ADVANCED SHEAR AND AXIAL LAWS OF BEARINGS IN THE NONLINEAR SEISMIC ANALYSIS OF R.C. BUILDINGS

F. Mazza¹

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Base-isolated reinforced concrete (r.c.) framed structures may be subjected to amplification of seismic demand in the superstructure and complex shear-axial interaction in the isolation system at sites located near an active fault. Advanced mathematical models to predict the highly nonlinear behaviour of elastomeric bearings (e.g. high-damping-laminated-rubber bearings, HDLRBs), when subjected to a combination of severe horizontal displacement and significant variation of the axial load, have been recently proposed in the literature and verified with experimental data. The aim of the present work is to evaluate the effects of these advanced formulations in shear, axial and rotational behaviour of HDLRBs on the nonlinear seismic analysis of r.c. base-isolated structures. New and retrofitted base-isolated five-storey r.c. framed buildings are designed in a high-risk seismic zone. Simplified and advanced load-deformation laws are compared on the basis of a three-degree-of-freedom model of the HDLRBs including horizontal and vertical displacements and rotation. Nonlinear incremental dynamic analysis of the test structures is carried out considering near-fault earthquakes recorded at different stations. Results show that property modification factors should be introduced in bounding analysis procedures to include the effects of advanced nonlinear modelling of the isolation system.

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Introduction

The effectiveness of base-isolation in protecting new buildings and also as a retrofitting strategy of existing buildings subjected to far-fault earthquakes is widely acknowledged, while more studies are needed in the case of near-fault earthquakes. Base-isolated structures may be subjected to amplification of seismic demand in the superstructure and complex shear-axial interaction in the isolation system at sites located near an active fault [1]. Fling-step and forward directivity effects, with long-period horizontal pulses in the velocity signals, and high values of the peak ground acceleration ratio, defined as the ratio between the peak values of the vertical and horizontal ground accelerations, are the main causes of these effects [2]. Experimental results have shown that HDLRBs behave nonlinearly when subjected to the combination of considerable horizontal displacements and axial loads, with horizontal stiffness decreasing with increasing vertical load and vertical stiffness decreasing with increasing lateral deformation [3].

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Other studies have highlighted the fact that increasing values of the horizontal displacement can induce a reduction in both critical buckling load [4] and horizontal stiffness [5]. Moreover, the vertical deformation consists of two contributions: i.e. axial shortening or lengthening due to imposed axial load and as a function of the horizontal displacement [6]. Recently, a nonlinear model that captures the tensile response of elastomeric bearings, including the cavitation and post-cavitation behaviour, has been also proposed [7]. Finally, rotation at the supports of HDLRBs can appreciably influence their lateral behaviour [8, 9].

While the analytical and experimental studies above-mentioned have provided considerable insight into how to predict the nonlinear behaviour of elastomeric bearings when subjected to extreme loading, more investigation is required to evaluate the effects of these advanced models of HDLRBs in the nonlinear seismic analysis of reinforced concrete (r.c.) framed buildings subjected to near-fault earthquakes. To this end, a new five-storey base-isolated structure is designed in line with the current Italian code in a high-risk seismic zone (NTC08, [10]). Next, an existing five-storey r.c. framed building, primarily designed (as fixed-base) in compliance with a former Italian seismic code for a medium-risk zone (DM96, [11]), is retrofitted by the insertion of HDLRBs. Then, a computer code is developed for the nonlinear seismic analysis of the test structures, considering a lumped plasticity model to describe the inelastic deformation of r.c. frame members of the superstructure, while nonlinear force-displacement laws are adopted for the isolation system. Specifically, four analytical models of HDLRBs are compared: i) viscoelastic linear (VEL model); ii) with coupling of the horizontal and vertical motion (CM1 model); iii) with cavitation and post-cavitation in tension (CM2C model); iv) with considerable displacements and rotations (CM2CR model). The horizontal and vertical components of near-fault ground motions are selected from the PEER database [12] and scaled within a nonlinear incremental dynamic analysis.

