Australian Earthquake Engineering Society 2011 Conference, 18-20 November, Barossa Valley, South Australia

Simple Seismic Design and Assessment of Buildings Incorporating The Displacement Controlled Phenomenon

E. Lumantarna[1], N. Lam[1] and J. Wilson[2]

1. Dept of Civil and Environmental Engineering, The University of Melbourne, Parkville, VIC 3010, Australia.

2. School of Engineering and Industrial Science, Swinburne University of Technology, Hawthorne, VIC 3122, Australia.

ABSTRACT

Seismic design, or assessment, of buildings based on conventional force-based principles typically involves trading-off strength with ductility to provide sufficient capacity of the building to absorb energy in a controlled manner when excited into motions by an earthquake. In conditions of moderate ground shaking which is generated by a small or medium magnitude earthquake the energy demand on the building could subside with increasing effective natural periods. Consequently, the amount of drift imposed on the building could be limited irrespective of the degradation in strength or stiffness of the lateral resisting elements. Seismic response behavior of this nature can be explained by what is known as the displacement-controlled phenomenon. The assumption of a peak displacement demand limit (which is function of the properties of the ground shaking) can potentially simplify the seismic design or assessment of a structure which is flexible or has the capacity to undergo large displacement without collapsing.

A generalised response spectrum model which features displacement-controlled phenomenon was first developed to provide seismic response predictions assuming linear elastic behaviour. The model has since been further developed from results generated by parametric studies involving extensive non-linear time history analyses to incorporate the effects of inelastic behavior and torsional actions. The maximum drift demand on a building, of symmetrical or asymmetrical construction, can be conveniently estimated using the developed model thus circumventing the need to undertake time consuming modelling and computations.

1. Introduction

Seismic assessment has traditionally been based on trading-off between strength and ductility to allow the energy demand from an earthquake to be absorbed and dissipated in a controlled manner. The energy demand from small to medium earthquakes could subside with the increasing period of the building. Consequently, the peak displacement demand of the building is constrained to an upper limit as illustrated by Figure 1 for a building subjected to a single pulse and periodic pulses. The assumption of a peak displacement demand limit (which is a function of the properties of the ground shaking) can potentially simplify the seismic design or assessment of a structure which is flexible or has the capacity to undergo large displacement without collapsing.

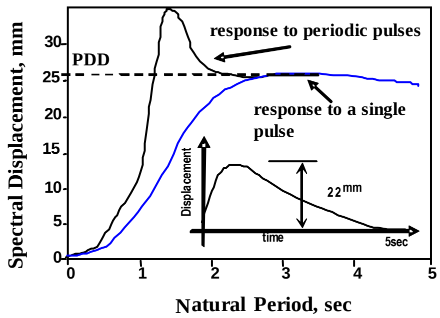


Figure 1 Displacement-controlled behaviour (Lam & Chandler, 2005)

The concept of displacement-controlled behaviour has been used for the seismic assessment of unreinforced masonry parapet walls (Lam *et al.*, 1995), unreinforced masonry walls subject to one-way bending (Doherty *et al.*, 2002) and free standing objects (Al Abadi *et al.*, 2006; Kafle *et al.*, 2011). In this study, the seismic assessment based on the displacement-controlled behaviour is extended to non ductile buildings, of symmetrical and asymmetrical constructions.

A generalised response spectrum model which features displacement-controlled phenomenon is presented in Section 2. The displacement response spectrum was developed to provide seismic response predictions assuming linear elastic behaviour. The developed predictions have been extended to account for the effects of inelastic behaviour (Section 3) and torsional actions (Section 4).

2. Displacement response spectrum and peak displacement demand in an earthquake

The peak displacement demand of a structure in an earthquake can be estimated from the maximum point of the displacement response spectrum. In this study, the proposed displacement response spectrum is presented in the *response spectral displacement* (*RSD*) format in the *bi-linear* form (Figure 2) and defined by the following expressions:

$$RSD(T) = \frac{RSD_{max}}{T_2} T \qquad \text{for } T \leq T_2 \qquad (1a)$$

$$RSD(T) = RSD_{max} \qquad \text{for } T > T_2 \qquad (1b)$$

Where RSD_{max} is the peak displacement demand and T_2 is the second corner period.

The proposed displacement response spectrum is truncated at the limiting period of 5 sec. The estimated values of RSD_{max} based on the period range of up to 5secs can be used for the stability assessment of a wide range of structures, such as free standing objects (Al Abadi *et al.*, 2006; Kafle *et al.*, 2011), unreinforced masonry walls (Doherty *et al.*, 2002; Lam *et al.*, 1995) and soft-storey structures (Lumantarna *et al.*, 2011).

