SEISMIC SAFETY OF MULTI-STOREY BUILDING FACADES IN URBAN AREAS

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ABSTRACT:

Mainstream earthquake engineering research on life safety in buildings is concerned mainly with structural failures which affect building occupants. However, past earthquake experience worldwide has seen falling debris from damaged building façades being a cause of widespread casualties and injuries particularly if the building is in a crowded urban locality.

Contemporary earthquake codes of practice typically provide force-based assessments for the strength of building façades as for other non-structural components. This does not address the risk of breakage or dislodgement of façade units arising from dynamic inter-storey drifts. Of particular concern is buildings in low and moderate seismic regions where façades are typically not well separated from the structure. Whilst low rise buildings are structurally more vulnerable (due to the higher inter-storey drifts which are contributed significantly by higher modes effects).

The authors have been awarded a small grant to undertake a one year pilot study on seismically induced damage to building façades. This paper presents interim research findings on two fronts: (i) inter-storey drift demand (which depends on the site seismic hazard level and the building response) (ii) inter-storey drift capacity (which is limited by the deformability of the façade). The developing methodology should enable the seismic performance of building façades to be conveniently checked in practice.

1. INTRODUCTION

Damage to building facades, vertical piping and the like in medium and high-rise buildings account for more than 50% of the damage repair bill (Brunsdon, 2000). Failures of facades in tall buildings in a congested urban environment can also cause injuries and deaths as well as costly disruptions to the continuous function of facilities. The conditions of the façade panels are very much dependent on inter-storey drifts which can permanently distort the connecting brackets and cause damage to the butt joints between adjoining panels. Such effects have not been adequately addressed by the current codified force-based (FB) provisions which are primarily concerned with inertia forces generated in the components. This paper is structured to address two key issues: (i) Prediction of the inter-storey drift demand (Section 2) and (ii) Prediction of the inter-storey drift capacity (Section 3). The outcome of this pilot investigation is the development of a simple design/assessment procedure which is aimed at ensuring a minimum level of protection against seismically induced damage in facades for new buildings as well as for existing buildings through retrofitting.

Seismic demand predictions begin with the seismic hazard level. In the Australian context, hazard level can be expressed in terms of the design peak ground velocity (PGV) or the acceleration coefficient, a, as defined on hazard contour maps in the current Australian Earthquake Loading Standard (AS1170.4). The PGV, or a, can be related to the displacement demand (Δ_e) of the building based on soil conditions and the dynamic properties of the building. The adopted relationships have been developed from research described elsewhere (eg. Lam, 2000 & 2001a&b; Koo, 2000), but key assumptions will be stated in the paper. This Δ_e demand which represents the overall displacement response of the building can be translated into the inter-storey drift angle. Reasonable estimates of the inter-storey drift demand can be obtained from dynamic analysis using a realistic displacement response spectrum. An alternative convenient scaling procedure is described in Section 2 to determine the seismically induced dynamic drift-angle based on existing wind analysis calculations.

2. PREDICTION OF INTER-STOREY DRIFT DEMAND

A PGV of 60mm/sec (i.e. a=0.08) as designated for most capital cities in Australia for a return period of 500 years is translated to a maximum response spectral displacement of 30mm for rock sites, as shown in the companion paper published in this volume (Lam, 2001b). A 50m sediment possessing an average shear wave velocity of 200m/sec is assumed to overlie hard Silurvian mudstone which implies a high soil-rock impedance contrast and a site natural period of 1 second. The maximum response spectral displacement allowing for the effects of soil resonance is estimated at about 120mm (Lam, 2000).

The effective displacement demand (Δ_e) is related to the maximum inter-storey drift angle (θ_{max}) by Eq.1:

$$\theta_{\max} = \lambda_{\max} \frac{\Delta_e}{H} \tag{1}$$

where $\lambda_{\max}$ is the dynamic drift angle factor and H is the building height.

