16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1468

Registration Code: S-X1464673591

A MODIFIED RESPONSE SPECTRUM ANALYSIS PROCEDURE TO DETERMINE NONLINEAR SEISMIC DEMANDS OF HIGH-RISE BUILDINGS WITH SHEAR WALLS

F. Najam(1) and P. Warnitchai(2)

(1) *PhD Student, Asian Institute of Technology (AIT), Thailand, Email: fawad.ahmed.najam@ait.ac.th*
(2) *Professor, Asian Institute of Technology (AIT), Thailand, Email: pennung.ait@gmail.com*

Abstract

The standard Response Spectrum Analysis (RSA) procedure prescribed in various codes and design guidelines is commonly used by practicing engineers to determine seismic demands for structural design purpose. In this procedure the elastic force demands of all significant vibration modes are first combined and then reduced by a response modification factor (R) to get the inelastic design demands. Recent studies, however, have shown that this standard RSA procedure significantly underestimates the true inelastic demands of high-rise buildings as higher vibration modes tend to remain elastic (or undergo very little nonlinearity), and therefore it may not be appropriate to reduce their demand contribution by the same factor. In this study, a modified RSA procedure based on equivalent linearization concept is presented. The underlying assumptions are that nonlinear seismic demands can be approximately obtained by summing up individual modal responses, and that the responses of each individual vibration mode can be approximately represented by those of an equivalent linear SDF system. Using three high-rise buildings with reinforced concrete shear walls (20-, 33- and 44-story high), the accuracy of this procedure and the standard RSA procedure are examined. In this examination, the inelastic demands computed by the Nonlinear Response History Analysis (NLRHA) procedure are used as benchmark. The modified RSA procedure is found to provide a reasonable degree of accuracy for all case study buildings under different input ground motions.

Keywords: Response Spectrum Analysis; Response Modification Factor; Nonlinear Model; RC Shear Wall; High-rise Buildings

1. Introduction

Over last few decades, structural design against earthquakes has passed through a continuous process of evolution. The story which started from a simple mass-proportional lateral load resisted by elastic action has now evolved in to an explicit consideration of design earthquakes applied to a detailed nonlinear finite-element model. Exponential growth in computational power in recent years is continuously narrowing the industry-academia gap by providing the cutting-edge research and technology to practicing engineers at their doorstep. As a result, structural designers nowadays are equipped with far more aids and tools compared to a couple of decades ago. Moreover, recent advancements in nonlinear modeling techniques have also opened a whole new research area dealing with constructing computer models with close-to-real behaviors. With such a range of options available, the choice of modeling scheme and analysis procedure for design decision-making often becomes a matter of *"the more the sweat; the more the reward"* for designer. However, this is not a complete depiction of this story. If on one side, these advancements are bringing more sophistication to design process (in terms of better structural idealization and faster numerical solvers), they are also making the process complex, extra skill-demanding, and sometimes unnecessarily intricate. Practicing engineers are always interested in simple and conceptually elegant procedures providing reasonably accurate estimates in lesser time and effort. For example, for a high-rise building project, setting up a full nonlinear structural model—sophisticated enough to capture all important aspects of material and component nonlinearity—may be an onerous task compared to linear elastic model. Nonlinear modeling not only requires great expertise and detailed insight of various complex interactions and phenomena (associated with individual inelastic components), but also demands significant computational effort and resources. Moreover, latest analysis guidelines require to use a large number of ground motions records representing the anticipated seismic hazard at the building site. The process of selecting representative ground motions, performing Nonlinear Dynamic Procedure (NDP) and post-processing of results may cost a significant amount of time. Also, an ordinary design office may not have necessary expertise and resources to undergo this complete process for each project. For most practical cases, the linear elastic analysis may serve the purpose of estimating design demands within their required degree of accuracy.

The most commonly used analysis tool to determine design forces and displacement demands is the Response Spectrum Analysis (RSA) procedure. It is based on the idea that vibration of a linear elastic system can be seen as a superposition of few significantly contributing vibration modes, resulting in useful physical insight of structural response. Although the development of fast numerical solvers, user-friendly software, and significant decrease in computational cost over last two decades have resulted in a number of new analysis procedures, RSA is still the most widely applied procedure, owing to its practical convenience. As prescribed in various codes and design guidelines, it provides a convenient way to determine elastic demands against a specified seismic hazard represented by a 5% damped acceleration response spectrum. First, the eigen-value analysis of linear elastic model is performed to determine the structural natural periods and vibration mode shapes. The spectral acceleration corresponding to few significant vibration modes are then converted in to equivalent force demands, and the demands are combined using a suitable modal combination rule (usually SRSS, if the natural time periods are well separated). Finally, considering the expected inelastic response of structure and over-strength, the design forces are determined by reducing the elastic force demands by a response modification factor, *R* (ASCE 7-05 [1]) or a behavior factor, q (EC 8[2]). The value of the factor depends upon the structural configuration and type of lateral load-resisting system.

2. Background and Motivation

The use of response modification factor (*R*), as recommended in the standard RSA procedure, is equivalent to reducing the demand contribution of each vibration mode with the same factor. In recent years, various studies based on modal separation of dynamic response have shown that each mode does not experience the same level of nonlinearity under a ground motion, and thus reducing the force demands of all modes by the same reduction factor may result in a significant underestimation of demands. The inelastic action mostly occurs under the response of fundamental vibration mode, while higher modes tend to undergo lower levels of nonlinearity. Various studies have identified this discrepancy in standard RSA and attempted to provide viable solutions ranging from different response modification factors for each mode (e.g. [3]) to modifying the modal properties for higher modes (e.g. [4]). Priestley and Amaris [5] attempted to address this issue for cantilever wall structures and showed that a fairly accurate estimate of nonlinear force demands can be obtained by combining the inelastic shear (corresponding to formation of a plastic hinge) from first mode with unreduced contributions from higher vibration modes (assuming them elastic). Based on the assumptions that inelastic action only limits the first-mode response and higher modes remain elastic, they proposed a Modified Modal Superposition (MMS) method recommending to apply a response modification factor only to first vibration mode. However, it was later observed that for frame structures, the response of higher modes may also experience inelasticity [6], and MMS may result in significantly overestimating story shears and other force demands in such cases. Sullivan et al. [4] proposed a new modal analysis approach to determine inelastic (transitory) modal properties for RC frame-wall buildings and showed that modal superposition of first-mode nonlinear response with higher-mode response determined using transitory modal properties provides significantly improved predictions of maximum base shear (compared to elastic higher modes at initial modal properties, as proposed in MMS).

