Critical comparison of design procedures and analysis methods for base isolated buildings

2015 NZSEE Conference

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ABSTRACT: Design codes for base isolated structures allow for the adoption of linear or non-linear analysis procedures.

Even though linear dynamic approaches should be limited to a narrow class of structures, they are often used for preliminary design. However, the use of linear effective properties of the system may not accurately account for the non-linearity of the isolation devices, leading to potentially un-conservative design, especially for large effective periods and high effective damping of the isolation system. On the other hand, non-linear time history methods may require considerable computational effort. The apparent complexity of such more rigorous approaches may have contributed to limit the use of base isolation to large and important/critical facilities, impairing its widespread use for ordinary constructions.

This paper aims to illustrate the potential inaccuracies of a linear dynamic approach, even when applied to a regular structural configuration, by comparing the differences in the predicted response on two regular moment resisting frames with alternative isolation systems, when using linear response spectrum and non-linear time history analyses. Numerical simulations under a suite of far field and near field records (including ground motions from the Christchurch Feb 2011 event) are used to identify the most relevant parameters of the isolation devices that control the overall response. The limitations of a linear dynamic approach are discussed and recommendations for further investigations to support the use of a linear elastic approach for ordinary and "simple" structures are provided.

1 INTRODUCTION

Major international codes provide rules for analysis and design of base isolated structures. Whilst no explicit limitations apply to the adoption of non-linear analyses procedures, restrictions apply to the implementation of linear analyses for the final design of the system. According to the American (ASCE 7-10) and the European (Eurocode 8, EC8) codes, linear analyses, either static or dynamic, may be employed if the isolation system can be modelled with equivalent linear visco-elastic or bilinear hysteretic behaviour. Non-linear dynamic analyses are deemed compulsory whenever the isolation system cannot be represented by an equivalent linear model, thus an adequate constitutive relationship should be formulated and implemented in the structural analysis scheme.

Linear design approaches include the equivalent static analysis and the response spectrum analysis. It is worth noting that the equivalent static analysis of base isolated structures, in practice, follows a displacement-based design approach, where the design forces are developed on the basis of the design displacement level and associated energy dissipation. Even though this approach cannot be adopted for the final design in all circumstances, is very often used in the preliminary design stages. Understanding the limitations of linear analyses would therefore be useful to appreciate the critical aspects which require attention in the final stage, even when non-linear analyses are used for design verification.

This paper presents preliminary results, as part of a more comprehensive research, on the reliability of linear procedures for the design/analysis of base isolated structures. For the sake of simplicity and following the most common practice, the study considered reinforced concrete moment resisting frame buildings only, with three types of isolation devices (High Damping Rubber Bearings, Lead Rubber Bearings and Friction Pendulum Systems) featuring different properties and design parameters.

The outcomes of the study can be expected, with some caution and reason, to apply (in qualitative and general terms) to other type of superstructures featuring similar mass and stiffness.

2 DYNAMIC ANALYSIS PROCEDURES

2.1 Limitations of equivalent linearization methods

In seismic structural codes, isolation systems can be modelled by means of force-displacement linear relationship when the structure moves as a rigid body, so higher oscillation modes are ignored. In this case the distribution of the inertia forces can be considered to be almost uniform. In case of highly non-linear isolation systems, which, as a rule, can be represented by bilinear force-displacement relationship, the seismic response should be evaluated by using a time-history analysis in order to control the influence of the higher modes.

Because the equivalent linear approach cannot characterize the isolation system based on its physical parameters, linear dynamic analyses are in general expected to be limited in their ability to capture the influence of higher modes on the overall response of buildings isolated, especially when using highly non-linear system. FEMA-274 (1997) point out: (C9.2.2.1 - "[…] *for highly damped isolation systems, the shear force distribution is nearly constant over the height of the structure, whereas for lightly damped systems this distribution is approximately triangular. The latter is indicative of response in the fundamental mode of vibration, whereas the former is indicative of higher mode response, which typically accompanied by higher accelerations in upper floors*").