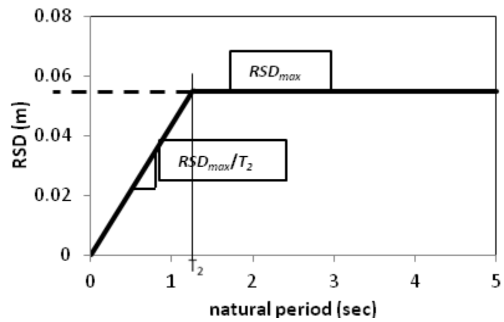


Figure 2 Displacement response spectrum in the bi-linear form

Extensive parametric studies have been undertaken by the authors (Lumantarna *et al.*, 2011) involving 168 records to evaluate existing predictive models for the peak displacement demand (RSD_{max}) and second corner period (T_2). The parametric studies revealed the estimated (median) values of the RSD_{max} parameter are highly consistent across the different models except at short distances (eg. 10km). The *mid-range* values (along with the *upper bound* and *lower bound* values) predicted by the suite of models for various magnitude-distance combinations are shown in Table 1.

Table 1 Median Model Predictions of Peak Displacement Demand RSD_{max}(mm)

	R = 10 km	R = 20 km	R = 30 km	R = 40 km	R = 50 km
M5.5	23 (15 – 30)	15 (10 – 20)	10 (5 – 15)	8	5
M 6	*68* *(45 – 90)*	34 (25 – 40)	20 (15 – 25)	15	10
M 6.5	*135* *(90 – 180)*	75 (55 – 90)	55 (45 – 60)	38 (30 – 45)	33 (25 – 40)

Notes :
(a) Mid range values are shown.
(b) Upper and lower bound values are shown in brackets where there are significant inter-model discrepancies.
(c) Values shown in *italics* are much less well constrained and are associated with scenarios of very low probability of occurrences in a region of low-moderate seismicity.

The value of T_2 is not unique and is well known to be sensitive to the moment magnitude of the earthquake. In contrast to the predictions for the RSD_{max} values, the predictions of T_2 values from the available predictive models (eg. AS1170.4, 2007; EN 1998-1, 2004; FEMA 450-1, 2003; Faccioli *et al.*, 2004; Lam *et al.*, 2000) are very diverse. However, the parametric studies undertaken by the authors (Lumantarna *et al.*, 2011) have found that the predictive equation (Eq. (2)) by Lam *et al.* (2000) is consistent with field observations.

$$T_2 = 0.5 + \frac{M-5}{2} \qquad (2)$$

where, M is the moment magnitude of the earthquake.

The maximum displacement demand (RSD_{max}) in more onerous soil conditions can be significantly amplified. Site response analyses have been undertaken based on shear wave analyses using accelerograms generated by stochastic simulations. A total of 1600 accelerograms on rock, shallow and deep soil sites were generated based on magnitude and epicentral distance combinations producing peak ground velocity PGV on rock which ranges from 20 to 100mm/sec. The accelerograms were generated by stochastic simulations using program GENQKE (Lam, 2002) and shear wave analyses using program SHAKE (Idriss & Sun, 1992). The range of PGV is consistent with hazard factor Z of 0.03 to 0.12 which is representative of seismic hazard in most capital cities in Australia including Melbourne, Sydney, Canberra, Adelaide, and Perth for 50 up to about 2500 year return period (AS1170.4, 2007). It was found from the site response analyses that the amplification generally ranges between 3 and 5.

Thus, the maximum response spectral displacement ($RSD_{max,soil}$) can be estimated as follows:

$$RSD_{\max,soil} \approx RSD(T_{2,soil}) \;\; 4 \qquad (3)$$

where, $RSD(T_{2,soil})$ is the response spectral displacement defined by Eq. (1).

The displacement response spectrum model on a soil site of the bi-linear model then can be defined as follows:

$$RSD(T) = \frac{RSD_{max,soil}}{T_{2,soil}} T \qquad \text{for } T \leq T_{2,soil} \qquad (4a)$$

$$RSD(T) = RSD_{max,soil} \qquad \text{for } T > T_{2,soil} \qquad (4b)$$

The construction of displacement response spectrum on a soil site is shown schematically in Figure 3.

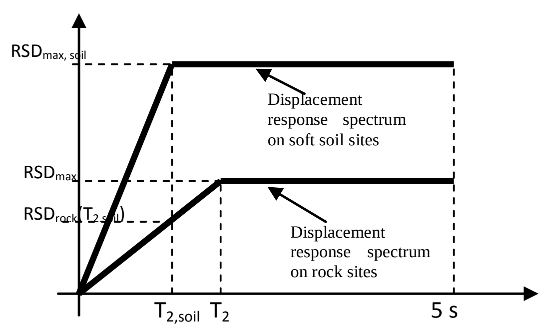
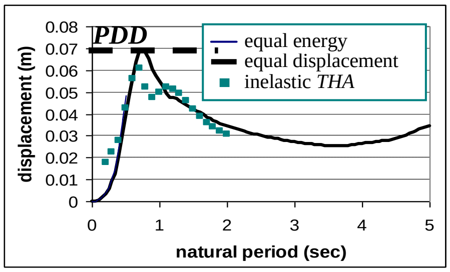