More recently, Ahmed and Warnitchai [7] studied the nonlinear response contributions of individual vibration modes by using the Uncoupled Modal Response History Analysis (UMRHA) procedure—originally formulated by Chopra and Goel [8]. It was shown that the complex nonlinear responses (floor displacements, inter-story drifts, story shears, story moments, and floor accelerations) of high-rise RC buildings can be approximately decomposed into contributions from a few nonlinear single-degree-of-freedom (SDF) systems. The UMRHA procedure can be considered as a natural extension of classical modal analysis to inelastic systems. Although the theoretical basis for modal analysis does not remain valid when a system enters into inelastic range, it is assumed that its nonlinear dynamic response can still be approximately described using the vibration mode shapes of the corresponding elastic system. This assumption allows us to develop an uncoupled formulation of dynamic equations of motion similar to the classical modal analysis, with a difference that the response of each vibration mode is now represented by that of a nonlinear SDF system. The governing equation of motion for any i^{th} vibration mode is as follows [8].

$$\ddot{D}_i + 2\xi_i\omega_i\dot{D}_i + F_{si}(D_i, \dot{D}_i)/L_i = -\ddot{x}_g(t) \quad (1)$$

Where $D_i(t)$ is the displacement response history of SDF system when subjected to ground acceleration $\ddot{x}_g(t)$, while ξ_i and ω_i are the initial viscous damping and initial natural (circular) frequency of that mode, respectively. L_i is defined as $\phi_i^t\boldsymbol{M}$ (where ϕ_i is the i^{th} vibration mode shape vector and $\boldsymbol{M}$ is the mass matrix). The solution of this equation requires the knowledge of a nonlinear force-deformation relationship denoted by $F_{si}(D_i, \dot{D}_i)$. Ideally, this relationship is expected to capture the complete inelastic cyclic behavior of the i^{th} vibration mode. Depending upon key structural characteristics and type of lateral load-resisting system, a suitable choice for $F_{si}(D_i, \dot{D}_i)$ may range from simple hysteretic models governed by few controlling parameters (e.g. bilinear, elastic-perfectly plastic) to relatively more detailed behaviors (e.g. stiffness and strength degrading models). The actual cyclic behavior associated with any vibration mode of a multi-degree-of-freedom (MDF) system can be determined by subjecting its detailed nonlinear structural model to gradually increasing and direction-reversing inertial force vector of that mode. The structural response under inertial force pattern $\boldsymbol{s}_i^* = \boldsymbol{M}\phi_i$ is expected to be dominated primarily by the i^{th} vibration mode, and therefore the resulting cyclic response can be mapped to a suitable hysteretic behavior to model the equivalent i^{th}-mode nonlinear SDF system. The uncoupled modal response histories from all significant vibration modes are converted to the corresponding responses of actual MDF system and are directly combined in time domain to get the overall dynamic responses (which are shown to match-well with those computed by the NLRHA procedure for real RC shear wall buildings [9, 10]). This also eliminates the need of using any modal combination rule as required in RSA. Using the UMRHA procedure, Ahmed and Warnitchai [7] further confirmed that each contributing nonlinear SDF experiences a different level of inelasticity under the same ground motion and therefore reducing each modal force demands with the same response modification factor as in standard RSA is not appropriate. It was also observed that a significant nonlinearity may arise from tensile cracking of RC shear walls, prior to the yielding of reinforcing steel bars in shear walls [7, 8 and 10]. In such cases, the ductility ratio μ—the ratio of maximum to yield displacement—may not be an appropriate indicator of nonlinear structural state. Also, for the purpose of constructing equivalent SDF systems in UMRHA, a typically assumed bilinear idealization for capacity curve may not be suitable in such cases, with no well-defined yield point. On the other hand, the roof drift ratio (the ratio of peak roof displacement to total height of building) is found to be a better and physically meaningful deformation measure to assess the extent of nonlinearity in a structure.

3. Formulation and Basic Concept of Modified Response Spectrum Analysis (MRSA)

The basic assumption of UMRHA—that a complex nonlinear structure can be approximately represented by a few nonlinear modal SDF systems—further leads to an idea that properly-tuned *"equivalent linear"* SDF systems can represent these nonlinear modal SDF systems. The underlying concept is that a fairly accurate estimate of inelastic response can be obtained by analyzing a hypothetical equivalent elastic system with modified properties. This approach—referred in literature as Equivalent Linearization, EL—can be applied for all significant vibration modes by converting their representative nonlinear SDFs into equivalent elastic counterparts. Resultantly, instead of direct scaling-down of elastic force demands (as in standard RSA), the modal superposition of these *"equivalent linear"* SDF systems is expected to provide fairly accurate estimates of nonlinear demands, provided its equivalent properties are the best representative of close-to-real nonlinear structural state. This scheme—referred onwards as Modified Response Spectrum Analysis, MRSA—although involves an additional step (i.e. estimation of equivalent linear properties), offers a significant reduction in effort and time compared to nonlinear dynamic analysis of an inelastic model. Considering it as a modification applied over individual elastic modal response, MRSA can be performed within already implemented standard RSA framework in various commercial software for linear elastic analysis. Fig. 1 presents the basic concept of the proposed MRSA, as guided by UMRHA.

The optimum definition of an *"equivalent linear"* system has remained a subject of immense research during the past few decades, as the accuracy of any EL procedure primarily depends on how well the equivalent properties (equivalent elastic period T_{eq}, and equivalent viscous damping ξ_{eq}) represent the actual nonlinear structure. As both phenomena (i.e. period elongation and additional hysteretic damping due to inelasticity) simultaneously affect the displacement response in an opposing manner, their individual accuracies with respect to the actual nonlinear system depend on each other. Often, the choice for equivalent period is made first and additional damping, required to achieve optimum response calibration between the two systems, is derived. The most common way to estimate the amount of hysteretic damping is by using the equal-energy principle—first introduced by Jacobsen [11]. In this approach, the dissipated energy per cycle (against a sinusoidal excitation) of the original nonlinear SDF system is equated to the energy dissipated by viscous damping of an equivalent linear system. Based on this equality, the equivalent hysteretic damping ratio (ξ_h) can be determined as follows.

$$\xi_h = \frac{1}{4\pi}\frac{E_D(x)}{E_{so}(x)} \tag{1}$$

where $E_D(x)$ is the energy dissipated per cycle in the nonlinear system at an amplitude x (and is determined as the area enclosed by hysteretic loops), and $E_{so}(x)$ is the strain energy associated with the equivalent linear system (and is calculated as the triangular area under the linear force-deformation behavior). It is important to note that $E_{so}(x)$ is dependent on the equivalent linear stiffness (K_e). This means that a choice of higher equivalent elastic stiffness will result in a lower equivalent hysteretic damping ratio, and conversely a lower assumed equivalent stiffness will provide a higher ξ_h in order to maintain equal-energy dissipation.

