RESPONSE SPECTRA OF RC ROCKING STRUCTURES ON INELASTIC BASE FLOOR SUPPORT

C. A. Zeris[1], P. Giannitsas[2] and K. Alexandropoulos[2]

[1] *Assistant Professor, School of Civil Engineering , National Technical University of Athens, Greece, Zografou, GR 1577. Email: zeris@central.ntua.gr*

[2] *Graduate Civil Engineer, School of Civil Engineering , National Technical University of Athens, Greece, Zografou, GR 1577*

ABSTRACT :
Single degree of freedom inelastic response spectra are re-evaluated for a class of simplified reinforced concrete (RC) building oscillator models that exhibit both rocking and lateral response and behave as rigid blocks above the ground story; rocking and lateral deformation at the ground level are resisted through RC columns with axial – rocking resistance interaction. This type of building oscillators are encountered in a wide variety of irregular RC building systems, either intentionally discontinuous (open ground floor for shops and parking) or, unintentionally discontinuous, due to the inclusion of stiff infill panels above the base floor. The inelastic response spectra of the simplified rocking models are evaluated using code OpenSees (developed at PEER) and are compared to lateral only inelastic spectra as traditionally assumed in design. Comparisons of lateral ductility and inelastic demand at the base columns are presented, as a function of the system's parameters, namely: natural periods T_L, T_θ and normalized resistances η_L, η_θ (yield strength normalized by the corresponding peak inertial force). It is shown that for a certain period range, the presence of rocking, coupled with the inelastic lateral response, increases both the transverse ductility demand as well as the axial resisting forces at the ground floor, compared to those obtained from conventional elastic analysis procedures.

KEYWORDS:
Rocking, Inelastic Spectra, Reinforced Concrete, Seismic Design, Soft Storey, Ductility.

1. INTRODUCTION AND STATEMENT OF THE PROBLEM

According to current design practice, simplified inelastic response spectra of equivalent single degree of freedom oscillators of the building are adopted for the prediction of the seismic forces and deformations of both new and existing reinforced concrete (RC) buildings. Following this approach, buildings are modeled as simple oscillators deforming laterally, over a wide range of natural periods and critical damping ratios, characteristic of structural response. For new buildings, the equivalent spectral response predictions at the global response level

(a)  (b) 

Figure 1. Overturning ground floor failure: (a) the Imperial County Services Building (Zeris, 1986), intentional soft storey; and (b) Office building with perimeter infilled frames, Mexico City earthquake, 1985.

are scaled down, making use of specified reduction factors that take into account the inherent ductility of the structure, to establish design forces. For existing building redesign, reduction factor versus ductility relations are adopted over the entire period range of expected inelastic response, in order to predict the target deformation for the particular design spectrum.

Following the post failure analysis of the Imperial County Services Building (Fig.1a) that suffered significant damages during the 1979 Imperial Valley earthquake (Zeris et al., 1984), it was shown that for certain classes of framed structures like this structure, that exhibit a relatively rigid superstructure on a relatively soft ground storey, the use of the usual simplified equivalent representation approach of lateral response will lead to a gross underestimation of the failure mode and the seismic forces and deformations, both at the global as well as the local level (Zeris, 1986) due to the effect of the rocking superstructure on the soft ground storey. Although this particular six story building was designed and detailed according to current seismic design procedures for ductile response, it suffered explosive failure of the ground columns under the rigid superstructure followed by partial collapse of the end bay; being heavily instrumented complete records of the response were obtained, thereby providing a full scale test of the seismic performance of this form of soft storey RC structures.

Soft story RC buildings such as this structure (also called pilotis frames) are widespread in existing RC frame construction worldwide, developed mainly in the 70s both for functional as well as seismic performance reasons and suffering damages since then, particularly in near field earthquakes (Mahin et al., 1976, Sezen et al., 2000, Karakostas et al., 2005). Buildings with soft storey characteristics also appear in modern RC construction, although current design norms prescribe more stringent rules for the reduction factors and the detailing of the soft storey. Even ordinary RC frames however, without such an intentional stiffness irregularity as an intentional soft storey at the base may also be sensitive to coupled rocking - lateral response due to the presence of perimeter non structural masonry infills, often partially discontinued at the ground for functional reasons, thereby introducing an irregularity with height which is usually ignored in the design and the analysis modeling process. Furthermore, the fact that infilled structures are designed following elastic design procedures relying on seismic force patterns representative of the bare frame structure only makes their inelastic response even more susceptible to such unanticipated effects as the case study above (Fig. 1b).

