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Drift Intolerant Façade Systems and Flexible Shear Walls. Do we have a Problem?

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1.0 INTRODUCTION

We have all heard the saying "a chain is only as good as its weakest link". In earthquake engineering, we frequently rely on this principle when employing the Capacity Design approach to proportion our structures and their elements. However, as Structural Engineers we are often guilty of being preoccupied with the response of the primary structure, at the expense of proper consideration of so called non-structural elements. These non-structural elements have the potential to become the weakest link in an otherwise well resolved building, and in the event of a major earthquake, create a significant life safety hazard.

For this reason, the Australian Standard for Earthquake Actions AS1170.4 (1993) has specific provisions for the design of non-structural parts and components. Items such as parapets, chimneys, curtain walls, ceilings and partitions are addressed. The method of assessment focuses on the determination of inertial forces generated by such elements, and the adequacy of their attachment to the primary structure.

AS1170.4 (1993) also establishes limits on allowable inter-storey drift, and requires the designer to ensure that cladding and façade attachments are capable of accommodating the calculated design storey drift (d_{st}). The new Draft Standard AS/NZS 1170.4 Draft D5212-5.1(2005) repeats these requirements, but introduces a new statement to the effect that all drift provisions are "deemed to be satisfied if the primary seismic force-resisting elements are structural walls that extend to the base". This additional provision will, in the presence of a shear wall system, relieve the designer from any requirement to consider drift compatibility between façade systems and building superstructure, irrespective of wall stiffness.

1.0 INTRODUCTION (cont)

This paper presents the author's concern regarding the attachment of drift intolerant façade systems to buildings in which the primary seismic force-resisting elements are relatively narrow and therefore flexible shear walls. The recent design of two buildings currently under construction in Adelaide is used to illustrate how cladding drift capacity can govern the lateral stiffness requirements of buildings incorporating narrow, flexible shear walls. It is the author's opinion that the proposed deemed to satisfy provision of the Draft Standard could, during a major earthquake, lead to situations in which the drift capacity of currently fashionable façade systems is exceeded, creating a significant life safety hazard from falling glass.

2.0 OVERVIEW OF EXAMPLE BUILDINGS

2.1 Building One – Five Storey Office, Adelaide

Example Building One is a five storey steel framed office building, currently nearing completion in Adelaide. The building is approximately 64m in length and 50m in width, creating a typical floor plate area of 3,200 square metres. Floor to floor height is 4.5 metres, except for ground to first floor level, which is 6.0 metres. Due to the nature of business conducted within the building, the client required that the building be classified as a post-disaster facility, and an importance factor of 1.25 was applied to seismic loads determined in accordance with AS1170.4 (1993). This is consistent with an annual probability of exceedence of 1/800 using the current Building Code of Australia (2005), and Draft Standard. Figure 1 illustrates the floor plan, and locates elements of structural interest.

Primary seismic force-resisting elements of the Building One consist of:

- The western full height loadbearing precast wall
- The insitu reinforced concrete lift shafts to the west
- The insitu reinforced concrete eastern stair core

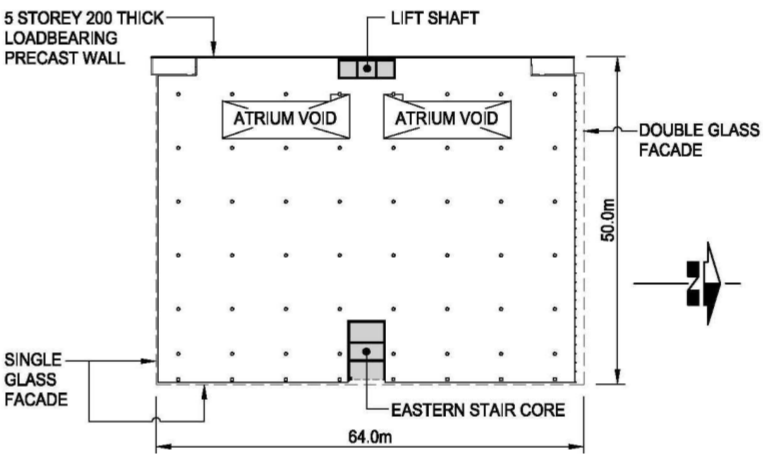

Figure 1 – Building One, Typical Floor Plan

2.0 OVERVIEW OF EXAMPLE BUILDINGS (cont)

2.1 Building One – Five Storey Office, Adelaide (cont)

The western façade consists of loadbearing precast concrete, forming a north-south shear wall running the full length of the building. All other facades consist of architectural glazing, partially constructed from captive glass housed in aluminium mullions, some of which incorporate movement joints (refer Figure 2), and partially built using butt jointed glass supported on "spider" corner fixings as shown in Figure 3.

