Hindawi
Advances in Civil Engineering
Volume 2019, Article ID 1035015, 21 pages
https://doi.org/10.1155/2019/1035015

Research Article

Evaluation of Simplified Analysis Procedures for a High-Rise Reinforced Concrete Core Wall Structure

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Received 6 September 2018; Accepted 15 January 2019; Published 12 February 2019

Academic Editor: Rosario Montuori

Nonlinear response history analysis (NLRHA) procedure is one of the most precise and accurate numerical method to compute the seismic demands of high-rise structures but is complex, rigorous, and time-consuming and requires a lot of expertise for nonlinear modelling and results interpretation. Therefore, practicing engineers in developing countries like Pakistan still use the simplified analysis procedures to compute the seismic demands. Among the simplified analysis procedures, equivalent lateral force and response spectrum analysis procedures are widely used for the design purpose. However, other procedures have also been proposed in the recent past to accurately capture the higher mode effects in mid-to-high-rise structures. In the current study, results of a forty-story core wall building are used to check the relative accuracy and ease of application of different simplified analysis procedures. Furthermore, a modal decomposition technique is used to separate the modal responses from the NLRHA results, and a mode wise comparison of different demand parameters for different simplified procedures is performed. The current study has been used to clearly identify the reasons of inaccuracies in different simplified procedures. Furthermore, a simplified analysis procedure is proposed to accurately estimate the seismic demands of high-rise buildings and the possible solutions to improve their predictions.

1. Introduction

Recent survey by the United Nations estimated that more than half the population of the world lives in the urban regions [1]. The United Nations also predicted that, by the end of 2050, about 64% of the developing countries and 86% of the developed countries population will be living in the cities. To accommodate the increase in population in the urban areas, either the urban areas (cities) should be expanded or the high-rise buildings should be constructed. High-rise buildings are generally more vulnerable to seismic hazard and wind actions as compared to low-rise buildings. Therefore, it is essential to use efficient lateral force-resisting systems to reduce lateral demands. Various lateral force-resisting systems have been developed in the past for high-rise buildings. High-rise reinforced concrete (RC) buildings with an RC core wall as a lateral force-resisting system present one such solution and provide the advantages of faster construction, flexible architecture, and the availability of more open spaces [2]. As the core wall is stiffer compared to the RC columns, most of the lateral forces are resisted by core wall. Thus, the columns remain flexible to carry the gravity loads.

Nonlinear response history analysis (NLRHA) procedure is one of the most precise and accurate numerical methods to compute the dynamic responses of high-rise structures; however, it is time-consuming, computationally expensive, and requires expertise in nonlinear modelling. A recent study of Mehmood et al. [3] shows that the computation time required to perform the NLRHA procedure to predict the seismic responses of a high-rise building under one ground motion is about 30 hours and the postprocessing takes another 5 hours. Therefore, several simplified analysis procedures were developed in the past to avoid the aforementioned issues associated with the NLRHA procedure. Some of the simplified procedures require nonlinear modelling, while for others, the linear elastic modelling option suffices. But these methods are less accurate as compared to the NLRHA procedure. Most of these simplified procedures were adopted in various codes such as UBC-97 [4], FEMA-356 [5], and ATC-40 [6]. Simplified analysis procedures include response spectrum analysis

(RSA) procedure, equivalent lateral force (ELF) procedure, modified modal superposition procedure (MMSP), modal pushover analysis (MPA) procedure, and weighted capacity design (WCD) method.

A number of simplified pushover analysis procedures have also been developed recently for various specific structural types such as bidirectional energy-based pushover (BEP) procedure [7], cyclic pushover analysis procedure [8], extended consecutive modal pushover procedure [9], improved modal pushover analysis procedure [10], extended energy-based pushover analysis procedure [11], simplified pushover analysis procedure for high-rise rocking buildings [12], envelope-based pushover analysis procedure [13], and adaptive modal pushover analysis procedure [14]. However, the current study is focused on the applications of more established simplified analysis procedures.

The simplest and straightforward design method is the ELF procedure which was adopted by UBC-97. This method mainly considers the seismic responses of a fundamental mode and also an extra force at top of a structure for higher modes. It has been found to be reasonably accurate in predicting the seismic forces of low-rise buildings; however, it underestimates the seismic responses of high-rise buildings in which higher modes contribute significantly.

