designed to limit calculated inter-storey lateral drift to H/500 of the storey height". This is aimed for the serviceability limit state of the building mainly for wind loading. Whilst the Standard for "Steel structures" AS 4100 (1998) recommends compliance with AS 1170.4 (2007).

AS 1170.4 (2007), clauses 5.4.4 and 5.5.4, specify that, “the inter-storey drift at the ultimate limit state, calculated from the forces determined according to strength and stability provisions shall not exceed 1.5% of the storey height for each level and “the attachment of cladding and façade panels to the seismic-force-resisting system shall have sufficient deformation and rotational capacity”. This requirement is for the ultimate limit state of the building for seismic performance and a 3600 mm height floor is equivalent to a relative building deflection of 54 mm.

The New Zealand Standard “Earthquake actions”, NZS 1170.5 (2004) specifies in clause 7.5 that, “a maximum inter-storey drift limit of 2.5 % is applicable for the ultimate limit state of 500 year RP event. In the case of a 2500 year RP near fault event, this limit has to be increased to 3.75%. Drift limits of 2.5% and 3.75% create demands of 90 mm and 135 mm respectively on façade systems, assuming a storey height of 3600 mm.

The Council on Tall Buildings, Group SB (1979), examined the serviceability wind drift criteria from industry and literature and found that drift limits ranging from 0.001H to 0.004H were used. However the Council states that buildings designed in the past have been known to perform satisfactorily when designed for drift limits from 0.002H to 0.005H. ASCE Task Committee found that most of the design for institutional, commercial, and residential building types used drift ratio in the order of 0.002H to 0.0025H for steel framed buildings.

The Australian Standard “Glass in buildings-Selection and installation” AS 1288 (2006), provides guidance for the strength and serviceability design of glass subject to out-of-plane wind loading but does not comment on in-plane effects. From discussions with industry experts, the glazed façades are also designed for in-plane racking performance due to wind loading which is usually H/500 for serviceability conditions. Methods to assess the drift demand on buildings for the purpose of assessing glazed façades are described in section 3.

3. ASSESSMENT METHODS FOR CALCULATING INTER-STOREY DRIFT DEMANDS

Seismic drift demand on buildings can be investigated in many ways using elastic or inelastic approaches with static or dynamic analyses. Seismic drift assessment methods in regular buildings are suggested using five tiers with increasing accuracy and complexity.

Tier # 1 – 1.5% inter-storey drift (AS 1170.4)

A 1.5% drift for a 3600 mm storey height corresponds to a lateral deflection Δ = 54 mm. If the capacity of the façade system is higher than this limit, then the façade is considered safe and it is not necessary to carry out further assessment.

Tier # 2 – RSD_{max} from response spectrum (AS 1170.4)

The RSD_{max} method is suitable for buildings dominated by the first mode and typically less than 10 storeys.

$$\text{Inter-storey drift} = \frac{RSD_{max} \times F_{PF} \times F_M \times F_T}{n \times h} \leq 1.5\% \qquad (1)$$

Where RSD_{max} = maximum displacement demand for site class

F_{PF} = Participation factor (1.0 – 1.5)

F_M = mode shape correction multiplier (1.0 - 2.0)

F_T = Torsional amplification factor (1.0 - 2.0)

n = number of storey

h = storey height

The participation factor can be assumed F_{PF} = 1.5 for buildings regular in elevation and F_{PF} = 1.0 for single storey buildings and soft-storey buildings. A value of F_m = 1 can be used for building less than 5 storeys and conservatively the value of F_m = 2 can be used for buildings between 5-10 storeys to account for the curved mode shape (Wilson and Lam 2006). The value of F_T =1 is recommended for symmetric buildings, whilst values of F_T = 1.6 and F_T = 2.0 are conservatively assumed for estimating peak drift demands for buildings that are asymmetric in one and two directions respectively (Lumantarna et al. 2008). Table 1a, 1b and 1c summarise the typical drift demands for different heights of building for 500 year RP (Z = 0.1g) and 1500 year RP (Z = 0.15g) events on different soil types for regular, one and two directional asymmetric buildings in accordance with AS 1170.4 (2007).