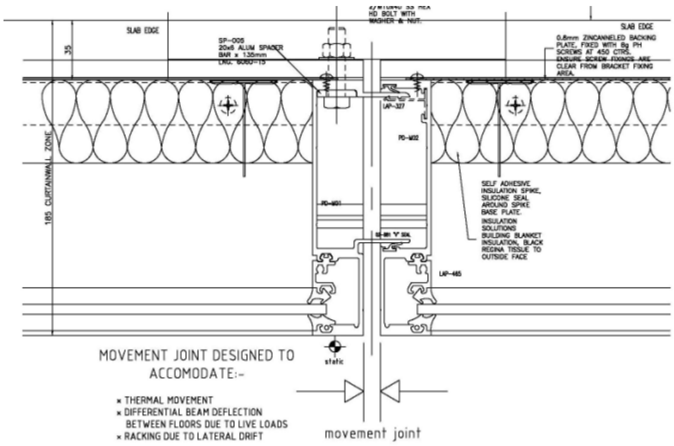

Figure 2 – Typical Movement Joint to Captive Glass Façade System

(a) (b)

Figure 3 - Building One, butt jointed glass façade supported on "spider" corner fixings (a) general view of façade facing street, (b) close up of double skin glass.

2.2 Building Two – 14 Storey Office, Adelaide

Example Building Two is a fourteen storey concrete framed office building currently under construction in Adelaide. The building is 67.5m in length and 23 metres wide, giving a floor plate area of approximately 1,500m^2. Typical floor to floor height is 3.525 metres. A combination of non-loadbearing precast and architectural glazing systems similar to those found on Building One are used to clad the structure. Resistance to lateral loads is provided by a combination of perimeter moment frame action together with an insitu reinforced concrete lift and service core. Figure 4 illustrates the relevant elements of the building.

2.0 OVERVIEW OF EXAMPLE BUILDINGS (cont)

2.2 Building Two – 14 Storey Office, Adelaide (cont)

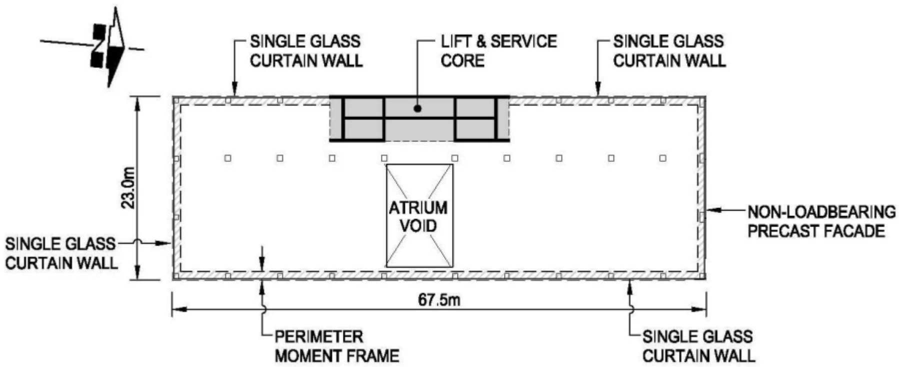

Figure 4 – Building Two, Typical Floor Plan

3.0 DRIFT CALCULATION

In accordance with AS1170.4 (1993), inter-storey drift estimates for both buildings were calculated using an elastic analysis and applying the design ultimate earthquake actions. The elastic deflections so obtained were then scaled up by K_d to approximate the inelastic deflected response of the structure.

In undertaking sway deflection calculations it is important that the element stiffnesses used reasonably estimate the effective stiffness at, or close to, member yield. Various recommended stiffness values can be found in the literature, the author having adopted those suggested by T. Paulay and N. Priestley (1992). Stiffness values estimated for the perimeter moment frame in Building Two were in the following range:

Beams $I_e = 0.40 – 0.50\ I_g$
Columns $I_e = 0.50 – 0.80\ I_g$

Lightly loaded columns in tension were assigned lower stiffnesses, whilst heavily loaded columns were assigned the higher stiffness values in the range indicated.

In recent years there has been significant debate regarding the appropriate design stiffness of slender cantilever shear walls subjected to earthquake loading. N. Priestley and T. Pauley (2002) in their discussion of the work by R. Fenwick and D. Bull (2000) highlight some of the issues. The various models proposed by different researchers strongly diverge for lightly loaded walls with low reinforcement ratios (<0.010). However, it would seem that irrespective of assumptions made in formulating the stiffness models, for practical load levels in medium rise structures (5-10 storeys), which incorporate moderate to high levels of reinforcement (reinforcement ratio 0.012 to 0.025), the models are all in reasonable agreement.

3.0 DRIFT CALCULATION (cont)

For the analysis of cantilever shear walls subjected predominantly to flexure in the example buildings, the following simple expression proposed by T. Paulay and N. Priestley (1992) has been adopted:

$$I_e = \left(\frac{100}{f_{sy}} + \frac{P_u}{f_c' A_g} \right) I_g$$

For the example buildings, effective stiffness was found to lie in the range of $I_e = 0.3 - 0.45\ I_g$. All shear wall footings are heavily piled into stiff clays. Additional deflections arising from footing rotation were considered to increase cantilever displacements in the order of an additional 10%.