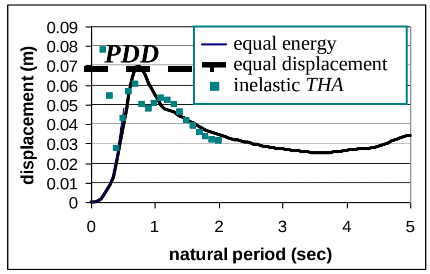
Figure 3 Displacement response spectrum on soil site

3. Peak displacement demand of non-ductile structure

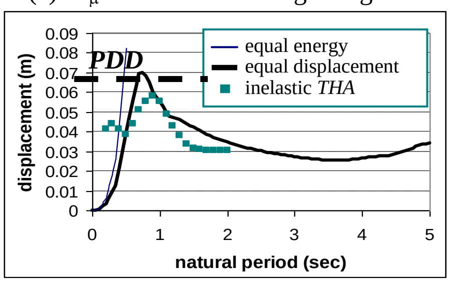
In this section, the use of displacement response spectrum is extended to estimate the *peak displacement demand* of non-ductile structures. Parametric studies based on non-linear time-history-analyses have been undertaken by the authors (Lumantarna *et al.*, 2010). Hysteretic models used in the studies represent the hysteretic behavior of non-ductile structures, such as unreinforced masonry wall and soft-storey columns, and feature strength degradation of up to 15% per unit increase in the ductility demand ratio.

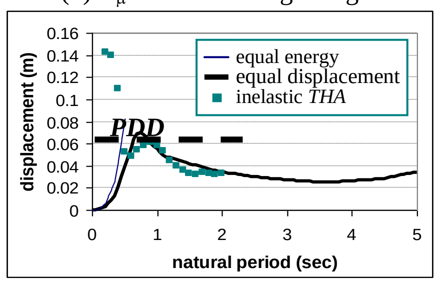
It was found from the studies that estimates based on the *equal-displacement* propositions in the displacement sensitive region of the response spectrum have been shown to be reasonably good and insensitive to considerable strength degradation (Fig. 4). In contrast, it is shown in Figure 4 that estimates by both the *equal-displacement* and *equal-energy* propositions can be exceeded significantly by the inelastic displacement demands in the acceleration and velocity sensitive region. These anomalies are exacerbated by strength degradation behaviour. The displacement demand behaviour depends very much on the corner period parameter which characterises the frequency properties of the ground shaking. However, it was found that with a modest strength reduction factor of 2, the system's inelastic displacement demand would typically be constrained by the peak displacement demand (***PDD***) which is defined by the maximum point on the elastic displacement response spectrum for 5% damping (RSD_{max}) irrespective of the initial natural period of the system.





(a) $R_\mu = 2$ without strength degradation (b) $R_\mu = 2$ with strength degradation





(c) $R_\mu = 4$ without strength degradation (d) $R_\mu = 4$ with strength degradation

Figure 4 Elastic and Inelastic Displacement demands with Friuli earthquake

4. Peak displacement demand of asymmetrical buildings

In situations where the center of resistance (CR) of the building is offset from the center of mass (CM) (Fig. 5), the building will translate and rotate when subject to earthquake excitations. The translation and rotation can result in displacement amplification at the edges of the building as shown in Figure 5(b). It is postulated herein that the displacement demand on a torsionally unbalanced building could be constrained by an upper limit which is referred herein as the peak displacement demand (***PDD***). It is noted that maximum displacement could occur at the flexible or the stiff edge of the building depending on the dominant mode of vibration. The peak displacement demand referred in this section represents the higher of the two values.

The peak displacement demand (***PDD***) of an asymmetrical building has been estimated in this study using the response spectral analysis method assuming linear elastic behaviour. The displacement response of a torsionally coupled building can be determined by calculating eigenvalues and eigenvectors of the equation of motion as shown by Eq. (5).

$$[\bar{\boldsymbol{k}} - \omega_n^{\ 2}\bar{\boldsymbol{m}}]\phi_n = \mathbf{0} \tag{5}$$

where ω_n are the natural frequencies of a torsionally coupled building; $\bar{\boldsymbol{k}}$ and $\bar{\boldsymbol{m}}$ is the stiffness and the mass matrix respectively, as defined by Eq. (6).

$$\bar{\boldsymbol{k}} = \begin{bmatrix} k_y & e_s k_y \\ e_s k_y & k_\theta \end{bmatrix} \quad (6a) \quad \text{and} \quad \bar{\boldsymbol{m}} = \begin{bmatrix} m & 0 \\ 0 & r^2 m \end{bmatrix} \quad (6b)$$

where k_y is the total lateral stiffness of the building in the y direction; k_θ is the torsional stiffness about the center of mass; m is the mass; r is the mass radius of gyration; and e_s is the offset of the center of resistance from the center of mass.

The mode shape vector ϕ_n is given by Eq. (7).

$$\phi_n = \begin{bmatrix} \phi_{yn} \\ \phi_{\theta n} \end{bmatrix} \quad (7)$$

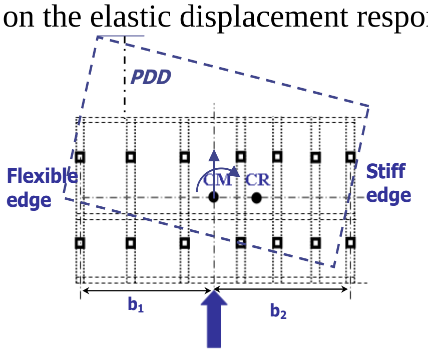
where ϕ_n is the natural mode shapes of vibration of the n^{th} mode, consisting of translational and rotational components.

