An Investigation of the inelastic response of RC pilotis systems

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1. Introduction and Statement of the Problem

Reinforced concrete (RC) pilotis buildings are widespread throughout densely populated and earthquake-vulnerable countries in the Mediterranean area and they are mainly characterised by an open space at the ground floor that could be considered for different functional reasons, such as parking, shops, driveways, gardens and garages. In most of the cases, the ground floor is used for commercial purposes with residential housing at the upper floors or it could be used as a garage in case of a fully residential pilotis building. The stiffness irregularity in elevation that this typology of structures possesses is due to a significant change in lateral stiffness between the open ground storey and the partially or fully masonry infilled RC upper frame.

Such (intentionally or not) created non-uniform distribution of mass, stiffness, lateral and rocking resistance capacity induces a non-ductile seismic response (i.e. soft-storey mechanism) of the structural system, thus increasing its vulnerability to earthquake. As already observed in the past seismic events (e.g., Lekidis et al., 1999, Karakostas et al., 2005), damage is more likely to concentrate at the weaker base, and considering the fact that this type of mechanism does not allow for force redistribution, the activation of the soft-storey mechanism drastically reduces not only the lateral ductility capacity of the building but also its resilience to lateral loading. The fact that pilotis are widespread in countries with a high seismic hazard over the entire Mediterranean area, enhance even more the vulnerability of the whole building population of densely populated cities like Athens (Greece) and Istanbul (Turkey).
Out of several cases reported in post-earthquake reconnaissance reports on pilotis failures, a well-known and documented failure of an intended pilotis system was the partial collapse of the Imperial County Services Building, an intentional pilotis system (Figures 1, 2). Its failure, among other issues, brought forward the contribution of superstructure rocking to the lateral seismic response of pilotis RC structures. The fact that this building was the first under the Strong Motion Instrumentation Program to be monitored extensively within the structure and at the free field, made this structure unique as one of the first full scale seismic tests of its time that was studied by several researchers (Zeris, 1986, Kreger et al., 1989). The post failure structural analysis of the building and the records, together with inelastic analyses of the end wall on columns showed that for this structural form the equivalent lateral single degree of freedom response representation cannot estimate reliably the failure mode, member forces and lateral deformations at global and local level (Zeris 1986).

In order to fully understand the rocking behaviour of RC pilotis buildings, a simplified soft-storey oscillator model has been created. The mathematical representation of the elastic response of the aforementioned oscillator, treated in Zeris (2015), is initially discussed. Subsequently the non-linear behaviour of the simplified model is investigated through time-history analyses and, more specifically, by means of an Incremental Dynamic Analysis (Vamvatsikos et al., 2002).

2. Rocking Assessment Methodology

The methodology adopted for the assessment of the rocking response of pilotis buildings broaden from the mathematical formulation of the elastic response of the rocking oscillator model (Zeris, 2015) to a more detailed analysis on a non-linear simplified rocking oscillator model. Global structural response parameters are considered to evaluate the effect that the presence of a rocking superstructure has on the overall structural response.

2.1 Elastic Coupled System

The response of a class of non-linear oscillators representing a structural system characterised by a relatively rigid superstructure on a soft ground storey is initially evaluated under the assumption of
elast resistant. The mathematical description of the problem is derived by means of a simplified rocking oscillator model described in Figure 3 and through the definition of the governing equation of motion in Eq. 1, accounting for both lateral translation and overturning (rocking) of the rigid superstructure relatively to the flexible ground, as indicated in the following:

\[
F_I(t) + F_D(t) + F_R(t) = F_\ell(t)
\]

where \(F_I\), \(F_D\), \(F_R\) and \(F_\ell\) are inertia, damping, elastic internal resistance and forcing vectors, respectively, as a function of time. Under the assumption of small overturning rotations, three governing differential equations of the system's independent DOF response are determined (Eq. 2), namely the lateral, vertical and rocking response of the wall centre of mass relative to the base, where \(u(t)\), \(v(t)\) and \(\theta(t)\) are lateral, vertical and rotational response histories of the centre of mass of the block relatively to the ground. The stiffness quantities involved are the lateral stiffness \(k_L\) and the rocking resisting axial stiffness \(k_\ell\) of the columns at each end of the wall.