There is concern that the high level of energy dissipation provides a reduction of the bearing displacements and of the shear force in the isolation system, which is an evident benefit, but also may lead to an increase of the shear forces along the height of the structure and, inevitably, of the inter-storey drifts and floors spectrum. This may be not appropriate when the intent of seismic isolation is to protect secondary systems, such as sensitive equipment. Thus, even in case of regular structural configurations, the impact of the higher modes must be checked up.

2.2 Response Spectrum Analysis

Modal linear analysis may be employed when the behaviour of the isolation system can be represented by means of equivalent properties (stiffness and damping) computed at a lateral displacement corresponding to the limit state under consideration. The effective stiffness (k_{eff}) corresponds to the secant stiffness at the design displacement and the effective damping (ξ_{eff}) of the bearing devices quantifies the energy dissipated under cyclic loads.

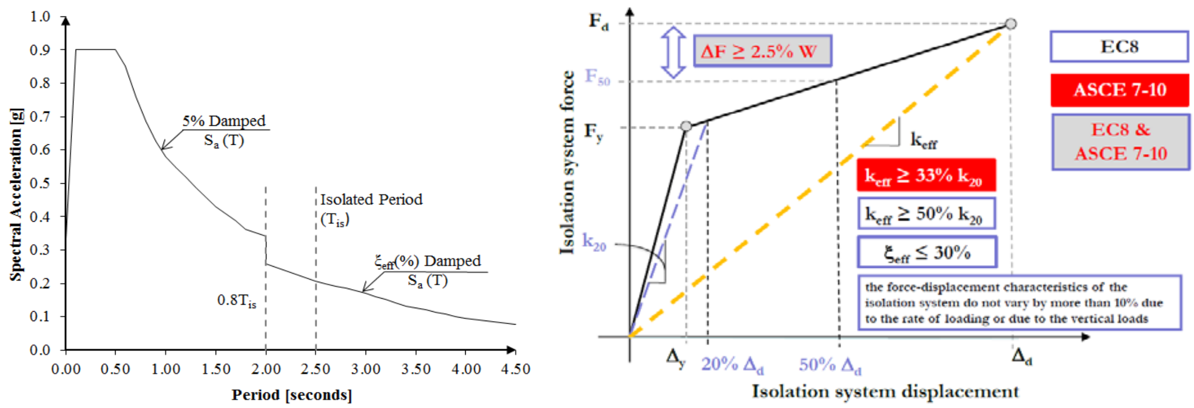


Figure 1. Acceleration damped spectral shape (a) and requirements for the adoption of a linear model for the isolators according to ASCE 7-10 and EC8 (b).

The equivalent damping associated with the isolation system response can be conveniently introduced by applying the appropriate damping coefficient in the corresponding modes (first two modes of the isolated structure). Modal damping values for higher modes are selected consistent with those that would be appropriate for response-spectrum analysis of the structure above the isolation system assuming a fixed base (Figure 1 (a)). The damping reduction factor is symbolized as "η" in EC8 and as "B factor" in ASCE 7-10. Limitations apply for the adoption of a linear model of the isolators (Fig. 1(b)).

2.3 **Response History Analysis**

If an isolation system cannot be represented by an equivalent linear model, the seismic response should be evaluated by nonlinear time history analysis (NLTHA), using a constitutive relationship of the devices which can adequately reproduce the behaviour of the isolation system in the range of deformations and velocities anticipated in the seismic design situation. A full 3D model is used with full representation of bi-directional loading and torsional response.

In ASCE 7-10, NLTHA requires the adoption of seven pairs of records scaled to the design spectrum so that the average of the SSRS spectrum of component pairs is not lower than the design value over the period range (0.2 $T_{1,DE}$ - 1.25 $T_{1,MCE}$), with T_1 equal to the effective period (T_{eff}) (at both the Design level Earthquake (DE) and the Maximum Considered Earthquake (MCE_R)) for a base isolated structure (evaluated independently at the two seismic intensity levels), as summarised in Table 1. The average of the 7 THs is used for design.

