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Effects of Building Asymmetry in Areas of Low-to-Moderate Seismicity

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ABSTRACT

Effects of building asymmetry in an earthquake are typically taken into account in a static analysis by incorporating an offset of the design seismic forces from the centre of resistance of the building. Whilst the analysis approach is simple the real behavior of the building in an earthquake can be very different to predictions from the calculations. Most commercial packages have the capability of 3-dimensional time-history analysis but few engineers are able to exercise good control of the analysis and be able to evaluate the results. This paper introduces a simple modelling approach which can be operated on a generic modelling platform such as EXCEL and MATLAB and yet capable of capturing the dynamic behavior of an asymmetrical building system. It can be shown by this alternative modelling approach that in certain conditions of moderate ground shaking the displacement at the edge of the building cannot exceed 1.6 times the peak displacement demand as indicated by the elastic response spectrum of the earthquake.

1. INTRODUCTION

Inertia forces generated in a building in seismic conditions are traditionally represented by quasi-static forces that are applied at the position of the centre of mass of the building which can be offset significantly from the centre of rigidity of the supporting lateral resisting elements. Clearly, the extent of distress to the building increases with increasing amount of offset (or eccentricity) of the quasi-static forces. However, the actual inertia forces are dynamic in nature and consequently the behavioural trends can be very different to those shown by results of static analyses. Many structural engineer designers resort to commercial packages which provide “solutions” to dynamic problems based on given geometry model of the building and a

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seismic action model which is normally represented in the form of a response spectrum. However, results generated by individual computer runs would not serve the purpose of identifying what factors are most influential and how results would change as values of certain parameters vary. It is always risky to rely on results generated by one computer model whilst with little knowledge of how results would change because of factors that have not been modelled. In perspectives, the quality of a design/assessment process is not on the 3D, or 4D, visualisation of the results of an isolated execution of a program but is instead on the ability to identify what matters most amid all the uncertainties in practice and to provide accurate information on the relevant parameters.

The more elaborate the computer model of a structure the more time consuming, and challenging, it can become in undertaking repetitive sensitivity analyses to support decision making. Whilst the use of dynamic analysis on fully developed structural frame, or finite element, models is now very common in contemporary structural design practices the capacity of the engineer to exercise judgement on the design, and to evaluate accuracies of the computer generated results, are generally limited to their understanding of the principles of statics. There are, indeed, significant differences on how asymmetry affects the behaviour a building in quasi-static and seismic (dynamic) conditions. The motivation behind the writing of the paper is the need to fill in this knowledge gap by showing the behavioural pattern from first principles, and by providing the key relationships, thereby facilitating dynamic simulations to be undertaken on a common platform such as EXCEL, or MATLAB. Linearly elastic behaviour of the lateral resisting elements has been assumed in analyses presented in this paper which is primarily aimed at illustrating certain important phenomena that are not well represented by quasi-static models. The trends that have been revealed in the linear elastic analyses have been found to be consistent with those observed from non-linear time-history analyses based on different levels of ductility (Lumantarna *et al.* 2013).

Research that have been undertaken in the past 30 years on the topic of seismically induced torsional behaviour in buildings is to ensure that the displacement demand of individual walls, and columns, in the building can be represented conservatively by quasi-static provisions as stipulated in design codes of practices (Dempsey and Tso 1982, Chandler and Hutchinson 1987; Chandler 1988; Rutenberg and Pekau 1987 & 1989). Early studies on seismically induced torsional behaviour in buildings were based on considering a restrictive class of moment resisting frames (e.g., Kan and Chopra 1977; Cheung and Tso 1987, Hejal and Chopra 1989). Results obtained from the 3D analyses of multi-storey building models in terms of their torsional rotation were found to be very consistent with results of analyses of simple single-storey building models. Similar comparative analyses have also been undertaken on a wider form of multi-storey buildings that are supported jointly by structural walls and moment resisting frames (Lam *et al.,* 1997). It is generally recognised that the torsional response behaviour of a multi-storey building can be simulated by the analysis of a single-storey model. Thus, a large amount of research investigations have been performed on single-storey building models. For similar reasons analyses presented in this paper are based on results of analyses of simple models that comprised only a single floor plate.

An important element of originality in this paper is the focus on conditions of displacement controlled behaviour which is particular relevant to the considerations of small-medium magnitude (local) earthquakes affecting regions of low to moderate seismicity (Lumantarna *et al.,* 2012). By displacement controlled behaviour, the maximum displacement experienced by a based excited linear elastic single-degree-of-freedom system, or pendulum, would be dependent on the amount of displacement experienced at the support and not sensitive to its natural period of vibration (refer upper part of Fig. 1). The phenomenon of acceleration, velocity and displacement controlled behaviour in the context of an elastic response spectrum has been known for a long time, and become well publicised by Newmark and Hall (1982). The extension of this principle to the rocking behaviour of free-standing objects in recent times has important implications to the engineering of structures, and non-structural components, in regions of low to moderate seismicity. It is shown schematically in the lower part of Fig. 1 that in displacement controlled conditions the amount of displacement at the upper end of a rocking object is insensitive to the height of the centre of gravity of the object. This phenomenon was confirmed in shaking table experiments conducted by the authors and co-workers (Kafle *et al.* 2011). The height of the centre of gravity of a free-stranding object can be considered as the vertical eccentricity about its base. In other words, the amount of rotation of the object in the vertical plane is actually insensitive to the eccentricity value. By analogy, the amount of horizontal rotation of the floor plate forming part of a building could also be insensitive to the eccentricity value contrary to what is shown by quasi-static models. A key objective of this paper is to reveal the significance of the effects of eccentricity in terms of the displacement demand behaviour of the building.