\[
\begin{bmatrix}
R(t) \\
V(t) \\
M(t)
\end{bmatrix}
= 
\begin{bmatrix}
0 & k_L & 0 & k_{\ell} \\
0 & 0 & k_v & 0 \\
k_{\ell} & 0 & 0 & k_{\ell}
\end{bmatrix}
\begin{bmatrix}
u(t) \\
v(t) \\
\theta(t)
\end{bmatrix}
= 
\begin{bmatrix}
k_L & 0 & -k_L\left(\frac{H}{2}\right) \\
0 & 2k_v & 0 \\
-k_L\left(\frac{H}{2}\right) & 0 & k_L\left(\frac{H}{2}\right)^2 + k_v\left(\frac{B}{2}\right)^2
\end{bmatrix}
\begin{bmatrix}
u(t) \\
v(t) \\
\theta(t)
\end{bmatrix}
\]

\[
\begin{bmatrix}
R_I \\
V_I \\
M_I
\end{bmatrix}
= 
\begin{bmatrix}
m & 0 & 0 \\
0 & m & 0 \\
0 & 0 & mr^2
\end{bmatrix}
\begin{bmatrix}
\ddot{u}(t) \\
\ddot{v}(t) \\
\ddot{\theta}(t)
\end{bmatrix}
= 
\begin{bmatrix}
\gamma BH & 0 & 0 \\
0 & \gamma BH & 0 \\
0 & 0 & \gamma BH(B^2 + H^2)/12
\end{bmatrix}
\begin{bmatrix}
\ddot{u}(t) \\
\ddot{v}(t) \\
\ddot{\theta}(t)
\end{bmatrix}
\]

\[
F_\ell(t) = -
\begin{bmatrix}
m \\
0 \\
0
\end{bmatrix}
\ddot{u}_\ell(t)
\]

Therefore, unlike the vertical term that, for small rotations is independent of the other DOFs, the lateral and rocking response of the pilotis is coupled by the off-diagonal terms of the stiffness matrix, as indicated in Eq. 2. Finally, in order to define all the quantities of the equation of motion (Eq. 1), the inertial force term and the external lateral excitation term are evaluated as indicated in Eq. 3 And Eq. 4 respectively, where \(m\) is the translational mass, \(I = mr^2\) is the mass moment of inertia of the rigid block superstructure about the axis normal to the plane of the wall through its centre of mass, \(\gamma\) is the unit weight of the wall with height \(H\) and width \(B\), \(\ddot{u}_\ell(t)\) is the base input excitation and \(h_c\) is the position of the centre of mass relative to the ground.

Considering out the vertical term that yields the standard vertical eigenperiod and eigenmode of the system, the next two fundamental eigenvalues of the coupled lateral-rocking soft storey oscillator can be established by solving the standard eigenvalue problem, as described in more in detail in Zeris (2015), for the mass and the stiffness matrices previously defined; the resulting expressions for the two coupled angular frequencies \(\omega_1\) and \(\omega_3\) of the system are indicated in Eq. 5, Eq. 6 and Eq. 7, in terms
of \( \omega_L \) and \( \omega_\theta \), where \( \omega_L \) is the uncoupled lateral angular velocity \( \omega_L = (k_L/m)^{0.5} \), \( \omega_\theta \) is the uncoupled rocking angular velocity \( \omega_\theta = (k_\theta/I)^{0.5} \), \( \gamma \) is the ratio of the uncoupled oscillators’ frequencies and \( \beta \) is a function of the oscillator slenderness (or aspect ratio) \( \alpha = H/B \).

\[
\omega_{1,3}^2 = \left[ \frac{k_L}{m} \left( 1 + \frac{1}{4 \beta^2} \right) + \frac{k_\theta}{m \beta^2 H^2} \right] \mp \sqrt{\left[ \frac{k_L}{m} \left( 1 + \frac{1}{4 \beta^2} \right) + \frac{k_\theta}{m \beta^2 H^2} \right]^2 - \frac{4 k_L k_\theta}{m^2 \beta^2 H^2}} / 2
\]

