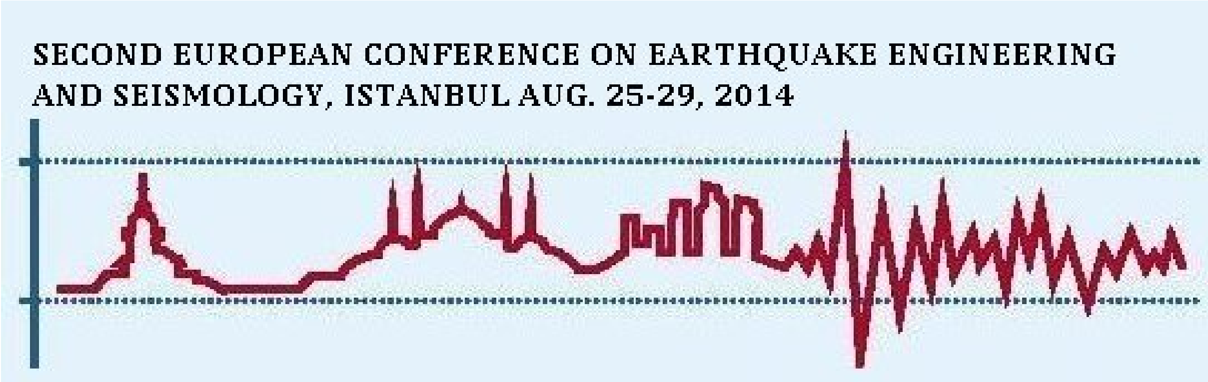

MINIMIZING THE SEISMIC RESPONSE OF SETBACK ASYMMETRIC BUILDINGS UNDER STRONG GROUND EXCITATIONS

G. Georgoussis[1)] *, A. Tsompanos*[2)] *and T. Makarios*[3)]

ABSTRACT

Studied in this paper is the inelastic rotational behaviour of asymmetric multi story building structures, irregular in elevation, under strong ground excitations in relation to the location of the modal centre of rigidity (m-CR). The concept of this point in mono-symmetric uniform over the height buildings has been outlined by the first of the authors in earlier papers, and its significance lies on the property that when its location is within a close distance from the axis passing through the centres of floor masses (mass axis) the torsional response of elastic systems is mitigated. This practically translational behaviour is preserved into the inelastic phase, when the lateral load resisting bents are detailed as planar structures under a code load. This is attributed to their concurrent yielding in the case of a ground motion. At present, a generalized definition of this point is given for buildings with large setbacks, which are classified as irregular in elevation buildings. The proposed procedure, for assessing the rotational behaviour of elastic systems, retains the simplicity of the methodology applied to uniform structures, and it is demonstrated that when the mass axis is passing through m-CR and the strength assignment of all lateral load resisting bents is stiffness proportional, the response of a such structural configuration is practically translational when the lateral load resisting bents are stressed beyond their elastic limits. The accuracy of the proposed procedure is first illustrated in mixed-bent-type eight-story elastic setback buildings, which are characterized by Eurocode (EC8-2004) as irregular in elevation structures, and comparisons are made with the accurate results obtained from the SAP2000 computer program. The inelastic response of these structures, when the strength of various bents is determined by a planar static analysis under a code lateral loading, is investigated under the Loma Prieta (1989) and Imperial Valley (1940) ground motions.

INTRODUCTION

It has been shown that asymmetric multistory buildings, uniform over the height, having resisting bents with stiffness matrices which are proportional to each other (proportionate buildings) and the centers of floor masses on the same vertical line, can be analyzed (i) by determining the response of the corresponding uncoupled multi-story structure and, (ii) for each mode of vibration of the latter structure, by analyzing an equivalent torsionally coupled single story system (Kan and Chopra 1977a, Hejal and Chopra 1989, Athanatopoulou et al 2006). This analysis, applicable also to shear type buildings with different static eccentricities at the various floor levels (Kan and Chopra 1977b), was