The rest of this paper is structured as follows:

- general layout of single-storey building models that have been employed in the investigation
- frequency behaviour of the coupled modes of vibration
- displacement and rotation behaviour associated with the two vibration modes
- Displacement demand behaviour at the edge of the floor plate which contains the key results to be presented in the paper
- Details of eigen-solutions
- Closing remarks over results that have been presented

2. SINGLE-STOREY BUILDING MODEL

A regular multi-storey building can be simplified into a single-storey building model as illustrated by the schematic diagram of Fig. 2. The simple model used for analysis is rectangular in plan of width 2B and with the centre of rigidity (CR) offset from the centre of the building, which is defined herein as eccentricity (e). Although the modelling was based on a regular building the trends so observed are generally applicable to the wider form of buildings. Other relevant dynamic and stiffness properties of the building model are as defined by the following parameters:

$M =$ mass; $J =$ torsional moment of inertia; $J = Mr^2$; $r =$ mass radius of gyration;
$K_x =$ Horizontal stiffness in the x - direction; $K_t =$ Torsional stiffness; $\frac{K_t}{K_x} = b^2$

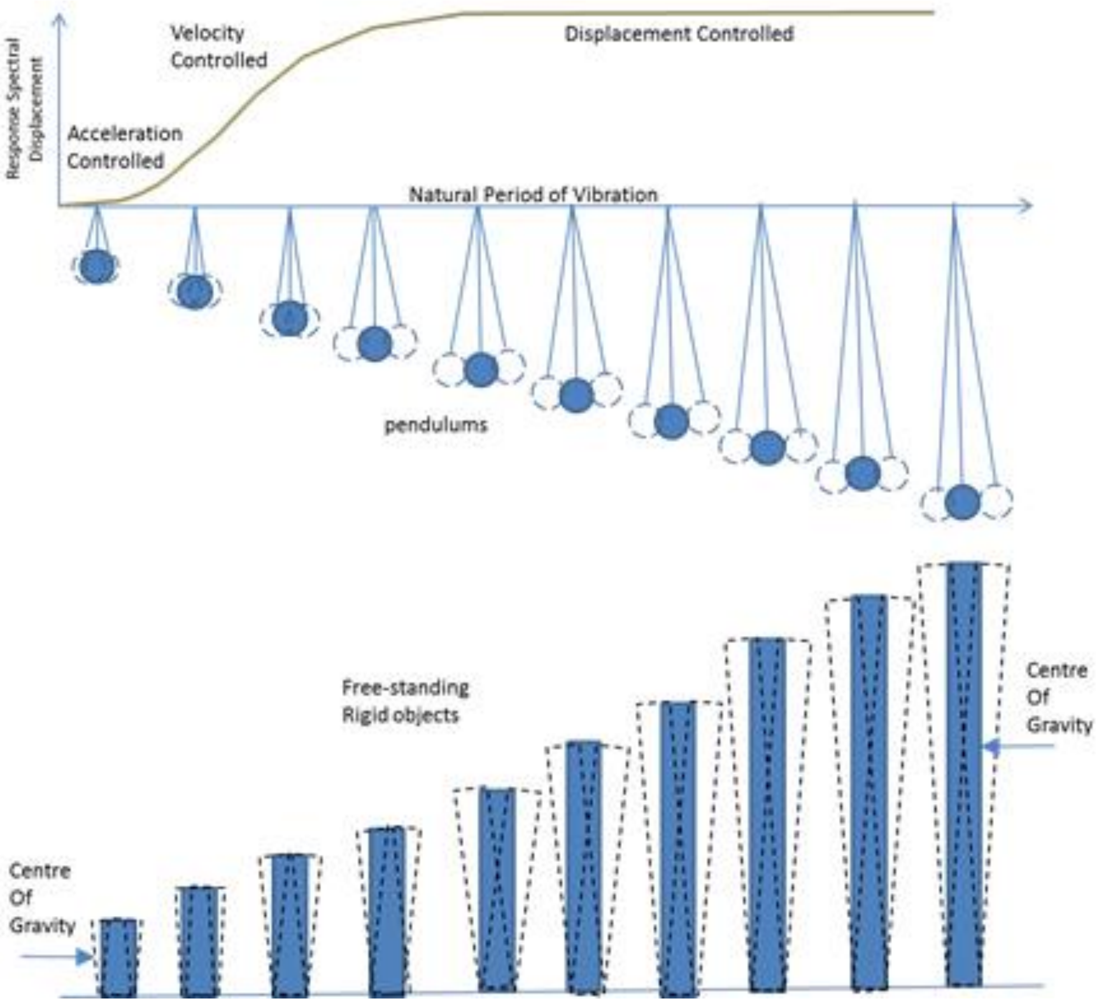


Figure 1 Schematic representation of displacement controlled behaviour

Parameter "b" is used to characterise the torsional stiffness of the model. The higher the b value the higher the torsional stiffness of the lateral supporting elements in relation to their collective translational stiffnesses. Imagine that the simplified model of the building is supported only by a pair of frames then 2b is the frame spacing, and b is the perpendicular offset of the frame from the CR position. The value of B which is the amount of offset of the edge of the floor plate from its centre varies from 1.25 (square plate) to 1.75 (rectangular plate with a very high aspect ratio) times the mass radius of gyration (r). Thus, it is conservative to assume that $b_r =$ B/r = 1.8.

