1. INTRODUCTION

Non-structural components in a building can be classified into two groups for seismic design purposes: (i) drift-sensitive components and (ii) acceleration-sensitive components. Examples of drift-sensitive components are ceiling-high partitions, vertical piping and facades. As these components are attached to the building structure at multiple locations up the height of the building, they deform according to the deflection profile of building. The risk of damage to these components is dependent on inter-storey drift. In contrast, acceleration-sensitive components, which are the subject of interests in this paper, are attached to the building floor (or wall) at single locations. Examples of acceleration sensitive components are mechanical-electrical components including boilers, pressure vessels, transformers, generators and air-conditioners. Office-partitions, heavy furniture items and building contents including storage racks and library book shelves are also acceleration-sensitive components.

Major codes of practice for earthquake resistant design of buildings contain provisions for estimating the design acceleration (hence inertia force) induced into the component by seismic floor motions (FEMA 356, 2000; IBC, 2000 and AS 1170.4,1993). Very high accelerations have been stipulated by such codes including the current Australian Standard for earthquake loading (eg. AS1170.4, 1993) and there has been problems with compliances in practice. The justification of certain code clauses has been questioned by practitioners as the basis of the provisions is often not fully transparent. In the new (revised) standard (draft document no. D5212-5.1: 2005), certain provisions have been relaxed. Even then, it is important that the rationale of the standard's provisions be conveyed clearly to practising professionals for facilitating effective implementation of the Standard and for directing further research and development. Review articles on non-structural components can be found in earthquake engineering literature (eg. Rodriguez *et* al, 2002 and Naeim F., 1999) but in the opinion of the authors, these articles are not ideal in fulfilling the stated purposes in the Australian context. A key objective of this paper is to contribute to this dissemination of recent research findings by the authors. An important feature of this article is the introduction of the displacement-based (DB) criterion of damage to building contents (which is distinguished from the conventional notion of seismic hazard being represented by the inertia force). The DB approach of modelling seismically induced damage, which has been advocated for designing building and bridge structures since the early 1990's, is extended herein for assessing seismically induced damage to non-structural components and building contents.

The rest of this paper is structured as follows: amplification of floor acceleration up the height of the building (Section 2), amplification of acceleration on the component (Section 3) and displacement-based modelling (Section 4).

2. AMPLIFICATION OF FLOOR ACCELERATION

The peak floor acceleration (*PFA*) can be expressed in terms of the peak ground acceleration (*PGA*) as defined by equation (1).

$$PFA = \alpha_{\text{ground-floor}}\, PGA \qquad (1)$$

where $\alpha_{\text{ground-floor}}$ takes into account of two factors: (i) the dynamic amplification of acceleration at the centre of inertia of the building; and (ii) the factor relating the acceleration of an upper floor to that at the building centre of inertia.

The first factor contribution to $\alpha_{\text{ground-floor}}$ is often taken as "2.5", being the ratio of the peak response spectral acceleration (of an elastic single-degree-of-freedom system) and the peak acceleration applied at the base of the system. In the revised standard this well known dynamic factor has been increased by 25% to the value of about "3" for soft-soil (site Class D and E) conditions.

The second factor is the participation factor of the building responding in pure translational motion. This factor is intuitively in the order of 1.5 given that the center of inertia of the building is at approximately two-thirds up the height of the building. In theory, the value of $\alpha_{\text{ground-floor}}$, being the product of the two factors (1.5 and 2.5), is in the order of 3.5 - 4. This factor when combined with the acceleration amplification of the component could result in a very high design inertia force on the component, citing results from analytical investigations based on estimated peak accelerations on the building floor (Lam *et* al, 1998).

It is noted that the participation factor of 1.5 assumed in the simple calculation presented above was based on the building with a uniform vertical distribution of mass. If the mass of the building is discretised at the floor levels only, the calculated participation factors would be lower, and hence lower values of $\alpha_{\text{ground-floor}}$ as indicated by equation (2).

$$\textit{Participation factor} = \sum m_i\, \delta_i / \sum m_i\, \delta_i^2 \qquad (2)$$

where δ_i is the normalised floor displacement and $\delta_i \leq 1.0$

The value of the *participation factor* calculated by equation (2) assuming equal mass on each floor and a linear deflection profile normalized to unity at the roof is presented in Table 1.