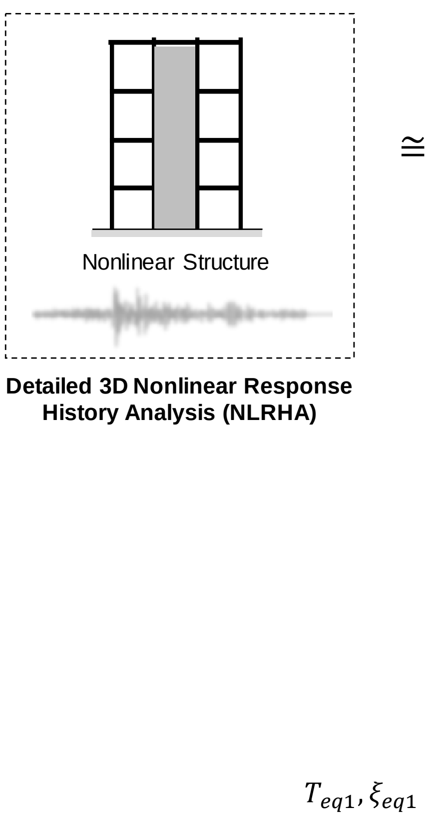


Fig. 1 – Basic concept of the proposed Modified Response Spectrum Analysis (MRSA) Procedure

Historically, equivalent linear methods can be classified into two main groups based on the adopted definition of the equivalent elastic period (T_{eq}), or equivalent elastic stiffness (K_{eq}). In the first group, the equivalent period (T_{eq}) is determined from the secant stiffness at the maximum nonlinear displacement. This definition of equivalent stiffness was first proposed by Rosenblueth and Herrera [12] and later widely adopted by several studies. Considering the capacity curve of any structure, the geometric importance and associated physical meaning of this choice is obvious. The capacity spectrum method of ATC 40 [13] also adopts the secant period at the performance point corresponding to the intersection of capacity curve and reduced demand spectrum (based on the amount of equivalent viscous damping, ξ_{eq}—determined as the sum of inherent viscous damping ξ_i and hysteretic damping ξ_h). The expression for ξ_h was derived from the equal-energy principle (equation 1) applied to idealized bilinear behavior. Fig. 2 shows a basic concept to convert a SDF system with a nonlinear force-deformation relationship into an equivalent linear system with T_{eq}, set as the secant period at maximum nonlinear amplitude, and corresponding equivalent hysteretic damping (ξ_h) from equation (1).

The second group, on the other hand, does not define the equivalent elastic period based on the secant stiffness. The underlying idea of most methods in this group is that an equivalent linear stiffness lying between the initial stiffness and the secant stiffness (at maximum amplitude of nonlinear system) can provide better response matching with that of nonlinear system. They are based on the consideration that the nonlinear system's characteristics at the maximum response, which is occurring only for an instant, may not represent the characteristics of an equivalent linear system. The secant stiffness corresponding to maximum amplitude is usually considered as an upper bound softening stiffness in these methods. The equivalent linear properties (T_{eq} and ξ_{eq}) are generally determined based on iterative analysis results from a large number of nonlinear SDFs modeled with different hysteretic behaviors and subjected to a large number of real ground motion records. The optimum pair of T_{eq} and ξ_{eq} in each case is identified by minimizing an error index representing the difference among responses of nonlinear and equivalent linear systems. The final proposed relationships are generally developed by optimal fitting or statistical analysis of obtained data. It is important to note that such iterative techniques to identify the best combination of T_{eq} and ξ_{eq} may sometimes result in multiple solutions, and the final output may be sensitive to the selected iterative scheme, error index, and characteristics of ground motions. Being based on empirical response calibration, these procedures may also lack theoretical rigor and may result in inaccurate estimates of period elongation and additional damping. A comprehensive review and comparison of various EL methods can be found in Liu et al. [14]; and Lin and Miranda [15].

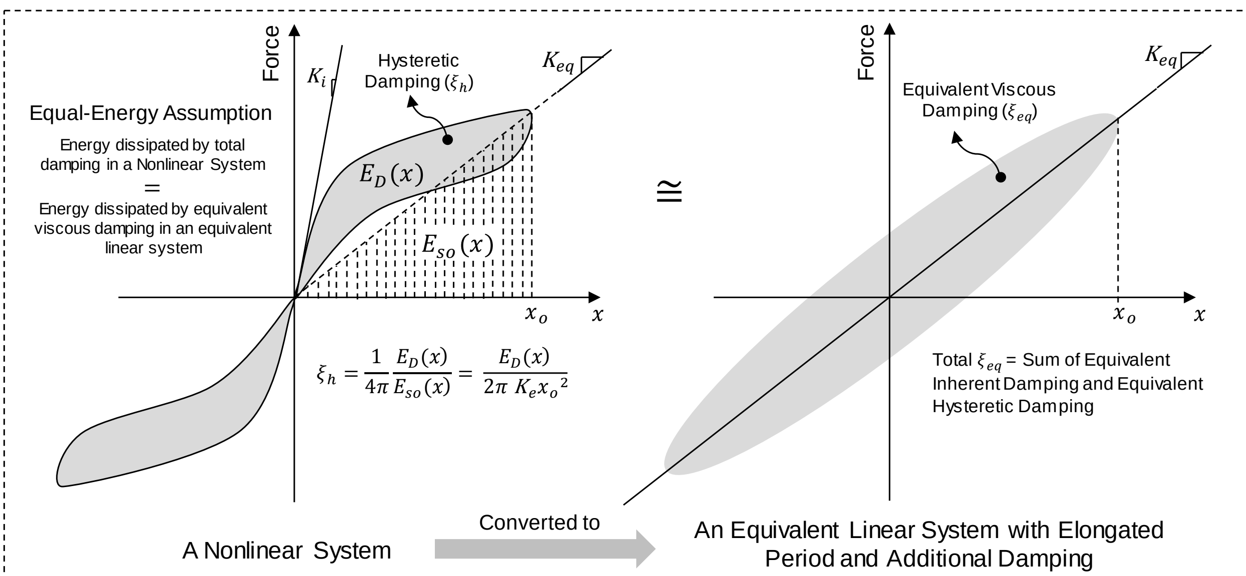


Fig. 2 – Representing a nonlinear system with an equivalent linear system having elongated period (T_{sec} at maximum amplitude) and additional damping (ξ_h determined from equal-energy dissipation principle)

4. Summary of Modified Response Spectrum Analysis (MRSA)

Generally, the equivalent linear properties are derived as a function of the ductility ratio (μ—the ratio of maximum to yield displacement), which is the most widely accepted measure of inelasticity. However, as mentioned in section 2, the cyclic response of high-rise RC buildings may exhibit significant nonlinearity well before the actual yielding of steel reinforcement. Cracking of shear walls by the lateral seismic loads may lead to significant stiffness softening at displacement much lower than that at the yield point. In such cases, the roof drift ratio (the ratio of peak roof displacement to total height of building) is found to be a more appropriate deformation measure to represent the nonlinear state of structure, as compared to the ductility ratio (μ). In this study, the equivalent linear properties are determined as a function of roof drift ratio. These relationships (relating T_{eq} and ξ_{eq} with roof drift ratio) are required as a prerequisite for MRSA, and can be generalized as design aids for convenient and wider applicability. As the expected roof drift ratio is not known at the start of design process, an iterative procedure is proposed in MRSA to estimate the equivalent linear properties. A step-by-step procedure for MRSA is proposed as follows.