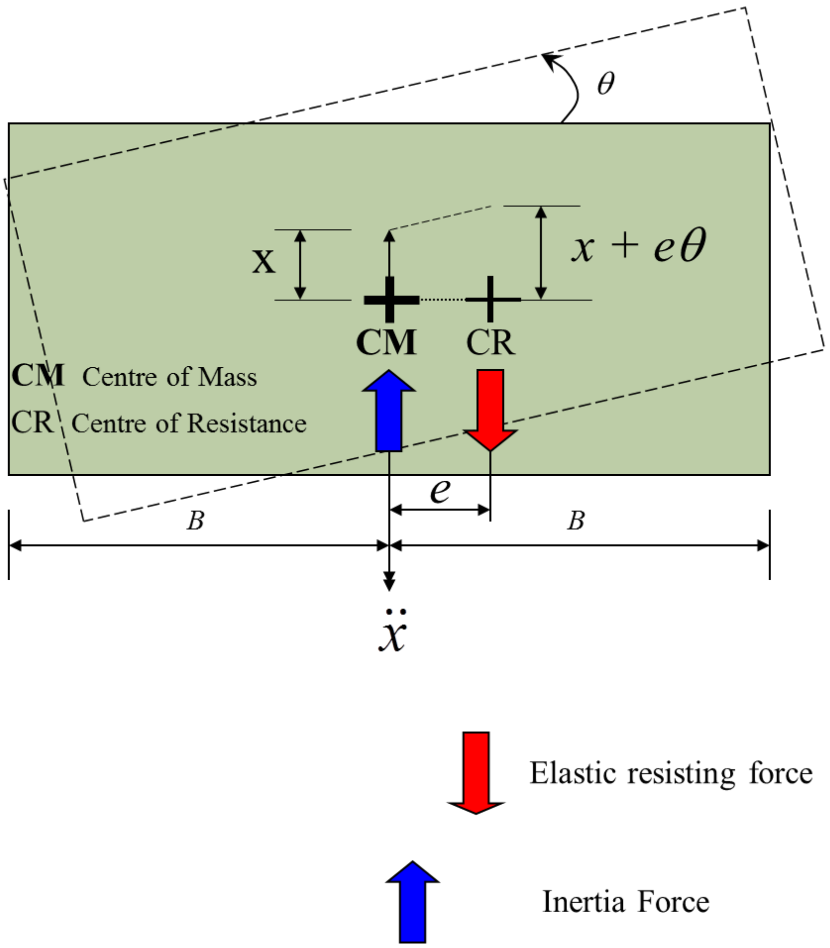


Figure 2 Schematic diagram showing the single-storey building model

3. FREQUENCY RATIOS OF THE COUPLED MODES OF VIBRATION

A building model featuring uniaxial asymmetry will have every translational mode of vibration split into two coupled torsional modes of vibration. Thus, the example (asymmetrical) single-storey building model features two coupled modes of vibration, and not just a single mode even though the model has only one rectangular floor plate. The two coupled modes are increased to three if the model features bi-axial asymmetry (i.e., asymmetry in both directions). The dynamic response behaviour of the structure in natural (free) vibration is not a simple harmonic function (as in the case of a symmetrical single-storey model) but the sum of two harmonics each of which is associated with a coupled torsional mode of vibration. The first mode has a natural angular frequency ($\Omega_{j=1}$) which is lower than that of the corresponding symmetrical building model (ω_x) whereas the second torsional mode of vibration has angular natural frequency ($\Omega_{j=2}$) which is higher than ω_x.

Values of the two frequency ratios $\lambda_{j=1,2}$ as defined by Eq. (1) is summarised in Fig. 3.

$$\Omega_{j=1,2} = \lambda_{j=1,2} \times \omega_x \quad (1)$$

It is shown that the value of λ_1 is always greater than unity whereas the value of λ_2 is always smaller than unity. Frequency values of the two coupled modes can be very close (i.e., λ values of both modes are close to unity) when the value of b_r (i.e., b/r) is also close to unity, and more so for small eccentricity values. The values of both λ_1 and λ_2 were obtained as solution to an eigenvalue problem forming part of the modal dynamic analysis of the building model the details of which will be illustrated in a later section of the paper. It is shown that the value of the eccentricity parameter (e/r) can have some effects on the dynamic response behaviour on the building system but its extent of influence depends on the value of the other parameter (b/r) and the mode of vibration. The influence of the eccentricity value on the dynamic torsional coupling behaviour of the asymmetrical building system is clearly not as distinctive as the influence of the same on static behaviour which is familiar to all structural engineers.