(a)

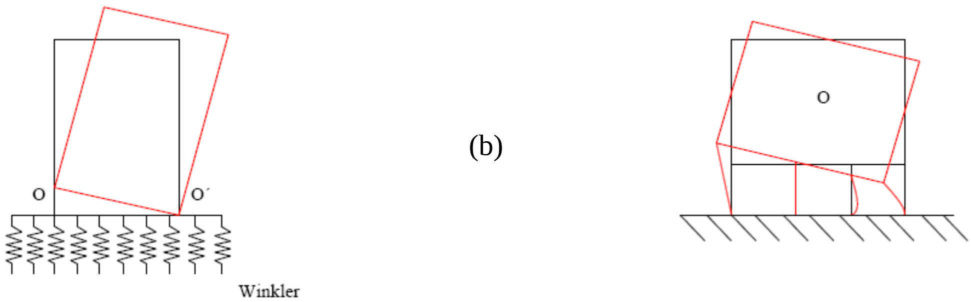


Figure 2. Rocking superstructure models under base excitation: (a) Rigid block on a Winkler foundation and (b) Structural rigid block on a soft ground story.

The problem of structural rocking has received considerable attention mainly in the response estimation of rigid structural blocks resting on rigid, elastic or inelastic foundation with or without the ability to uplift and/or slide (Fig.2.a), since the early work of Housner (1963), who investigated the overturning stability of rigid blocks. Psycharis et al. (1983) developed a mathematical representation of the problem modeling a tensionless foundation with equivalent springs. Asla et al. (1980) provided both a numerical as well as an experimental investigation of rocking systems with uplift, quantifying the physical parameters of the problem. Yim et al. (1980) analyzed stochastically the rocking block problem on a tensionless rigid base while Yim et al. (1984) developed inelastic response spectra of the rigid oscillator on a tensionless Winkler foundation. More recent numerical studies by Makris et al. (2003) have shown that the conventional equivalent single degree of freedom oscillator approach for rocking system design is unconservative, developing the mathematical formulation of a rocking spectrum and providing stability limits for rocking rigid oscillators allowed to overturn and (de)stabilize under their own weight. Overall, studies of this particular class of building oscillators has shown the strong influence of the rocking component in the short period range, depending on the input frequency content, the aspect ratio of the block (namely its height to width ratio), yet a potentially beneficial role of the foundation uplift (as long as stability is maintained) in reducing the global deformations, compared to the case fully fixed to the foundation. Unlike the above structural model, the building specific problem of coupled inelastic lateral rocking response of a rigid superstructure on soft ground storey (Fig. 2b) has received less attention so far, even though the inadequate response of such soft story rocking structures has often been observed in past and recent near field earthquakes. Due to the fact that such types of structures exist worldwide in the vicinity of active faults, and are sensitive to the coupled response either intentionally or unintentionally, a more detailed parametric study of their inelastic behavior was deemed necessary, as described herein.