5)

\[
\omega_{1,3}^2 = \left( \omega_L^2 \left( 1 + \frac{1}{4 \beta^2} \right) + \omega_\theta^2 \right) \mp \sqrt{\left( \omega_L^2 \left( 1 + \frac{1}{4 \beta^2} \right) + \omega_\theta^2 \right)^2 - 4 \omega_L^2 \omega_\theta^2} / 2
\]

6)

\[
\gamma^2 = \frac{\omega_\theta^2}{\omega_L^2} = \frac{T_1^2}{T_\theta^2} = \frac{1}{2 \alpha^2 \beta^2} \frac{k_v}{k_L} = \frac{6 \ k_v}{1 + \alpha^2} \ k_L
\]

7)

\[
\beta = \sqrt{\frac{1 + (1/\alpha)^2}{12}}
\]

Fig. 4 Interdependence between uncoupled period ratio and stiffness ratio.

Fig. 5 Variation of the ratio between coupled fundamental period and uncoupled lateral period, \( T_1/T_{lat} \) with \( k_v/k_L \)

Fig. 6 Variation of the ratio between coupled rocking period and uncoupled lateral period, \( T_3/T_{lat} \) with \( k_v/k_L \).

Uncoupled lateral and rocking periods (Fig. 4), \( T_{lat} \) and \( T_\theta \), as well as coupled fundamental and rocking periods (Fig. 5 and Fig. 6), \( T_1 \) and \( T_3 \), are showed against the ratio between vertical axial/rocking stiffness and lateral stiffness, \( k_v/k_L \), through which, as expected, the interdependence of the uncoupled lateral and rocking mode periods to the system’s vertical to lateral stiffness ratio is demonstrated. Furthermore, the results clearly indicate a coupling of modes together with an influence of the aspect ratio on the elastic response the pilotis model. As \( k_v/k_L \) increases, going towards a case of a pilotis with laterally but not axially cracking or yielding ground column elements, the magnitude of the ratio \( T_1/T_{lat} \) decreases reaching asymptotically the value of 1. No superstructure rocking influence is appreciated in the higher stiffness ratio range and no fundamental period shift is showed.
However, in the lower stiffness ratio range, the influence of the aspect ratio $\alpha$ in the fundamental period shift is clearly demonstrated. On the other hand, considering the ratio between the coupled third mode period $T_3$ and the uncoupled lateral period, the trend decreases as $k_\theta/k_L$ increases, showing a shortening of the higher coupled period especially for the pilotis characterised by the lower aspect ratio $\alpha = 1$, the case of a square superstructure.

As the preceding analysis has shown, the coupled system modal frequencies are dependent on the stiffness and aspect ratio of the oscillator to different extents. However, these terms are not entirely independent once the uncoupled lateral and rocking periods $T_{lat}$ and $T_\theta$ are defined. Therefore to visualise these variations in absolute period terms, the first and third mode periods of the 3DOFs system are plotted in terms of $T_{lat}$ and for five values of $T_\theta$, for aspect ratio 1.0, in Fig. 7 and 8, respectively. It is seen that, as an example, for the case of a flexible square superstructure with $T_{lat} = 0.9$ sec, for which $T_\theta$ assumes a value of 0.9 sec, the first mode period shifts up to about 150% of the lateral-only first period $T_{lat}$. Similar results can be found also considering the pilotis with higher aspect ratios $\alpha$. On the contrary, considering the third coupled period $T_3$, it seems to be more dependent on the rocking period $T_\theta$ for higher values of uncoupled lateral periods. For the stiffer values of $T_\theta$, $T_3$ tends to be equal to the uncoupled rocking period itself, as the uncoupled lateral period increases. Not very significant influence of the aspect ratio of the pilotis is appreciated for the results on the rocking coupled period $T_3$.