Having determined the natural frequencies and the mode shapes of vibration, the contribution of the n^{th} mode (u_n) of vibration to the total displacement "u" of the building can be conservatively estimated by Eq. (8) assuming a constant value of the displacement demand (which is the value of RSD_{max} as defined by Fig. 5b):

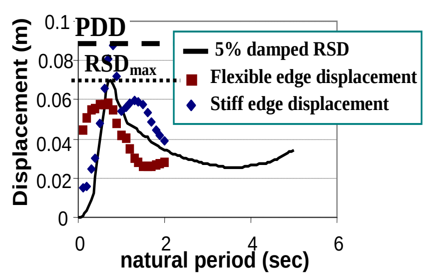
$$u_{yn} = PF_n \ \phi_{yn} \ RSD_{max} \quad (8a)$$

$$u_{\theta n} = PF_n \ \phi_{\theta n} \ RSD_{max} \quad (8b)$$

where ϕ_{yn} and $\phi_{\theta n}$ are the translational and rotational components of the n^{th} mode of vibration; PF_n is the participation factor of the n^{th} mode; and RSD_{max} is maximum point on the elastic displacement response spectrum.



(a) floor plan



(b) Peak displacement demand

Figure 5 Amplification of displacement demands in an asymmetrical building

The displacement demand values at the flexible edge $\Delta_{flex,n}$ and stiff edge $\Delta_{stiff,n}$ of the building for the n^{th} mode of vibration can be expressed in terms of the translational and rotational displacement (as defined by Eq. (9a) and (9b)).

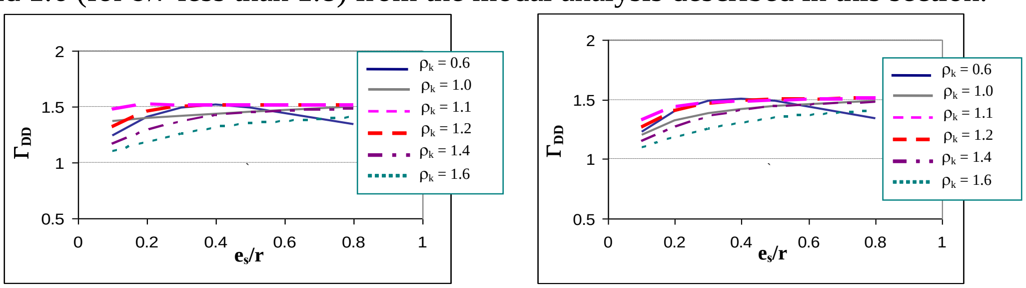
$$\Delta_{flex,n} = u_{yn} - u_{\theta n} \ b_1 \quad (9a)$$

$$\Delta_{stiff,n} = u_{yn} + u_{\theta n} \ b_2 \quad (9b)$$

where b_1 and b_2 are the offsets of the flexible and stiff edge of the building respectively from its center of mass. The displacement at the edges of the building for each mode of vibration can be combined according to the *square-root-of-the-sum-of-the square* SRSS method or the *complete quadratic combination* (CQC) for torsional sensitive systems with closely spaced natural frequencies (Chopra, 2000).

Field surveys on soft-storey buildings (Wilson *et al.*, 2005) revealed that the normalised eccentricity (e) and uncoupled natural frequency ratio values ($\rho_k = 1/r\ (k_\theta/k_y)^{1/2}$) of the building (both parameters have been normalised with respect to the mass radius of gyration r) ranges in between 0.05 to 0.6 and 0.8 to 1.6 respectively. The offset of the flexible, or stiff edge, of the building from its center of mass normalised with respect to the mass radius of gyration (b/r) was less than 1.8. The displacement demand values at the flexible and stiff edges of the building for each mode of vibration were calculated and combined using the SRSS and the CQC rules, in order that the value of the peak displacement demand (*PDD*) on individual edge elements in the building can be determined.

The torsional amplification factor Γ_{DD} is presented in Figure 6 for a range of combinations of e and ρ_k values likely to be found in real buildings. Γ_{DD} is defined as the ratio of the peak displacement demand value on the edge elements to the maximum response spectral displacement value (ie. $\Gamma_{DD}=PDD/RSD_{max}$). Figure 6 indicates that the SRSS rule provides conservative estimates of the peak displacement demands of the critical edge elements. Importantly, the value of Γ_{DD} was found to be insensitive to variations in the value of parameter: e and ρ_k. The value of Γ_{DD} was found to vary up to around 1.6 (for b/r less than 1.8) from the modal analysis described in this section.