**Nonlinear modelling of elastomeric bearing**

The nonlinear response of an HDLRB subjected to lateral loading (F), acting simultaneously with a compressive or tensile axial (vertical) load (P), exhibits a horizontal ($u_h$) and vertical ($u_v$) displacement, where $u_v$ is assumed as positive in the vertical upward direction, and a rotation ($\theta$) of the top plate. Nonlinear behaviour of the elastomeric bearings can be modelled as an in-parallel combination of elementary mechanical systems comprising: i) linear (elastic) axial and rotational springs, to describe the effective stiffness; ii) viscous dashpot to describe the energy dissipation; iii) nonlinear (elastic) axial and rotational springs, responsible for the stiffness variation at large horizontal displacements and rotations, respectively; iv) nonlinear (inelastic) axial spring to describe the hysteretic behavior in tension. In detail, four alternative analytical models are presented in this work, based on the combination of a number of established physical models allowing for the cyclic behavior of HDLRB to be numerically simulated: i) viscoelastic linear (i.e. VEL model), with a linear elastic axial spring acting in parallel with a linear viscous dashpot both in the horizontal and vertical directions and with a linear elastic rotational spring (Fig. 1a); ii) coupling the horizontal and vertical motion (i.e. CM1 model), with nonlinear elastic axial springs both in the horizontal and vertical directions (Fig. 1b); iii) with cavitation and post-cavitation in tension (Fig. 1c), assuming a nonlinear inelastic axial spring in the vertical direction different from that of the CM1 model (i.e. CM2C model); iv) representing the effects of large horizontal displacements and rotations (Fig. 1d), with a nonlinear elastic axial spring in the horizontal direction different from those of the CM1 and CM2C models and a nonlinear elastic rotational spring (i.e. CM3CR model).
VEL model: viscoelastic linear

Elastomeric bearings are usually made with nonlinear hysteretic materials, but for a simplified design procedure an equivalent viscoelastic linear representation is often preferred. To this end, a three-spring-two-dashpot model can idealize an HDLRB (Fig. 1a), assuming uncoupled elastic (i.e. $F_{K0}$ and $P_{K0}$) and damping (i.e. $F_{C0}$ and $P_{C0}$) axial forces proportional to horizontal and vertical displacement ($u_H$ and $u_V$) and velocity ($\dot{u}_H$ and $\dot{u}_V$), respectively, and spring moment ($M_{K0}$) proportional to the rotation ($\theta$)

\[
F = F_{K0} + F_{C0} = K_{H0}u_H + C_{H0}\dot{u}_H \approx K_{H0}u_H + (\xi_H K_{H0} T_{1H}/\pi)\dot{u}_H \quad (1a)
\]

\[
P = P_{K0} + P_{C0} = K_{V0}u_V + C_{V0}\dot{u}_V \approx K_{V0}u_V + (\xi_V K_{V0} T_{1V}/\pi)\dot{u}_V \quad (1b)
\]

\[
M = M_{K0} = K_{00}\theta \quad (1c)
\]

where: $K_{H0}$ and $K_{V0}$ are the nominal values of the horizontal and vertical stiffness at the design displacements and zero axial load; $K_{00}$ is the rotational stiffness at zero shear strain; $\xi_H$ ($\xi_V$) and $T_{1H}$ ($T_{1V}$) represent, respectively, the equivalent viscous damping ratio and the fundamental vibration period in the horizontal (vertical) direction.

CM1 model: coupling of horizontal and vertical motion

Nonlinear seismic analysis of base-isolated structures requires the use of simple but reasonably accurate models, taking into account axial forces significantly affecting the horizontal response and softening in the vertical direction at large lateral deformations. Starting from experimental results [3], the three-spring-two-dashpot model can be modified assuming coupled nonlinear elastic springs in the horizontal and vertical directions (Fig. 1b), with a modified vertical displacement ($u_{V*}$) taking into account the axial shortening or lengthening due to second order geometric effects:
\[ F = F_{K1} + F_{C0} = K_{H1}u_H + C_{H0}\dot{u}_H \cong K_{H0}\left[1 - \left(\frac{P}{P_{cr}}\right)^2\right] u_H + \left(\xi_H K_{H0}T_{1H}/\pi\right)\dot{u}_H \quad (2a) \]

\[ P = P_{K1} + P_{C0} = K_{V1}u_V + C_{V0}\dot{u}_V \cong \frac{K_{V0}}{1 + 48\left(\frac{u_H}{\pi D}\right) + \frac{\xi_V K_{V0}T_{1V}}{\pi} + u_V} \quad (2b) \]

\[ M = M_{K0} = K_{60}^\theta \quad (2c) \]

