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Displacement Controlled Behaviour of Structures subject to Moderate Ground Shaking

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ABSTRACT

Seismic assessment and design procedures are typically founded on the concept of trading off strength with ductility (displacement) to provide sufficient capability for the structure to absorb and dissipate seismically induced energy in an earthquake. However, in the regions of low to moderate seismicity, the kinetic energy demand of an earthquake will generally subside when the structure has been displaced to a certain limit. Consequently, the seismic performance of a structure can be controlled by its displacement capacity as opposed to its energy dissipation capability. This paper presents a seismic performance assessment method which accounts for the displacement-controlled phenomenon associated with moderate ground shaking. Parametric studies based on non-linear time-history analyses have been undertaken to identify the behavioural trends. Hysteretic models used in the studies were based on the observations of results from recent cyclic testings carried out at the University of Melbourne and the University of Adelaide. The effects of asymmetry in building systems on its seismic response behaviour have been investigated forming one of the main thrusts of the research. Results from parametric studies have been integrated to develop a simple and yet reliable alternative procedure for the seismic assessment of structures in regions of low and moderate seismicity like Australia.

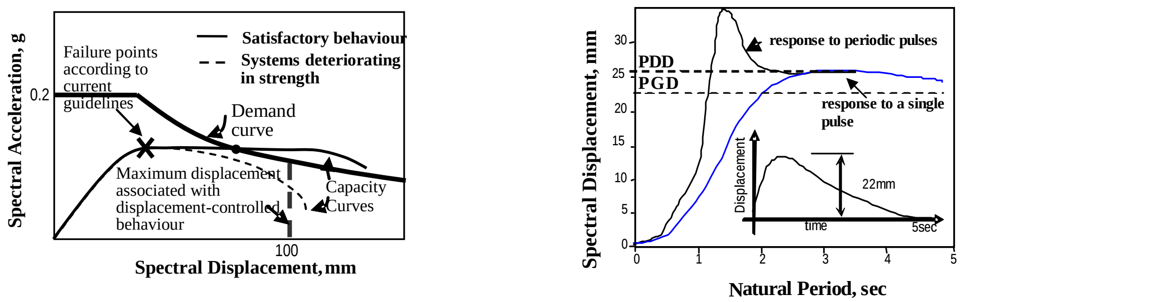
Keywords: earthquake; displacement-controlled behaviour; non-linear time history analyses; asymmetric building

1. Introduction

The aseismic design and performance assessment of a structure is traditionally based on the trading-off strength demand with the ductility (displacement) demand. Thus, the ductility capacity of a structure is critical to its performance in an earthquake when the strength demand is too high to accommodate. This trading-off can be represented conveniently by the capacity-spectrum (refer Figure 1a). The structure is deemed to be safe by the *Capacity Spectrum Method* if the capacity curve and the demand curve intersect as shown in Figure 1a. Given that the ductility capacity of the structure is so critical to its survival, structures in high seismicity region are typically designed to accommodate large displacement. Importantly, the lateral strength of the structural system must be maintained (and degrade by no more than, say, 20%) when undergoing the designated displacement. Effectively, the displacement capacity and the residual strength capacity of the structure are both critical to its survival when subject to the strong shaking of the projected earthquake scenario. This is because the trading-off phenomenon is underpinned by the concept of the conservation of energy in that the kinetic energy developed in the responding structure must be absorbed as strain energy and be eventually dissipated in a controlled fashion.

Studies undertaken in recent years by the authors and collaborators revealed that the kinetic energy demand generated by the moderate ground shaking of a small-medium magnitude (M<7) earthquake will subside as the structure "softens" with increasing displacement demand (Lam and Chandler, 2005; Wilson and Lam, 2006). The diminishing energy phenomenon means that assessment methodology founded on the conventional concept of trading-off strength with ductility will not necessarily give a realistic prediction of the seismic performance of the structure in terms of its ultimate survival in an earthquake of this nature. A simple comparison of the displacement demand on the structure with its displacement capacity is a more direct, and effective, approach of evaluating its seismic performance. In other words, the structure can be deemed safe if the displacement demand can be accommodated whilst the gravity load carrying capacity is also maintained. This alternative approach to seismic assessment based on the displacement-controlled phenomenon has a fundamental distinction from the conventional approach of trading-off strength with ductility. Whilst ductile design and detailing is relevant to both approaches, the associated performance requirements can be quantified very differently even for the same level of seismicity.