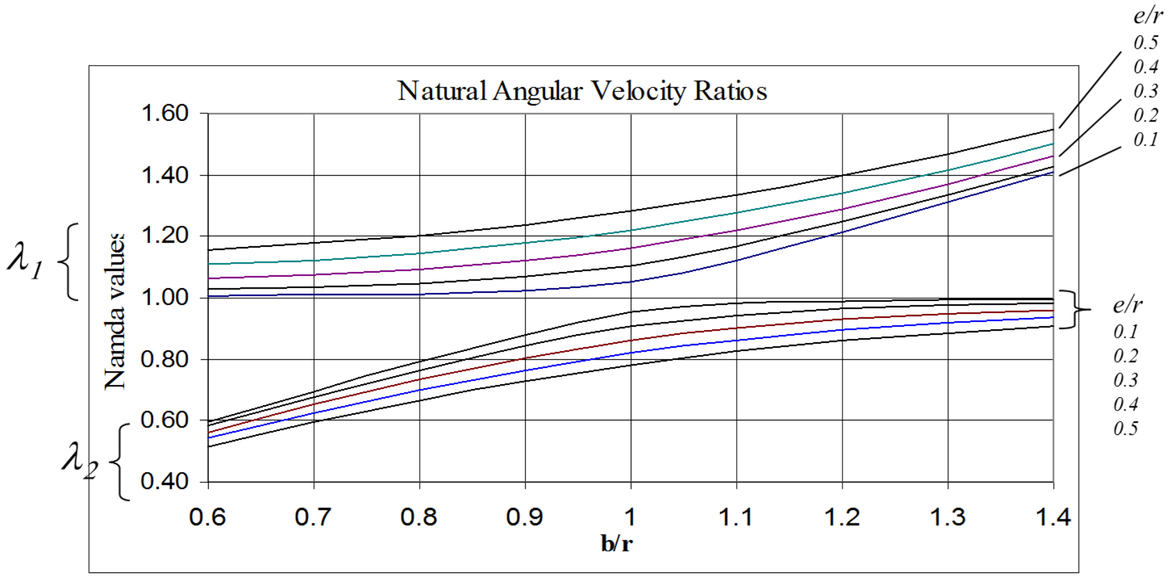


Figure 3 Frequency ratios of the two coupled vibration modes

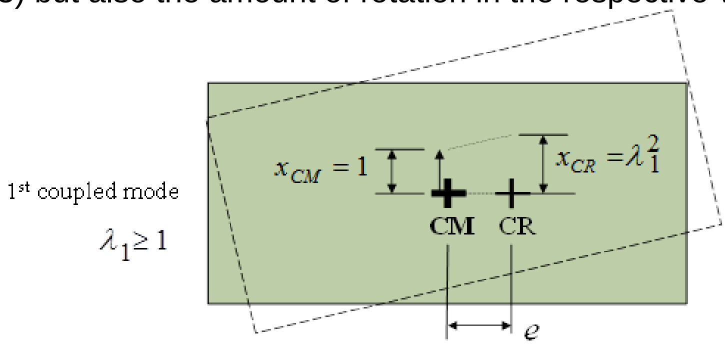
4. RATIO OF TRANSLATION AND TWIST

Another important outcome of the dynamic modal analysis is the shape of deflection of the building in natural vibration. With the analysis for torsional behaviour (of a single-storey building model) the “deflection shape” as obtained from modal analysis is essentially the amount of rotation of the floor plate in the horizontal plane associated with each of the two vibration modes. It was found that the amount of rotation of the building floor is dependent on the value of λ^2 and the amount of offset of the centre of rigidity (CR) of the building from its centre of mass (CM) as illustrated by the schematic diagrams of Figs. 4a and 4b. Relationships defining the normalised displacement value at the CM and CR positions for the two coupled vibrational modes as illustrated in Figs. 4a and 4b are as follows:

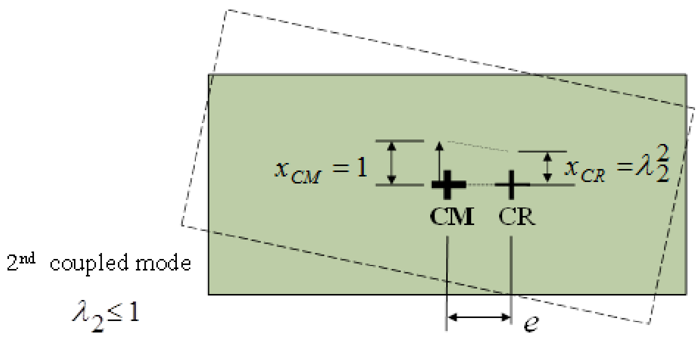
$$\begin{Bmatrix} \frac{x}{r} \\ \theta \end{Bmatrix} = \begin{Bmatrix} 1 \\ \left(\frac{\lambda_j^2 - 1}{\frac{e}{r}} \right) \end{Bmatrix} \tag{2a}$$

$$\frac{x_{CM}}{r} = 1 \ \text{ or } x_{CM} = r; \quad x_{\text{CR}} = x_{CM} + \theta.e = r + \left(\frac{\lambda_j^2 - 1}{\frac{e}{r}} \right) \times e = \lambda_j^2.r \tag{2b}$$

By proportion when x_{CM} = 1, x_{CR} = λ^2_j. The displacement of the building is shown to have been normalised with respect to the amount of displacement at its CM (i.e., x_{CM} = 1). The normalised displacement value at the CR is accordingly equal to λ^2 for the respective mode of vibration (Figs. 4a and 4b). The sense of rotation as shown in Figure 4a may appear counter intuitive to those who base their reasoning on their understanding of static behaviour. The opposite sense of rotation in the vibrational modes as shown in the figures is reflective of the reversible and cyclic nature of dynamic behaviour. The values of λ_1^2 and λ_2^2 are not only controlling frequency behaviour (Figure 3) but also the amount of rotation in the respective vibration mode.