Table 1 Estimated factors of *ground-floor* amplification at roof level

Total no. of floors (including roof)	Assumed dynamic amplification factor at centre of inertia of building	Participation factor (Eqn.2)	Ground-floor amplification $\alpha_{\text{ground-floor}}$
1	2.5	1.00	2.5
2	2.5	1.20	3.0
3	2.5	1.28	3.2
4	2.5	1.33	3.3
infinity	variable	1.50	variable

Although the table seems to show a trend of increasing value of $\alpha_{\text{ground-floor}}$ with increasing number of storeys in the building, this increase is in reality offset by the decrease in the dynamic amplification of the building as a whole (as values shown in the 2nd column of the table would decrease with increasing number of stories in the building) in accordance with the classical hyperbolic function in the velocity-controlled region of the acceleration response spectrum. It is noted that values presented in this table represent the most onerous conditions at the roof of the building whereas floor accelerations in the rest of the building are considerably lower.

The $\alpha_{\text{ground-floor}}$ factor of "2" stipulated at the roof level by AS1170.4 (1993), with notation "a_x", is clearly inadequate in certain conditions. The factor of "3" stipulated by the revised standard and by IBC (2000) is generally consistent with the values shown in Table 1, and is unlikely to be significantly exceeded in practice. Although provisions in the New Zealand Standard (NZS1170.5:2004) appear much more rigorous, they are founded on similar principles.

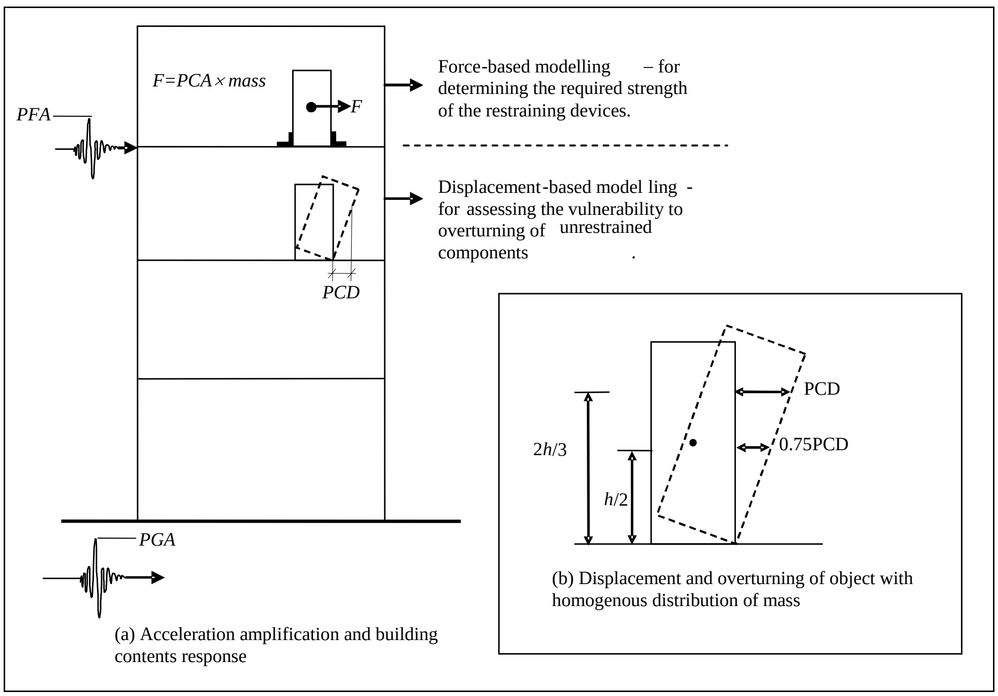


Fig. 1: Force-based and Displacement based modelling of non-structural components

3. AMPLIFICATION OF COMPONENT ACCELERATION

The acceleration at the centre of inertia of a rigid component which is fully restrained on the building floor is normally taken to be equal to the acceleration of the floor. In such situations, only equation (1) is required for defining the design inertia force on the component. However, a further amplification factor $\alpha_{\text{floor-component}}$ as defined by equation (3) is required for components that are flexible or have not been fully restrained.