No. of Storeys	Inter-storey drift (mm)					
	Class B		Class C		Class D	
	500	1500	500	1500	500	1500
3	17	25	24	36	38	54
8	13	19	18	27	28	42
10	10	15	14	21	23	34

Table 1a Maximum drift demand on façade systems (regular buildings)

No. of Storeys	Inter-storey drift (mm)					
	Class B		Class C		Class D	
	500	1500	500	1500	500	1500
3	27	40	38	57	54	81
8	20	30	29	43	45	68
10	16	24	23	34	36	54

Table 1b Maximum drift demand on façade systems (one directional asymmetric buildings)

No. of Storeys	Inter-storey drift (mm)					
	Class B		Class C		Class D	
	500	1500	500	1500	500	1500
3	34	50	48	71	54	108
8	25	38	36	53	54	81
10	20	30	29	43	45	68

Table 1c Maximum drift demand on façade systems (two directional asymmetric buildings)

Tier # 3 – Δ_e from response spectrum (AS 1170.4)

The Δ_e method also is suitable for buildings dominated by the first mode and typically less than 10 storeys. The natural period of the buildings is required. The effective moment of inertia (I_e) should be used to represent the cracked sections rather than the gross sectional properties (I_g). The use of I_g instead of I_e would potentially lead to a substantial underestimate of the inter-storey drift (McBean 2008). Paulay and Priestley (1992) recommended $0.40I_g$ for beams and $0.60I_g$ for normal columns as effective moments of inertia. In this study an average $0.50I_g$ has been used for all elements to provide a reasonable estimate of the building natural period. The typical displacement demand corresponding to the effective stiffness (Δ_e) is shown in Figure 1.

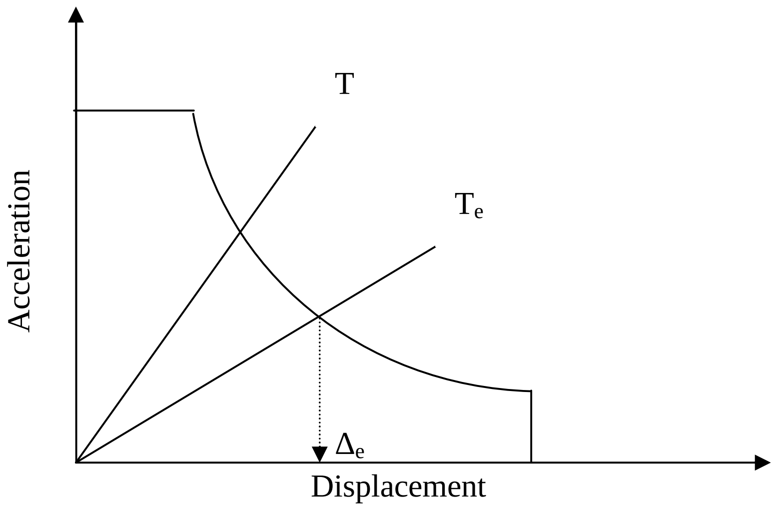


Figure 1 Displacement for effective stiffness of a building

The inter-storey drift can be estimated from the Tier # 3 method as follows:

$$\text{Inter-storey drift} = \frac{\Delta_e \times F_{PF} \times F_M \times F_T}{n \times h} \leq 1.5\% \quad (2)$$

Where Δ_e = displacement demand corresponding to effective period (T_e) and the remaining factors are the same as explained in Tier # 2. Table 2 compares the typical drift demands for the 500 year RP events with Z = 0.1g on different soil types for regular buildings (I_g and $0.50I_g$) in accordance with AS 1170.4 (2007).

No. of Storeys	Period T (s)	$0.50I_g$ period T_e (s)	Inter-storey drift (mm)					
			Class B		Class C		Class D	
			I_g	$0.50I_g$	I_g	$0.50I_g$	I_g	$0.50I_g$
3	0.3	0.43	3.4	4.7	4.2	6.7	4.2	8.4
5	0.5	0.71	3.3	4.7	4.7	6.7	7.0	10.6
8	0.8	1.13	6.7	9.4	9.5	13.4	15.1	21.3
10	1.0	1.41	6.7	9.5	9.5	13.4	15.0	21.3

Table 2 Maximum drift demand on façade systems (regular buildings)

Tier # 4 – Modal analysis using response spectrum method (AS 1170.4)

The method is suitable for buildings greater than 10 storeys where higher mode effects are important. The soil class and modal properties including natural period of the building for effective stiffness are required to undertake the analysis. The well known modal combination rule, the square root of the sum of the squares method is used to calculate the inter-storey drifts. The method is applied for three high-rise buildings. Table 3 summarises the details of the buildings. Table 4 compares the typical drift demand on regular buildings for 500 year RP event for soil classes in accordance with AS 1170.4 (2007).