a) Using a linear elastic model, determine the initial modal properties (initial natural periods (T_i), vibration mode shapes, modal participation factors).
b) From the displacement response spectrum (developed for the initial viscous damping ξ_i), determine the spectral displacement (S_D) corresponding to the mode's initial period (T_i), and convert it into the initial elastic roof drift ratio.
c) The determination of equivalent linear properties requires a few iterations over the roof drift ratio, starting from the initial elastic roof drift ratio determined in the previous step. From developed T_{eq} *vs. Roof Drift Ratio* and ξ_{eq} *vs. Roof Drift Ratio* relationships (e.g. Figure 5), determine the elongated period (T_{eq}) and increased damping (ξ_{eq}) against the initial elastic roof drift. Then, from a new displacement spectrum corresponding to the increased damping (ξ_{eq}), again read the spectral displacement at elongated trial period (T_{eq}), and convert it into roof drift ratio. Repeat this process until the starting value of roof drift ratio for an iteration converges to the resulting roof drift ratio at equivalent linear properties. Those T_{eq} and ξ_{eq} corresponding to the final iteration are used further in next steps.
d) Repeat the steps (b) and (c) for other significant vibration modes. A convenient assumption is to assume no period elongation and additional damping for higher modes, owing to their tendency to remain elastic, or undergo little nonlinearity. This assumption will reduce the proposed MRSA to MMS [5]. However, for high-rise structures expected to undergo significant nonlinearity in their higher vibration modes, the elastic-higher-modes assumption may result in overestimation of force demands, and therefore equivalent linearization for higher modes should also be considered.
e) With the final T_{eq} and ξ_{eq} of each vibration mode, the standard response spectrum procedure is carried out. The spectral acceleration (S_A) corresponding to equivalent linear properties is used instead of initial S_A (at T_i and ξ_i). Alternatively, for cases where the analysis is carried out on a commercial software, the results of standard RSA procedure for each mode can be modified by the ratio of S_A at equivalent linear properties to S_A at initial properties. Most commercial software provides the facility to apply scale factors to load cases prior to analysis.

5. Case Study Buildings and Ground Motions

The proposed analysis scheme is tested by using three existing case study high-rise buildings in Bangkok, the capital city of Thailand. These buildings (20-, 33- and 44-story high) were selected after a detailed review of more than 200 existing buildings, primarily as part of a project funded by Bangkok Metropolitan Administration (BMA) aiming to evaluate the seismic risk and potential losses in Bangkok. The primary gravity-load-carrying system in these buildings is RC slab-column frames, while the lateral load is mainly resisted by a number of RC walls and cores. All case study buildings have masonry infill walls and have mat foundation resting on piles. Salient structural and architectural features of the selected buildings are given in Table 1. For detailed nonlinear response history analysis (NLRHA) and cyclic modal pushover analysis, full 3D inelastic finite element models of case study buildings were created in Perform 3D [16]. All elements of RC shear walls were divided in to a large number of vertical nonlinear concrete and steel fibers to simulate the combined axial-flexural behavior. Concrete fibers were modeled using Mander's unconfined stress strain model [17] approximated by a tri-linear envelope. Steel fibers were modeled with a non-degrading type bilinear hysteretic model including strain hardening. Lumped fiber modeling was used for RC columns with steel and concrete fibers at both ends (having a length equal to effective depth of column cross-section) with elastic frame element in-between. Concrete slabs were modeled using elastic thin shell elements. Masonry infill walls were also included in model as equivalent diagonal struts. The properties of the equivalent struts were determined according to the guidelines in FEMA 356 [18]. A modal damping ratio of 2.5% for each significant vibration mode is assumed in this study. To perform standard RSA and MRSA, corresponding linear elastic models for these case study buildings were also created in ETABS [19].

Table 1 – Basic geometry and characteristics of case study buildings

Building No.			**B1**	**B2**	**B3**
Height (m)			60	116	152
No. of stories			20	33	44
Typical Story Height (m)			2.8	3.5	3.5
Height of Podium (m)			14	22	43
Natural periods of first three translational vibration modes (sec)	Strong Direction	T_1	1.44	2.81	2.79
		T_2	0.38	0.60	0.71
		T_3	0.17	0.31	0.33
	Weak Direction	T_1	2.12	3.21	3.61
		T_2	0.63	0.97	1.12
		T_3	0.21	0.47	0.31
Typical Floor Area (m x m)	Podium		47 x 33	58 x 33	74 x 34
	Tower		33 x 33	33 x 31	34 x 34
RC Wall section area/building footprint area (%)			0.40	1.22	0.90
RC Column section area/building footprint area (%)			1.20	2.20	1.80

Bangkok has long been considered as being free from seismic risk due to the absence of nearby seismic sources. However, recent seismic hazard studies indicate that the city is still at risk from damaging large-magnitude earthquakes originating from distant seismic sources. Moreover, the deep soil basin of Bangkok has an ability to amplify the ground acceleration about 3 to 4 times, and the amplified ground motions tend to have a rather long predominant period [20]. In this study, three ground motion sets (each with 6 acceleration time histories), originally selected by a detailed seismic hazard analysis of Bangkok, are used. They are selected to represent various possible ground motions with different spectral shapes and predominant periods. Set 1 represents ground motions from distant large earthquakes (M8+) modified to match with the uniform hazard spectrum (UHS) of Bangkok. Set 2 represents relatively shorter predominant period ground motions from crustal earthquakes with magnitude M 7—7.5 occurring at a distance of around 100 km from Bangkok. Set 3 represents very long period ground motions originating from megathurst earthquakes (M8.5+) occurring at a distance greater than 600 km from Bangkok. These set 3 records are characterized as long period, long duration, large displacement, but low acceleration. Fig. 3 shows the target and matched spectra of all 3 ground motion sets.