In EC8, NLTHA requires the adoption of pairs of records (minimum 3, and 7 are required to design for average outcomes) scaled to the design spectrum (Table 2) so as the average value at T = 0s is not lower than the design ground acceleration and that the minimum value of the spectrum amongst all the considered records is at least equal to 90% of the design spectrum acceleration over the period range (0.2 T_1 - 2.0T_1), with T_1 equal to T_{eff} for a base isolated structure. In 3D models, records are applied simultaneously, following the same combination rule adopted for a RSA (i.e. 100% principal direction + 30%) orthogonal component.

Table 1. Earthquake records scaling criteria – ASCE 7-10

Criteria	Note	General Remarks
$SRSS = Average_{7-pairs} = \left\{\sqrt{S_{ix}{}^2 + S_{iy}{}^2}\right\}_{0.2T_{1,DE}}^{1.25T_{1,MCE}} \geq S_d$	Calculate SRSS spectrum for each pair, and average amongst 7-pairs.	The resultant of the two components is compared against the design value.

S_{ix}: spectra for the x-component of record pair i
S_{iy}: spectra for the y-component of record pair i
S_d: design spectrum

Table 2. Earthquake records scaling criteria – EC8

Criteria	Note	General Remarks
$Average\left(S_{i0}\right) \geq PGA$	Min PGA to control low period ordinates	- 3 to 6 record pairs: design for peak demands
$Min\left(S_i\right)_{0.2T_1}^{2.0T_1} \geq 90\% S_d$	Min scatter over the period range for scaling	- 7 or more pairs: design for average values

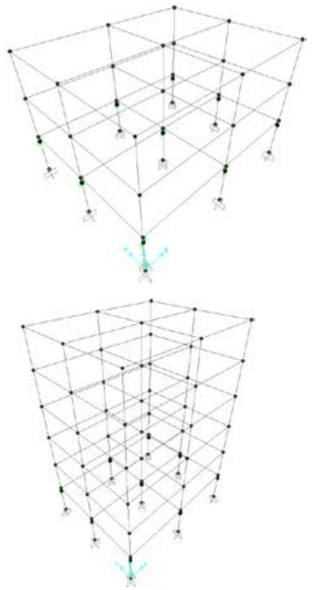
S_{i0}: spectral acceleration at 0s for the each record i
PGA: Design peak ground acceleration
T_1: Effective period
S_i: spectral acceleration for the each record i
S_d: design spectral ordinate

3 **CASE STUDIES**

Two RC framed building models (Figure 2), differing in the number of storeys (3 and 6 suspended floors respectively) and implementing three alternative Isolation System (IS) types, were considered and analysed using the program SAP2000 (v.17.1).

Independently from the type of isolation system, the bearings were centrally placed at the top of the basement columns. To account for different energy dissipation properties, a modal damping of 5% was assigned to the superstructure (assumed to respond elastically), while there was no need to make explicit the damping of the isolation system, because the hysteretic model of the device takes it into account.

According to EC8, the design of both the structure and the isolators is based on the demand obtained for the Design Earthquake (DE) level (e.g. 1/500 year or 10% probability of exceedance within a design life of 50 years). The design level spectrum is assumed equal to that defined in NZS 1170.5 for an IL2 structure (e.g. an Office building), considering soil class D and a hazard factor of Z=0.3 (e.g. Christchurch).



3-Storey Moment Resisting Frame:

- Number of floors = 3
- Interstorey height = 3.6m
- Longitudinal span length: 7.5m x 2
- Transversal span length: 6.0m x 2
- Seismic weight: 6,067 kN (1st: 39%, 2nd: 33%, 3nd: 28%)
- Equivalent fixed base period = 0.44 sec