$$PCA = \alpha_{\text{floor-component}} \; PFA \tag{3}$$

where PCA is the peak acceleration of the component which is used for calculating the force required to restrain the component (refer Figure 1a).
The value of $\alpha_{\text{floor-component}}$ depends on a number of factors including the nature of the excitations applied to the component, the natural period and damping properties of the

component assemblage, and the ductility of the components. In the extreme condition of a flexible component subject to a fully periodic motion of the building floor, the value of $\alpha_{\text{floor-component}}$ can be as high as 10 for 5% damping (ie ζ=0.05) based on equation (4a) which models the conditions of resonance.

$$\alpha_{\text{floor-component}} = 1/2\zeta \tag{4a}$$

If the motion of the building floor is taken to be similar to the motion of the ground, the value of $\alpha_{\text{floor-component}}$ can be estimated using equation (4b) as for single-degree-of-freedom building structures excited by motions of the ground.

$$\alpha_{\text{floor-component}} = 2.5 \tag{4b}$$

Provisions in the new Standard are generally based on equation (4b) but with modifications to account for factors including the allowance for over-strength and ductility of the restraining devices.

In AS1170.4:1993, the value of $\alpha_{\text{floor-component}}$ for spring-mounted components (with the different notation "a_c") varies between 1 and 2 depending on the natural period of the component in relation to the fundamental natural period of the building. Whilst this factor alone appears inadequate in allowing for conditions of resonance, other co-existing factors have been introduced to result in an overall conservative set of provisions.

4. DISPLACEMENT-BASED MODELLING OF COMPONENT FAILURE

The acceleration-based (force-based) provisions as described, whilst appearing thorough and logical, might be totally inappropriate for components that are un-restrained (ie. free-standing components) which is commonly the case in regions of low and moderate seismicity. Such components might displace "rock" when excited but not necessarily overturn if the centre-of-gravity (cg) of the component has not been displaced beyond its pivotal edge (Lam and Wilson, 2001). The seismic performance of unrestrained objects is then highly dependent on the displacement behaviour of the building floor as opposed to the acceleration behaviour (Lam and Gad, 2002). Refer Figure 1a for the diagrammatic illustrations of the force-based and displacement-based modelling concepts.

Experimental and analytical investigations have been undertaken by the authors to model rocking motions of rigid free-standing objects (Al Abadi *et* al, 2004 & 2005). Whilst rocking behaviour is often perceived to be highly non-linear (ie. periodic behaviour highly dependent on amplitude) it was found that rocking motions can be approximated by a pendulum or a linear elastic single-degree-of-freedom system if the cg of the object is within 50% from the limit for overturning (Figure 2a). Expressions for the equivalent natural period of uniform free-standing objects have been developed by Al Abadi *et* al (2005) using linearisation techniques. This equivalent period is mainly dependent on the height of the object according to the derived expressions (not shown herein). For example, an equivalent period of approximately 1 sec is calculated for a 0.75 m tall object, and 1.6 sec for a 2 m tall object.

Linearisation enables overturning risk to be assessed by comparing the displacement limit for overturning with the displacement demand imposed by the floor motions (as represented by the floor displacement response spectrum). In the most onerous condition of the rocking component resonating with the building floor, the peak displacement demand at the centre of inertia of the component (PCD) can be estimated by equation (5) based on the highest level (peak position) of the elastic displacement response spectrum.

$$PCD = PCA\ (T_{building}/2\pi)^2 \tag{5}$$

Equation (5) may be combined with equations (1) – (4) in obtaining a conservative estimate for the displacement at the cg of the object. For a uniform object, the cg is at the mid-height of the object whereas the centre of inertia is at 2/3 up the object height. Thus, the displacement calculated from these equations should be factored down by ¾ to obtain the displacement of the object at its cg (Figure 1b). An interesting observation from equation (5) is that the displacement demand on an object does not increase indefinitely with increasing object period (hence increasing object height). In a building with $T_{building}$ = 1 sec, for example, the displacement demand on a 2 m tall object (1.6 sec period) is not higher than that of a 0.7 m tall object (1 sec period), as the displacement demand is highest at 1 sec. This is an interesting phenomenon as the vulnerability of an object to overturning is then mainly a function of its width at the base. Consequently, the risk to overturning does not increase indefinitely with its aspect ratio as implied by a force-based evaluation.