![Fig.7 Variation of the coupled fundamental period $T_1$ as a function of the SDOF lateral-only period $T_{lat}$, for different values of uncoupled rocking period (case $\alpha = 1$).](image)

![Fig.8 Variation of the coupled rocking period $T_3$ as a function of the lateral-only period $T_{lat}$, for different values of uncoupled rocking period (case $\alpha = 1$).](image)

2.2 Numerical Model and Base Seismic Input Characteristics

In order to investigate the previously described coupled system under base excitation, a 3DOFs bi-dimensional system shown in Fig. 9 is modeled using the OpenSees platform (McKenna et al. 2000) and a Tcl programming language based algorithm. The rigid superstructure has been modeled by means of high stiffness and strength linear elements having axial-only (truss, along the perimeter) and flexural/axial (elasticBeamColumn, for the edge to centroid connection) characteristics (Fig. 10). Two vertical truss elements are placed at the base, connecting the two lower edges of the superstructure to the foundation, in order to model the system rocking springs. Moreover, concerning the lateral system strength and stiffness, a horizontal truss is positioned laterally and is connected to one of the two lower
superstructure edges. The wall tributary mass in all three DOFs is lumped at the centroid. Second-order effects are not considered in the spring rocking oscillator model.

The considered seismic input used for the time history analyses is the accelerogram (A299-1) recorded during the Athens 1999 earthquake (Greece) near the mountain of Parnitha, which caused several damages in the RC pilotis buildings that had been constructed during the 1970s in Athens (Fig. 11). The record has a Peak Ground Acceleration (PGA) of about 0.16 g and a predominant period $T_p$ of 0.2 sec (Fig. 12). In all the cases considered, Rayleigh damping of 5% of critical was assumed in the first and third modes of the coupled response of the spring model.

### 2.3 Inelastic coupled system

The inelastic response of the aforementioned oscillator is now investigated under seismic excitation at the base. In order to evaluate the effect of the superstructure rocking influence on the response of the oscillator, inelastic displacement ductility demands will be compared to those obtained under lateral response only.

#### 2.3.1 Lateral Response

The non-linear representation of the equivalent oscillator lateral response can be derived by equating the internal forces (inertia, damping and resisting force of the ground storey equivalent lateral spring)
to the external forces acting at the centre of mass of the rigid superstructure, as indicated in the following Eq. 8:

$$m \cdot \ddot{u}(t) + 2 \xi m \omega_L \cdot \dot{u}(t) + k_L \left( u(t) - H \cdot \frac{\theta(t)}{2} \right) = -m \cdot \ddot{u}_y(t)$$  \hspace{1cm} (8)

where $u(t)$ is the lateral response, $\theta(t)$ is the rocking response and $\xi$ is the fraction of the critical damping. Moreover, Eq. 8 can be normalised (Bertero et al. 1978) by considering the term $m u_y$, with $u_y$ being the yield lateral displacement of the oscillator model. The yield lateral deformation $u_y$ is a system parameter obtained from the lateral inelastic constitutive equation of the ground storey lateral spring, $u_y = R_y/k_L$, where $R_y$ is its lateral yield resistance. Hence, the aforementioned equation of motion of the inelastic oscillator can be rearranged, as indicated in Eqs. 9 and 10:

$$\ddot{u}_L + 2 \xi \omega_L \cdot \dot{u}_L + \frac{F_R(u, \theta, t)}{m \cdot R_y / k_L} = -\ddot{u}_y(t), \quad F_R(u, \theta, t) = R_y \cdot f_R(u, \theta, t)$$  \hspace{1cm} (9)

$$\ddot{u}_L + 2 \xi \omega_L \cdot \dot{u}_L + \omega^2 L \cdot f_R(u, \theta, t) = \frac{\omega^2 L \cdot g(t)}{\eta_L}, \quad \eta_L = \frac{R_y}{m \cdot \ddot{u}_y^{\max}}$$  \hspace{1cm} (10)

where $u_y(t) = u(t)/u_y(t)$ is the lateral displacement ductility, $\eta_{lat}$ is the normalised lateral resistance of the oscillator (Zeris et al., 2008).