The inter-storey drift angle (θ_j) contributed by an individual mode of vibration, j, can be expressed in terms of Eqs.2a and 2b:

$$\theta_j = PF_j\left(\frac{MAX(\delta_i-\delta_{i-1})_j}{\underset{n}{H}}\right)RSD_j = \left(\frac{\{\delta\}_j^T\{M\}\{1\}}{\{\delta\}_j^T\{M\}\{\delta\}}\right)\left(\frac{MAX(\delta_i-\delta_{i-1})_j}{\underset{n}{H}}\right)RSD_j \qquad (2a)$$

$$\theta_j = \left(\frac{\sum_{i=1}^{n} m_i\delta_i}{\sum_{i=1}^{n} m_i\delta_i^2}\right)_j\left(\frac{MAX(\delta_i-\delta_{i-1})_j}{\underset{n}{H}}\right)RSD_j \qquad (2b)$$

where PF_j is the modal participation factor, δ_i is modal displacement, m_i is the storey mass and RSD_j is the modal response spectral displacement.

By letting $\theta_j = \lambda_{\max j}\frac{RSD_j}{H}$

$$\lambda_{\max j} = \frac{n(MAX(\delta_i-\delta_{i-1}))_j\left(\sum_{i=1}^{n} m_i\delta_i\right)_j}{\left(\sum_{i=1}^{n} m_i\delta_i^2\right)_j} \qquad (2c)$$

By modal combination of the first three vibration modes: j =1-3 using the "square-root-of-the-sum of the squares" method:

$$\theta_{\max} = \sqrt{\theta_1^2+\theta_2^2+\theta_3^2} \qquad (2d)$$

$$\theta_{\max} = \sqrt{\lambda_{\theta 1}^2+\lambda_{\theta 2}^2\left(\frac{RSD_2}{RSD_1}\right)^2+\lambda_{\theta 3}^2\left(\frac{RSD_3}{RSD_1}\right)^2}\,\frac{RSD_1}{H} \qquad (2e)$$

$$\text{or } \lambda_{\max} = \sqrt{\lambda_{\theta 1}^2+\lambda_{\theta 2}^2\left(\frac{RSD_2}{RSD_1}\right)^2+\lambda_{\theta 3}^2\left(\frac{RSD_3}{RSD_1}\right)^2} \qquad (2f)$$

For three example buildings shown in Figs.1a-1c, $\lambda_{\max}$ factors were determined using Eqs.2a-2f and the corresponding values are listed in Table 1. The modal properties for two of the examples (Figs.1a & 1b) were obtained from vibration monitoring of two real buildings in Singapore (Brownjohn, 2000). For the third example (Fig.1c) which is an irregular building featuring a transfer plate (Su, 2000), modal analysis was conducted. The ratios $\left(\frac{RSD_2}{RSD_1}\right)$ and $\left(\frac{RSD_3}{RSD_1}\right)$ were taken to be 1.0 and 0.5 respectively based on the assumptions stated in Fig.2. A consistent $\lambda_{\max}=3.4$ was obtained for the two buildings shown in Figs.1a-1b and a higher factor of 4 for the irregular building shown in Fig.1c.