2. DESCRIPTION OF THE MODEL OSCILLATOR

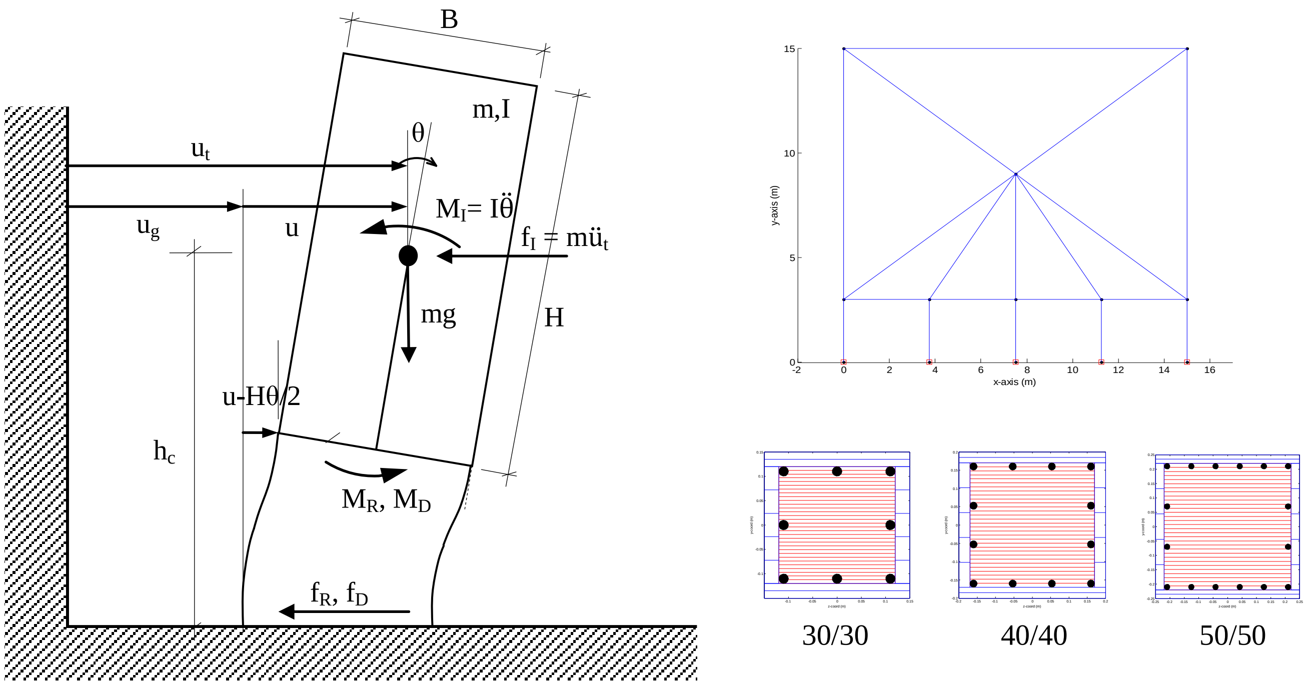


Figure 3. Model of the inelastic rocking oscillator considered: Basic model parameters and numerical implementation with typical column support configuration (dimensions in cm).

2.1 Derivation of the response parameters

In order to perform a parametric study of the rocking oscillator on a soft storey (Fig.3), the two coupled equations of dynamic equilibrium governing the horizontal and rocking response, are presented in non-dimensional form. For small rotations, the equilibrium of the oscillator of Fig.2 in the lateral direction under base excitation $\ddot{u}_g(t)$ is obtained by equating the internal and external forces (inertial, damping and resisting):

$$m \cdot \ddot{u}(t) + 2\xi m\omega_L \cdot \dot{u}(t) + k_L \cdot (u(t) - H \cdot \theta(t)/2) = -m \cdot \ddot{u}_g(t) \ , \quad \omega_L = \sqrt{k_L/m} = 2\pi/T_L \qquad (2.1)$$

where u(t) is the lateral response at the center of mass of the rigid block relative to ground, m is the mass of the rigid superstructure, ω_L is the initial elastic angular velocity of the system, equal to $2\pi/T_L$ the initial linear elastic natural period in lateral response, ξ is the fraction of critical damping, assumed to be 5% for concrete, k_L is the lateral stiffness of all resisting columns at the ground story and, following elastic response, the resisting term $k_L \cdot (u - H \cdot \theta/2) = F_R(t)$ is equal to the base shear. By replacing in Eqn. 2.1 the latter term with a more general nonlinear function of the resistance $F_R(u,\theta,t)$ as a function of the base story lateral deformation with time, Eqn. 2.1 is extended to the more general equation of motion of the nonlinear oscillator. Following Bertero et al.

(1978), Eqn. 2.1 is normalized on both sides dividing by $m \cdot u_y$, where u_y is the yield deformation, a system parameter defined from the lateral inelastic constitutive equation of the base storey as $u_y = R_y/k$, where R_y the lateral yield strength at the base. Expressing the base excitation history $\ddot{u}_g(t)$ as the peak ground acceleration $\ddot{u}_g^{max}$ times a normalized ground motion time history g(t), Eqn. 2.2 below is rearranged as Eqn. 2.3:

$$\ddot{\mu}_L + 2\xi\omega_L \cdot \dot{\mu}_L + \frac{F_R(u,\theta,t)}{m \cdot R_y/k_L} = -\frac{\ddot{u}_g(t)}{u_y} \quad , \quad F_R(u,\theta,t) = R_y \cdot f_R(u,\theta,t) \quad \Rightarrow \qquad (2.2)$$

$$\ddot{\mu}_L + 2\xi\omega_L \cdot \dot{\mu}_L + \omega_L^2 \cdot f_R(u,\theta,t) = -\frac{\omega_L^2 \cdot g(t)}{\eta_L} \quad , \qquad \eta_L = \frac{R_y}{m \cdot \ddot{u}_{max}^g} \qquad (2.3)$$

where μ_L is the center of mass lateral deformation ductility history ($\mu_L = u(t)/u_y$) and $\ddot{\mu}_L, \dot{\mu}_L$ its corresponding time derivatives, $f_R(u,\theta,t)$ is a non dimensional function of the base shear cyclic hysteretic dependence on the inelastic deformation and rotation with time and η_L the normalized lateral resistance of the oscillator, namely R_y divided by the peak inertia force of a rigid body under the base excitation history $\ddot{u}_g(t)$ (namely, $m \cdot \ddot{u}_g^{max}$).