where: \( D \) is the diameter of the bearing; \( \alpha_{K0} = K_{V0}/K_{H0} \) is the nominal stiffness ratio; \( \alpha_b = h_b/t_r \), \( t_r \) being the total thickness of the rubber and \( h_b \) the total height of the bearing; \( S_2 = D/t_r \) is the secondary shape factor. A reduced critical buckling load is also introduced, with a bilinear approximation of the area-reduction method which takes into account the finite buckling capacity of a bearing at zero overlap area [4]:

\[ P'_{cr} = 0.2P_{cr} \text{ for } \frac{A_t}{A} \leq 0.2, \quad P'_{cr} = P_{cr} \frac{A_t}{A} \text{ for } \frac{A_t}{A} > 0.2, \quad P_{cr} = \frac{\pi G S_1 S_2 A}{2\sqrt{2}} \quad (2d,e,f) \]

where: \( G \) is the shear modulus of the rubber; \( S_1 = D/(4t) \) is the primary shape factor, \( t \) being the thickness of the single layer of rubber; \( A \) is the bonded rubber area while \( A_t \) is the reduced area due to lateral displacement.

**CM1C model: cavitation and post-cavitation in tension**

Experimental studies have investigated the effect of cavitation on tensile response of elastomeric bearings and found that the post-cavitation stiffness is not zero [7]. Moreover, cavitation in an elastomeric bearing is accompanied by irreversible damage due to the formation of microcracks in the rubber. It is expected, therefore, that when an elastomeric bearing is loaded beyond the point of cavitation and unloaded, it returns along a new path with hysteretic energy dissipation. The inelastic behaviour in tension can be introduced by modifying the three-spring-two-dashpot model with a nonlinear vertical spring to account for the effect of post-cavitation hardening when tensile axial load is applied (Fig. 1c)

\[ F = F_{K1} + F_{C0} = K_{H1}u_H + C_{H0}\dot{u}_H \cong K_{H0}\left[1 - \left(\frac{P}{P_{cr}}\right)^2\right] u_H + \left(\xi_H K_{H0}T_{1H}/\pi\right)\dot{u}_H \quad (3a) \]

\[ P = P_{K1} + P_{C0} = K_{V1}u_V + C_{V0}\dot{u}_V = \frac{K_{V0}}{1 + \left(1 - e^{-k(u_v-u_{vc})}/(kt_r)\right) \text{ for } P \geq P_c, \quad P = P_{K1} + P_{C0} \text{ for } P < 0 \quad (3b,c) \]

\[ M = M_{K0} = K_{60}^\theta \quad (3d) \]

where: \( u_{vc} \) is the vertical displacement at the onset of cavitation; \( P_c = 3GA \) is the cavitation force; \( k = 20 \) describes the post-cavitation variation of the tensile stiffness. Finally, the degradation in the cyclic tensile loading is evaluated by introducing a reduced cavitation strength

\[ P_{cn} = P_c \left\{ 1 - 0.75 \left[ 1 - e^{-\left(\frac{u_v-u_{vc}}{u_{vc}}\right)} \right] \right\} \quad (3e) \]
CM2CR model: large displacements and rotations

The post-critical behaviour of an HDLRB is unstable, so its response can only be predicted when the nonlinearities are fully accounted for [5]. The horizontal and rotational stiffness lessens with increasing horizontal displacement, thereby requiring modified expressions of the nonlinear axial (horizontal) and rotational springs (Fig. 1d):

\[
F = F_{K2} + F_{C0} = K_{H2}u_H + C_{H0}\dot{u}_H \approx K_{H1}\left[1 - 0.325\tanh\left(\frac{\alpha u_H}{t_r}\right)\right] u_H + \frac{\zeta_H K_{H0} T_H}{\pi} u_H \tag{4a}
\]

\[
P = P_{K2} = P_c \left[1 + \left(1 - e^{-k(u_x - u_{x_c})/(kt)}\right)\right] \text{ for } P \geq P_c , \quad P = P_{K1} + P_{C0} \text{ for } P < P_c \tag{4b,c}
\]

\[
M = M_{K1} = K_{01} \theta = K_{00} \left[1 - \frac{25.4 - t}{D} \left(\frac{\alpha u_H}{t_r}\right)\right] \theta \tag{4d}
\]

\(\alpha\) being a dimensionless constant with a value of 1. It should be noted that \(u_H\) in Eq. 4a and 4d is expressed in mm.