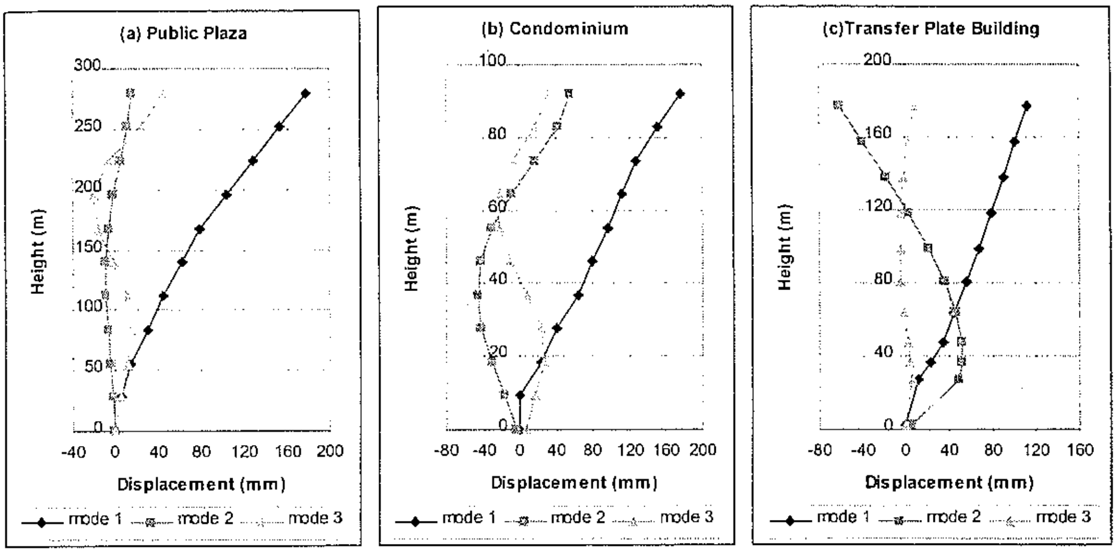


Figure 1: Modal deflections of example buildings with a common effective displacement of 120mm, (a) Public Plaza and (b) Condominium, after Johnbrown (2000) and (c) Transfer Plate Building, after Su (2000).

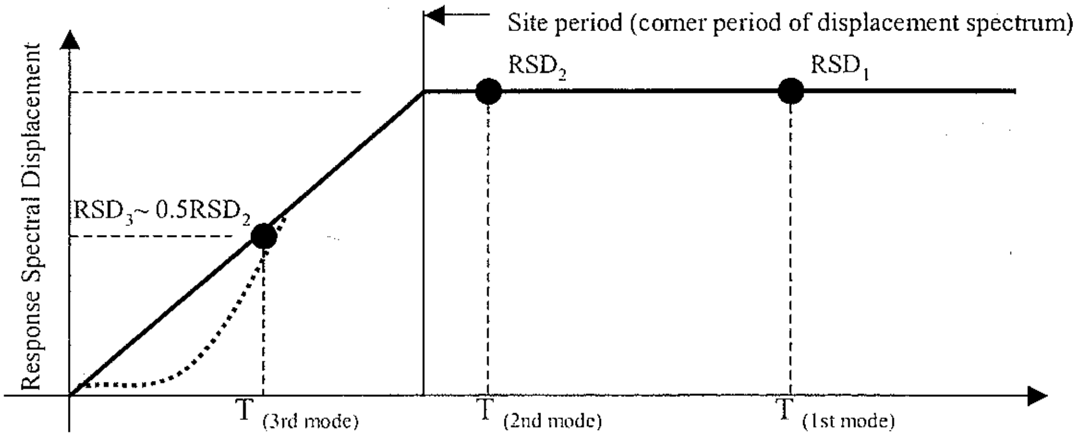


Figure 2: Assumed relative response spectral displacement between individual modes of vibration.

The overall state of deformation in a building can be expressed conveniently in terms of the average drift angle (θ_{ave}) which is defined as the roof displacement (associated with the fundamental vibration mode) divided by the building height (H). θ_{ave} can be related to Δ_e as follows :

$$\theta_{ave} = \frac{\lambda_{ave}\Delta_e}{H} \quad (with\ \lambda_{ave} \approx 1.5) \qquad (3)$$

The λ_{ave} values as calculated for the example buildings are shown in Table 1 along with the λ_{max} values. The ratio $\theta_{max}/\theta_{ave}$ is shown to range between 2.3 and 2.8. Thus, it is reasonable to take an average value of 2.5 as an initial approximation. However, a more conservative assumption of $\theta_{max}/\theta_{ave}=3$ provides some allowance for building irregularity.