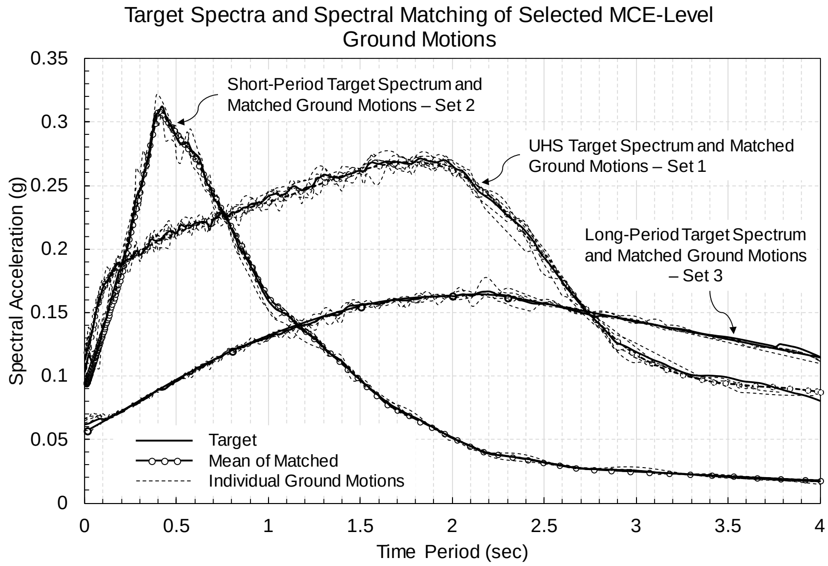


Fig. 3 – 5%-damped spectra of ground motions used in this study

6. Equivalent Linear Properties for Case Study Buildings

The determination of right equivalent linear properties is a key step in the proposed MRSA procedure. The intent of this study is to prove the concept (i.e. the application of equivalent linearization concept for RSA procedure); it is not intended to propose any refinement in existing EL methods. Therefore, the discussion on what should be the most realistic equivalent linear properties is beyond the scope of this paper. For the purpose of evaluating the basic concept, it is more rational to stick to the simplest and more conventional approach, instead of going after empirical and relatively more ambitious efforts to determine T_{eq} and ξ_{eq}. Therefore, the secant period at maximum amplitude (T_{sec}) is opted as equivalent linear period for the case study buildings and the hysteretic damping is determined from the equal-energy dissipation principle, applied to actual cyclic behavior of the case study buildings. First, the case study buildings were subjected to gradually-increasing monotonic inertial load vectors (after the application of gravity loads) proportional to the first three translational modes in both strong and weak directions. Point-by-point conversion of secant stiffness (K_{sec}) in to T_{sec} on monotonic modal pushover curves (for each mode) resulted in T_{sec}/T_i *vs. Roof Drift Ratio* relationships as shown in Fig. 5(a). The following expression is used to determine T_{sec} for any i^{th} mode.

$$T_{sec,i} = 2\pi \sqrt{\frac{M_i^*}{\Gamma_i \, K_{sec,i}}} \tag{2}$$

Where M_i^* is the effective modal mass, Γ_i is the modal participation factor of i^{th} mode and $K_{sec,i}$ is the secant stiffness (V_{bi}/u_{ri}—the ratio of base shear to roof displacement) from i^{th} mode pushover curve. To determine the hysteretic damping, cyclic modal pushover analysis was then carried out under direction-reversing inertial load vectors. Loading cycles were applied and reversed in directions with a gradual increment in control displacement (after each cycle) until significant damage occurred. The resulting cyclic roof drift ratio Δ_{roof}/H versus base shear coefficient V_b/W curves for strong directions are presented in Fig. 4.

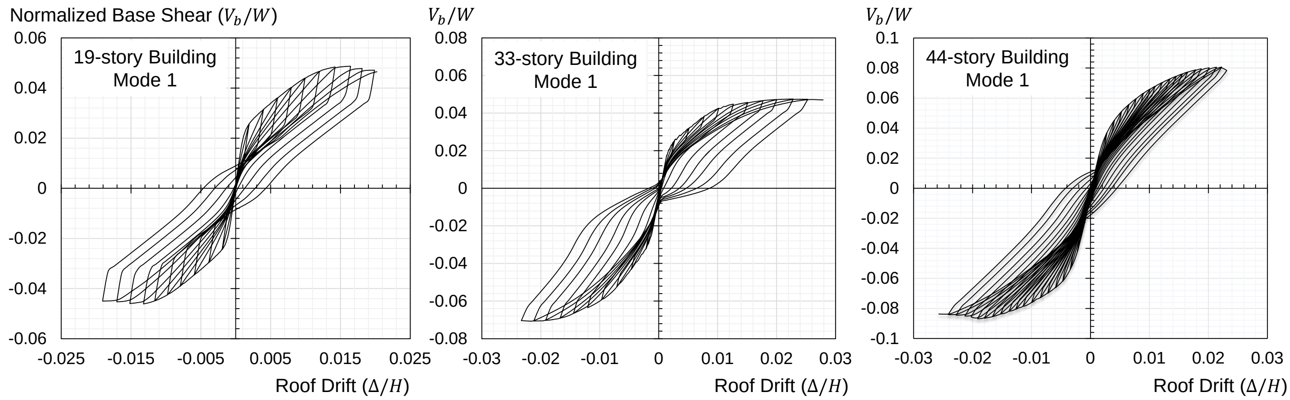
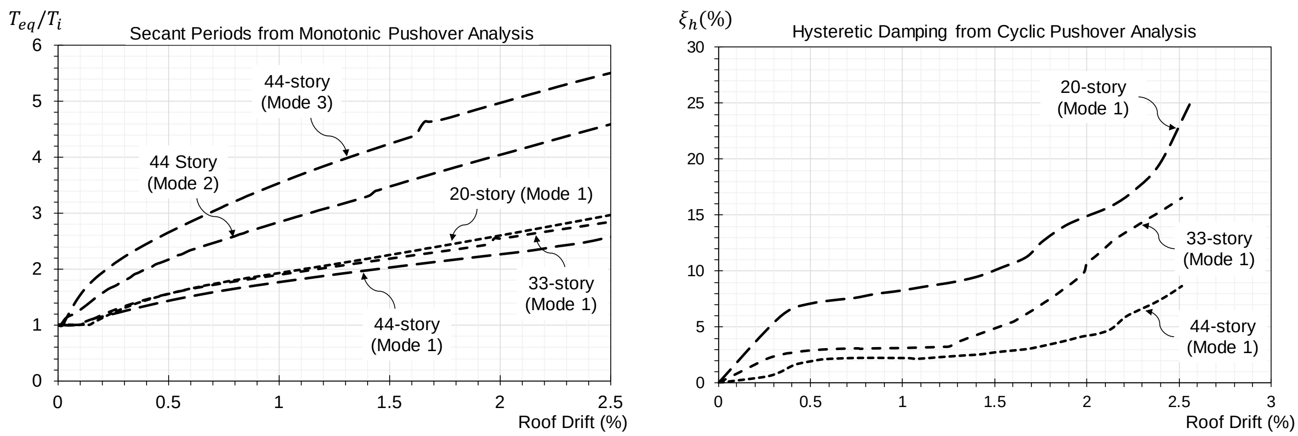


Fig. 4 – First-mode cyclic behavior (V_b/W vs. Δ/H) of the case study buildings (in strong direction)



(a) T_{eq}/T_i *vs. Roof Drift Ratio* for case study buildings (b) Equivalent hysteretic damping (ξ_h) *vs. Roof Drift Ratio*

Fig. 5 – Equivalent linear properties of the case study buildings determined from Cyclic Pushover Analysis

The actual area, $E_D(x)$ enclosed by each cyclic pushover loop (with stiffness from preceding loop) at maximum roof drift ratio, is determined. The strain energy $E_{so}(x)$ associated with equivalent linear system (with secant stiffness) is also calculated for all points on cyclic pushover envelope. Additional hysteretic damping (ξ_h) is then determined using equal-energy dissipation assumption (equation 1). For first mode in strong directions of 3 case study buildings, the ξ_h *vs. Roof Drift Ratio* relationships are also shown in Fig. 5(b). These relationships will be used to iteratively determine the equivalent linear properties in MRSA applied to case study buildings in subsequent section.