New and retrofitted base-isolated structures

A five-storey r.c. framed building is considered for both new (i.e. N.BI) and retrofitted (i.e. R.BI) structures with a base-isolation system (Fig. 2), considering three values for the nominal stiffness ratio \(\alpha_{K0}\) of the HDLRBs (i.e. 400, 1200 and 2400). In order to take the plastic deformations along the beams into account, each of the \(m_i\) is discretized into four sub-elements of equal length; in this way, the potential critical sections correspond to end (exterior and interior), quarter-span and mid-span sections. Dead and live gravity loads used in the design of the N.BI structures are assumed equal to: 6.1 kN/m² and 3 kN/m², for the isolated floor; 4.4 kN/m² and 3 kN/m², for the top floor; 5.7 kN/m² and 3 kN/m², for the other floors. Masonry infills are taken into account by considering a gravity load of 2.7 kN/m² along the perimeter.
Seismic loads are evaluated in line with NTC08 [10], assuming: high-risk seismic zone (peak ground accelerations in the horizontal direction, PGA_{H}=0.283g and 0.345g, at the life-safety, LS, and collapse-prevention, CP, limit states, respectively); elastic response of the superstructure (behaviour factor, q_{H}=1.0); medium-dense subsoil type (class C, with subsoil parameter S_{H}=1.41 and 1.31 at the LS and CP limit states, respectively). A cylindrical compressive strength of 25 N/mm^2 for the concrete and a yield strength of 450 N/mm^2 for the steel are assumed for the r.c. frame members. On the other hand, the original (fixed-base) framed building of the R.BI structures is designed in line with DM96 [11], for a medium-risk seismic region (seismic coefficient: C=0.07) and a typical subsoil class (main coefficients: R=e=β=1). Dead gravity loads used in the design of the R.BI are different from those of the N.BI structures and equal to: 4.5 kN/m^2, for the top floor; 5.2 kN/m^2, for the other floors. Then, steel yield strength of 375 N/mm^2 is considered. Floor masses and cross-sections of the r.c. frame members for the N.BI and R.BI structures are reported in Figs. 2a and 2b, respectively.

The design of the fifteen isolators, with the same dimensions, is carried out in line with the current Italian seismic code at the CP limit state (NTC08, [10]). The base-isolation systems of the N.BI and R.BI test structures are designed on the same values of the fundamental vibration period (i.e. T_{H}=2.5s) and equivalent viscous damping ratio (i.e. ξ_{H}=10%) in the horizontal direction. Different fundamental vibration periods are obtained in the vertical direction (see Table 1), depending on the selected α_{K0} value, related to a same value of the equivalent viscous damping ratio in the same direction (i.e. ξ_{V}=5%). In Table 1, initial stiffnesses (i.e. K_{H0} and K_{V0}) of the HDLRBs are reported together with the following geometrical properties: diameter of the bearing (D); primary (S_1) and secondary (S_2) shape factors. Results of verifications are also presented in Table 1: i.e. shear strain of the elastomer due to seismic displacement (γ_s); maximum total shear strain (γ_{tot,max}); minimum ratio between critical buckling load (P_{cr}) and maximum compression axial load (P). Finally, the design of the superstructure complies with the LS limit state, satisfying minimum conditions for the longitudinal bars of r.c. frame members in line with NTC08 [10], for the N.BI structures in low ductility class, and DM96 [11], for the R.BI structures.

Table 1. Properties of the HDLRBs for the new (N.BI) and retrofitted (R.BI) base-isolated structures (units in kN, cm and s).

<table>
<thead>
<tr>
<th>α_{K0}</th>
<th>T_{1V}</th>
<th>K_{H0}</th>
<th>K_{V0}</th>
<th>D</th>
<th>S_1</th>
<th>S_2</th>
<th>γ_s</th>
<th>γ_{tot,max}</th>
<th>(P_{cr}/P)_{min}</th>
</tr>
</thead>
<tbody>
<tr>
<td>N400</td>
<td>0.125</td>
<td>5.46</td>
<td>2186</td>
<td>70.0</td>
<td>8.7</td>
<td>2.8</td>
<td>1.0</td>
<td>4.4</td>
<td>2.0</td>
</tr>
<tr>
<td>R400</td>
<td></td>
<td></td>
<td></td>
<td>71.7</td>
<td>8.6</td>
<td>2.8</td>
<td>1.1</td>
<td>4.5</td>
<td>2.0</td>
</tr>
<tr>
<td>N1200</td>
<td>0.072</td>
<td>5.46</td>
<td>6557</td>
<td>55.3</td>
<td>16.7</td>
<td>3.6</td>
<td>1.6</td>
<td>5.0</td>
<td>2.7</td>
</tr>
<tr>
<td>R1200</td>
<td></td>
<td></td>
<td></td>
<td>58.6</td>
<td>16.7</td>
<td>3.4</td>
<td>1.6</td>
<td>5.0</td>
<td>2.7</td>
</tr>
<tr>
<td>N2400</td>
<td>0.051</td>
<td>5.46</td>
<td>13113</td>
<td>48.8</td>
<td>30.2</td>
<td>4.1</td>
<td>2.0</td>
<td>4.8</td>
<td>3.9</td>
</tr>
<tr>
<td>R2400</td>
<td></td>
<td></td>
<td></td>
<td>52.8</td>
<td>30.2</td>
<td>3.8</td>
<td>2.0</td>
<td>4.7</td>
<td>4.0</td>
</tr>
</tbody>
</table>