[1)] Professor, School of Pedagogical and Technological Education (ASPETE), Athens, Greece, ggeorgo@tee.gr
[2)] Scientific Collaborator, School of Pedagogical and Technological Education (ASPETE), Athens, Greece
[3)] Lecturer, University of Thessaloniki, Thessaloniki, Greece, makariostr@civil.auth.gr

extended by the first of the authors (Georgoussis 2009, 2010, 2012) to non- proportionate buildings, by introducing the concept of the modal center of stiffness (m-CR). This is the center of the element modal stiffnesses of an equivalent single story modal system and may be seen as its 'equivalent center of rigidity'. For medium height structures however, the response of which during a ground motion is basically dependent on the first mode displacements, it is adequate to restrict all the subsequent calculations on the first mode dynamic data. Therefore, the location of m-CR may simply be determined from the first mode element frequencies of the bent-subsystems that provide the lateral resistance of a given structure and the aforesaid methodology, which is based on the grounds of the properties of proportionate building, may also be applied to elastic mixed-bent-type eccentric structures for assessing basic dynamic data. These element frequencies are determined from the corresponding individual bents when they are assumed to have, as planar frames, the mass of the complete structure. In recent papers, for the case of structures composed by very dissimilar bents, a higher accuracy of the aforementioned methodology can be attained with the use of the effective element frequencies, which are based on the element frequencies, but, also, take into account the ratio of the effective modal mass of the individual bent to the corresponding mass of the uncoupled multistory system (Georgoussis 2013a, 2014).

The main property of the centre of the effective element stiffnesses, which defines m-CR, is that when it lies on the mass axis, the response of uniform building structures, asymmetric in plan, is basically translational not only into the elastic phase, but also when the structure is stressed beyond the elastic limits, provided that the strength assignment of its resisting bents is stiffness proportional. In other words, this response is obtained when the building is detailed as a planar structure under a code load (Georgoussis et al (2013b) and Georgoussis (2014)). This is attributed to the almost concurrent yielding of all resisting elements, which preserves the translational response, attained at the end of the elastic phase, to the post elastic phase. This response is evident in eccentric single story systems. Reviewing the literature, it can be seen that systems, with coincident the centres of mass and rigidity and elasto-plastic elements having a strength distribution proportional to the stiffness distribution (usually called torsionally balanced (TB) models) present a purely translational inelastic response under strong ground excitations. For this reason they are used as 'reference' models in relevant studies (e.g. Correnza et al, 1994; Chandler et al, 1996; Wong and Tso 1994). This behaviour is attained because yielding is initiated at the same instant for all elements and the element force balance about CM is preserved into the inelastic phase, leading to a translational response throughout the ground shaking (recent qualitative overviews have been presented by De Stefano and Pintucchi, 2008 and by Anagnostopoulos et al, 2013). The problem of controlling the rotational response of multistory structures has also been investigated in the past. For example, Aziminejad et al (2008) and Aziminejad and Moghadam (2009) in their studies examined the problem of element strength distribution on the rotational response of multistory buildings by using a proper configuration of the centers of mass, strength and stiffness according to the findings obtained from single story systems with elements having strength dependant stiffness (Myslimaj and Tso, 2002, 2004).

As it has been shown (Georgoussis et al (2013b) and Georgoussis (2014)) that in uniform multi-story buildings, in which the mass axis passes through m-CR, their response into the inelastic phase is basically translational when the strength assignment of the various bents is based on a planar static analysis under a set of lateral forces simulating a 'seismic loading', this property of m-CR can be used by the practicing engineer as guidelines to form a structural configuration which will sustain minimum rotational response, simply by allocating the resisting elements in such a way that this point lies close to the mass axis.

The aim of this study is to present data that demonstrate that the same guidelines can also be applied to asymmetric multistory buildings with an abrupt mass discontinuity, which are classified by Eurocode 8 as irregular structures. In such cases, Eurocode 8 and other modern country codes specify a full 3-dimensional dynamic analysis, even for low height structures. There are not recommendations of how the practicing engineer can assess the fundamental frequency by a simple formula or methodology and there are not provisions which allow the structural detailing by a pseudo-static structural design against an equivalent lateral load. Only in the case of buildings with a fairly even distribution of mass (regular buildings) the codes provide simple expressions for calculating the fundamental frequency and allow for a pseudo-static structural design. The main request however from the structural engineer, particularly at the preliminary stage of design, is to have a tool (a simple methodology) of forming a plan configuration of minimum torsional response in the case of strong ground motions. It is demonstrated that such a structural design can be attained by a pseudo-static analysis, provided that a frequency assessment can be obtained with accuracy (to define accurately the base shear from the acceleration design spectrum) and the plan configuration provides the m-CR point close to the mass axis.