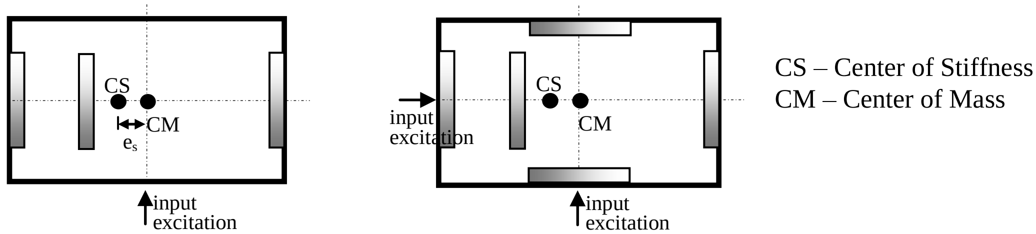


(a) SRSS rule (b) CQC rule

Figure 6 Torsional amplification factor Γ_{DD} using response spectral analysis

Parametric studies were undertaken based on linear and non-linear time history analyses of single building models supported by three lateral resisting frames presented in Figure 7(a). Refer to Lumantarna *et al.* (2010) for details of accelerograms. The frame elements were so disposed within the building in order that the values of eccentricity (e) and uncoupled natural frequency ratio (ρ_k) varied within the range 0.1 to 0.5 and 0.8 to 1.3, respectively. The initial stiffness of individual frames in the model was calibrated such that the uncoupled natural period of vibration of the building ranges in between 0.2 to 2secs. The strengths of the frames were assumed to be equal. The notional yield strength of each frames was adjusted in order that the strength of each frame was exceeded by the elastic strength demand by a factor (F_y=Elastic strength/R_μ) of 2 to 4. Hysteretic models of the building were based on the estimated hysteretic behaviour of non-ductile structural components including soft-storey columns and unreinforced masonry walls. All the analyses were based on 15% degradation in strength per unit increase in the value of μ.

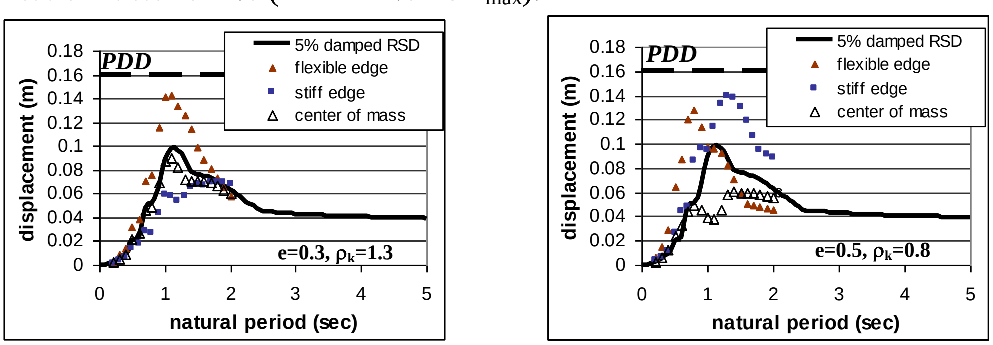
Meanwhile, control analyses assuming no strength degradation have also been undertaken.



(a) Model A (b) Model B

Figure 7 Single-storey building models

The peak displacement demand of elements at the edges and center of mass of the building are plotted in Figure 8 against their initial uncoupled natural period. Maximum displacement demand generally occurred at the stiff edge of a torsionally flexible building (which was characterised by low values of ρ_k). On the other hand, the maximum displacement demand at the flexible edge was generally amplified in a torsionally stiff building (which was characterised by high values of ρ_k). This trend is consistent with the literature which reported favourable response for the flexible edges for the torsionally flexible building. However, contrary to the trend observed from the literature (eg. Rutenberg and Pekau, 1987; Chandler and Hutchinson, 1988), the peak displacement demand values based on elastic behaviour were generally shown to be insensitive to variations in the value of *e* and ρ_k. It is shown that the displacement demand values were all constrained by the *PDD* limit that has been estimated by applying a torsional amplification factor of 1.6 (*PDD* = 1.6 RSD_{max}).

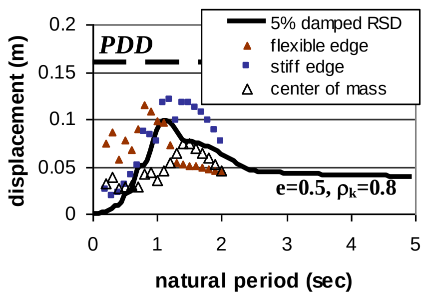


(a) e = 0.3, ρ_k = 1.3 (b) e = 0.5, ρ_k = 0.8

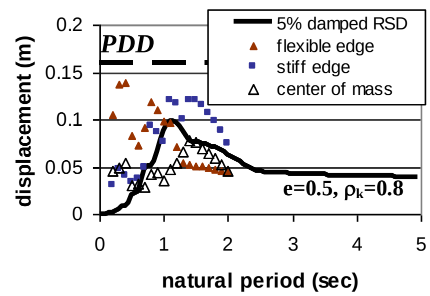
Figure 8 Elastic displacement demands of asymmetrical building with generated accelerogram class C