Neglecting the higher order lateral damping force contributing terms and the gravity effects on the rocking stiffness, the dynamic equilibrium of the rigid wall oscillator under rotational response due to rocking about the center of mass under the base excitation $\ddot{u}_g(t)$ is provided by equating the internal and external moments (inertial, damping and resisting) in Eqn. 2.4 below (see also Psycharis et al, 1983):

$$I \cdot \ddot{\theta}(t) + mh_c \cdot \ddot{u}(t) + 2\xi I\omega_\theta \cdot \dot{\theta}(t) + k_\theta \cdot \theta(t) = -mh_c \cdot \ddot{u}_g(t) \quad , \quad \omega_\theta = \sqrt{k_\theta/I} = 2\pi/T_\theta \qquad (2.4)$$

where I is the moment of inertia of the rigid wall, ω_θ is the initial elastic angular velocity of the system equal to $2\pi/T_\theta$, the linear elastic natural period in rotational response, ξ is the fraction of critical damping (assumed to be the same as laterally), h_c is the elevation of the wall center of gravity relative to the base storey and k_θ is the elastic rocking stiffness provided by the base story columns (sum of axial and flexural terms, subtracting the higher order correction term $m \cdot g \cdot h_c$). Following elastic response, the resisting term $k_\theta \cdot \theta = M_R(t)$ is equal to the base storey resistance to bending. In the general case however of an RC building, $M_R(u,\theta,t)$ is expressed as an inelastic function of the wall rotation and relative lateral deformation with time through the coupling of the axial and flexural terms (both stiffness and resistance) of the supporting columns. After dividing both sides of Eqn. 2.4 by the yield rotation of the oscillator θ_y, namely the rotation inducing tensile yield in the outermost tensile column, including gravity compression, at a corresponding rocking capacity of the system M_y, Eqn. 2.5 and 2.6 are obtained for the coupled lateral - rotational portion of the response:

$$\ddot{\mu}_\theta + \frac{mh_c u_y}{I \cdot \theta_y} \cdot \ddot{\mu}_L + 2\xi\omega_\theta \cdot \dot{\mu}_\theta + \frac{M_R(u,\theta,t)}{I \cdot M_y/k_\theta} = -\frac{mh_c \cdot \ddot{u}_g(t)}{I \cdot \theta_y} \quad , \quad M_R(u,\theta,t) = M_y \cdot f_M(u,\theta,t) \Rightarrow \qquad (2.5)$$

$$\ddot{\mu}_\theta + \frac{mh_c u_y}{I \cdot \theta_y} \cdot \ddot{\mu}_L + 2\xi\omega_\theta \cdot \dot{\mu}_\theta + \omega_\theta^2 \cdot f_M(u,\theta,t) = -\frac{\omega_\theta^2 \cdot g(t)}{\eta_\theta} \quad , \quad \eta_\theta = \frac{M_y}{m \cdot h_c \cdot \ddot{u}_{max}^g} \qquad (2.6)$$

where μ_θ is the rotational ductility ($\mu_\theta = \theta(t)/\theta_y$) and $\ddot{\mu}_\theta, \dot{\mu}_\theta$ its corresponding time derivatives, $f_M(u,\theta,t)$ is a non dimensional function of the base rocking cyclic hysteretic dependence on the inelastic deformation and rotation with time and η_θ is the normalized bending strength of the oscillator, namely M_y divided by the peak inertia moment about the centre of mass of a rigid body under the base excitation history $\ddot{u}_g(t)$ (namely, $m \cdot h_c \cdot \ddot{u}_g^{max}$).

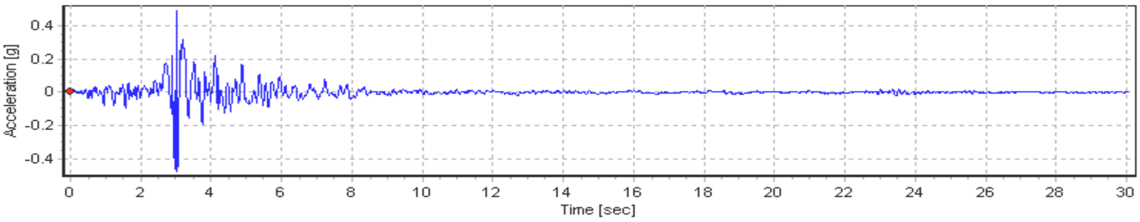
2.2 Determination of lateral / rocking spectra for systems without stiffness and strength coupling