The methodology to assess the first four frequencies and the location of m-CR in structural models composed by dissimilar bents and having a mass discontinuity at mid-height is demonstrated in typical 8-story buildings. The response of these models, detailed as planar systems under a code lateral loading, is examined under two characteristic ground motions (Loma Prieta (1989) and Imperial Valley (1940), selected from the strong ground motion database of the Pacific Earthquake Engineering Research (PEER) Center (hppt://peer.berkely.edu) and scaled to a PGA=0.5g. The results obtained by the accurate SAP2000 nonlinear computer program clearly reveal the virtually translational response of the systems in which the mass axis is passing close to m-CR.

DESCRIPTION OF METHODOLOGY - TORSIONALLY BALANCED SETBACK BUILDINGS

Consider a mono-symmetric multistory building with a setback as shown in Fig. 1. The building is uniform over the height H_b, which defines the base structure and has a uniformly distributed mass, equal to m_b per floor, and a radius of gyration equal to r_b. Above this level, it has a setback forming a uniform tower structure of a reduced floor plan with a height equal to H_t, a mass per floor equal to m_t and a radius of gyration equal to r_t. Each floor consists of a rigid slab (deck) and at present the centers of mass (CM) at each floor are assumed to lie on the same vertical line (CM axis) which is passing through the centroids of all decks. The effect of stiffness discontinuity is not examined in this paper and the structural system that provides the lateral stiffness against horizontal loads is assumed to consist of different types of bents (rigid frames, shear walls, coupled wall systems) which extend up to the top of the building. Any columns that are curtailed at the height H_b are assumed to support only the gravity loads of the increased floor size of the base structure without any contribution to the lateral resistance of the buildings.

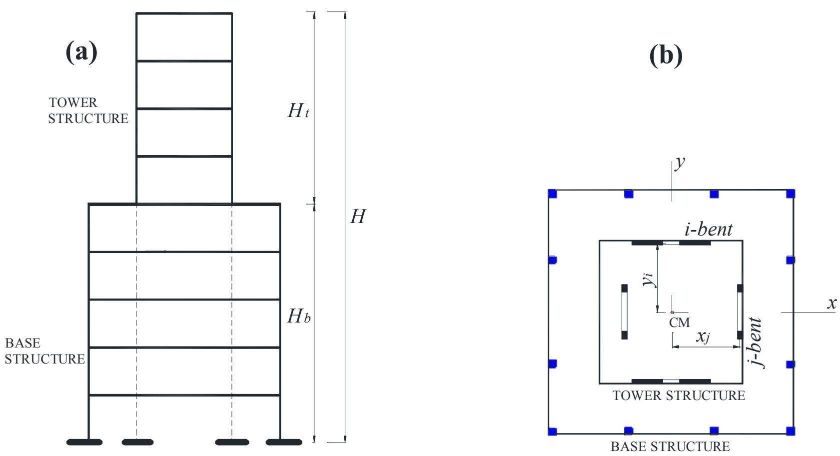

Figure 1. (a) Multistory setback building with (b) an asymmetric structural configuration.

The methodology to analyze elastic setback buildings like that of Fig.1 is outlined in an earlier paper (Georgoussis 2011). The backbone of this method is similar to that applied to uniform over the height systems (Georgoussis 2009, 2010, 2012). In brief, the peak elastic response of medium height buildings can be derived by analyzing two equivalent single-story modal systems, each of which has a mass equal to the k-mode effective mass, M_k^* (k=1,2), of the uncoupled multi-story structure, and is supported by elements with a stiffness equal to the product of M_k^* with the first mode (when k=1) or second mode (when k=2) squared frequencies of the corresponding real bents of the assumed multi-story structure. For an excitation along the y-direction M_k^* can be taken equal to that corresponding to same direction (M_{yk}^*) and at present, because the example models described further below are composed by dissimilar bents , the effective element frequencies are used to determine the stiffnesses of elements of the equivalent single-story systems, as shown below. The undamped equation of motion of the aforementioned k-mode equivalent single story system, in a coordinate system with the origin at the center of mass (Fig. 1(b)), is as follows:

$$\mathbf{M}_k^*\ddot{\mathbf{U}}_k + \mathbf{K}_k^*\mathbf{U}_k = -\mathbf{M}_k^*\boldsymbol{\iota}\ddot{u}_g \tag{1}$$

where

$\mathbf{M}_k^* = M_{yk}^*\begin{bmatrix} 1 & 0 \\ 0 & r_{ek}^2 \end{bmatrix}$ is the effective k-mode mass matrix,

$\mathbf{U}_k = \langle u_k \quad \theta_k \rangle^{\mathrm{T}}$ is the corresponding modal displacement vector at CM

$\mathbf{K}_k^* = \begin{bmatrix} k_y^* & k_{yw}^* \\ k_{wy}^* & k_w^* \end{bmatrix}_k$ is the effective k-mode stiffness matrix

$\boldsymbol{\iota}^{\mathrm{T}} = \langle 1 \quad 0 \rangle^{\mathrm{T}}$ is the influence vector, and (2)

$$k_{yk}^* = \Sigma k_{jk}^* = M_{yk}^*\Sigma\overline{\omega}_{jk}^2$$

$$k_{wk}^* = \Sigma x_j^2 k_{jk}^* + \Sigma y_i^2 k_{ik}^* = M_{yk}^*\Sigma(x_j^2\overline{\omega}_{jk}^2 + y_i^2\overline{\omega}_{ik}^2)$$

$$k_{ywk}^* = k_{wyk}^* = \Sigma x_j k_{jk}^* = M_{yk}^*\Sigma x_j\overline{\omega}_{jk}^2$$

$$\overline{\omega}_{jk}^2 = \omega_{jk}^2\frac{M_{jk}^*}{M_{yk}^*} \qquad \overline{\omega}_{ik}^2 = \omega_{ik}^2\frac{M_{ik}^*}{M_{xk}^*}$$

The quantities $\overline{\omega}_{jk}$ and $\overline{\omega}_{ik}$ are the effective element frequencies of the j and i-bents, aligned along the y- and x-directions at distances x_j and y_i respectively. It is evident that when the lateral stiffness of a given building is composed by the same type of bents (e.g. flexural shear walls), they are respectively equal to the element frequencies ω_{jk} and ω_{ik}. As shown in Georgoussis (2011), the radius of gyration of the equivalent single story system r_{ek} may be given as $\bar{r}_{ek}r_b$, where $\bar{r}_{ek}$ represents a ratio of Rayleigh's quotients, which can be approximated as

$$\bar{r}_{ek} = \frac{\omega_{yk}}{\omega_{ryk}} \quad (\text{or } \bar{r}_{ek} = \frac{\omega_{xk}}{\omega_{rxk}}) \tag{3}$$

where ω_{yk} (or ω_{xk}) is the k-mode frequency of the uncoupled multistory structure in the y-direction (or x-direction) and ω_{ryk} (or ω_{rxk}) the corresponding frequency of the same structure when the mass in the floors of the tower section is reduced to $m_{rt} = (r_t / r_b)^2 m_t$. It has been shown (Georgoussis, 2011) that in common setback buildings, the ratio $\bar{r}_{ek}$ is very little dependent on type of the lateral load resisting system (frame, wall, dual system). Therefore, any of the expressions of Eq. (3) may be used for practical applications, but it is advisable to use the mean value of these expressions, since this averaging procedure utilizes the response of the structure in both directions.

Note here that the coupled Eq. (1), for the first mode (k=1) single-story system, provides the response quantities of the first two modes of vibration. Therefore, when the plan configuration produces an uncoupled stiffness matrix in Eq. (1), the first two modes of vibration (translational and rotational) are decoupled and the response for a low height building will be practically translational. In fact, this condition specifies that the first mode center of rigidity (m-CR) of the corresponding single-story system coincides with CM. As the x-coordinate of m-CR can be determined from the condition: $k^*_{ywk}\,(=k^*_{wyk})=0$, that is:

$$x_{m-CR}=\frac{\Sigma(x_j\overline{\omega}^2_{j1})}{\Sigma(\overline{\omega}^2_{j1})} \tag{4}$$

medium or low height structures (where the first two modes of vibration virtually determine their response) in which the location of m-CR coincides with CM ($x_{m\text{-}CR}$=0), are expected to sustain a practically translational response and such structural configurations may be seen as torsionally balanced systems. The main objective of the paper is to demonstrate that these balanced setback buildings retain this translational response into the post elastic phase when they are detailed as planar structures under a code horizontal load.