The peak inelastic displacement demand on elements in asymmetrical buildings based on R_μ = 2 and 4 are presented in Figure 9 for systems with and without strength degradation. The peak displacement demand values from analyses incorporating inelastic behaviour (but with no degradation in strength) were shown to be generally lower the peak displacement demand values based on elastic behavior (compare Figs 8(b) with 9(a) & (c)). The dissipation of energy by hysteretic means is considered to have reduced the element displacement demands. However, much higher displacement demand values have been observed from analyses which have incorporated strength degradation behaviour (Figs. 9(b) & (d)). The much higher displacement demand is as a result of accumulation of residual displacement. In such conditions, the notional ***PDD*** limit (***PDD*** = 1.6 RSD_{max}) can be exceeded.

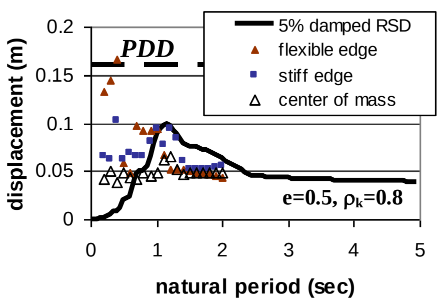
Despite significant strength degradation, the peak displacement demands of the edge elements in asymmetrical buildings were well constrained by the notional peak displacement demand limit (***PDD***) if the uncoupled natural period of the building was higher than the dominant period of earthquake excitations (Figs. 9(b) & (d)). It was shown further that with a modest strength reduction factor (R_μ) of 2, the element inelastic displacement demands were also constrained within the ***PDD*** limit irrespective of the uncoupled natural period of the building. The robustness of the notional ***PDD*** limit in estimating the peak element displacement demand of the building has been demonstrated herein.



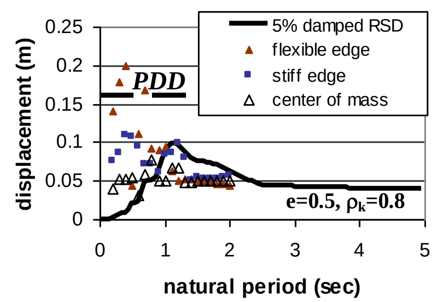
(a) without strength degradation, $R_\mu = 2$



(b) with strength degradation, $R_\mu = 2$



(c) without strength degradation, $R_\mu = 4$

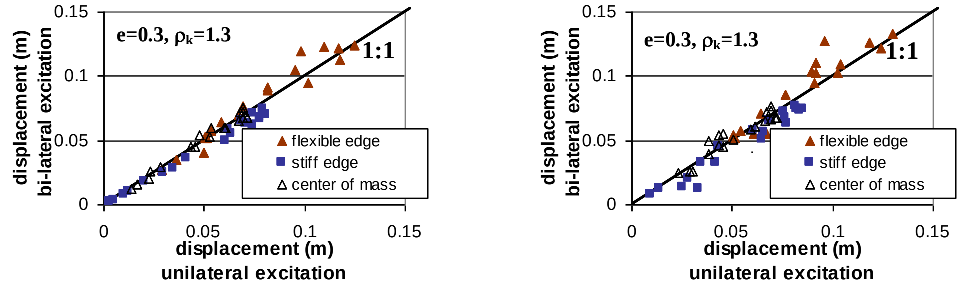


(d) with strength degradation, $R_\mu = 4$

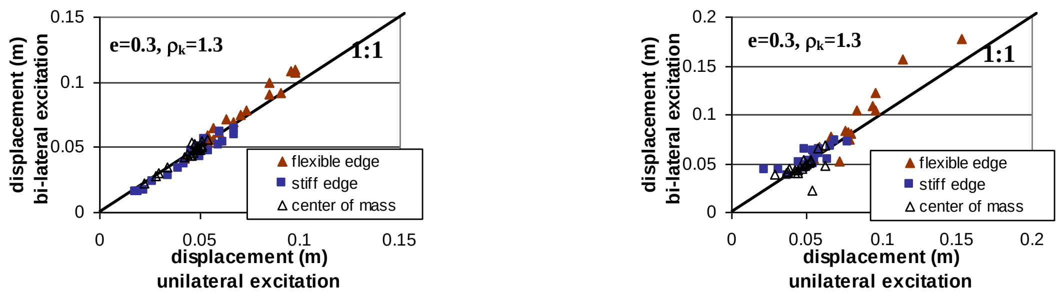
Figure 9 Inelastic displacement demands of asymmetrical building with San Fernando earthquake

The torsional amplification factor of 1.6 and results of THA as presented in Figure 9 were based on simplified building models featuring parallel frames and uni-lateral input excitations (model A in Fig. 7(a)). Contributions by frames which were oriented in the orthogonal directions have been neglected. The peak element displacement demand values of the simplified model (model A) have also been compared with displacement values of models of which orthogonal frames subject to bi-directional excitations (model B in Fig. 7(b)). The uncoupled natural periods of vibration of the building in the two orthogonal directions were assumed to be equal. The notional yield strength of each frame in the orthogonal direction has also been adjusted in order that the strength of each frame was exceeded by the strength demand by a factor of 2 to 4 (R_μ = 2 - 4). The building was subject to excitations of equal intensity in both directions.