7. Evaluation of MRSA Procedure

The accuracy of MRSA is now compared with that of standard RSA for the case study buildings, using seismic demands computed by NLRHA as benchmark. Individual modal response histories are also obtained using UMRHA and are compared with the corresponding elastic demands (at equivalent linear properties) determined by MRSA. Although the proposed iterative procedure to determine equivalent linear properties is intended for any number of significant vibration modes, the initial elastic roof drift ratios for several cases (mostly higher modes) are so small, resulting in no significant period elongation and additional damping. In such cases the initial elastic properties (T_i and ξ_i) are used for estimating seismic demands in MRSA.

7.1 Mode-by-mode Response Comparison with UMRHA

Fig. 6 illustrates an example comparison of NLRHA demands (story displacements, inter-story drift ratios, story shears, and story overturning moments) with the combined response envelopes obtained from UMRHA, considering the first three vibration modes of a 44-story building subjected to a ground motion from set 1. A reasonable response matching confirms the applicability of UMRHA to decompose complex nonlinear responses into contributions from a few important vibration modes, where each mode behaves like a nonlinear SDF system. More details and verification of the UMRHA procedure with real buildings can be seen in [9] and [10]. Fig. 7 shows the decomposed modal responses for the same example case. The envelopes of individual modal contributions to story displacements, inter-story drift ratios, story shears and story overturning moments are shown. Given that the combined response envelops match well with NLRHA (Fig. 6), these individual nonlinear modal responses can be compared with corresponding equivalent elastic modal demands, to gauge the accuracy of MRSA. Although this modal decomposition results may provide various useful insights about complex nonlinear response, this study uses the results just to validate the EL modal contributions as shown in Fig. 8.

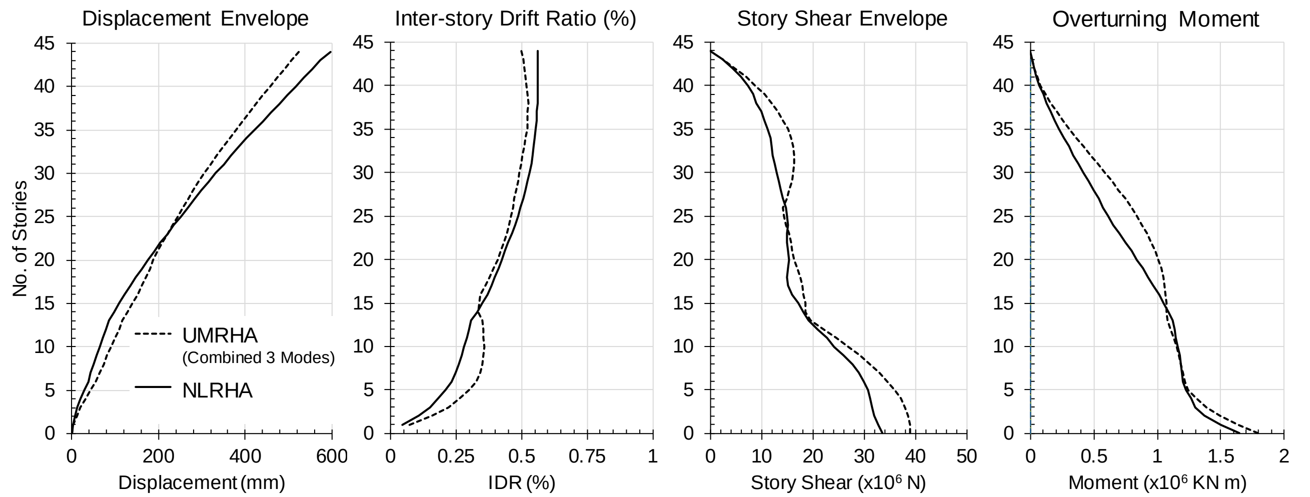


Fig. 6 – Comparison of seismic demands from NLRHA and UMRHA (3 modes combined) for a 44-story case study building

Continuing the same example, the story shear comparison between UMRHA and equivalent linear modal demands (under a ground motion from each set) is shown in Fig. 8. A reasonable match at individual mode level (except mode 2 against ground motions from set 2) shows that MRSA is able to provide reasonably accurate prediction of nonlinear seismic shear demands for every important mode.

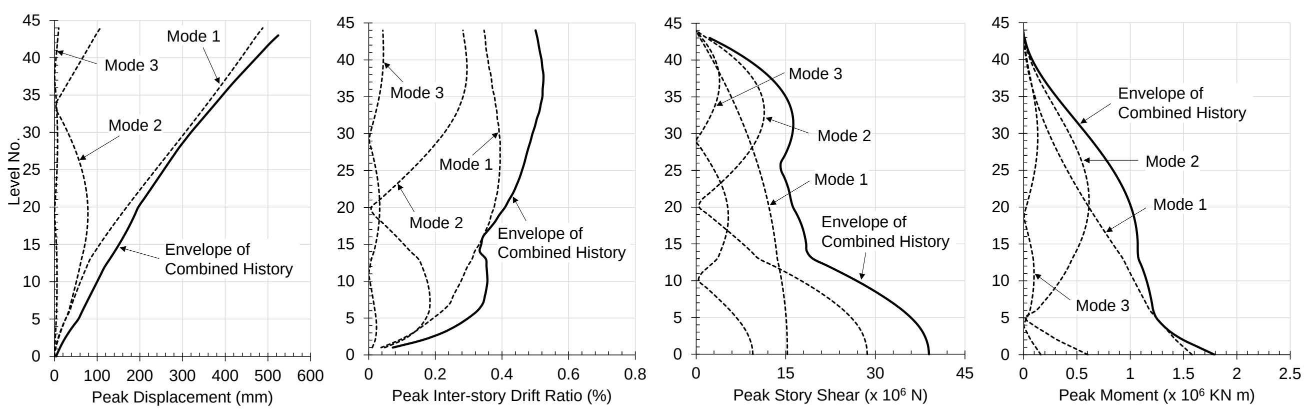


Fig. 7 – Individual modal contributions as obtained from UMRHA (strong direction, 44-story building)