STUDIED SYSTEMS

To illustrate the application and accuracy of the proposed method, the setback model structure shown in Figure 2 was analyzed. This is an 8-story monosymmetric building, which consists of a 4-story base structure with a floor plan of 22x15m and a top structure with an equal number of stories and a floor plan of 15x10m. The lateral load resisting system, extending up to the top of the building, is composed by dissimilar bents: two structural walls (Wa and Wb) and a moment resisting frame (FR) are aligned along the y-direction and a pair of coupled-wall bents (CW) is oriented along the x-axis of symmetry. The structural walls Wa and Wb are of cross sections 30x500cm, the moment resisting frame FR consists of two 75x75cm columns, 5m apart, connected by beams of a cross section 40x70cm while the CW bents are composed two 30x300cm walls, 5m apart, connected by lintel beams of a cross section 25x90cm at the floor levels. The latter bents are located symmetrically to CM at the edges of the floors of the tower structure, that is at distances equal to $\pm 5\text{m}$ .The mass of the base floors is m_b=264kNs^2/m, the radius of gyration about CM is r_b= 7.687m and the corresponding quantities of the tower structure are equal to m_t=120kNs^2/m and r_t= 5.204m respectively. The story height is 3.5m and the modulus of elasticity (E) is assumed equal to 20x10^6 kN/m^2, typical for concrete structures. The centers of mass of the floor slabs lie on a same vertical line, which passes through the centroids of all the orthogonal floor plans of the example structure.

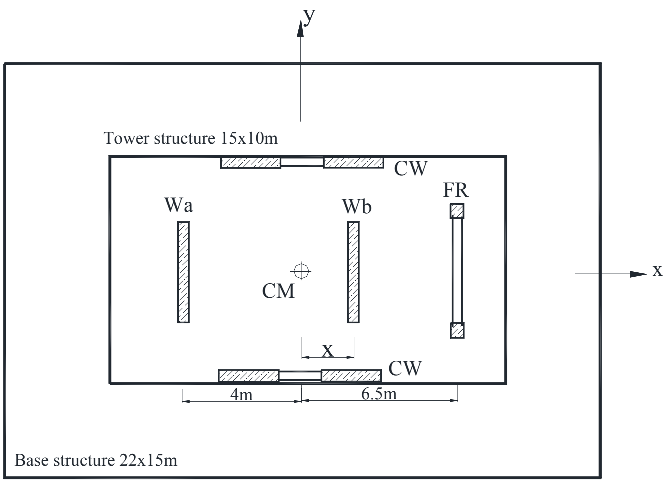

Figure 2. Example setback building.

To investigate the accuracy of the proposed method to a broader range of building structures, different structural configurations of the example structure are examined as follows: wall Wa and frame FR are assumed to be located at a fixed positions, the first on the left of CM in a distance equal to 4m and the second on the right of CM at a distance of 6.5m, while the second wall Wb is taking all the possible locations along the x-axis within the limits of the tower section.

At first the periods/frequencies of the assumed models, for all possible locations of Wb, are examined. The accuracy of the proposed approximate procedure to predict periods of vibrations is investigated by comparison with the results derived from the computer program SAP2000-V11. In the computer analyses, the out of plane stiffness of the bents was neglected and in the wide column analogy used to simulate the CW bents the clear span of the coupling beams was increased by the depth of the beams (Coull and Puri, 1968). To apply the proposed method, the first pair of frequencies of the various bent-subsystems is required, and also their effective modal masses. Denoting with M the total mass of the structure (M=4m_b+4m_t=1536kNs^2/m), these quantities for the bents of the assume structure were found by means of the SAP2000 program as follows:

Walls Wa and Wb: ω_{w1}=5.092/s, ω_{w2}=23.994/s and $\overline{M}^*_{wa1} = M^*_{w1}/M = 0.577$, $\overline{M}^*_{w2} = 0.275$.