Performance assessment of the structure based on the displacement-controlled phenomenon can be very convenient to apply if the peak displacement demand (PDD) of single-degree-of-freedom systems can be identified readily. Figure 1b shows the displacement response spectra of a single pulse and that of a series of multiple (periodic) pulses which represents the scenario of resonance on a flexible soil site. The ***PDD*** can be identified readily for both cases based on the assumption of linear elastic behaviour. The excitation of a real earthquake can be described as a combination of the two types of idealized pulses. Consequently, displacement response spectra as observed in field recordings from moderate ground shakings are similar in form to that shown in Figure 1b.



(a) Acceleration-Displacement response diagram (b) Displacement response spectrum, (Lam and Chandler, 2005)

Figure 1 Displacement-controlled behaviour

When the ***PDD*** has been identified, the seismic assessment of a building employing the displacement criterion seems to be straightforward. However, there are important issues to address. One of these issues is in making use of the elastic displacement response spectra (and the ***PDD***) for constraining the displacement demand of the non-ductile systems which have been excited into the post-elastic range. Parametric studies have been undertaken by the authors based on non-linear time history analyses (*THA*) of hysteretic models that are representative of the common non-ductile structures including unreinforced masonry walls and soft-storey buildings. Hysteretic modelling is described in Section 2. And the earthquake excitations employed in the non-linear *THA* are described in Section 3. Finally, the use of the elastic displacement response spectra and the ***PDD*** for constraining the displacement demand of non-ductile systems is evaluated by parametric studies in Section 4 and extended to the assessment of asymmetrical building models in Section 5. Results from the parametric studies have been integrated to develop a simple seismic assessment procedure for structures in regions of low to moderate seismic regions such as Australia.

2. Hysteretic models for non-ductile systems

Extensive studies have been undertaken to investigate the seismic performance of soft-storey columns (Wilson et al., 2005) and URM walls (Griffith et al., 2004, Griffith et al., 2007). The observed initial natural period and yield strengths are summarised in Table 1.

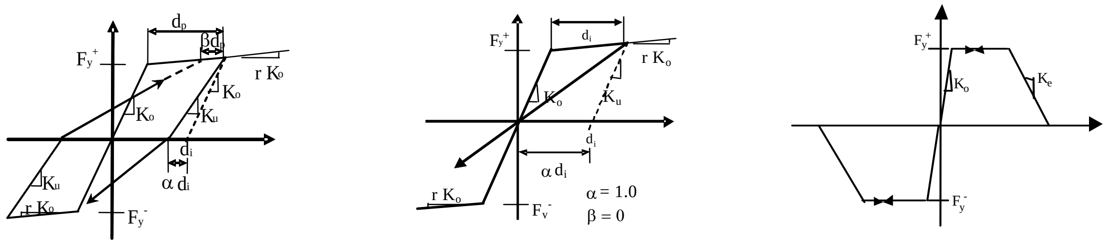
Table 1 Seismic capacity and seismic demand of typical structures

	Initial period sec	**Notional "yield" strength g's**	**Seismic demand g's**
URM walls one-way bending **two-way bending**	0.1 - 0.8	0.15 - 0.75 0.8 – 3	0.2 - 2
Soft-storey buildings	0.2 – 2	0.07 – 0.3	0.2 - 0.3

* Seismic demand prediction is based on PGA of 0.06g to 0.12g as stipulated by the new Australian Standard (AS1170.4, 2005) for 500 year return period.

* The prediction of seismic demands imposed on URM walls takes into account the filtering effect of multi-storey buildings.

It is also shown that the yield strength can be exceeded by the seismic demand by a factor of 2 – 4 for soft-storey columns and a factor of up to 3 for out-of-plane two-way bending of URM walls. This factor can be increased to a much higher value in conditions of one-way bending which applies to parapet walls and gable end walls (that have not been effectively tied to the rest of the structure). Standard hysteretic models including the modified Takeda model (Figure 2a) and the origin-centered model (Figure 2b) have been used to represent the hysteretic behaviour of URM walls (Figure 2c).