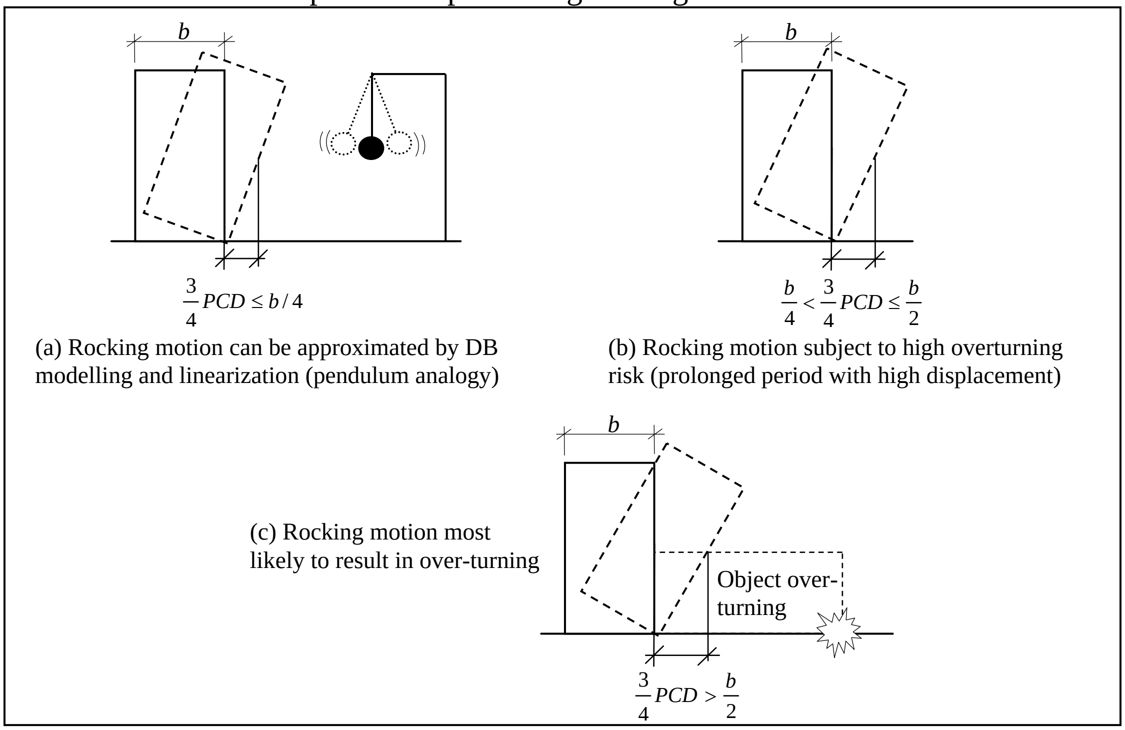
An object which has been displaced beyond the 50% margin is subject to high overturning risks. Such risk is further escalated as the rocking motion “slows down” when approaching the overturning limit and hence the exposure to high overturning risk is prolonged (Figure 2b). It is therefore recommended that if rocking motion of the component is allowed the displacement of the cg must have at least 50 % margin of safety in order that approximations by the linear model can be applied. An object which has been displaced beyond the overturning limit (ie. cg displaced beyond the pivotal edge) is deemed to have overturned (Figure 2c).

The force-based and displacement-based criteria could be jointly used to assess the seismic performance of non-structural components in a building. A component can be deemed safe if either performance criterion is satisfied (ie. the object either does not overturn or does not rock at all). It is noted, however, that the simple approach described in this paper to check for overturning is generally appropriate for buildings with natural period which is lower than the site natural period but might exaggerate overturning risks for components in taller buildings (eg. buildings exceeding 10 storeys) based on the findings by Franke *et* al, (2005).

5. CONCLUSIONS

i. The rationale for modelling the amplification of the acceleration of the building floor and that of the component has been presented.

ii. Various codes provisions in addressing these amplification mechanisms have been reviewed.
iii. The displacement-based approach of assessing the overturning risk of unrestrained components experiencing rocking motions have been introduced.



(a) Rocking motion can be approximated by DB modelling and linearization (pendulum analogy)

(b) Rocking motion subject to high overturning risk (prolonged period with high displacement)

(c) Rocking motion most likely to result in over-turning

Fig. 2: Displacement limits for rocking

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