Frame FR: ω_{f1}=3.033/s, ω_{f2}=8.483/s and $\overline{M}^*_{f1} = 0.744$, $\overline{M}^*_{f2} = 0.138$

Coupled walls CW: ω_{cw1}=5.372/s, ω_{cw2}=19.592/s and $\overline{M}^*_{cw1} = 0.625$, $\overline{M}^*_{cw2} = 0.220$.

The first two effective modal masses of the uncoupled structure, in the y-direction, normalized with respect to the total mass, are respectively equal to $\overline{M}^*_{y1} = 0.588$ and $\overline{M}^*_{y2} = 0.265$. From these data, the first two effective element frequencies of all the bents (as defined in last line of equations (2)) can be determined. For walls Wa and Wb: $\overline{\omega}_{w1} = 5.044/s$ and $\overline{\omega}_{w2} = 24.442/s$. For frame FR: $\overline{\omega}_{f1} = 3.412/s$ and $\overline{\omega}_{f2} = 6.122/s$, and for the coupled walls CW the effective frequencies are equal to the element frequencies as above. The radius of gyration of the equivalent single story system r_{ek} (k=1, 2), computed as described in the previous section, was found equal to r_{e1}=0.742*r_b=5.704m and r_{e2}=0.871*r_b=6.695m for the first and second mode equivalent single story systems respectively.

The inelastic response of the assumed model structures was investigated under two characteristic ground motions (Loma Prieta (1989) and Imperial Valley (1940)), selected from the strong ground motion database of the Pacific Earthquake Engineering Research (PEER) Center (hppt://peer.berkely.edu) and scaled to a PGA=0.5g (unidirectional excitations along the y-axis). For all the possible locations of Wb, inelastic analyses, by means of the computer program SAP2000-V11, were performed to evaluate top rotations and base shears and torques. The strength assignment of all bents is based on planar static analyses, along the x and y directions, under an external lateral loading with the floor forces determined from Equation (4.11) of Eurocode 8 and summing to a base (design) shear equal to V_d=3072kN (approximately equal to 20% of the total weight of the structure). The aforesaid equation simply approximates the fundamental mode shape with a linear displacement profile. More specifically, allowing for plastic hinges at the bases of walls Wa and Wb and detailing frame FR according to the strong column-weak beam philosophy (that is, allowing plastic hinges at the ends of the beams and at the foot of the ground floor columns), this static analysis leads to the following results: (i) the bending (yield) capacity at the plastic hinges at the base of walls Wa and Wb is equal to 22770kNm and, (ii) the bending (yield) capacity of the plastic hinges at the ends of the beams of FR (from the top downwards) is equal to 456, 575, 562, 560, 536, 483, 386, and 244kNm respectively, while the corresponding capacity at the plastic hinges at the base of the ground columns of FR equals 301kNm. Similar is the strength detailing of the coupled wall bents CW, where the bending (yield) capacity of the plastic hinges of the beams at the interface of wall (from the top downwards) is equal to 554, 593, 644, 689, 711, 685, 579, and 362kNm respectively, and the corresponding capacity at the plastic hinges at the base of the walls equals 14717kNm. All the nonlinear response history analyses were performed by means of the program SAP2000-V11, using inelastic link elements at the assumed locations of plastic hinges. The moment-rotation relationships of these elements were assumed bilinear with a post-yielding stiffness ratio of the generalized load-deformation curve, equal to 4%. The aforesaid analyses were performed using the numerical implicit Wilson-θ time integration method, with the parameter θ taken equal to 1.4.

OBSERVED NONLINEAR SEISMIC RESPONSE

The first four periods of vibration of the example structures of Fig. 2, computed by the proposed approximate method (green lines) for different locations of the Wb (indicated by the normalized coordinate $\bar{x} = x/r_b$), are shown in Fig. 3, together with the accurate computer values (black lines). For the first (thick solid lines), the second (thick dotted lines) and the fourth (thin dotted lines) mode of vibration, there is a close agreement between the two sets of periods. For the third mode of vibration (thin solid lines) the proposed procedure is gradually overestimating the accurate computer data as the wall Wb moves to the left of CM with a deviation reaching the value of 20%. It is worth noting here that the first pair of the approximate periods (first and second) is derived from the first mode (k=1) equivalent single story system, while the second pair of the approximate periods (third and fourth) is obtained from the second mode (k=2) equivalent single story system.