(a) Modified Takeda model (b) Origin-centered model (c) URM one-way bending (Doherty et al., 2002)

Figure 2 Hysteresis model

Calibrations have been undertaken based on testings of URM walls and RC columns carried out at the University of Adelaide (Griffith et al., 2007) and the University of Melbourne (Rodsin, et al., 2004) respectively using: i) the origin-centered model, ii) the modified Takeda model (α=0 - 0.5, β= 0) for URM walls and iii) the modified Takeda model (α=0.5, β=0.2 and 0.6) for soft-storey columns. URM walls typically feature higher rate of strength degradation (60% degradation in strength when ductility demand μ is 4.5) than soft-storey columns (30% degradation in strength when μ is 4.5).

3. Earthquake accelerograms

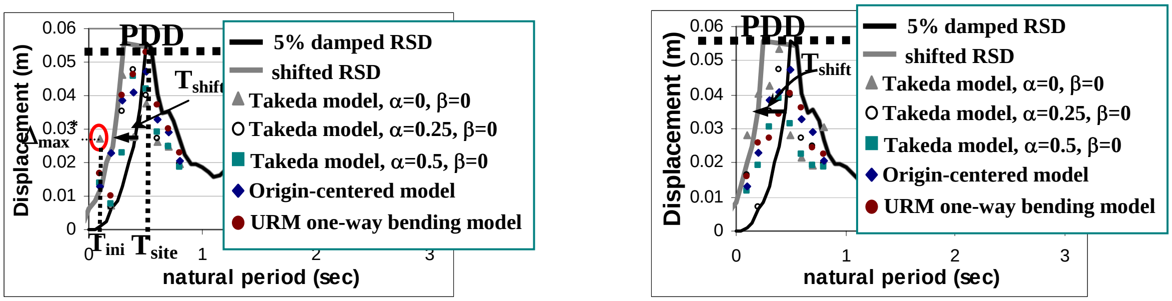
Four simulated and two recorded accelerograms on a class C and D site as stipulated by the Australian Standard (AS1170.4, 2005) were employed in the parametric studies (Table 2). The four simulated accelerograms were generated by stochastic simulations using program GENQKE (Lam et al., 2000) and shear wave analyses using program SHAKE (Idriss & Sun, 1992) based on earthquake scenarios that are consistent with a peak ground velocity of 60 mm/sec on rock sites and a seismic hazard factor of 0.08g (Lam et al., 2005). All accelerograms employed in the analysis feature displacement-controlled behaviour in view of their displacement response spectra.

Table 2 List of accelerograms

No	Event	Direction	Station	M	R	PGV	Site
1	Generated (AS1170.4)			6.5	40	60	Class C
2	Generated (AS1170.4)			6.5	40	60	Class D
3	Generated (AS1170.4)			5.5	17	60	Class C
4	Generated (AS1170.4)			5.5	17	60	Class D
5	Friuli aftershock (11/09/76)	N-S	Buia	5	7		Class C
6	San Fernando	021		6.6	25		Class D

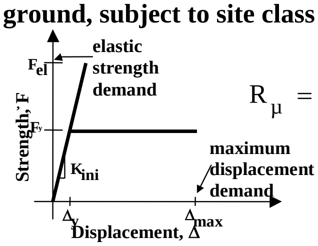
4. Displacement response behaviour

Parametric studies involving non-linear *THA* were undertaken on single-degree-of-freedom (*SDOF*) systems with hysteretic models introduced in Section 2. The calculated maximum displacement demands of the *SDOF* systems are correlated with their respective initial natural period in Figures 3 & 5 for URM walls (5% damping) and soft-storey buildings (8% damping). It is noted that the yield strength in each of the SDOF systems have been adjusted in order that the ductility reduction ratio R_μ equals 2 and 3 for URM walls and 2 and 4 for soft-storeys.