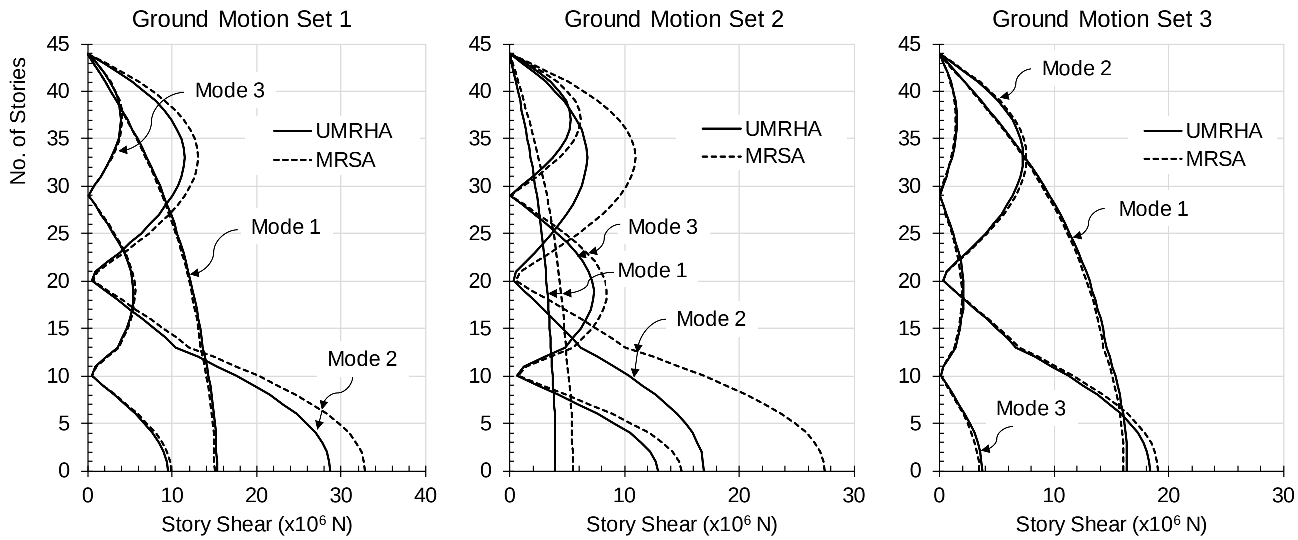


Fig. 8 – Comparison of modal story shears from UMRHA and MRSA at equivalent linear properties

7.2 Combined Response Comparison with NLRHA

Using the 44-story case study building as example (B3 in Table 1), Fig. 9 shows a comparison between seismic demands computed by the standard RSA, MRSA, and NLRHA procedures for all 3 sets of ground motions. In the standard RSA, the response modification factor (R) and deflection amplification factor (C_d) are taken as 4.5 and 4, respectively, in accordance with ASCE 7-05 (Table 12.2-1 [1]). A significant underestimation of force demands by the standard RSA procedure is evident. This underestimation may result in unsafe design of new buildings or inaccurate performance assessment of existing buildings. MRSA, on the other hand, is providing reasonably matching results with slight overestimation in some cases. However, being on conservative side, and considering the ease offered by MRSA procedure, this overestimation may be tolerated within certain acceptable limits. The satisfactory performance of MRSA shows that it can be considered (and developed further for general use) as a simplified analysis option in cases where it is not practical to perform detailed NLRHA. The required computational effort and convenient application offered by MRSA is almost the same as standard RSA (with an additional step of estimating right equivalent linear properties).

The wider applicability of MRSA depends heavily on an efficient generalization scheme to determine equivalent linear properties for a wide range of structural systems, hysteretic behaviors, and other controlling factors. The use of more accurate EL methods (instead of T_{sec} and ξ_h from equal-energy principle) may also result in better response prediction of MRSA. Moreover, the use of convenient graphical aids can help practicing engineers to quickly perform iterations to finalize T_{eq} and ξ_{eq}. The facility of modifying the modal load cases in commercial software by user-defined factors can be used to automate MRSA by manually calculating necessary modification factors (ratios of S_A at equivalent linear properties to that at initial properties). A possible way to avoid manual modification of software generated results is the use of stiffness modifiers for achieving equivalent linear period, resulting in a softened elastic structural model expected to mimic the behavior of detailed nonlinear model. Considering the impact this idea may have on common design office practice, further improvement potentials in terms of accuracy and convenient applicability should be explored in future works.