### 2.3.2 Rocking Response

The non-linear representation of the equivalent oscillator rocking response is herein calculated, similarly to the calculation of the lateral equation of motion, by performing the rotational equilibrium of the forces involved with respect to the position of the centre of mass and relatively to the ground. The equation of dynamic rotational equilibrium is given in Eq. 11:

$$I \cdot \ddot{\theta}(t) + m h_c \cdot \ddot{u}(t) + 2 \xi I \omega_0 \cdot \dot{\theta}(t) + k_\theta \cdot \dot{\theta}(t) = -m h_c \cdot \ddot{u}_y(t), \quad k_\theta = 2k_v (B/2)^2$$  \hspace{1cm} (11)

where $I$ is the moment of inertia of the rigid block superstructure for in-plane rotation about the axis normal to the frame plane through its centre of mass, $\theta(t)$ is the rotational response, $h_c$ is the elevation of superstructure centre of gravity with respect to the foundation, $k_\theta$ is the initial elastic rocking stiffness under rotation of the superstructure provided by the two vertical ground storey springs, equal to $k_\theta = 2k_v (B/2)^2$ for the uncoupled rocking oscillator and $k_\theta = 2k_v (B/2)^2 + k_L (H/2)^2$ for the coupled rocking oscillator. The stabilising contribution provided by the gravity term $mgh_c$ is not considered for the rocking oscillator under investigation, since the overturning resisting characterised by the presence of the two ground storey vertical springs is much more affecting the response of the overall oscillator. Similar to the normalisation performed for the lateral response dynamic equation, Eq. 11 is normalised by the yield rotation $\theta_y$, as indicated in the following Eqs 12 and 13, below:

$$\ddot{\mu}_0 + \frac{m h_c}{I \cdot \theta_y} \cdot \dot{\mu}_0 + 2 \xi \omega_0 \cdot \dot{\mu}_0 + \frac{M_R(u, \theta, t)}{I \cdot \theta_y / k_\theta} = -\frac{m h_c \cdot \ddot{u}_y(t)}{I \cdot \theta_y}, \quad M_R(u, \theta, t) = M_y \cdot f_R(u, \theta, t)$$  \hspace{1cm} (12)

$$\ddot{\mu}_0 + \frac{m h_c}{I \cdot \theta_y} \cdot \dot{\mu}_0 + 2 \xi \omega_0 \cdot \dot{\mu}_0 + \omega^2 \cdot f_M(u, \theta, t) = -\frac{\omega^2 L \cdot g(t)}{\eta_\theta}, \quad \eta_\theta = \frac{M_y}{m h_c \cdot \ddot{u}_y^{\max}}$$  \hspace{1cm} (13)
where $\mu_0 = \theta(t)/\theta_y$ is the rotational ductility, $f_\mu(u,\theta,t)$ is a non-dimensional function of the base rocking cyclic dependence on the inelastic rotation, $M_y$ is the rollover yield moment resistance, equal to $M_y = R_y B$ for the unclipped rocking oscillator and $M_y = R_y B + R_y H/2$ for the coupled rocking oscillator and $\eta_0$ is the normalised rocking strength of the oscillator (Zonis, 2008).

The spectrum of the rocking response of the inelastic coupled oscillator has been evaluated by considering the previously described numerical model and investigating the response under base excitation, for different values of $T_{lat}$ (0.1 to 1.5), $T_\theta$ (0.1 to 1.0), $\eta_{lat}$ (0.1 to 0.5) and $\eta_\theta$ (0.2 to 0.6), which represent the modal and the resisting quantities that describe the rocking oscillator. Furthermore, three different geometric configurations are considered for the oscillator, having three different values of the aspect ratio $\alpha$ (1.0, 2.0 and 3.0).

The maximum displacement ductility demand from the 3DOFs rocking oscillator response ($\mu_{rock}$) is compared to the maximum displacement ductility demand from the SDOF model ($\mu_{lat,only}$). This comparison clearly highlights the effect of the presence of rocking response in a simplified model that represents the behaviour of building characterised by the presence of soft ground storey. Since modern design codes are based on structural performance which is based on displacement, displacement ductility and/or curvature ductility, the aforementioned investigation is helpful to understand if the current SDOF representation used in the design process is also conservative for buildings where the rocking/rotation of the rigid superstructure might affect the response of the overall structural system.