(a) 1st coupled mode of vibration



(b) 2nd coupled mode of vibration

Figure 4 Normalised displacement at CM and CR of the two coupled vibration modes

For those whose mindset are framed by the results of static analyses the offset of the CR from the CM (ie the eccentricity) is the most dominant factor controlling torsional actions. However, this notion is contradicted by the following interesting observations from results of dynamic modal analysis:

(i) the value of λ is only weakly dependent on the value of e/r as shown by Fig. 3,
(ii) With higher b/r values the influence of e/r becomes very minor
(iii) for a given value of λ the amount of rotation is actually smaller, and not larger, when the value of the eccentricity is increased (Fig. 4a),
(iv) the two vibration modes feature opposite sense of horizontal rotation which is a phenomenon that cannot be replicated by quasi static actions (Figs. 4a & 4b).

5. DISPLACEMENT ESTIMATES AT DESIGNATED POSITIONS

The displacement time-history at any selected position on the floor plate is the sum of contributions by the two coupled modes of vibration. For each contributing mode the displacement time-history is the product of three factors:

(i) Normalised displacement value at the position of interest (Fig. 4);
(ii) Participation factor, PF (given by Eq. (4d));
(iii) Displacement time-history of a single-degree-of-freedom system which has natural angular frequency Ω equal to that of the respective coupled mode of vibration, $U_{\Omega,,\zeta}(t)$ where ζ is the viscous damping ratio.

In summary, the normalised displacement value is equal to unity at the CM position; λ^2 at the CR position; and $x_{\pm B}$, as defined by the following expressions at the two edges (refer Figs. 5a and 5b):

$$x_{+\mathrm{B}} = 1 + \left(\lambda_1^2 - 1\right)\left(\frac{+B}{e}\right) \quad \text{where } \lambda_1^2 - 1 \geq 0 \text{ at edge } +\mathrm{B} \text{ in the 1st mode} \tag{3a}$$

$$x_{-\mathrm{B}} = 1 + \left(\lambda_2^2 - 1\right)\left(\frac{-B}{e}\right) \quad \text{where } \lambda_2^2 - 1 \leq 0 \text{ at edge } -\mathrm{B} \text{ in the 2nd mode} \tag{3b}$$

Expressions for determining the time-history of the displacement at the CM, CR and edges +B and –B are accordingly listed in the following:

At CM
$$x_{CM}(t) = \sum_{j=1}^{2} 1 \times PF_j \times U_{\Omega_j,\xi}(t) \tag{4a}$$

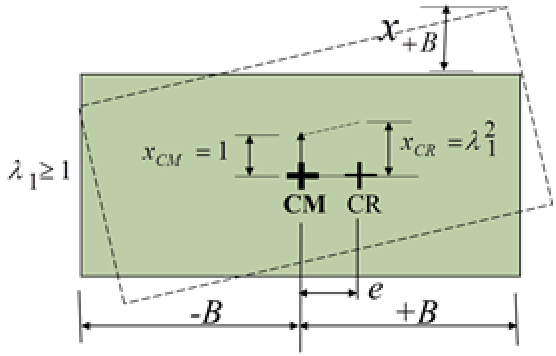
At CR
$$x_{CR}(t) = \sum_{j=1}^{2} \lambda_j^2 \times PF_j \times U_{\Omega_j,\xi}(t) \tag{4b}$$

At the two edges +B and –B:

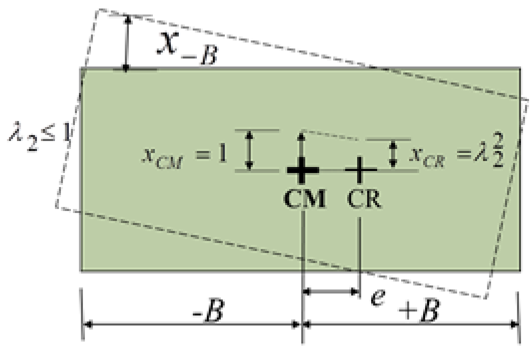
$$x_{\pm B}(t) = \sum_{j=1}^{2}\left(1 + \left(\lambda_j^2 - 1\right)\left(\frac{\pm B}{e}\right)\right) \times PF_j \times U_{\Omega_j,\xi}(t) \text{ or } \sum_{j=1}^{2}\left(1 + \frac{\lambda_j^2 - 1}{\mathrm{e_r}} B_r\right) \times PF_j \times U_{\Omega_j,\xi}(t) \tag{4c}$$

where,

$$PF_j = \frac{\begin{Bmatrix} 1 & \dfrac{\lambda_j^2 - 1}{e_r} \end{Bmatrix} \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \begin{Bmatrix} 1 \\ 0 \end{Bmatrix}}{\begin{Bmatrix} 1 & \dfrac{\lambda_j^2 - 1}{e_r} \end{Bmatrix} \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \begin{Bmatrix} 1 \\ \dfrac{\lambda_j^2 - 1}{e_r} \end{Bmatrix}} = \frac{1}{1 + \left(\dfrac{\lambda_j^2 - 1}{e_r} \right)^2} \qquad \text{(4d)}$$