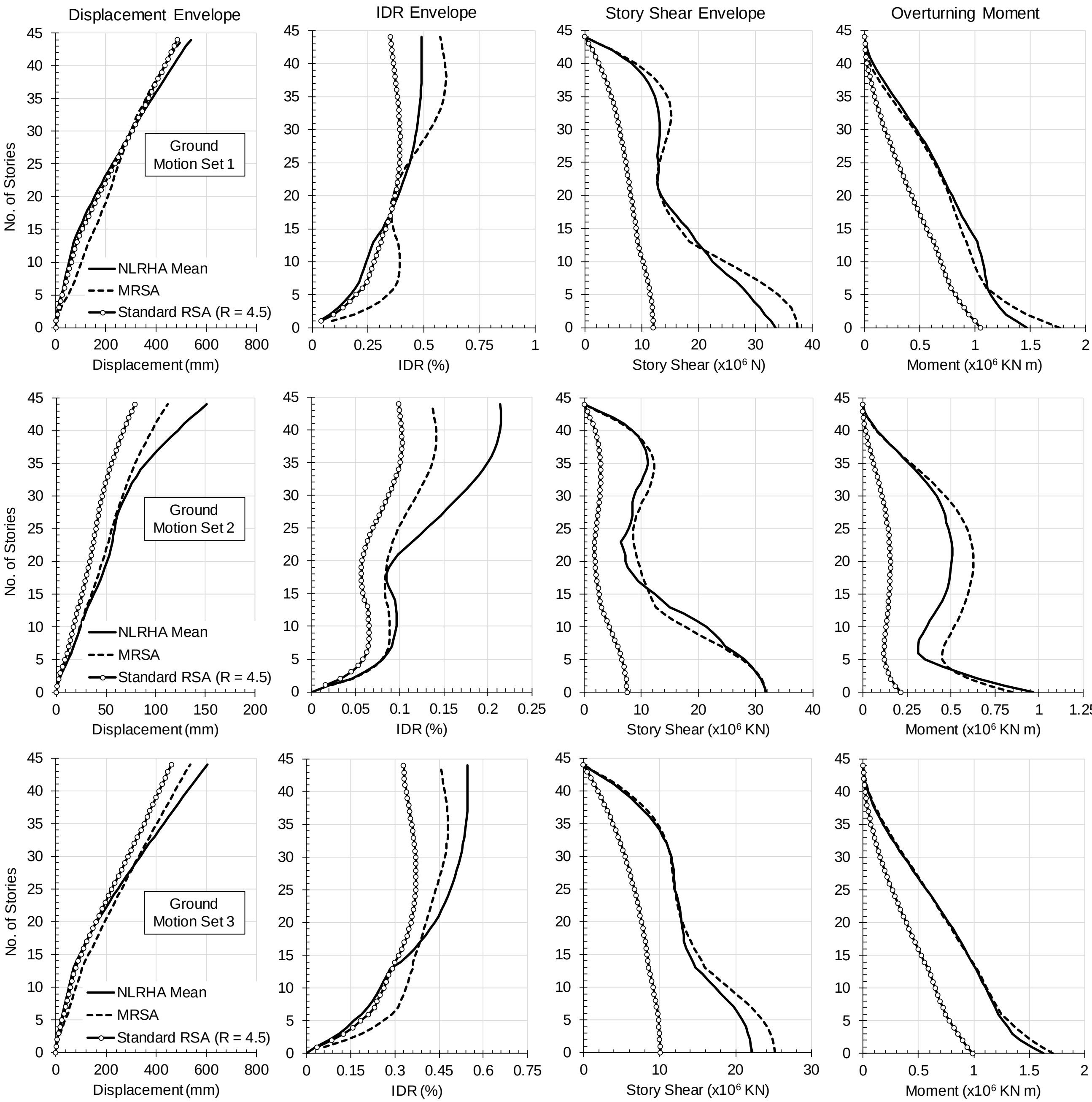


Fig. 9 – Comparison between standard RSA and MRSA with benchmark NLRHA values for all three sets of ground motions (Strong direction of 44-story case study building)

8. Conclusions

This study presents a modified version of response spectrum analysis based on equivalent linearization approach. The underlying idea is that a properly tuned linear elastic SDF model (with elongated natural period and with additional damping) can approximately represent the nonlinear behavior of a vibration mode, and hence can provide a reasonable estimate of nonlinear seismic demands of that mode. Using the most conventional equivalent linearization approach (i.e. setting equivalent linear stiffness to the secant stiffness of nonlinear system at maximum response amplitude and using additional damping determined from the equal-energy dissipation principle), this modified response spectrum analysis (MRSA) procedure is applied to three case study high-rise buildings with shear walls. It is shown that the MRSA procedure can estimate nonlinear seismic demands of these buildings with reasonable accuracy, either for those of individual vibration modes or for their sum (total demands). The MRSA procedure also retains the convenience offered by the standard RSA procedure for practicing engineers; it does not require nonlinear analysis nor nonlinear modeling. This study is only a first step towards the development of more versatile MRSA procedure. Several improvements can be made in the future to make this MRSA procedure applicable to buildings and structures of various types, configurations, and materials used.

References

[1] ASCE 7-05 (2006): Minimum design loads for buildings and other structures. ASCE/SEI 7-05, Reston, Va.
[2] EN 1998-1 (2005): Eurocode 8: Design of structures for earthquake resistance–Part 1: General rules, seismic actions and rules for buildings.
[3] Maniatakis CA, Psycharis IN, Spyrakos CC (2013): Effect of higher modes on the seismic response and design of moment-resisting RC frame structures, *Engineering Structures*, Volume 56, November 2013, Pages 417–430.
[4] Sullivan TJ, Priestley MJN, Calvi GM (2007): Estimating the Higher-Mode Response of Ductile Structures. *Journal of Earthquake Engineering*, 12:456–472, 2008. ISSN: 1363-2469.
[5] Priestley MJN and Amaris AD (2002): Dynamic amplification of seismic moments and shears in cantilever walls. European School for Advanced Studies in Reduction of Seismic Risk. *Research Report No. ROSE-2002/01*, Pavia, Italy.
[6] Priestley MJN (2003): Does capacity design do the job? An examination of higher mode effects in cantilever walls. *Bulletin of NZ National Society for Earthquake Engineering*, 36(4).
[7] Ahmed M, Warnitchai P (2012): The cause of unproportionately large higher mode contributions in the inelastic seismic responses of high-rise core-wall buildings, *Earthquake Engineering and Structural Dynamics*, vol. 41, no. 15, pp. 2195–2214.
[8] Chopra AK, Goel RK (2002): A modal pushover analysis procedure for estimating seismic demands for buildings. *Earthquake Engineering and Structural Dynamics*. 31(3):561–82.
[9] Ahmed M (2011): Optimal reduction of inelastic seismic demands in high-rise RC core wall buildings using energy dissipating devices, *PhD Thesis*, Asian Institute of Technology (AIT), Thailand.
[10] Mehmood T (2015): Investigation of nonlinear seismic response of high-rise RC wall structures using modal decomposition technique, *PhD Thesis*, Asian Institute of Technology (AIT), Thailand.
[11] Jacobsen LS (1930): Steady forced vibrations as influenced by damping. *ASME Trans* 1930;52(15):169–81.
[12] Rosenblueth E, Herrera I (1964): On a kind of hysteretic damping. *Journal of the Engineering Mechanics Division*, 1964;90 (4):37–48.
[13] ATC 40 (1996): Seismic evaluation and retrofit of concrete buildings. Redwood City, California, 1996.
[14] Liu T, Zordan T, Briseghella B, Zhang Q (2014): Evaluation of equivalent linearization analysis methods for seismically isolated buildings characterized by SDOF systems, *Engineering Structures* 59 (2014) 619–634.
[15] Lin YY, Miranda E (2009): Evaluation of equivalent linear methods for estimating target displacements of existing structures, *Engineering Structures*, Volume 31, Issue 12, Pages 3080–3089, ISSN 0141-0296.
[16] Computers and Structures, Inc. (CSI) (2006): PERFORM-3D: Nonlinear analysis and performance assessment for 3-D structures, Version 4, Berkeley, California.
[17] Mander JB, Priestley MJN, Park R (1988): Theoretical stress-strain model for confined concrete. *Journal of structural engineering*, 114(8):1804–1826.
[18] FEMA 356 (2000): Pre-standard and Commentary for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency Washington.
[19] Computers and Structures, Inc. (CSI) (2011): ETABS Nonlinear v9.7.4, Extended Three Dimensional Analysis of Building System, 2011. Berkeley, California, USA.
[20] Poovarodom N, Jirasakjamroonsri A, Warnitchai P (2017): Development of new design spectral accelerations for Bangkok considering deep basin effects. Submitted to *16th World Conference on Earthquake Engineering 16WCEE 2017*, Paper No 2141, Santiago, Chile.