Building Reference and location	Description	Height (m)	Number of floors	Natural period (sec)			Reference
				Mode 1	Mode 2	Mode 3	
1 Singapore	Frame-tube Office block	280	66	5.4	1.5	0.7	(Brownjohn and Pan 2001)
2 Melbourne	Central core steel frame Office block	152	40	3.8	1.0	0.6	(Swaddiwudhipong et al. 2002)
3 Singapore	Concrete Office block	91	26	1.5	0.4	0.2	(Brownjohn and Pan 2001)

Table 3 Summary of the buildings

No. of Storeys	Inter-storey drift (mm)					
	Class B		Class C		Class D	
	I_g	$0.50I_g$	I_g	$0.50I_g$	I_g	$0.50I_g$
26	2.7	3.0	3.8	4.2	5.9	6.2
40	2.8	3.2	3.9	4.5	6.2	7.2
66	1.7	2.0	2.4	2.9	3.8	4.5

Table 4 Maximum drift on buildings (Z = 0.1g and 500 YRP)

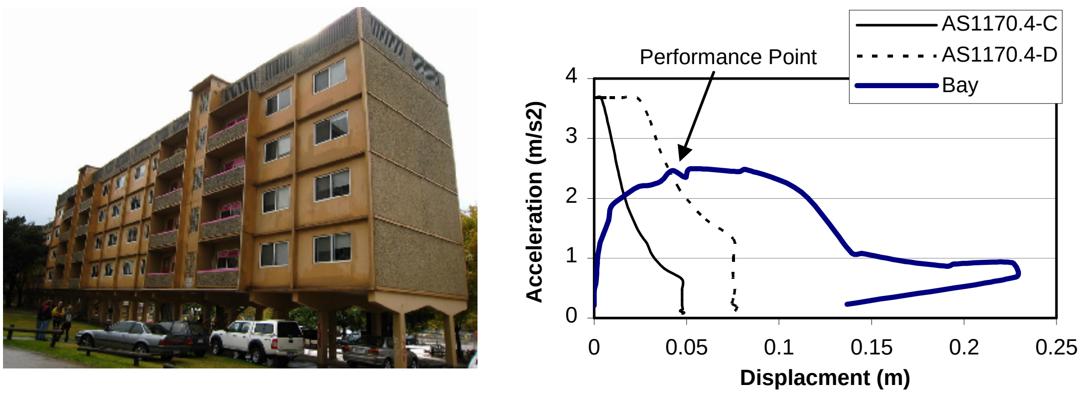
The time-history analysis approach of determining earthquake actions can result in more efficient designs in comparison with using response spectra specified by the Standard (Lam and Wilson 2005).

Tier # 5 – Capacity spectrum method

The capacity spectrum method can be used to estimate the likely inelastic displacement demand of an equivalent single degree of freedom structure. The maximum drift demand on the building can then be estimated from Equation 2 by substituting $\Delta_{inelastic}$ for Δ_e.

$$\text{Inter-storey drift} = \frac{\Delta_{inelastic} \times F_{PF} \times F_M \times F_T}{n \times h} \leq 1.5\% \qquad (3)$$

The capacity spectrum method is illustrated using a real case study example involving a soft-storey building. The building is illustrated in Figure 2a and has 5 storeys with the ground floor open and much more flexible and laterally weaker than the upper storeys. Consequently, under lateral loading the deformations are concentrated at the ground floor level with the columns being forced to drift laterally whilst maintaining the gravity load of the upper storeys. Wibowo et al (2008) undertook a unique pushover experimental study of this building while measuring the lateral force versus displacement behaviour of the building. This has been plotted on an ADRS format diagram as shown in Figure 2b, from where the performance point can be estimated for 500 year RP event (Z = 0.1g) for soil classes. The inelastic displacement demand on a class 'D' site for this building is $\Delta_{inelastic}$ = 45 mm and the inter-storey drift can be calculated from Equation 3 assuming F_{PF} = 1.0, F_M = 1.0 and n = 1.