Table 1: Calculated drift angles for the example buildings shown in Fig.1

Example Buildings	λ_{max}	λ_{ave}	$\theta_{max}/\theta_{ave}$
Public Plaza (Fig.1a)	3.4	1.5	2.3
Condominium (Fig.1b)	3.4	1.5	2.3
Transfer Plate Building (Fig.1c)	4.2	1.5	2.8

The significance of the average drift angle (θ_{ave}) is that it can be estimated from a quasi-static analysis since θ_{ave} is contributed only by the fundamental mode of vibration. Consequently, θ_{ave} (and hence θ_{max}) can be obtained from wind analysis calculation based on simple scaling. The dynamic analysis as described above assists in the development of this scaling procedure which comprises the following steps (see also diagrammatic illustration in Fig.3):

(i) Identify the average drift angle induced by the design wind forces ($Wind\theta_{ave}$).

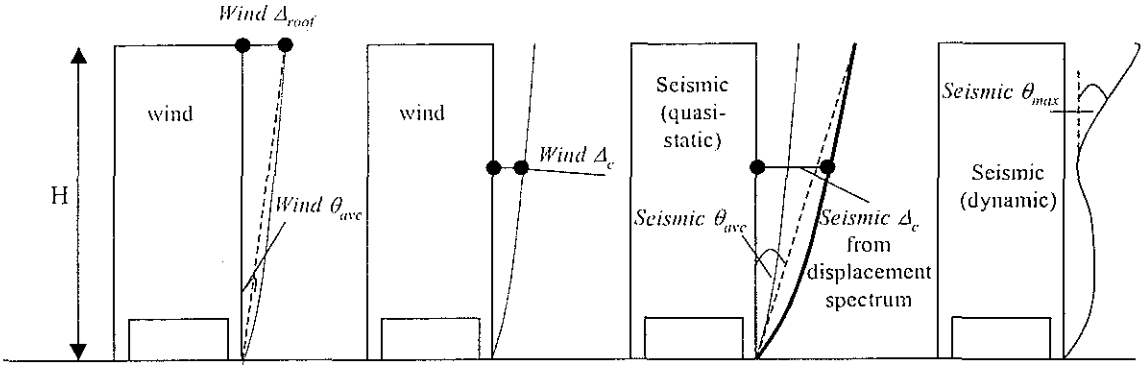
(ii) Calculate the effective displacement associated with the wind induced deflection ($Wind\Delta_e$) which is approximately $\sum\delta^2/\sum\delta$.

(iii) Apply scaling: $$Seismic\theta_{ave} \approx Seismic\Delta_e \frac{Wind\theta_{ave}}{Wind\Delta_e} \approx 120\times\frac{1.5}{H} \qquad (4)$$

where $Seismic\Delta_e$ can be read off from a displacement response spectrum.

(iv) $$Seismic\theta_{max} \approx 3\times Seismic\theta_{ave} \approx 3\times\frac{180}{H} \approx \frac{540}{H} \qquad (5)$$

where the factor 3 (or 2.5) is based on the observations from Table 1.



(i) Calculate average wind induced drift angle θ_{ave}

(ii) Calculate average wind induced effective displacement Δ_e

(iv) Estimate maximum dynamic drift angle θ_{max}

Figure 3: Procedure to extrapolate wind induced drift demand to seismically induced drift demand.

3. PREDICTION OF INTER-STOREY DRIFT CAPACITY

Inter-storey drift may impose both in-plane and out-of-plane forces on facades. The response of facades to such loads depends on several factors, including the stiffness and strength of the facade panels and the connections between the panels and the structure. The detailing of the connections between the façade and the supporting structure is influenced by face wind loading, thermal expansion, fire protection (prevention of fire spreading between floors), acoustic and architectural requirements, weather resistance well as ease of construction. With a large number of proprietary systems, which utilise different materials for the connections (eg. steel, aluminium and ceramics) detailed testing of individual systems and their sub-assemblages is required to study their potential seismic performance.

Fig. 4a shows a typical connection detail for a glazing façade used in a medium rise building in Melbourne. If the inter-storey drift between two consecutive floors is Δ, in the out-of-plane direction, the glazing panels may distort either in a curve or as a rigid body as shown in Fig. 4b. It is likely that the framing of the glazing units is stiff enough to result in rigid body rotation with the deformation taking place at the ends of the panels. The form and location of deformation will depend largely on the relative strength of the bolts and the support brackets (refer to Figs. 4c and 4d) as well as the strength of the butt joints between the panels. As an initial check, bolt failure would be imminent if the strength of the bolt is less than (M_p/a), where M_p is the yield strength of the bracket (Fig. 4c).