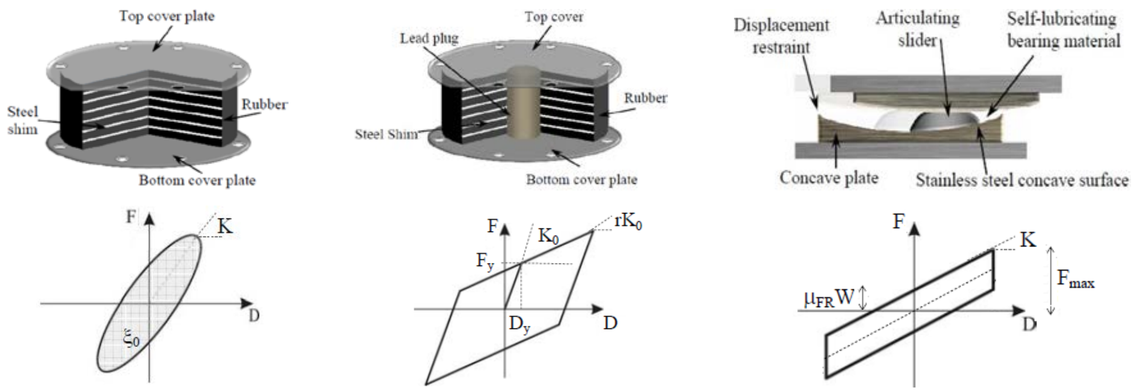
6-Storey Moment Resisting Frame:

- Number of floors = 6
- Interstorey height = 3.6m
- Longitudinal span length: 7.5m x 2
- Transversal span length: 6.0m x 2
- Seismic weight: 12,122 kN (1st: 19%, 2nd-5nd: 17%, 6nd: 13%)
- Equivalent fixed base period = 1.00 sec

Figure 2. Case Study buildings: summary of geometric layout.

3.1 Base Isolation Systems

Three different types of base isolation devices were investigated, namely High Damping Rubber Bearings (HDRB), Lead Rubber Bearings (LRB), both combined with low-friction flat sliders, and Friction Pendulum Bearings (FPB). HDRBs can be modelled with a visco-elastic behaviour (Kelly, 1993), an elasto-plastic with hardening model describes the cyclic behaviour of LRBs (Skinner et al. 1993), whilst a rigid-plastic with hardening behaviour can be used to model FPBs (Al-Hussaini et al. 1994).



(a) (b) (c)

Figure 3. Modelling cyclic behaviour of HDRB (a), LRB (b) and FPB (c).

The following items should be checked for the isolation system in the design procedure:

- The yield strength of the isolation system should be greater than the wind load.
- No tension is allowed in the isolators at the design displacement.
- The isolation system should not collapse at the maximum design displacement.

3.1.1 *High Damping Rubber Bearings (HDRB)*

For visco-elastic Isolation Systems, it can be assumed that the damping ratio (ξ_{IS}) does not depend on the design IS displacement (Δ_{IS}). This implies a great flexibility in the selection of the design IS displacement.

HDRB isolators can always be modelled as linear equivalent by expressing their characteristics in terms of horizontal effective stiffness (k_{eff}) and equivalent viscous damping (ξ_{eff}).

Table 3. 3-storey MRF: Reference design parameters for High Damping Rubber Bearings (HDRBs) (Soft compound rubber (S) with modulus of elasticity G=0.4 N/mm^2 and thickness t_e=176 mm)

Design displacement	Isolation period	HDRB's – Sliders	Effective stiffness	Effective damping	Vertical stiffness	Rubber Diameter	Total Height
340 mm	2.47 sec	6-3	0.64 kN/mm	10%	776 kN/mm	600 mm	329 mm

Table 4. 6-storey MRF: Reference design parameters for High Damping Rubber Bearings (HDRBs) (Normal compound rubber (N) with modulus of elasticity G=0.8 N/mm^2 and thickness t_e=176 mm)

Design displacement	Isolation period	HDRB's – Sliders	Effective stiffness	Effective damping	Vertical stiffness	Rubber Diameter	Total Height
335 mm	2.44 sec	6-3	1.29 kN/mm	10%	1154 kN/mm	600 mm	329 mm

HDRBs were combined with low-friction flat sliders which allow relative movements with a relatively low transmission of forces (friction coefficient $\mu_{FR} \cong 3\%$).

3.1.2 *Lead Rubber Bearings (LRB)*

For elasto-plastic Isolation Systems, the damping ratio (ξ_{IS}) depends on the design IS displacement (Δ_{IS}) through the post-yield hardening ratio and lead ductility ratio which are the main design parameters. As a consequence, an iterative design process is required to get the horizontal effective stiffness (k_{eff}) and effective damping (ξ_{eff}).