(a) 1st coupled mode of vibration



(b) 2nd coupled mode of vibration

Figure 5 Normalised edge displacement of the two coupled vibration modes

A computer program operated on common platforms like EXCEL or MATLAB is well capable of simulating the dynamic torsional response behaviour of an asymmetrical building model by making use of Eqs. (4a) – (4d).

In a response spectrum analysis the time-history of the ground excitation is not given. The properties of the ground acceleration are expressed in terms of the response spectrum. The maximum estimated displacement value of a single-degree-of-freedom system ($U_{\Omega j,\zeta}$(max)) is represented by the response spectral displacement *RSD (T, ζ)* which is function of natural period of vibration (*T*) and damping ratio (ζ). In displacement controlled conditions, the value of response spectral displacement is taken as constant at the highest point of the displacement response spectrum which is denoted as RSD_{max}. *Square-root-of-the-sum-of-the-square* (SRSS) combination rule can be employed for combining the contributions from the two coupled vibration modes. Although more rigorous combination rules could be used it was revealed in a recent study that reasonably accurate estimates of the maximum displacement demand can be obtained by employing the simpler *SRSS* combination rule. Maximum displacement estimates at CM, CR and at the two edges +B and –B can be determined using the following simplified relationships:

At CM $$\frac{x_{CM}(\max)}{RSD(T,\xi)} = \sqrt{\sum_{j=1}^{2}\left[PF_j\right]^2} \qquad (5a)$$

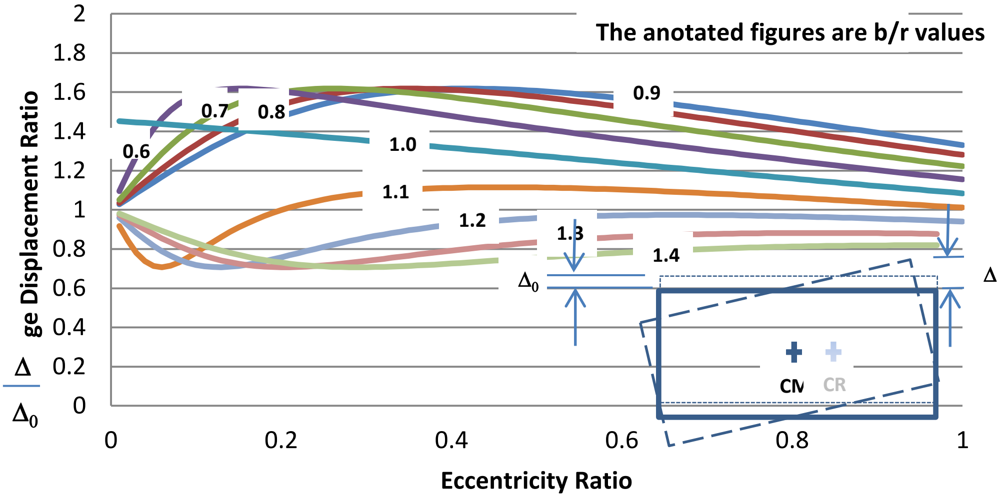
At CR $$\frac{x_{CR}(\max)}{RSD(T,\xi)} = \sum_{j=1}^{2}\left[\lambda_j^2 \times PF_j\right]^2 \qquad (5b)$$

At the two edges +B and –B:

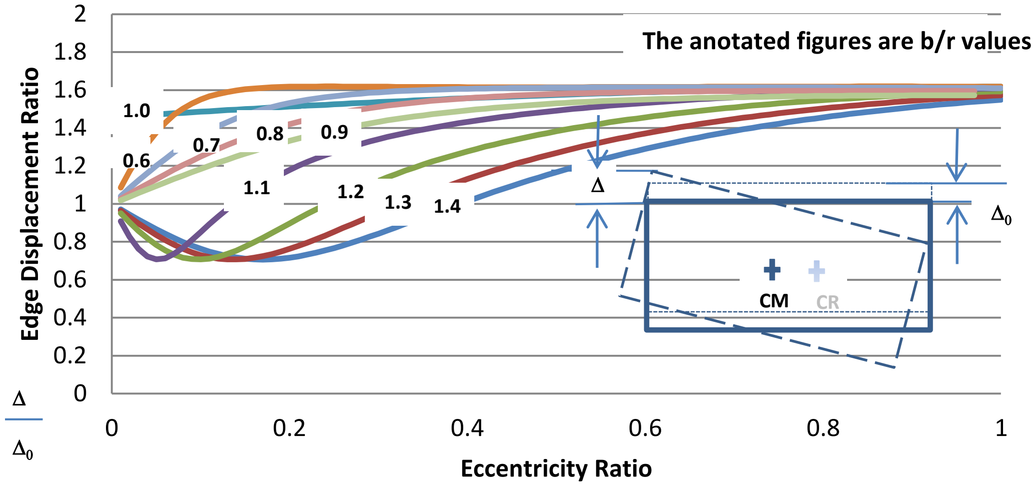
$$\frac{x_{\pm B}(\max)}{RSD(T,\xi)} = \sum_{j=1}^{2}\left[\left(1+\frac{\lambda_j^2-1}{e_r}B_r\right)\times PF_j\right]^2 \qquad (5c)$$