**Numerical results**

A numerical study is carried out to investigate the main effects produced by advanced nonlinear modelling of elastomeric bearings on the dynamic response of new and retrofitted r.c. base-
isolated structures subjected to near-fault earthquakes. For this purpose, four models of HDLRBs (i.e. VEL, CM1, CM1C and CM2CR previously described) are compared, assuming three values of the nominal stiffness ratio $\alpha_{K0}$ of the isolation system (i.e. 400, 1200 and 2400). A lumped plasticity model is used to describe the inelastic behaviour of r.c frame members of the superstructure [1]. Incremental dynamic analysis (IDA) of the test structures is carried out through a series of nonlinear dynamic analyses under near-fault EQs scaled to a submultiple ($a_g$) of the corresponding PGA value. Specifically, three near-fault EQs, each recorded at three stations, are selected from the PEER database [12], in line with the design hypothesis adopted (i.e. high-risk seismic region and medium subsoil class). For each motion, time-histories of the horizontal components of acceleration are projected along the direction of the strongest observed pulse [2]. Long-duration horizontal velocity pulses characterize the Chi-Chi (Taiwan, 1999) and Northridge (California, 1994) motions. On the other hand, high values of vertical-to-horizontal peak ground acceleration ($\alpha_{PGA}=PGA_v/PGA_h$) are evident in the Imperial Valley (California, 1979) and Northridge (California, 1994) earthquakes. The main data of the selected earthquakes are shown in Table 2: i.e. earthquake; recording station; closest distance to the fault ($\Delta$); horizontal ($PGA_h$ and $PGA_h$) and vertical ($PGA_v$) peak ground accelerations; orientation of the strongest observed pulse ($\phi$), in degrees clockwise from North. The results are evaluated as maximum of those separately obtained for each set of three accelerograms.

Table 2. Main parameters of the selected near-fault ground motions [12].

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Recording station</th>
<th>$\Delta$</th>
<th>$PGA_h$</th>
<th>$PGA_h$</th>
<th>$PGA_v$</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chi-Chi (Taiwan), 1999</td>
<td>TCU068</td>
<td>0.3 km</td>
<td>0.50 g</td>
<td>0.36 g</td>
<td>0.52 g</td>
<td>144°</td>
</tr>
<tr>
<td></td>
<td>TCU059</td>
<td>17.1 km</td>
<td>0.16 g</td>
<td>0.17 g</td>
<td>0.07 g</td>
<td>45°</td>
</tr>
<tr>
<td></td>
<td>TCU065</td>
<td>0.6 km</td>
<td>0.79 g</td>
<td>0.58 g</td>
<td>0.26 g</td>
<td>113°</td>
</tr>
<tr>
<td>Imperial Valley (California), 1979</td>
<td>El Centro D.A.</td>
<td>5.1 km</td>
<td>0.35 g</td>
<td>0.48 g</td>
<td>0.77 g</td>
<td>253°</td>
</tr>
<tr>
<td></td>
<td>El Centro A.#7</td>
<td>0.6 km</td>
<td>0.34 g</td>
<td>0.47 g</td>
<td>0.58 g</td>
<td>56°</td>
</tr>
<tr>
<td></td>
<td>El Centro A.#5</td>
<td>4.0 km</td>
<td>0.53 g</td>
<td>0.38 g</td>
<td>0.59 g</td>
<td>228°</td>
</tr>
<tr>
<td>Northridge (California), 1994</td>
<td>Rinaldi</td>
<td>6.5 km</td>
<td>0.87 g</td>
<td>0.47 g</td>
<td>0.96 g</td>
<td>209°</td>
</tr>
<tr>
<td></td>
<td>Newhall F.S.</td>
<td>5.9 km</td>
<td>0.58 g</td>
<td>0.59 g</td>
<td>0.55 g</td>
<td>21°</td>
</tr>
<tr>
<td></td>
<td>Newhall W.P.C.</td>
<td>5.5 km</td>
<td>0.42 g</td>
<td>0.36 g</td>
<td>0.30 g</td>
<td>34°</td>
</tr>
</tbody>
</table>