Figure 10 shows correlation plots of displacement demand values of building models under uni-lateral and bi-lateral excitations. The displacement demand values of model A and model B were generally found to be well correlated. It is shown that the displacement demand behaviour of realistic asymmetrical buildings (those incorporating orthogonal frames) subject to bi-directional excitations can be represented conservatively by the analyses of the simplified models (which feature uni-lateral excitations), provided that the center of resistance is in alignment with the center of mass in the orthogonal direction (model B in Fig. 7(b)).



(a) without strength degradation, $R_\mu = 2$ (b) with strength degradation, $R_\mu = 2$



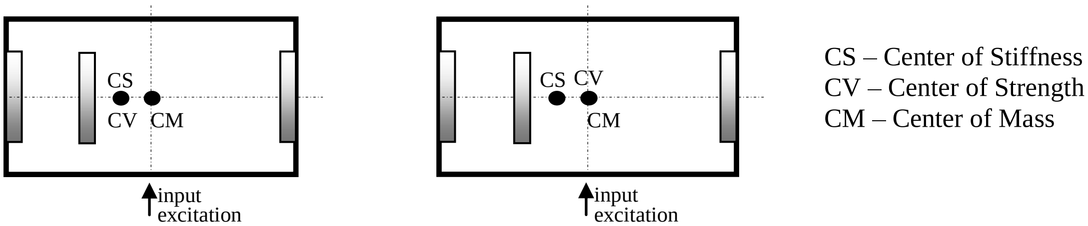
(c) without strength degradation, $R_\mu = 4$ (d) with strength degradation, $R_\mu = 4$

Figure 10 Displacement demands of building model A subject to uni-lateral and building model B subject to bi-directional excitations (San Fernando earthquake)

Asymmetrical buildings that have been designed according to the seismic design codes normally have the structural elements well positioned to account for the potential response behavior. The structural elements are normally designed for earthquake actions applied at the notional center of mass of the building with an additional eccentricity (AS1170.4, 2007; ICC, 2006; EN1998-1, 2004). Consequently, asymmetrical buildings that have been designed in accordance with the seismic design procedures will have the center of strength located close to the center of mass of the building.

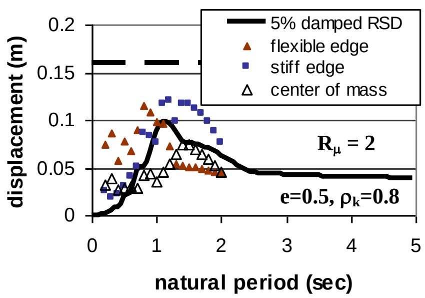
To investigate the displacement response behaviour of code designed buildings, non-linear time history analyses were performed on building model C (Fig. 11(b)) with the center of strength (CV) coinciding with the center of mass (CM). Frame elements in model C (Fig. 11(b)) were positioned identically as for model A (Fig. 11(a). However, the strength of the frames in model C were distributed such that the center of strength CV coincided with the center of mass CM.

The maximum displacement demand behavior associated with building models A and C were investigated by correlating the maximum displacement demand values against the uncoupled natural periods of vibration of the buildings in Figure 12. The displacement behavior of the center of mass of the buildings is shown to be similar between models A and C. However, the reduction in strength eccentricity is shown to generally reduce the torsional response of the building especially when the building has been excited well into inelastic range. The observed trend is expected as the displacement behaviour of a system excited well into inelastic range is governed more by the tangential stiffness as opposed to the initial stiffness of the system. In view of the observed trends, model A is considered to be conservative in representing the torsional response behaviour of buildings.

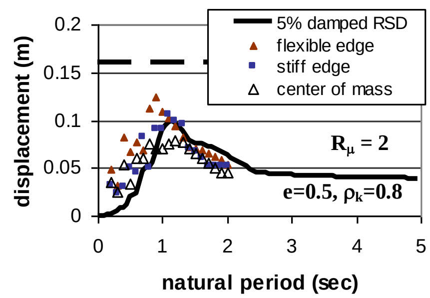


(a) Model A (b) Model C

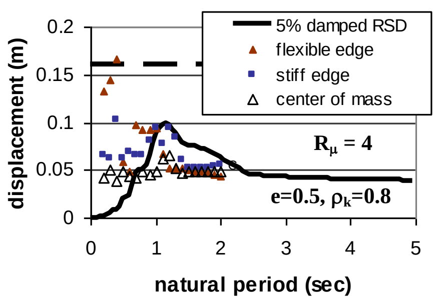
Figure 11 Single storey building models



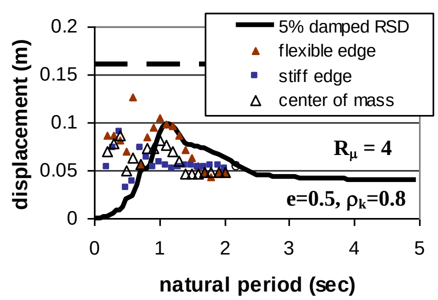
(a) Model A (CV = CS), $R_\mu = 2$



(b) Model C (CV = CM), $R_\mu = 2$



(c) Model A (CV = CS), $R_\mu = 4$



(d) Model A (CV = CM), $R_\mu = 4$

Figure 12 Displacement demands of building model A and building model C with strength degradation (San Fernando earthquake)