Table 5. 3-storey MRF: Reference design parameters for Lead Rubber Bearings (LRBs) (yield displacement d_y=10mm; yielding force F_y=50kN; post-yielding stiffness K_{el}/K_{pl} = 0.10; F_y/W=5%)

Design displacement	Isolation period	LRB's – Sliders	Effective stiffness	Effective damping	Vertical stiffness	Rubber Diameter	Total Height
260 mm	2.40 sec	6-3	0.67 kN/mm	16%	932 kN/mm	500 mm	310 mm

Table 6. 6-storey MRF: Reference design parameters for Lead Rubber Bearings (LRBs) (yield displacement d_y=10mm; yielding force F_y=100kN; post-yielding stiffness K_{el}/K_{pl} = 0.10; F_y/W=5%)

Design displacement	Isolation period	LRB's – Sliders	Effective stiffness	Effective damping	Vertical stiffness	Rubber Diameter	Total Height
250 mm	2.37 sec	6-3	1.35 kN/mm	16%	1232 kN/mm	500 mm	377 mm

The selected types of device meet the requirements for the adoption of a linear model. Also in this case, LRBs were combined with low-friction flat sliders (friction coefficient $\mu_{FR} \cong 3\%$).

3.1.3 *Friction Pendulum Bearings (FPB)*

For Friction Pendulum Isolation Systems, the damping ratio (ξ_{IS}) depends on the friction coefficient (μ_{FR}), design isolator displacement (Δ_{IS}) and period of the base-isolated building (T_{IS}) which is completely defined by the radius of curvature (R). As a consequence, an iterative design process is required, since input and output are mutually correlated through T_{IS}.

Table 7. Reference design parameters for Friction Pendulum Bearings (FPBs)

Design displacement	Isolation period	Radius of curvature	Friction coefficient	Effective damping
325 mm	2.66 sec	2500 mm	5.5%	20%

The selected type of device meets the requirements for the adoption of a linear model.

3.2 **Ground Motion Selection**

Following the recommendations of NEHRP (2011) for seismically isolated structures, a set of seven (pairs of horizontal acceleration components) strong motion earthquake records were selected such that the average spectral response of each set is comparable to the design spectrum at the periods of interest. The suite was developed according to the New Zealand Hazard Model specifically developed by Stirling et al. (2002, 2012) for the Canterbury region and consists of:

- four unscaled near-fault (pulse-like) ground motions (Table 8) from earthquakes of M_w6.25-to-6.75 recorded at distances from 0-to-20km (e.g., near-fault ground motions with high amplitude and long period velocity pulses characteristic of forward rupture directivity). Records from the M_w6.3 2011 (Feb, 22) Christchurch Earthquake were included in this set.
- three scaled far-field (non-pulse-like) ground motions (Table 9) from earthquakes of M_w7-to-7.5 recorded at distances less than 45km (e.g., ground motions with high frequency content). These records were scaled according to the recommendations of NZS 1170.5:2004 (Table 10).

The maximum displacement of the isolation system, the base shear and storey shear at any level, were calculated as average of the 7 THs.

Table 8. Unscaled near-fault ground motions.

Event	Year	Mw	Station	PEER No.
Imperial Valley-06	1979	6.53	El Centro Differential Array	184
Christchurch	2011	6.3	CBGS	-
Christchurch	2011	6.3	CHHC	-
Christchurch	2011	6.3	CCCC	-

Table 9. Scaled ground motions with high frequency content (scaling factor, k_1, between parenthesis).

Event	Year	Mw	Station	PEER No.
Cape Mendocino (2.1)	1992	7.01	Eureka – Myrtle & West	826
Landers (3.90)	1992	7.28	North Palm Springs	882
Hector Mine (2.44)	1999	7.13	Amboy	1762

Table 9. Earthquake records scaling criteria – NZ code.

Criteria	Note	General Remarks
$\begin{cases} Average_{7-pairs}\left(S_{ix}\right)_{0.4T_1}^{1.3T_1} \geq S_d \\ Average_{7-pairs}\left(S_{iy}\right)_{0.4T_1}^{1.3T_1} \geq S_d \end{cases}$	Calculate spectrum for each pair, and average amongst 7-pairs.	The average of each component is compared against the design value.