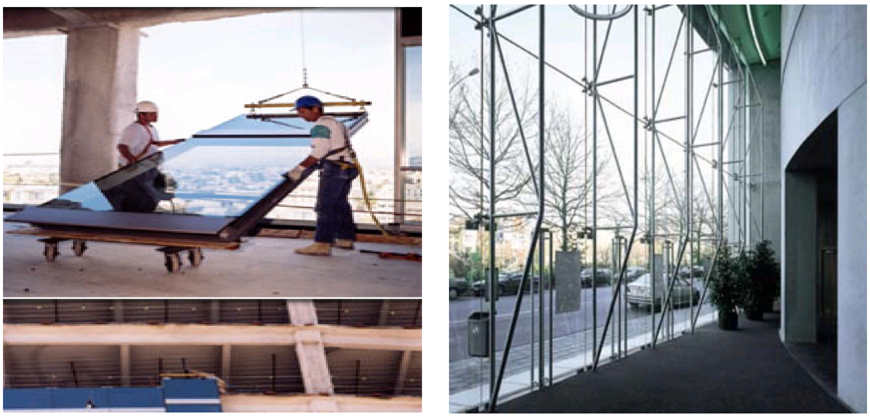


(a) (b)

Figure 2 (a) The tested soft storey building, (b) Pushover curve of the bays in weaker direction

4. IN-PLANE DRIFT CAPACITY OF GLAZED FAÇADE SYSTEMS

The glazed façades can be classified into two main types, namely framed glazed and frameless glazed façade systems. The unitized curtain wall system is a more contemporary framing method which comprises a glass vision panel and spandrel panel mounted in a prefabricated aluminium frame and illustrated as a complete unit in Figure 3a. Alternatively a new contemporary frameless glazed façade system is available which provides transparency and improved aesthetics, known as point fixed or bolt fixed glazed curtain wall system. Point fixed glazing systems are often connected with bolts to steel support structures, (which are exposed architectural elements) to combine structural stability with aesthetic expression. A typical bolted glazing system supported by trusses is shown in Figure 3b.



(a) (b)

Figure 3(a) Assembling of a stick curtain wall system and (b) Bolted glazing supported by truss system

Bouwkamp (1960) observed that in-plane deformation of window panels under lateral loading takes place in two phases, as shown schematically in Figure 4. First, the window frame deforms and the glass plate translates within the frame until contact occurs at two opposite corners of the glass panel (Figure 4b). The glass panel then further rotates until its opposite corners coincide with the adjacent frame corners (Figure 4c). Sucuoglu and Vallabhan (1997) found that the total lateral deformation of the window panel due to rigid body motion of the glass panel in the window frame can be expressed in terms of the geometric properties of window panel components as:

$$\Delta = 2c\left(1+\frac{h}{b}\right) \tag{4}$$

Where Δ is the lateral drift capacity of the glass frame and c, h and b are physical dimensions as defined in Figure 4.

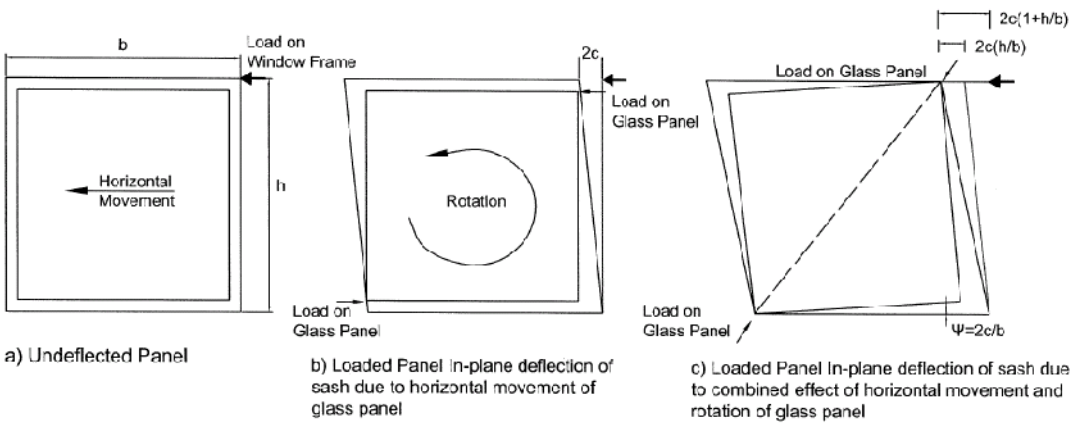


Figure 4 Movement of glass panel within window frame for a glazed window (Sucuoglu and Vallabhan 1997)