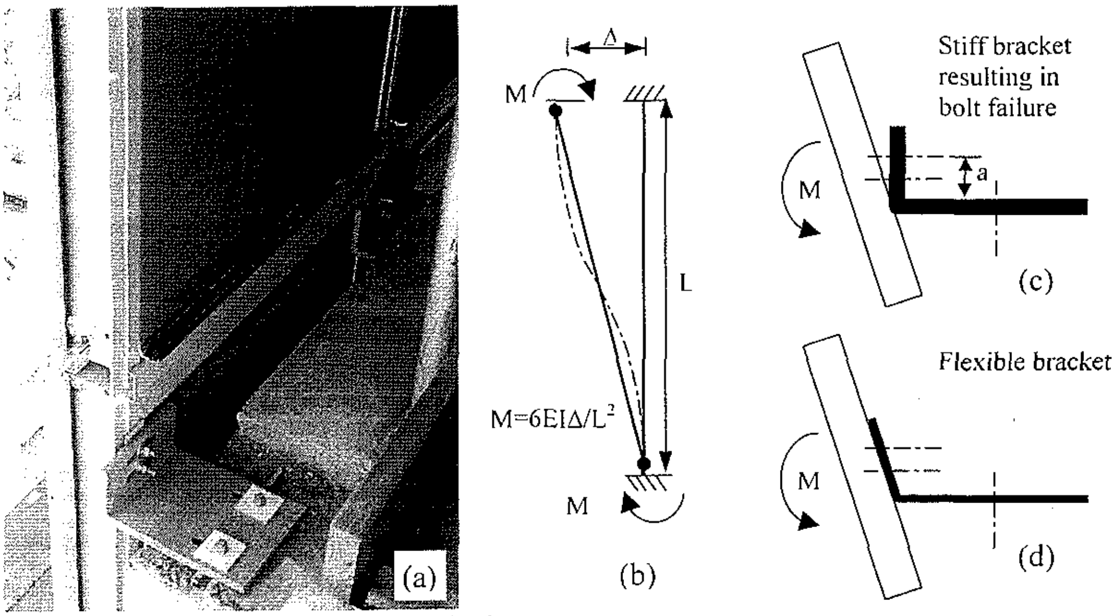


Figure 4: Connections between glazing panels and structure: (a) a photo of a typical bracket support; (b) possible idealisation of panel deformation; (c) bolt failure; and (d) distortion of bracket.

An experimental program is currently underway to test typical connections between glazing systems and the structure. The tests are conducted to obtain: (i) pull-out strength of bolts from glazing panels, and (ii) moment capacity of the connecting brackets. A comprehensive testing program will follow which will incorporate testing of full-scale units in both in-plane and out-of-plane directions. In addition, a variety of panel-to-panel and panel-to-structure connections will be tested and analysed. The drift capacity obtained from these tests will be compared with the drift demands as estimated in Section 2 to assess the performance of typical facade systems under earthquake loading.

4. CONCLUSION

The emphasis in earthquake codes and design procedures is on the structural system and particularly prevention of collapse. A great deal of success and confidence has been achieved in this area. However, damage to buildings, particularly to non-structural components in areas of low to medium seismicity is still a major concern. The cost of repair and interruption to business could be severe. The damage to non-structural components, such as facades, is primarily a result of inter-storey drift.

This paper has described a simple and convenient procedure to determine the seismic inter-storey drift demand in buildings. This procedure is based on scaling drift angles obtained from wind analyses and the use of relevant displacement response spectra. This method is particularly attractive in quickly assessing existing structures where wind deflections are available. As part of the on going research programme, testing on typical façade systems is currently underway to determine their drift capacity.

5. ACKNOWLEDGEMENT

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