S_{ix}: spectra for the x-component of record pair i
S_{iy}: spectra for the y-component of record pair i
S_d: design spectrum
T_1: Effective period

Figure 4 shows the comparison of the 5% elastic displacement design spectrum for Christchurch according to NZS 1170.5 (2004) with the 5% elastic response spectra for each ground motion as well as their average. The principal or stronger component of each record was defined as the component that has the larger spectral response at periods of interest.

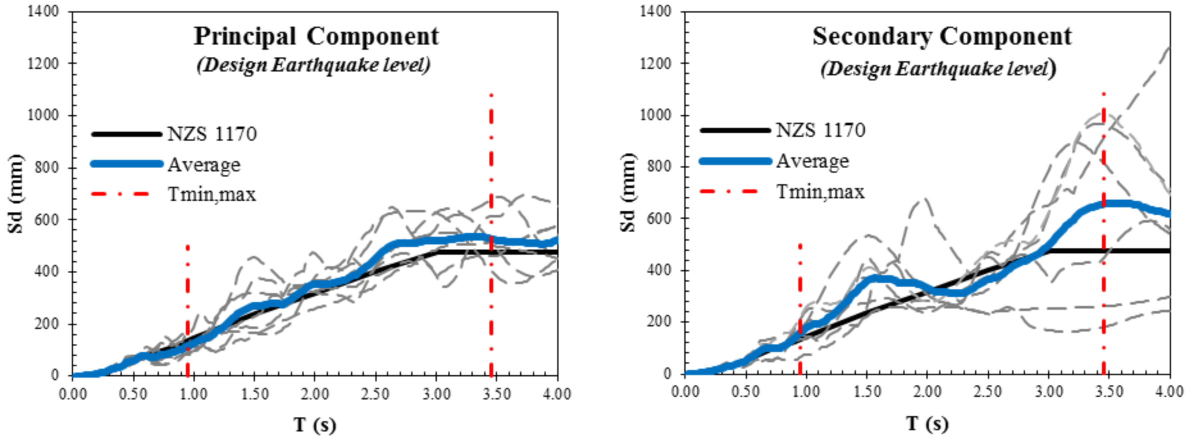


Figure 4. Comparison of selected ground motions with the NZS 1170.5 (2004) elastic displacement response spectra (500-year return period).

3.3 Summary of Results

From the results obtained for these simple case studies (Figs. 5 and 6) it can be observed that:

- The distribution of the inertia forces from RSA, obtained as the CQC of the individual modal responses, can be considered to be almost uniform for all the isolation systems, namely proportional to the storey mass, even for tall base-isolated buildings.
- NLTHA provides distribution of accelerations along the height of the structure which are almost uniform for HDRBs, while, for both LRBs and FPBs, some amplification will take place when the number of storeys increases (the highest values in case of FPBs in upper floors). It seems that the high level of energy dissipated, for LRBs by plastic flow of the lead core and for FPBs by sliding on a curved frictional surface, provides significant non-linearity for tall superstructures.
- The value of the maximum displacement of the isolation system/interface and the overall displacement profile along the height of the structure predicted with both analysis approaches are overall in reasonably good agreement. However, the average drifts resulting from time-history analyses tended to be higher than the RSA values (up to 30% for FPBs), especially at the mid-height floor levels. This can be related to higher mode effects, due to the non-linearity of the isolation systems, which are inadequately represented by elastic analyses.
- It is worth noting that the calculated inter-storey drift demands from RSA do not account for any modification factor, as for example required in NZS1170.5:2004 - Cl 7.3.1.

- The effects of the higher modes are more noticeable when the number of storeys increases. The uniform acceleration profile produced from RSA seems less appropriate and possibly not conservative to predict the storey shear force (differences in the results ranging from a minimum of 15% for HDRBs to a maximum of 25% for FPBs in the upper floors). The inertia force distribution defines the overturning moments on a structure and the distributions from the response spectrum analysis will produce a smaller overturning moment than the distributions from the time history analysis. This could lead to an un-conservative design for both the structural (e.g. higher than expected internal actions and ductility demand and in the structural elements) and the non-structural components (e.g. higher than expected floor accelerations and drifts).