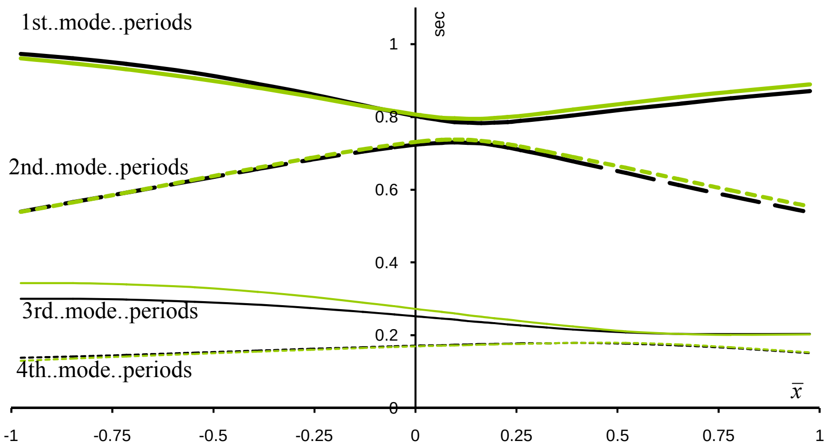

Figure 3. Periods of vibration of the example structural configurations.

The response of the inelastic setback structures, as described in the previous section, under the Loma Prieta (component Corralitos 000, 1989) and Imperial Valley (component ElC180, 1940) excitations, are shown in Fig. 4. In order to compare elastic and inelastic behaviors, the elastic responses of the assumed models under the same excitations are also presented in this figure. Three response parameters, obtained by time history analyses assuming a 5% damping ratio, are shown: top rotations, θ, normalized base shears and normalized base torques. The red lines represent the peak elastic response (top rotations: θ_e, are shown by dashed lines, normalized base shears: $\overline{V}_e = V_e/V_d$ by solid lines and normalized base torques: $\overline{T}_e = T_e/r_b V_d$ by dotted lines) and the corresponding black lines represent the peak inelastic behavior (θ_{in}, $\overline{V}_{in} = V_{in}/V_d$, $\overline{T}_{in} = T_{in}/r_b V_d$). In general terms the response shown in the aforementioned figure is similar to that demonstrated in uniform building systems (Georgoussis et al 2013b, Georgoussis 2014). Further than that, it may be seen that the response of the inelastic systems is smoother and the overall rotational behavior is smaller than that obtained by the elastic behavior. This finding confirms earlier observations on single story systems that after yielding asymmetric systems have the tendency to deform further in a translational mode (e.g. Kan and Chopra, 1981; Ghersi and Rossi, 2001).

Minimum elastic top rotational response is obtained when the moving Wb wall is located at the normalized coordinate $\bar{x} = 0.163$, while for the inelastic systems such a response is observed when Wb is taking the same location for the case of Loma excitation, and the slightly increased coordinate of $\bar{x} = 0.195$ for the case of Imperial excitation. Envisaging the diagrams of normalized torques it can be seen that their minimum values are also observed at the same locations of wall Wb. Note here that according to Eq. (4) minimum torsional response is expected when the moving wall Wb is positioned at $\bar{x} = 0.133$. The actual distance between the theoretical and computer derived locations is less than 0.23m. At such locations of Wb, the mass axis is very close to the first mode center of rigidity, implying that the elastic response of the system along the y-direction is virtually translational. As the strength distribution has been determined by a planar static analysis, this response results in an almost in-phase yielding of the bents aligned in the y-direction, leading to a minimum rotational response in the post elastic phase of response. It is worth reminding here a conclusion derived by Lucchini et al (2009) on the behavior of single story buildings: their nonlinear response depends on how the building enters the nonlinear range, which in turn depends on its elastic properties (i.e. the stiffness and mass distributions), and on the capacities of its resisting elements (i.e. the strength distribution).