(a) $R_\mu = 2$ (b) $R_\mu=3$

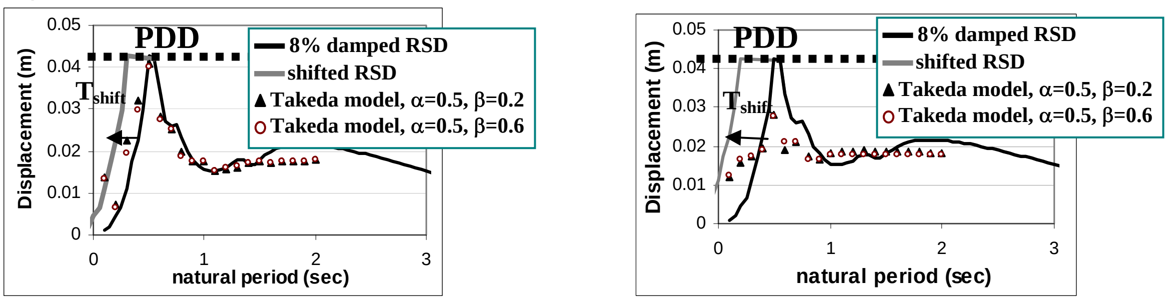
Figure 3 Displacement-controlled behaviour of non-linear responding systems (URM walls on the ground, subject to site class C earthquake, M=6.5, R=40 km) (* Refer to Figure 4 for description)



$$R_\mu = \frac{F_{el}}{F_y}$$

K_{ini} is proportioned such that the initial period ranges between 0.1 to 2 sec
F_y is scaled such that the Rμ is 2 and 3 for URM walls, 2 and 4 for soft-storey columns

Figure 4 Schematic envelope of force-displacement response



(a) $R_\mu=2$ (b) $R_\mu=4$

Figure 5 Displacement-controlled behaviour of non-linear responding systems (soft-storey columns, subject to site class C earthquake, M=6.5, R=40 km)

An important finding from the parametric studies as shown in Figures 3 & 5 is that the maximum inelastic displacement demands (***PDD***) are mostly constrained below the highest point of the elastic displacement response spectrum for 5% damping (URM walls) and 8% damping (soft-storeys). Another interesting feature that can be observed in Figure 3 & 5 is the enveloping of results (for $T < T_{site}$) by the elastic displacement response spectrum that has been shifted (to the left). The extent of the shift is shown to be dependent on the site natural period (T_{site}) and the ductility reduction factor (R_μ). Equation (1) is proposed to model this trend.

$$T_{shift} = \frac{R_\mu + 1.5}{9} T_{site} \qquad (1)$$

5. Peak displacement demands in building with asymmetry

In buildings where the centre of strength (or stiffness) is significantly offset from the centre of mass additional amplification of the displacement demand at the edges of the building due to torsion have been observed (refer Figure 6a & 6b). The torsional amplification factor (***Γ_{DD}***) is defined as the ratio of the ***PDD*** at an edge of the building and the highest point of the elastic displacement response spectrum for a single-degree-of-freedom system (RSD_{max}). That is, Γ_{DD} **= PDD/RSD_{max}**. The maximum displacement demand can occur at either the stiff edge (the edge closer distance to the center of resistance) or the flexible edge (the edge further distance to the center of resistance). The PDD referred herein represents the higher value of the two. Using single-storey buildings with two degrees-of-freedom 2DOF idealisation (plan view shown in Figure 6a), the maximum displacement at the flexible and stiff edge can be determined by solving the eigenvalue problem shown by equation (2).

$$\left[\bar{\mathbf{k}} - \omega_n^2 \bar{\mathbf{m}}\right] \phi_n = 0 \qquad (2)$$

$\bar{k}$ and $\bar{m}$ are the stiffness and the mass matrix respectively, defined by equation (3a) and (3b)