The geometric aspect ratio of the system (H/B), the two normalized strength parameters, η_L and η_θ, and the lateral and rotational oscillation periods T_L and T_θ, are the parameters of the simplified dynamic system in Fig.3. Once the base input time history g(t) and the hysteretic force and bending moment relations f_M and f_R are specified as inelastic hysteretic shapes (e.g. elastic perfectly plastic, bilinear hardening, pinched bilinear and so on), complete rocking / lateral response spectra can be obtained for a series of input motions and a family of

oscillators, over a complete set of lateral - rocking periods and lateral - rocking yield strength ratios.
In the intermediate case, the coupled system of equations above can be solved numerically for independent values of M_y, R_y, k_θ and k_L, strictly obtained from the basic parameters above: furthermore, in order to account for the possible effect of multiple three-dimensional frames interacting with the rocking wall in the plane, as analyzed herein, a fifth parameter is also used independent of the geometric shape, namely the ratio of the point of application of the inertia force to the system dimension H (ratio $\alpha = h_c/H$), where α typically ranges between 0.5 and 2/3. In order to evaluate the coupled system response under earthquake excitation, the structure was modeled in OpenSees (McKenna et al., 2000) using elastic line elements with large numerical stiffness and strength for the rocking wall (with its masses distributed at the corners). Zero length hinge elements in the vertical and lateral sense at the wall base corners were used to model the lateral and rocking spring stiffness, using uniaxial material hysteretic characteristics, in this case, bilinear or bilinear pinched were used in the solution, following the program conventions (this is referred to as '*The Spring Model*'). For comparisons against code BISPEC (Hachem et al., 1999) and a parametric investigation of lateral ductility spectra (η_L-μ_L-T_L) with their rocking lateral counterparts (η_L-μ_L-η_θ-T_L-T_θ) using this model, see Alexandropoulos et al. (2008).

2.3 Determination of lateral / rocking spectra for RC buildings with stiffness and strength coupling

In the case of an RC building in coupled lateral-rocking response, as the case investigated herein, the solution of the above system of equations is further complicated during the oscillator inelastic response by the fact that both the lateral and the rocking soft storey stiffness and strength (namely the yield strength parameters R_y and M_y) are strongly coupled during the entire response history. On the one hand, the stiffness at the base result from the lateral, bending and axial stiffnesses while on the other, the lateral and bending strength results from the axial and flexural strength of the individual columns. All of the above remain strongly coupled during the entire lateral and rocking response history, due to the axial load-bending moment interaction of the columns and the deformation dependent extent of concrete cracking or crushing and reinforcement yielding at the critical sections of the columns. It is the fact that neglecting such coupling in the nonlinear time history response prediction that will lead to significant errors in the response, both in terms of lateral ductility demand but also - more importantly - in terms of predicting with accuracy the axial load - bending demands of the soft storey columns (Zeris et al., 1991) and is therefore investigated herein.



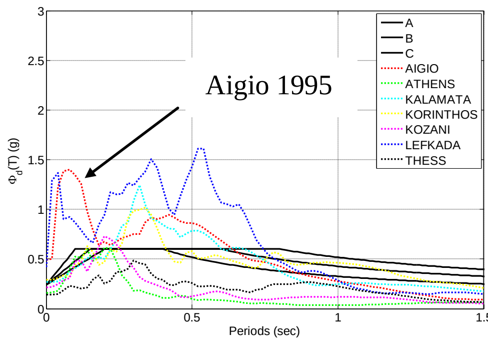


Figure 4. The Aigio 1995 record and elastic design response spectrum, compared to current Code design spectrum requirements (NEAK, 2000).

In order to express realistically this RC nonlinearity dependence in the spectral evaluation, the base storey of the model oscillator was further modeled using distributed inelasticity beam column elements with internally monitored fiber section models ('*The Concrete Model*', described herein). Different column dimensions and reinforcing ratios were considered while a wide range of actual material characteristics were adopted for these base column models, extending the *Spring Model* as shown in Fig.3. In this case, the ground storey elements were modeled using element *nonlinearBeamColumn* developed by Spacone et al. (1996). Three different column sizes were considered, namely 30/30cm, 40/40cm and 50/50 cm, with a constant soft storey height of 3.0m. Block aspect ratios were equal to 1/3, 1.0, 2.0 and 3.0 while several equal size column configurations were investigated, ranging from two spans and up to six spans, with limits in the column spacing. Reinforcement was equally distributed at the four sides to a total steel ratio of 2.0 % of the gross concrete area. The concrete unconfined compressive strength was 25 MPa and the steel yield strength was 500 MPa. Concrete was modeled using the softening/cracking hysteretic model proposed by Mander et al. (1988), while steel characteristics were bilinear with kinematic hardening. Two levels of gravity load were initially imposed on the columns, namely 10% and 30% of the axial load capacity of the column, being equal to the gross area times the compressive concrete strength. In order not to introduce additional parameters, no second order effects were considered in these analyses. Due to space limitations, out of different earthquakes considered, only the case of the M_s = 6.5 Aigio 1995 record shown in Fig.4, with a peak ground acceleration of 0.49g, is depicted herein (Lekidis et al., 1999). A more detailed parametric investigation is presented elsewhere (Alexandropoulos and Giannitsas, 2008).