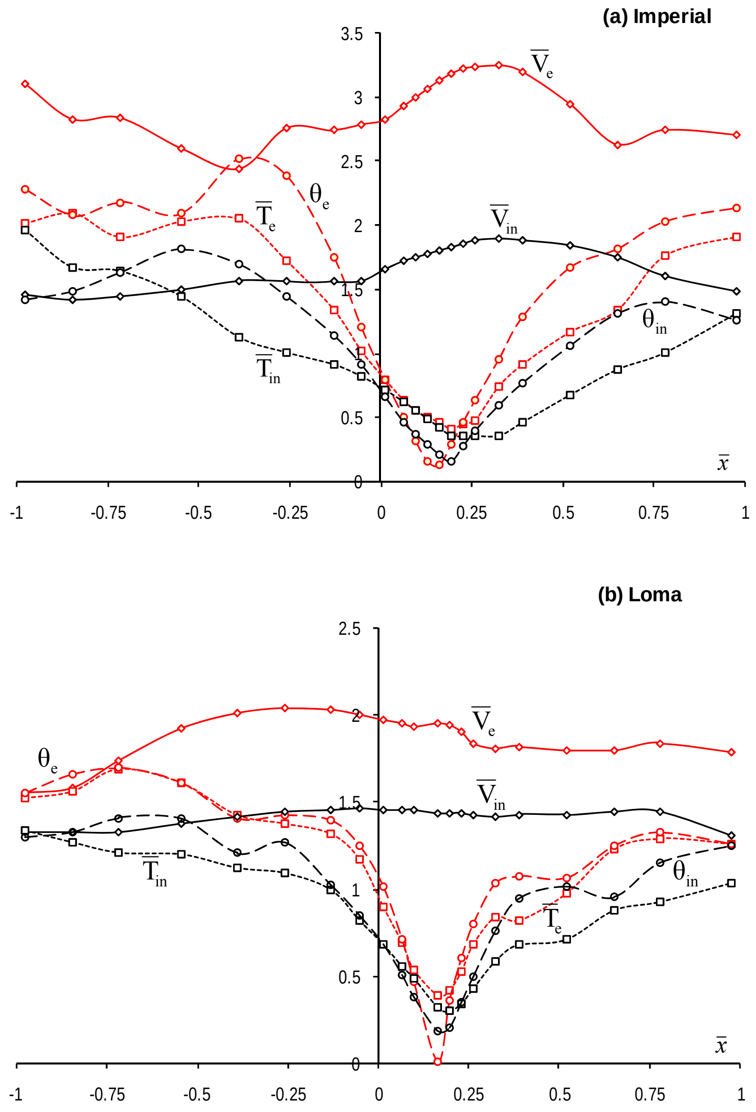

Figure 4. Top rotations (x10^{-2}, rads) and normalized base shears and torques.

CONCLUSIONS

Frequencies of eccentric, medium height setback buildings, composed by dissimilar bents, can be estimated with reasonable accuracy from the analysis of two equivalent, single-story modal systems, the masses of which are determined from the first two vibration modes of the uncoupled multi-story structure and the stiffnesses of the resisting elements are determined from the corresponding individual bents when they are assumed to have, as planar frames, the mass of the complete structure.

This simple analysis also provides with reasonable accuracy the location of the first mode center of rigidity. The main property of this point is that when it lies on the mass axis, the response of elastic building structures is basically translational. This behavior is preserved in the inelastic phase, when the strength assignment of the lateral load resisting bents is derived from a planar static analysis, as a consequence of the almost concurrent yielding of these bents. This is demonstrated in common 8-story setback buildings, which are classified by Eurocode 8 as irregular in elevation structures, under two characteristic ground motions. Therefore, as it is quite easy to determine the modal center with simple hand calculations, the proposed procedure can be used as a guideline to determine the optimum structural arrangement in terms of minimum torsional response. This procedure, in the preliminary stage of a structural application, can be implemented in two steps: First, by calculating a safe estimate of the frequencies of the building (using the aforementioned simplified analysis), in order to evaluate accurately the base shear from the design acceleration spectrum and, second, by detailing the building under a code lateral loading, but taking care to have the modal center of rigidity as close as possible to the mass axis.

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