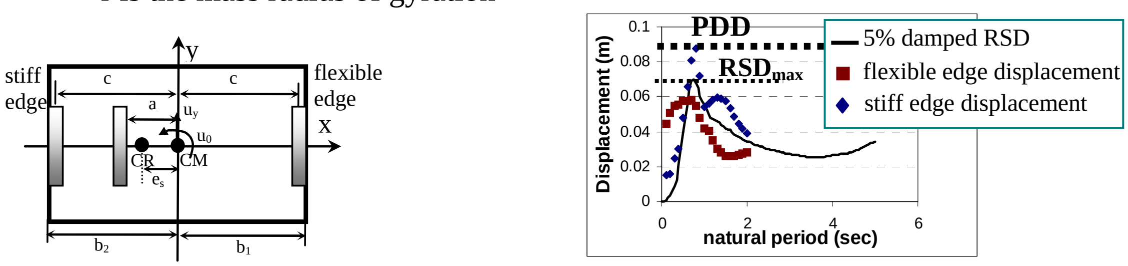
$$\bar{\mathbf{k}} = \begin{bmatrix} k_y & ek_y \\ ek_y & k_\theta \end{bmatrix} \quad (3a) \quad \text{and} \quad \bar{\mathbf{m}} = \begin{bmatrix} m & 0 \\ 0 & r^2 m \end{bmatrix} \quad (3b)$$

where, k_y is the total lateral stiffness in the y direction

k_θ is the torsional stiffness about the center of mass

m is the mass of the single-storey building model

r is the mass radius of gyration



(a) floor plan (b) displacement demands from THA

Figure 6 Single storey building model with 2 degrees of freedom

The amplification factor Γ_{DD} can be calculated using equation (4) if linear elastic behaviour is assumed.

$$\Gamma_{DD} = \text{larger of } \sqrt{\left(PF_1 - \hat{\theta}_1 \frac{b_1}{r}\right)^2 + \left((1 - PF_1) + \hat{\theta}_1 \frac{b_1}{r}\right)^2} \quad \text{or} \qquad (4a)$$

$$\sqrt{\left(PF_1 + \hat{\theta}_1 \frac{b_2}{r}\right)^2 + \left((1 - PF_1) - \hat{\theta}_1 \frac{b_2}{r}\right)^2} \qquad (4b)$$

where, PF_1 is the participation factor for mode 1

$\hat{\theta}_1$ is the rotation component of mode 1 multiplied by the mass radius of gyration r

$b_{1,2}$ is the distance from the center of mass to the flexible and stiff edge

PF_1 and $\hat{\theta}_1$ are defined by the equations 5(a) and 5(b):

$$PF_1 = \frac{e^2}{e^2 + (1-\Omega_1^{\ 2})^2} \qquad (5a) \quad \text{and} \qquad \hat{\theta}_1 = -\frac{e(1-\Omega_1^{\ 2})}{e^2 + (1-\Omega_1^{\ 2})^2} \qquad (5b)$$

Ω_1 is the 1st coupled circular frequency which can be related to the uncoupled frequency ratio ($\rho_k = 1/r \sqrt{k_\theta / k_y}$) and the distance from the center of resistance to the center of mass normalised to the mass radius of gyration (e) as shown by the following equation:

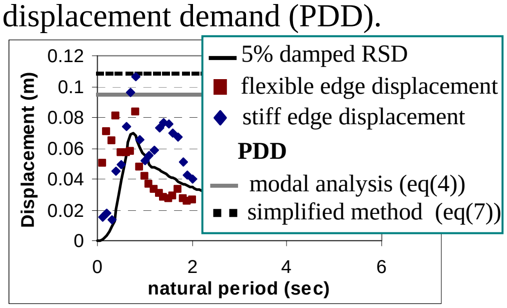
$$\Omega_1 = \frac{1+\rho_k^{\ 2}+e^2}{2} - \sqrt{\frac{\left(1+\rho_k^{\ 2}+e^2\right)^2}{4} - \rho_k^{\ 2}} \qquad (6)$$

Using eqs (4), (5) and (6), the Γ_{DD} can be determined for any combination of e (eccentricity) and ρ_k (uncoupled frequency ratio).