5. Concluding remarks

A simple seismic assessment of buildings in regions of low to moderate seismicity has been developed based on the concept of the displacement-controlled behaviour. A generalised displacement response spectrum model in bi-linear form was presented to provide seismic response predictions assuming linear elastic behaviour. The model has been extended to account for inelastic behaviour and torsional actions. Parametric studies based on non-linear time-history-analysis revealed that the peak displacement demands of non-ductile structures can be conservatively estimated by the maximum point of the displacement response spectrum. Further studies on asymmetrical single-storey building models have shown that the peak displacement demands of asymmetrical buildings that an amplification factor of 1.6 can be applied to estimate the peak displacement demands of (***PDD*** = 1.6 RSD_{max}) of buildings with one-asymmetry.

It has been demonstrated in this study that the maximum drift demand on a building can be estimated based on the properties of the ground shaking without the need to undertake time consuming modelling and computations. The maximum drift demand can be compared to the displacement capacity for a quick assessment of its potential seismic risk to collapse.

References

Al Abadi, H., Lam, N.T.K., Gad, E. 2006. A Simple Displacement Based Model for Predicting Seismically Induced Overturning, *Journal of Earthquake Engineering* 10(6): 775-814.

AS 1170.4 2007. Structural Design Actions – Part 4 Earthquake Actions, Sydney: Standards Australia.

Chandler, A.M., Hutchinson, G.L. 1988. A modified approach to earthquake resistant design of torsionally coupled buildings, *Bulletin of the New Zealand National Society of Earthquake Engineering*, 21: 140–152.

Chopra, A. K. 2000. *Dynamics of Structures – Theories and Applications to Earthquake Engineering* (2nd edition), Prentice Hall, New Jersey.

Doherty, K., Griffith, M., Lam, N.T.K., Wilson, J.L. 2002. Displacement-based analysis for out-of-plane bending of seismically loaded unreinforced masonry walls, *Journal of Earthquake Engineering and Structural Dynamics* 31(4), 833-850.

EN 1998-1 2004. Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, BSI.

Faccioli, E., Paolucci, R., Rey. J. 2004. Displacement Spectra for Long Periods, *Earthquake Spectra* 20(2), 347 – 376.

Federal Emergency Management Agency 2003. NEHRP Recommended Provisions for Seismic Regulations for New Building s and Other Structures Part I : Provisions (FEMA 450-1), Washington, D.C.

Idriss, I.M. and Sun, J.I. 1992. Users Manual for SHAKE-91, Maryland USA and Dept. of Civil & Environmental Engineering, University of California, Davis, USA

Kafle, B., Lam, N.T.K. Gad, E.F., Wilson, J.L. 2011. Displacement Controlled Rocking Behaviour of Rigid Objects," *Journal of Earthquake Engineering and Structural Dynamics*, EQE-10-0162.R2 (published on line in *Early View* section of journal).

Lam, N.T.K. 2002. *Program GENQKE users' manual*, Department of Civil & Environmental Engineering, University of Melbourne, Melbourne, Australia.

Lam, N., Chandler, A. 2005. Peak displacement demand of small to moderate magnitude earthquake in stable continental regions, *Earthquake Engineering and Structural Dynamics* 34: 1047-1072.

Lam, N.T.K., Wilson, J.L., Hutchinson, G. L. 1995. The seismic resistance of unreinforced masonry cantilever walls in low seismicity areas, *Proceedings Pacific Conference of Earthquake Engineering,* Melbourne, Australia, pp. 225-244.

Lam, N., Wilson, J., Chandler, A., Hutchinson, G. 2000. Response spectral relationships for rock sites derived from the component attenuation model," *Earthquake Engineering and Structural Dynamics* 29, 1457 – 1489

Lumantarna, E., Lam, N., Wilson, J. 2011. Bi-linear Displacement Response Spectrum Model for Engineering Applications in Low and Moderate Seismicity Regions, *Journal of Earthquake Engineering*, in review.

Lumantarna, E., Lam, N., Wilson, J. and Griffith, M. 2010. Inelastic Displacement Demand of Strength Degraded Structures, *Journal of Earthquake Engineering* 14(4): 487-511.

Rutenberg, A., Perkau, O.A., 1987. Seismic code provisions for asymmetrical structures: a re-evaluation, Engineering Structures 9: 255 – 264.

Wilson, J., Lam, N., Rodsin, K. 2005. Seismic performance of multi-storey apartment buildings with a soft-storey. *Australian Structural Engineering Conference,* Newcastle, Australia.