3. EVALUATION OF DUCTILITY AND NORMALIZED AXIAL FORCE SPECTRA

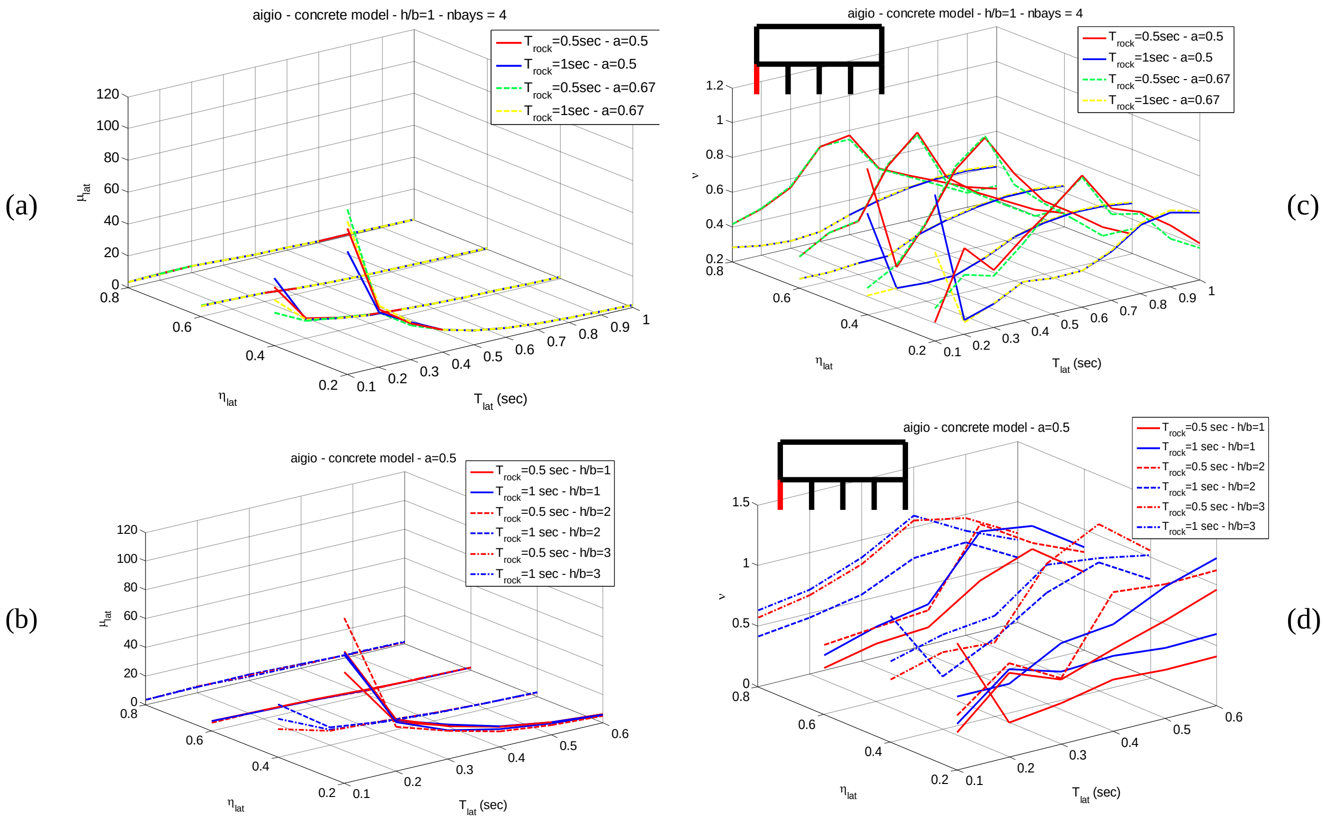


Figure 5. T_L and η_L Spectra of lateral ductility and normalized axial load at the corner column for different T_θ and block aspect ratio H/B (*Concrete Model*).