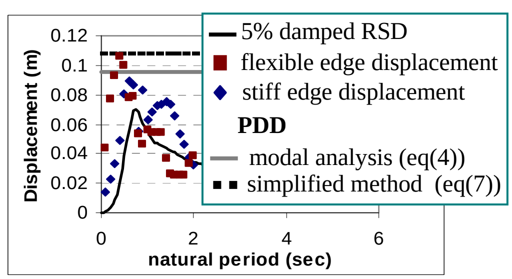
Field surveys undertaken on URM buildings (Griffith et al., 2004) and soft-storey structures (Wilson et al., 2005) indicated that the eccentricity ratio $\boldsymbol{e}$ and uncoupled frequency ratio ρ_k vary in the range 0.1-0.8 and 0.6-1.7 respectively. Both quantities have been normalised with respect to the mass radius of gyration ($\boldsymbol{r}$). The distance between the center of mass and the flexible, or stiff, edge of the building (normalised with respect to $\boldsymbol{r}$) was found to be less than 1.8. Equation (2) has been used to estimate the values of Γ_{DD} for all combinations of e and ρ_k that are likely to be encountered in real buildings (based on data collected from the field surveys) assuming linear elastic behaviour. The maximum value of Γ_{DD} was limited to around 1.6 and hence a simplified method for estimating PDD for a torsionally eccentric building is given by equation (7).

$$\boldsymbol{PDD} = 1.6\ \boldsymbol{RSD_{max}} \qquad (7)$$

Parametric studies based on non-linear *THA* were then undertaken to evaluate estimates of PDD using equations (4) or (7). The parametric studies covered the value $\boldsymbol{e}$ ranging 0.1 - 0.5 and ρ_k ranging 0.8 - 1.3. Results of the evaluation are shown in Figures 7 and 8 and demonstrate that both equations (4) and (7) provide realistic estimates of the peak displacement demand (PDD).

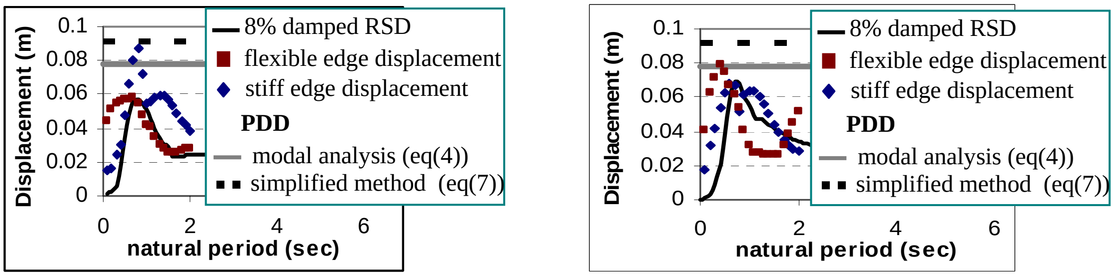


(a) Takeda model, α=0, β=0, R_μ=2



(b) Takeda model, α=0, β=0, R_μ=3

Figure 7 Displacement demands of torsionally unbalanced building with e=0.5, ρ_k=0.8, subject to Friuli earthquake, M=5, R=7km (hysteretic model fitted to hysteretic behaviour of URM walls)



(a) Takeda model, α=0.5, β=0.2, R_{μ}=2 (a) Takeda model, α=0.5, β=0.2, R_{μ}=4

Figure 8 Displacement demands of torsionally unbalanced building with e=0.5, ρ_k=0.8, subject to Friuli earthquake, M=5, R=7km (hysteretic model fitted to hysteretic behaviour of soft-storey columns)

6. Concluding remarks

The concept of displacement-controlled behaviour of structures when subject to moderate ground shakings generated by small-medium magnitude earthquakes has been introduced. The displacement-controlled phenomenon enables the seismic performance of the structure to be assessed based on comparison of the displacement demand with the capacity of the structure to displace whilst maintaining its gravity load carrying capability. This is an important distinction from the conventional seismic assessment approach which is based on trading-off strength demand with the ductility demand on the structure.

Parametric studies based on non-linear *THA* on representative hysteretic models of SDOF systems revealed that the PDD is mostly constrained by the highest point on the elastic displacement response spectrum for 5% damping (URM walls) and 8% damping (soft-storeys). Importantly, *THA* of asymmetrical building models support the use of an additional displacement amplification factor of 1.6 to model the increase in displacement resulted from torsional response behaviour of the building.

7. References

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