4.0 PERMISSIBLE DRIFT

4.1 AS1170.4 (1993) Drift Limits

The design storey drift in both the current and draft versions of AS1170.4 is not permitted to exceed 1.5% of the storey height. For our example buildings, this establishes the following upper drift limits.

Example Building	Floor – Floor Height (mm)	Permissible Design Storey Drift (mm)
One	4,500	68
Two	3,525	53

Table 1

These drift levels are set to ensure that the design storey drift does not exceed that which is consistent with the available element ductility based on structural detailing requirements of both AS1170.4 (1993), and the various material Standards used during design.

4.2 Façade Drift Capacity

Architectural glazing systems currently in use throughout Australia have evolved to suit marketplace demands from an aesthetic, economic, and functional perspective. In terms of structural design, they readily accommodate building movement arising from elastic and long term floor deflection, wind, thermal expansion of cladding elements, shrinkage and creep of both floor plates and columns, together with other movements that may arise during the service life of the structure. When evaluated in terms of inter-storey drift, these movements, in aggregate, create an inter-storey displacement demand on façade systems in the order of 10-15mm (0.3% drift). Commercially available curtain walls envisage serviceability movements in this range. Specific details such as split mullions and flexible attachments are available to increase the available movement, but add both cost and complexity to the façade system.

4.0 PERMISSIBLE DRIFT (cont)

4.2 Façade Drift Capacity (cont)

Beyond the available serviceability drift limits of 10-15mm, additional racking arising from the inelastic response to earthquake actions soon leads to interaction between the glazing frame and glass, inevitability resulting in damage to the glass. Further drift eventually reaches the drift capacity of the facade system, after which glass fallout can occur creating a life safety hazard.

From discussions with industry, typical curtain wall systems would seem to reach this ultimate condition at an inter-storey drift of approximately 30-40mm. Systems incorporating butt jointed glass supported on "spiders" would seem to be able to tolerate only half of this movement without loss of structural integrity. In spite of the Australian Standard for Testing of Building Facades, AS/NZS 4284 (1995), specifically requiring seismic testing on façade systems, little data is available on real drift performance of commercial systems. Available data would suggest that drift limits set for superstructure performance within AS1170.4 (refer Table 1) are well beyond that currently envisaged by façade manufacturers.

Koffel, W. et al (2005) discuss the vulnerability of architectural glass in recent U.S. earthquakes, and present a number of strategies currently being researched to improve their seismic performance.

5.0 DRIFT DESIGN

A first pass analysis was conducted for both buildings based on strength requirements alone of AS1170.4 (1993). The structural elements so proportioned were then used to calculate design inter-storey drift values, using the procedure outlined in Section 3.0 above. These drift estimates were then compared to the drift capacity limits established by the proposed façade detailing. The drift limits used for design were as shown in Table 2 below:

Building	Façade Drift Capacity
One	20mm (0.45%)
Two	30mm (0.85%)

Table 2

It was found for Building One that the initial drift estimate in the east-west direction was close to, but exceeded the 20mm limit set. The stiffness of the eastern stair core was then increased to reduce the maximum inter-storey drift to 21mm, which was accepted. This required a change in geometry of the shear walls used within the stair core. North-south drift was well under the target value owing to the great stiffness of the boundary wall.

5.0 DRIFT DESIGN (cont)

Similarly for Building Two, the calculated drift based on the structure proportioned for strength alone, exceeded the target value of 30mm in the east-west direction. It was impossible for planning reasons to effectively stiffen the response of the narrow western core. Supplementary stiffness was therefore introduced by creating a substantial perimeter moment frame at the northern and southern ends of the building. The proportions of the frame were adjusted to achieve the target drift, with the maximum drift occurring at Level 2.

6.0 DISCUSSION AND CONCLUSION

It is well known that shear walls provide an excellent strategy for improving the seismic response of buildings. M. Fintel (1995) conducted a detailed study of shear wall performance in real earthquakes commencing with the Chilean earthquake of May 1960, and concluded that to the best of his knowledge, "not a single concrete building containing shear walls has collapsed". Shear walls control drift, thereby reducing displacement demand on gravity systems and non-structural components.

However, the author has identified two recent projects containing flexible shear walls, in which the inter-storey drift capacity of glazed curtain wall façade systems was exceeded by design solutions based on strength alone. Both buildings required design revisions to ensure inter-storey drifts were reduced to values consistent with drift capacity based on the façade detailing.

If we are to effectively manage the life safety hazard from falling glass, it may be necessary to introduce slenderness limits on shear walls, above which the designer is required to verify drift performance by calculation. The author suggests that further work be undertaken to establish such limits.

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