Figs. 6a and 6b presents two sets of estimates of the edge displacement ratio $\frac{x_{\pm B}(\max)}{RSD(T,\xi)}$ as obtained using Eq. (5c) for the two edges. One of the edges is on the far side of the CR (also known as the “flexible side”) and the other edge is on the near side of CR (also known as the “stiff side”). It is shown in Fig. 6a that the displacement at the flexible edge does not increase indefinitely with increasing eccentricity contrary to what a quasi-static model would show but asymptote to a upper limit of 1.6. The torsional stiffness of the model as reflected in the b/r value is shown to be much more influential to the torsional response behaviour than the eccentricity value. For example, decreasing the eccentricity value by half is shown to result in, at most, only 20% reduction in the edge displacement demand value. By contrast, a slight increase in value of the torsional stiffness parameter (b/r) from 1.1 to 1.2 would bring about just as much reduction. In a quasi-static analysis the flexible edge is naturally subject to higher displacement demand given that the floor plate would rotate in a sense that would result in higher drifts at the flexible edge. In a dynamic analysis the floor plate would also rotate in the opposite sense that might result in the stiff edge to be subject to higher drifts depending on the value of b/r.

It has been found further that the torsionally induced displacement amplification factor (i.e., edge displacement ratio of an asymmetrical and a symmetrical building model subject to identical ground excitation) would also asymptote to the upper limit of 1.6 in the (more general) velocity controlled conditions provided that the value of b/r is greater than 1.0.



(a) Displacement of the flexible edge on the far side of CR



(b) Displacement of the stiff edge that on the near side of CR

Figure 6 Edge Displacement Ratios in displacement controlled conditions

In comparison, the edge displacement ratio in static conditions feature indefinite linear increase in value with increasing value of e/r as shown in Figure 7. This type of behaviour is implicitly assumed in all static analyses. It is also evident from results presented in this paper that incorporating the edge displacement amplification factor of 1.6 in the design of a building would make 3D dynamic analyses and the provision of accidental eccentricity un-necessary for the purpose of assessing seismic performances provided that no individual structural elements in the building have their ultimate behaviour that are very sensitive to biaxial lateral actions.

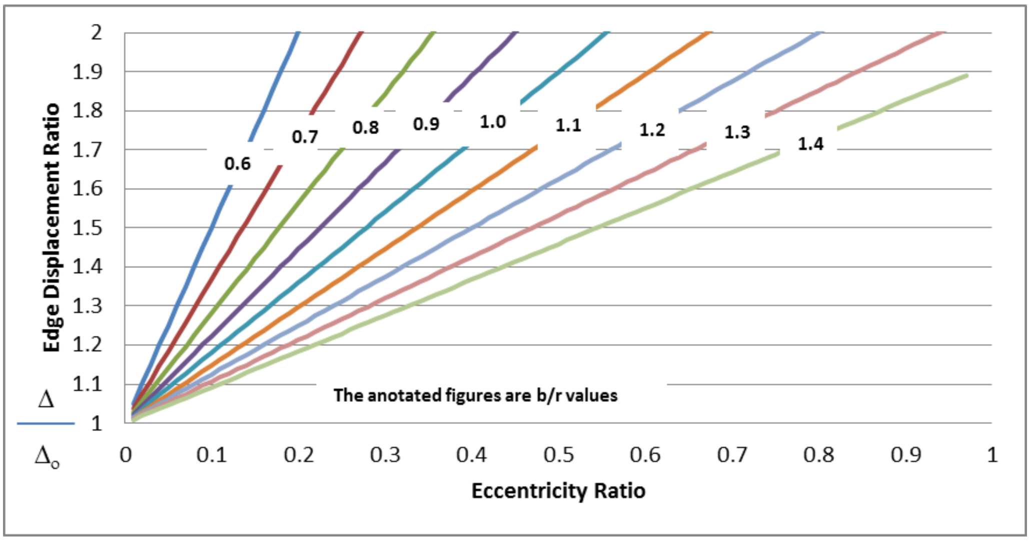


Figure 7 Edge Displacement Ratios according to results from static analyses

6. DYNAMIC EQUATIONS OF EQUILIBRIUM AND EIGENVALUE ANALYSIS

For completeness sake this section presents details of how the eigenvalues (λ_j) and eigenvectors (normalised displacement) as presented in the earlier sections of the paper were derived.