Typical spectral results for the five column configuration are considered in Fig. 5. It is observed (Fig. 5a-b) that the lateral ductility of the center of mass of the block oscillator is strongly influenced by the rocking parameters (namely the block aspect ratio H/B and T_θ) primarily in the case of laterally weak systems, having a normalized lateral strength ratio η_L for the Aigio 1995 record of 0.20 (namely a base shear coefficient at yield equal to 0.2*0.49, or 10% of the structure's weight, which is lower than current Code design levels in this seismicity region (zone II, EAK 2000). Furthermore, such an influence of rocking is only visible in the stiff systems with T_L of 0.30 sec or less.

Despite this observed lack of sensitivity in the lateral ductility and contrary to the above observation, however, the axial load sensitivity to the rocking aspect ratio H/B is significant; as the comparisons in Fig. 5c-d indicate, for H/B equal to 1.0 (Fig. 5c) there is a strong tendency for the normalized compressive axial load demand at the corner to increase with T_L, with the maximum compressive axial load varying in the period range considered between values as low as 0.4 at the low and high period range and reaching a peak demand of up to 1.20 in the central period range, around the spectral peak of the earthquake (see Fig. 4), namely at T_L between 0.3 and 0.5 sec. This occurrence of the peak axial load depends on the lateral resistance ratio η_L, with the relatively stronger and stiffer models exhibiting higher axial load demands (with little influence of α). Furthermore, the type of dependence is only evidenced for the case of a rocking period of 0.5 sec, with the more flexible structure (in rocking, with T_θ equal to 1.0 sec) behaving differently in the same period range, without as much sensitivity in the normalized force. Similar observations - but at different fundamental frequencies are obtained considering the comparison of the same response indices at different aspect ratios (1.0, 2.0 and 3.0), rather than α, indicated in Fig. 5d, where, compared to Fig. 5c, the influence of the block aspect ratio is investigated.

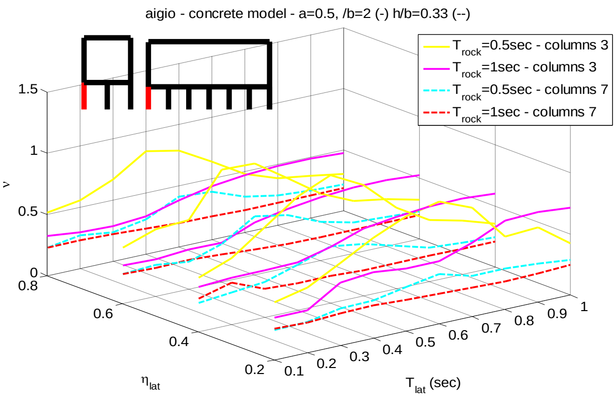


Figure 6. Significance of the building aspect ratio (H/B), T_θ and the number of supporting columns to the normalized corner column axial load (*Concrete Model*).

Considering the above significance of H/B, the beneficial effect of the number of supporting columns is demonstrated in Fig.6, where comparisons are made of the normalized corner column maximum compression for two different base storey configurations, namely two and six spans, over the same lateral period and normalized strength range as above. Two different aspect ratios H/B, namely 2.0 and 1/3 and two rocking periods, namely 0.5sec and 1.0sec, are considered. It is seen that the six span configuration exhibits minor variations of axial load at the corner above the gravity level, and only in the case of a stiff rocking system. The building with a larger aspect ratio is more sensitive to rocking phenomena, exhibiting amplifications of the gravity axial load in all cases, while behaving differently depending on the rocking flexibility: for the flexible building (T_θ equal to 1.0 sec) the axial load demands increase for longer periods (and reducing strength), while for the stiff rocking system (T_θ equal to 0.5 sec) peak axial loads occur in the intermediate to short period range, independent of the normalized resistance of the system.

4. CONCLUSIONS

The basic equations of the coupled lateral / rocking oscillator were developed for a set of soft storey structures comprising a rigid superstructure on a flexible base with inelastic characteristics specific of RC construction. The model was analyzed in the time domain and inelastic spectral predictions of the lateral ductility and the normalized axial loads at the corner support elements were presented, using nonlinear element formulations that model more reliably the characteristics if RC response, compared to simplified models currently adopted in seismic design of RC buildings. It was demonstrated that the rocking contribution to lateral ductility was considerable in the short period range and for laterally weak systems, at strength levels characterizing existing RC building construction designed by older seismic design norms. It was further established that despite this localized sensitivity of lateral ductility to rocking, the internal force predictions (in the form of the maximum normalized axial load demand under the block at the corner) exhibited a much stronger sensitivity to rocking, depending in various degrees to the system parameters, namely the lateral period, the lateral normalized strength, the rocking period, the block aspect ratio and the number of supporting elements: for height to width (H/B) aspect ratio of 2.0 and above and relatively few number of spans (between two and four), the peak axial load demand reached values as high as 120% of the axial load capacity of the member, close to periods near the principle frequency content of the earthquake investigated.