Firstly, maximum ductility demand of beams is plotted in Fig. 3 for increasing values of the dimensionless acceleration $\alpha_{a_g}(=a_g/PGA)$, comparing results obtained from different nonlinear models of the base-isolation system for new (i.e. N.BI, Figs. 3a,c,e) and retrofitted (i.e. R.BI, Figs. 3b,d,f) test structures. Ductility demand at the end sections of the first floor and mid-span sections of the first and fifth floors is presented, assuming different values of the nominal stiffness ratio $\alpha_{K0}$ of the base-isolation system (i.e. 400, 1200 and 2400). For each structure, the IDAs are interrupted at the same final value of the dimensionless acceleration $\alpha_{a_g}(=a_g/PGA)$. As shown, the N.BI structures present ductility demand for r.c frame members of the superstructure higher than that of the R.BI ones, with an amplification at the first level in particular (Figs. 3a-3b and 3e-3f), in the case of near-fault earthquakes with significant pulse-type nature of the horizontal component, and at the top level (Figs. 3c-3f), for high values of the peak ground
acceleration ratio $\alpha_{PGA}$. This happens because the R.BI structures are demonstrate a tension reinforcement ratio of the r.c. frame members always greater than that of the corresponding N.BI structures. Moreover, the simplified VEL model of the HDLRBs is generally the most conservative, producing upper-bound values of ductility demand, contrary to the advanced model accounting also for large displacements and rotations (i.e. CM2CR), which corresponds to the lower-bound values. The insertion of cavitation and post-cavitation in tension (i.e. CM1C) does not influence the response of the superstructure, practically producing the same results of the CM1 model where only coupling of horizontal and vertical motion is considered.

![Figure 3](image)

**Figure 3.** Effects of nonlinear modelling of the base-isolation system on ductility demand of beams for new (N.BI) and retrofitted (R.BI) base-isolated test structures.
Next, maximum response parameters for isolators of the N.BI and R.BI structures are plotted in Fig. 4, assuming different values of $\alpha_{K0}$ (i.e. 400, 1200 and 2400). Thus, maximum total shear strain ($\gamma_{\text{tot, max}}$) and dimensionless axial load in compression ($P/P_{\text{cr}} \text{max}$) and tension ($P/P_{\text{cr}} \text{max}$) are plotted for the Chi-Chi (Figs. 4a,b), Imperial Valley (Figs. 4c,d) and Northridge (Figs. 4e,f) EQs, respectively. Note that the HDLRBs of the R.BI are more sensitive than those of the N.BI structures to nonlinear modelling assumptions. In particular, the advanced CM2CR model is the most conservative for the Chi-Chi and Imperial Valley EQs producing upper-bound values of $\gamma_{\text{tot, max}}$ and ($P/P_{\text{cr}} \text{max}$), respectively.

Figure 4. Effects of nonlinear modelling of the base-isolation system on response parameters of the HDLRBs for new (N.BI) and retrofitted (R.BI) base-isolated test structures.
Conclusions

A computer code is implemented to evaluate main effects of simplified and advanced nonlinear models of HDLRBs on the seismic response of new and retrofitted r.c. base-isolated structures subjected to the horizontal and vertical components of near-fault earthquakes. The nonlinear IDA of the test structures, designed for low, intermediate and high values of the nominal stiffness ratio $\alpha_{K_0}$, is carried out considering four analytical models for the HDLRBs. The N.BI structures present higher values of ductility demand for r.c. frame members of the superstructure than those obtained for the R.BI ones, while the opposite is observed for the response parameters of the base-isolation system. An upper-to-lower bound approach is proposed to account for the sensitivity of design parameters to the nonlinear modelling of the base-isolation system. In conclusion, the VEL model may govern the sizing or strengthening of the superstructure for the N.BI and R.BI structures, respectively. On the other hand, the CM2CR model may be necessary to select stiffness and strength properties of the HDLRBs accurately.

Acknowledgments

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