Fig.13 Ratio between maximum displacement ductility from rocking response Vs. lateral-only ($\max_\mu_{rock}/\max_\mu_{lat,only}$), for $\alpha = 1$, $\eta_{lat} = 0.2$ and different values of $\eta_\theta$ and range of $T_\theta$ ($T_{rock}$).
Fig. 14 Ratio between maximum displacement ductility from rocking response Vs. lateral-only (max_\(\mu_{\text{rock}}\)/max_\(\mu_{\text{lat,only}}\)), for \(\alpha = 1\), \(\eta_{\text{lat}} = 0.4\) and different values of \(\eta_\theta\) and range of \(T_\theta(T_{\text{rock}})\).

Fig. 15 Ratio between maximum displacement ductility from rocking response Vs. lateral-only (max_\(\mu_{\text{rock}}\)/max_\(\mu_{\text{lat,only}}\)), for \(\alpha = 3\), \(\eta_{\text{lat}} = 0.4\) and different values of \(\eta_\theta\) and range of \(T_\theta(T_{\text{rock}})\).
As Figures 13 to 15 clearly show, quite substantial increases of the displacement ductility demand appear either at the lower period range, between 0.3 and 0.5 sec, and at the higher period range, between 0.7 and 1.0 sec. The highest peaks correspond to the cases where the rocking period $T_\theta$ is higher compared to the corresponding lateral-only period $T_{lat}$. The displacement ductility demand is shown in Figure 14 for a normalised lateral resistance value equal to 0.4 (compatible with new designs), by varying the normalised bending strength of the building. This last quantity seems not to have much influence in the response of the rocking oscillator model. This could be expected, as for the specific spring model, the bending resistance of the vertical elements is not coupled with the eventual fluctuation of the axial force demand which would most likely affect the structural response. Slight influence can be appreciated by considering, for the same building characterised by the same aspect ratio $\alpha$, different values of normalised lateral resistance $\eta_{lat}$, as shown in Figure 13 ($\eta_{lat} = 0.2$) and Figure 14 ($\eta_{lat} = 0.4$). In fact, displacement ductility demands increase based on the response of the lateral-only oscillator, shown at different period ranges.

Finally, the influence of the geometry of the building is considered by considering different aspect ratios (Figure 14 and Figure 15), it seems that the slenderness of the building also influences the structural response, as also established in the linear elastic oscillator response, especially in both period ranges, 0.3 and 0.5 sec and 0.7 and 1.0 sec, where the relative ductility demands are increased from 2.0 to 2.5 and 1.5 to 2.0 (of $\mu_{lat,only}$), respectively, for the building with $\alpha = 3$, compared to $\alpha = 1$.

2.3.3 Incremental Dynamic Analysis for the Inelastic Rocking Spring Model

Incremental dynamic analysis (Vamvatsikos et al., 2002) have been performed in order to evaluate the seismic response under increasing seismic loading for all the different rocking oscillators considered. The influence of the different parameters describing the oscillator model will be discussed with respect to the response of the SDOF representation, as previously made for the elastic and the inelastic description of the rocking oscillator under investigation.

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</table>

Table 1 Record selection and parameters
A ground motion database has been created with the purpose of selecting a set of records to be used for the non-linear time history analyses on the inelastic models. The records, selected from the PEER NGA online ground motion database, have been chosen on the basis of the following parameters: magnitude between 5.5 and 7.0 (characteristic of Mediterranean events), distance $R_{rup}$ and $R_{JB}$ between 16 km and 35 km (no directivity effects considered) and average shear wave velocity in the first 30 m of soil between 360 m/sec and 1000 m/sec (this includes soil A and B from the EC8 soil classification). Among all the selected records (from 38 seismic events), a number of 10 accelerograms (Table 1) have been considered for the analysis, with elastic response spectra for 5% damping shown in Fig. 16. Peak Ground Acceleration (PGA) is selected as the seismic intensity measure considered for the evaluation of the IDA curves and, for this purpose, the 10 selected records are scaled in order to match a number of 10 target horizontal elastic response spectra (Type 1, Soil B), characterised by 10 different values of PGA, from 0.1 g to 1.0 g. The scaling technique considered is the non-linear scaling by means of wavelets (Mukherjee et al., 2002). The aforementioned procedure has been performed by means of an algorithm programmed in Matlab and the following seismic parameters are calculated: Peak Ground Acceleration (PGA), Velocity (PGV) and Displacement (PDG), Root Mean Square (RMS) of acceleration, velocity and displacement, Arias Intensity (AI), Housner Intensity (HI), A95 parameter and predominant period ($T_p$).