The agreement between RSA and NLTHA is generally good. The main reason for RSA underestimation may be related to the use of a single effective stiffness which ignores that, for bilinear systems, the initial elastic stiffness will be more highly coupled with the structural modes than the post-yielding stiffness; therefore the dynamic response prior to the activation of the isolators may govern the design of the structure.

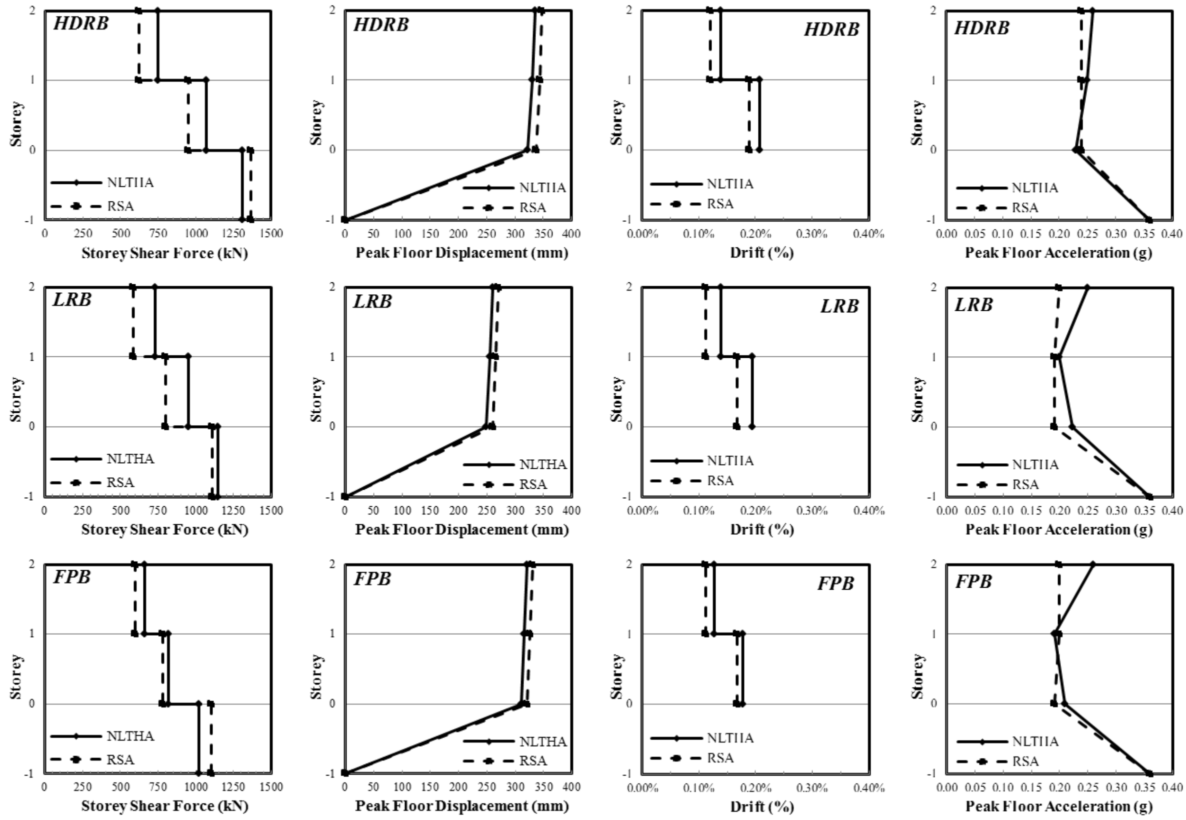


Figure 5. 3-Storey MRF – RSA vs NLTHA: storey shear forces, peak floor accelerations, inter-storey drifts and peak floor displacements.