REFERENCES

Alexandropoulos, K. and Giannitsas, P. (2008). Evaluation of inelastic response spectra of RC buildings with lateral and rocking response under seismic excitation. Post Graduate Thesis, Graduate PrograSchool of Civil Engineering, National Technical University of Athens (in Greek).

Aslam, M., Godden, W.G. and Scalise, D.T. (1980). Earthquake rocking response of rigid bodies. *Journal of the Structural Division*, ASCE, **106**, 377-392.

Bertero, V.V., Mahin, S.A. and Herrera, R.A. (1978). Aseismic design implications of near-fault San Fernando earthquake records. *Earthquake Engineering and Structural Dynamics*, **6**, 31-42.

Hachem, M. and Mahin, S. (1999). BISPEC. Nonlinear spectral analysis program. Pacific Earthquake Engineering Research Center, Univ. of California, Berkeley, (*http://www.ce.berkeley.edu/~hachem/bispec.html*).

Housner, G. W. (1963). The behavior of inverted pendulum structures during earthquakes. *Bulletin of the Seismological Society of America*, **53:2**, 404-417.

Karakostas, C., Lekidis, V., Makarios, T., Salonikios, T., Sous, I. and Demosthenous, M. (2005). Seismic response of structures and infrastructure facilities during the Lefkada, Greece earthquake of 14/8/2003. *Engineering Structures*, **27:2**, 213-227.

Lekidis, V., Karakostas, Ch., Dimitriu, P., Margaris, V., Kalogeras, I. and Theodulidis N. (1999). The Aigio (Greece) seismic sequence of June 1995: Seismological, strong-motion data and effects of the earthquake on structures. *Journal of Earthquake Engineering*, **3:3**, 349-380.

Mahin, S.A., Bertero, V.V., Chopra, A.K. and Collins, R.G. (1976). Response of the Olive View Medical Center main building during the San Fernando earthquake, *Report No. EERC 76-22*, Earthquake Engineering Research Center, University of California, Berkeley.

Makris, N. and Konstantinidis, D. (2003). The rocking spectrum and the limitations of practical design methodologies. *Earthquake Engineering and Structural Dynamics*, **32:2**, 265-289.

Mander, J.B., Priestley, M.J.N. and Park, R. (1988). Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering*, ASCE, **114:8**, 1804–1826.

Mc Kenna, F., Fenves, G., Jeremic, B. and Scott, M. (2000). OpenSees: Open System for earthquake engineering simulation. PEER, (*http://opensees.berkeley.edu*).

Ministry of Environment, Planning and Public Works. (2000). EAK 2000, Greek Earthquake Resistant Design Code, Athens, Greece (in Greek).

Psycharis, I.N. and Jennings P.C. (1983). Rocking of Slender rigid bodies allowed to uplift. *Earthquake Engineering and Structural Dynamics.* **11**, 57–76.

Sezen, H., Elwood, K.J., Whittaker, A., Mosalam, K.M., Wallace, J.W. and Stanton, J.F. (2000). Structural engineering reconnaissance of the August 17, 1999, Kocaeli (Izmit), Turkey, earthquake, *PEER Report 2000/09*, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Spacone, E., Filippou, F.C. and Taucer F.F. (1996). Fiber beam-column element for nonlinear analysis of R/C frames. Part I: Formulation. *Earthquake Engineering and Structural Dynamics*, **25**, 711–725.

Yim, S.C.S., Chopra, A.K. and Penzien, J. (1980). Rocking response of rigid blocks to earthquakes. *Earthquake Engineering and Structural Dynamics*, **8:6**, 565–587.

Yim, S.C.S. and Chopra, A.K. (1984). Earthquake response of structures with partial uplift on Winkler foundation. *Earthquake Engineering and Structural Dynamics*, **12:2**, 263-281.

Zeris, C.A. and Altmann, R. (1984). Implications of damages to the Imperial County Services Building for earthquake - resistant design. *Eighth World Conf. of Earthquake. Engineering*, **IV**, San Fransisco, 823-830.

Zeris C. (1986). Three dimensional nonlinear response of reinforced concrete buildings, *Ph.D. Dissertation*, University of California, Berkeley.

Zeris, C. and Mahin, S. (1991). Behavior of reinforced concrete structures subjected to uniaxial excitation. *Journal of the Structural Division*, ASCE, **117:9**, 2640-2656.