Equation of translational equilibrium

$$M\ddot{x} + K(x + e\theta) = 0 \quad (6a)$$

Equation of rotational equilibrium (taking moment about CM)

$$J\ddot{\theta} + K(x + e\theta)e + K_t\theta = 0 \quad (6b)$$

Equation of translational equilibrium

$$m\ddot{x} + K_x(x + e\theta) = 0 \quad (7a)$$

Divide both sides of equation by m and r

$$\frac{\ddot{x}}{r} + \omega_x^2\left(\frac{x}{r} + \frac{e}{r}\theta\right) = 0 \quad (7b)$$

Given that $\ddot{x}_r = \frac{\ddot{x}}{r}; x_r = \frac{x}{r}; e_r = \frac{e}{r}$:

$$\underline{\ddot{x}_r + \omega_x^2\{x_r + e_r\theta\} = 0} \quad (7c)$$

Equation of rotational equilibrium (taking moment about CM)

$$J\ddot{\theta} + K_x(x + e\theta)e + K_t\theta = 0 \quad (8a)$$

$$Mr^2\ddot{\theta} + K_x\left[(x + e\theta)e + \frac{K_t}{K_x}\theta\right] = 0 \quad (8b)$$

Divide both sides of equation by *M* and r^2 and given that $\omega_x^2 = \frac{K_x}{M}$:

$$\ddot{\theta}+\omega_x^2\left\{\left[\frac{x}{r}+\frac{e}{r}\theta\right]\frac{e}{r}+\frac{b^2}{r^2}\theta\right\}=0 \quad (8c)$$

Given that $b_r=\frac{b}{e}$:

$$\ddot{\theta}+\omega_x^2\ \{[x_r+e_r\theta]e_r+b_r^2\theta\}=0 \quad (8d)$$

$$\ddot{\theta}+\omega_x^2\ \{(e_r)x_r+(b_r^2+e_r^2)\}\theta=0 \quad (8e)$$

Recall the reduced equations of dynamic equillibrium

$$\begin{bmatrix}1 & 0\\ 0 & 1\end{bmatrix}\begin{Bmatrix}\ddot{x}_r\\ \ddot{\theta}\end{Bmatrix}+\omega_x^2\begin{bmatrix}1 & e_r\\ e_r & b_r^2+e_r^2\end{bmatrix}\begin{Bmatrix}x_r\\ \theta\end{Bmatrix}=\begin{Bmatrix}0\\ 0\end{Bmatrix}; \quad (9a)$$

$$\omega_x^2\begin{bmatrix}1 & e_r\\ e_r & b_r^2+e_r^2\end{bmatrix}\begin{Bmatrix}x_r\\ \theta\end{Bmatrix}-\Omega_j^2\begin{bmatrix}1 & 0\\ 0 & 1\end{bmatrix}\begin{Bmatrix}x_r\\ \theta\end{Bmatrix}=\begin{Bmatrix}0\\ 0\end{Bmatrix} \quad (9b)$$

Where $\Omega_{j=1,2}$ are the natural angular velocities of the coupled modes of vibration (i.e., the *eigenvalues*)

Let $\lambda_j^2=\frac{\Omega_j^2}{\omega_x^2}\Rightarrow\ Det\begin{vmatrix}1-\lambda_j^2 & e_r\\ e_r & (b_r^2+e_r^2)-\lambda_j^2\end{vmatrix}=0\Rightarrow\ (1-\lambda_j^2)\left[(b_r^2+e_r^2)-\lambda_j^2\right]-e_r^2=0$ (9c)

Solution for values of λ_j^2 can be obtained by solving the roots of a quadratic equation.

It can be shown using the elementary expression: $\frac{-b\pm\sqrt{b^2-4ac}}{2a}$ that

$$\lambda_j^2=\frac{1+(b_r^2+e_r^2)}{2}\pm\sqrt{\left[\frac{1-(b_r^2+e_r^2)}{2}\right]^2+e_r^2} \quad (9d)$$

Solution for the eigenvector is accordingly obtained as follows:

$$\begin{bmatrix}1-\lambda_j^2 & e_r\\ e_r & (b_r^2+e_r^2)-\lambda_j^2\end{bmatrix}\begin{Bmatrix}x_r=1\\ \theta\end{Bmatrix}=\begin{Bmatrix}0\\ \theta\end{Bmatrix}\Rightarrow \quad (10a)$$

$$(1-\lambda_j^2)+e_r\theta=0\Rightarrow\theta=\frac{\lambda_j^2-1}{e_r}\ \text{or}\ \frac{\lambda_j^2-1}{e/r} \quad (10b)$$

7. CLOSING REMARKS

Results generated by the dynamic analyses of the single-storey models provided a great deal of insights on how asymmetry, and the eccentricity in particular, affects torsional response behaviour of buildings in seismic conditions. The key observation from results generated from the response spectrum modal analyses is that the edge displacement demand of the building would not increase indefinitely with increasing eccentricity value but would asymptote at 1.6

times the peak displacement limit (assuming no asymmetry in the building). In other words, if the peak displacement limit as indicated by the elastic response spectrum is 100 mm the edge displacement is predicted not to exceed 160 mm. Another observation is that edges on both sides of the building could sustain similar amount of distress which is contrary to what a static analysis would show. Both observations may appear counter intuitive to engineers whose mindset have been fixed upon their experience in seeing results from static analyses. The reason of presenting the rocking behaviour of free-standing objects (Figure 1) is to proof the point that static analysis results do mis-represent real behaviour which is reinforced by comparison of the trends of edge displacement ratios as obtained from dynamic analyses (Figure 6) and static analyses (Figure 7).

8. ACKNOWLEDGEMENTS

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