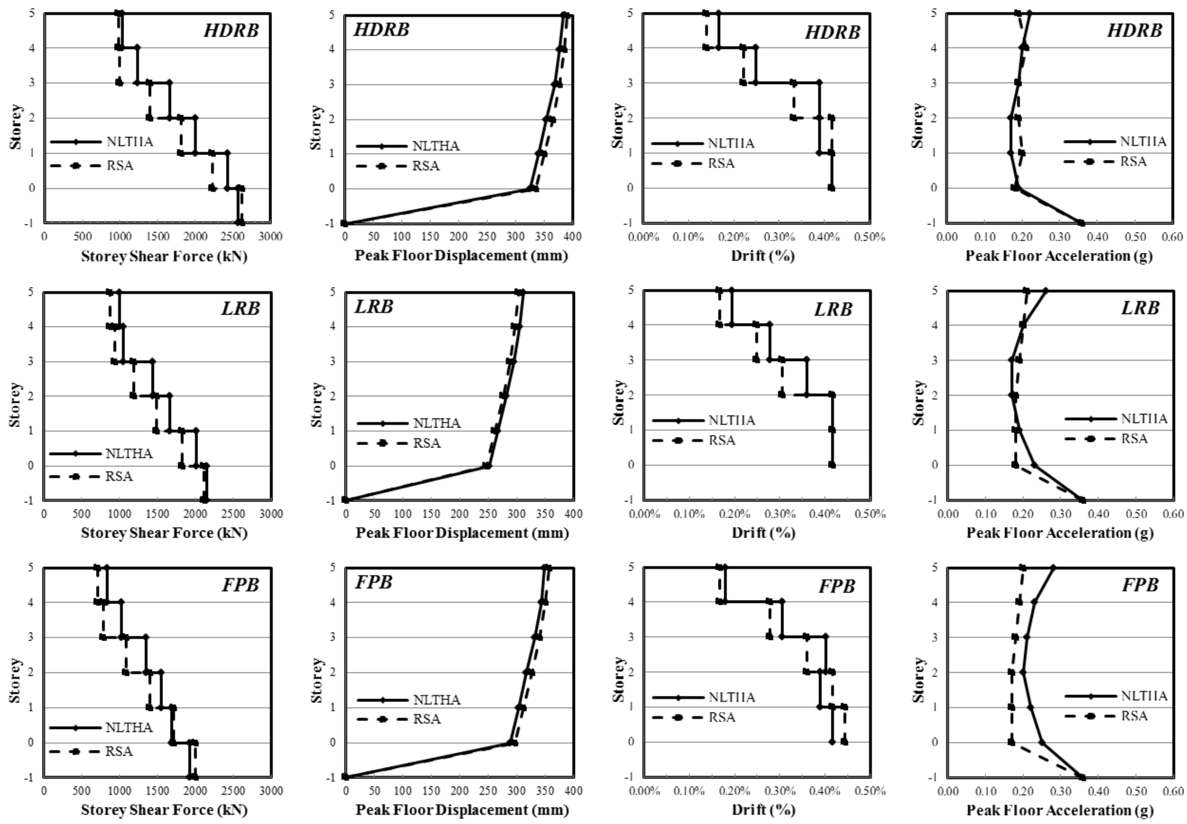


Figure 6. 6-Storey MRF – RSA vs NLTHA: storey shear forces, peak floor accelerations, inter-storey drifts and peak floor displacements.

4 RECOMMENDATIONS FOR FUTURE INVESTIGATIONS

Although this preliminary study investigates the limitations of linear dynamic approaches for base-isolated buildings, there are still many factors and uncertainties that need to be addressed, such as the following:

- The hysteretic behaviour of isolation systems is assumed to be bilinear. Other hysteretic models may be also considered.
- Potential torsional effects of 3-Dimensional base-isolated buildings need to be investigated.
- The inelastic behaviour of the superstructure should be investigated to define an appropriate value of the strength reduction factor to account for ductile behaviour.
- Vertical ground motions effects should also be examined, which could cause the over-turning of base-isolated buildings. Specifically, for friction pendulum bearings, normal force during the earthquake has significant effect on the friction coefficient, thereby influences the global performance of isolation system.
- How to apply the scaling procedure for earthquake ground motions so that the results from response spectral analysis and time history analysis are comparable.
- How to properly determine the damping reduction factor to perform linear analysis could be another interesting research topic.

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