![Fig.16 Earthquake selection spectra.](image1)

![Fig.17 Spectral matching (Friuli 1976) for increasing PGAs](image2)

A typical IDA excitation spectral input is depicted in Fig. 17. For what concerns the structural parameter selected for the definition of the IDA curves and considering the fact that the simplified 3DOFs rocking oscillator model is intended to represent the structural response of pilotis, which is characterised by the activation of the soft (ground) storey mechanism, interstorey drift referred to the ground storey columns is adopted for the calculation of the aforementioned IDA curves.

The results expressed in terms of Incremental Dynamic Analysis (IDA) show a clear amplification of the drift demand for the coupled rocking oscillator with respect to the lateral-only response. This increase in the deformation demand is significant for oscillators characterised by lateral period $T_{lat}$ between 0.1 sec and up to 0.6 sec (Fig. 18-20), exhibiting coupled to SDOF drift prediction increases between 300% and 130%, respectively. As $T_{lat}$ increases, the influence of the characteristic rocking period $T_0$ is reduced. No influence in the response by considering different aspect ratios is highlighted herein.
Fig. 18 IDA curves, $T_{lat} = 0.1$ sec, $\alpha = 1$, $\eta_0$ elastic

Fig. 19 IDA curves, $T_{lat} = 0.5$ sec, $\alpha = 1$, $\eta_0$ elastic
Conclusions

The work presented herein deals with the evaluation of the seismic vulnerability of a simplified rocking oscillator model representing the structural response of buildings characterised by the presence of a rigid superstructure sitting on a soft ground storey. The vulnerability of this soft storey mechanism is addressed in both elastic and inelastic representations of the model. The parameters considered to characterise the simplified oscillator are defined in terms of modal characteristics as well as lateral and axial capacity of the truss elements defined to model the behaviour rocking oscillator at its ground storey. Lateral ($T_{lat}$) and rocking ($T_{\theta}$) periods together with the normalised lateral resistance and the normalised bending strength are the parameters identifying the simplified numerical model. Different geometrical aspect ratios $\alpha$, defining the slenderness of the rigid superstructure, are taken into account in the performed analyses.

The structural response of the simplified oscillator model is investigated and addressed in the elastic sense by initially evaluating the effect that the coupling of the lateral and rocking behaviour have on the modal response. This coupling in the structural response results in a clear shifting of the fundamental period with respect to the commonly adopted SDOF representation indicated in modern seismic design codes. Moreover, when investigating the response in the non-linear range, ductility and ground storey drift demand between rocking and lateral-only response are compared.

Incremental Dynamic Analysis (IDA) is performed in order to analyze the building model in a statistical term by considering different seismic inputs previously scaled to a target horizontal response.
spectrum. The results highlight the unconservative assumption of the SDOF representation and this is demonstrated in an increase of the (lateral) displacement ductility and drift demand related to the ground storey, when rocking is included within the structural model. IDA results show an important difference between the rocking and the lateral-only response for oscillators characterised by an uncoupled lateral period $T_{lat}$ much lower than the associated uncoupled rocking period $T_{θ}$, periods from which the lateral and vertical mechanical characteristics of the ground storey truss elements depend. A higher vulnerability of the rocking oscillator against the lateral-only oscillator is demonstrated.

The assessment of the rocking response of a simplified RC building oscillator with soft ground storey will be further investigated by performing a simulated design procedure, for different levels of ductility, in order to evaluate the reinforcement characteristics of the ground storey columns. A more detailed numerical model will be created and the influence of different structural quantities, i.e. geometry, axial force ratio and confinement addressed. Fragility functions will be evaluated based on different global and local structural indices. A more refined and reliable approach in the selection of the ground motion database is required and it shall be implemented for the calculation of the fragility curves for the RC rocking oscillator model.

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