Sucuoglu and Vallabhan (1997) suggested that, this expression is valid when the glass panel is glazed with a soft sealant which permits the relative motion of glass panel with respect to the window frame. As the sealant hardens due to ageing, the lateral drift capacity of the widow panel reduces significantly. However, neoprene gaskets and other soft sealants used in modern glazing systems possess sufficient resilience to accommodate the relative motion of glass panels in window frames. Equation 4 indicates that the in-plane drift capacity of the glazed frame, before glass breakage is only dependent on the edge clearance and the aspect ratio. Typical edge clearances used in general practice range from 6 mm to 13 mm. Table 5 compares the in-plane drift capacity of 3600 high frame glazed curtain wall with different widths and edge clearances using Equation 3.

Height (h) (mm)	Width (b) (mm)	Aspect ratio (h/b)	In-plane drift capacity for typical edge clearances (mm)			
			c = 6	c = 8	c =10	c =12
3600	3000	1.2	26	35	44	53
3600	2400	1.5	30	40	50	60
3600	1800	2.0	36	48	60	72
3600	1200	3.0	48	64	80	96

Table 5 Typical in-plane drift capacity of framed glazed curtain walls

The in-plane drift capacity of a curtain wall with an aspect ratio of h/b = 2 and for the typical minimum edge clearance of c = 6 mm is therefore $\Delta = 36$ mm as summarised in Table 5. This drift capacity satisfies the drift demand of buildings analysed in Tier # 3 and Tier # 4. Some of the in-plane drift capacity figures in Table 5 also satisfy the demand of 54 mm as given in Tier # 1. It shows that for buildings with higher drift demands (especially in soft-storey buildings) the in-plane drift capacity of the framed façades can be modified by increasing the edge clearance or aspect ratio. Equation 4 can be modified for uneven clearances between vertical and horizontal glass edges and the frame (ASCE 7-02 2002) as:

$$\Delta = 2c_1\left(1 + \frac{h_p c_2}{b_p c_1}\right) \tag{5}$$

Where h_p = height of the rectangular glass panel, b_p = width of the rectangular glass panel, c_1 = clearance (gap) between the vertical glass edges and the frame, and c_2 = clearance (gap) between the horizontal glass edges and the frame.

The seismic performance of point fixed (frameless) glazing is likely to be quite different from conventional framed systems. McBean (2008) commented that the capacity of point fixed glazing is at least half of the capacity of framed glazing. There is very limited published research available on the behaviour of frameless glass façade systems under in-plane actions earthquake loading and a reliable and rational testing program is required to assess the drift performance of such systems (Sivanerupan S et al. 2008).

5. CONCLUSIONS AND SUMMARY

The seismic assessment of glazed façade systems requires an estimate of the likely drift demand from the building. Codified and industry provisions for in-plane drift limits on glazed façade systems are reviewed. Analysis results indicate that the inter-storey drift is much less than the 1.5% limit in AS 1170.4 (2007) for most buildings in Australia for the 500 year RP event except for soft storey structures. It reveals that standard methods should be used for estimating the in-plane seismic drift demands on glazed façade systems. A tiered approach has been presented for estimating the seismic drift demand of glazed façade systems with increasing levels of sophistication and accuracy. Applications of these methods are illustrated with number of examples and conservative factors are presented for considering the torsional behaviour of buildings.

The drift capacity of a framed glazed system is dependent on the edge clearance and the aspect ratio before glass breakage. Results indicate that framed glazed façade systems with typical minium edge clearance in regular buildings are not vulnerable except soft-storey buildings for moderate earthquake events such as 500 year RP event. However for torsionally unbalanced buildings, the maximum drift limit of 54 mm for a typical 3600 high floor can be achieved by increasing the edge clearance or aspect ratio of the glazed frame.

Despite its growing popularity, there is very limited published research on the behaviour of frameless glass façade systems under the in-plane earthquake loading. The seismic performance of point fixed (frameless) glazing is likely to be quite different from conventional framed systems. A reliable and rational testing and analytical work is required to assess the drift performance of point fixed glazed façade systems. This is also part of the on-going research undertaken by the authors to evaluate the vulnerability of glazed façades under in-plane seismic loading.

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