Response spectrum analysis (RSA) procedure, on the other hand, explicitly takes into account the higher mode actions and is the most commonly adopted procedure to compute the seismic demands of buildings in many countries. In this method, first, the elastic seismic forces of each mode are computed independently from the elastic response spectrum, and then, modal seismic forces are added by using a modal combination rule to obtain the total elastic responses. These elastic demands are then divided by the force reduction factor (R) to obtain the inelastic seismic demands. Research work of Eibl and Keintzel, Priestley and Amaris, Sullivan et al., and Munir and Warnitchai [15–18] show that RSA procedure underestimates the inelastic seismic demands of high-rise buildings.

Priestley et al. [19] proposed the MMSP in which only the 1st mode structural responses are divided by the force reduction factor (R) and then combined with elastic structural responses of higher modes to obtain the maximum response. The results of this method have been checked with the results from NLRHA procedure for cantilever walls having 2 to 20 stories, and it is found to slightly underestimate the seismic demands of shorter structures and overestimate the seismic demands of taller structures.

A simplified lateral force analysis procedure called the nonlinear static pushover (NSP) analysis procedure is adopted by various codes such as FEMA-356, ASCE-41-06 [20], and ATC-40. This method is based on the structural responses of the fundamental mode only. In this method, the structure is pushed laterally to a target displacement after the gravity loads are applied. These pushover lateral forces are proportional to fundamental modal inertial forces. The seismic responses of the building are then determined at the target displacement.

Chopra and Goel [21] suggested the modal pushover analysis (MPA) procedure to overcome shortcomings of NSP procedure based on the fundamental mode only. In this technique, firstly, all significant modes in which the building will vibrate are determined. An equivalent single degree of freedom (SDOF) system for each mode is subjected to the specified ground motion to determine the target displacement for each mode. The SDOF system is controlled by a few modal properties which are determined from the hysteretic behavior of the structure. The maximum structural responses are then determined at the target displacement for each mode. These individual responses are then added by a modal combination rule to obtain the overall structural responses. Goel and Chopra [22] also modified the MPA procedure by suggesting that the higher modes remain in the elastic region which is called the modified modal pushover analysis (MMPA) procedure.

The developing countries like Pakistan still use the simplified analysis procedures for the design of buildings against lateral forces due to limited expertise. These procedures are simple and easy to apply as compared to the NLRHA procedure to design the buildings for lateral forces, but their accuracy is still in question. The above discussed simplified analysis procedures have been compared individually with NLRHA procedure in the past to check their accuracy, but still there is a lack of literature in which all these simplified procedures have been checked especially for high-rise core wall buildings. This paper aims at comparing all simplified procedures for their relative accuracy, computational efforts, and computation time for high-rise core wall structures and to detail merits and demerits of various simplified analysis procedures. Furthermore, based on an in-depth analysis of modal responses using modal decomposition techniques, a modified simple analysis procedure is proposed. For this purpose, a high-rise core wall building is chosen as a case study building as explained in the next section.

2. Case Study Building

To check the relative accuracy of various simplified analysis procedures, a high-rise reinforced concrete (RC) core wall building is chosen as a case study building. According to Emporis Standards Committee, a multistory building with 12 to 39 stories having unknown height or a multistory building having a total height of 43 to 122 m (140 to 400 ft) is considered as a high-rise building [23]. The chosen case study building is a residential building having a total height of 126.50 m (415 ft) and 40 stories as shown in the Figures 1(a) and 1(b). It has a lobby level height of 6.0 m (20 ft), while above the lobby level, typical story height is 3.0 m (10 ft). It has three basement-level parking floors with 3.0 m (10 ft) story height. The lateral force-resisting system in the building comprises of fourteen peripheral columns and central reinforced concrete core wall, while the vertical load resisting system comprises of a 20.20 cm (8 inch) thick posttensioned concrete slab laying on the central core wall and the peripheral columns.

Klemencic Magnusson Associates and Arup used the Los Angeles tall building structural design council's alternative design procedure for tall buildings to design this building [24, 25]. This building is in a high seismic zone, which is

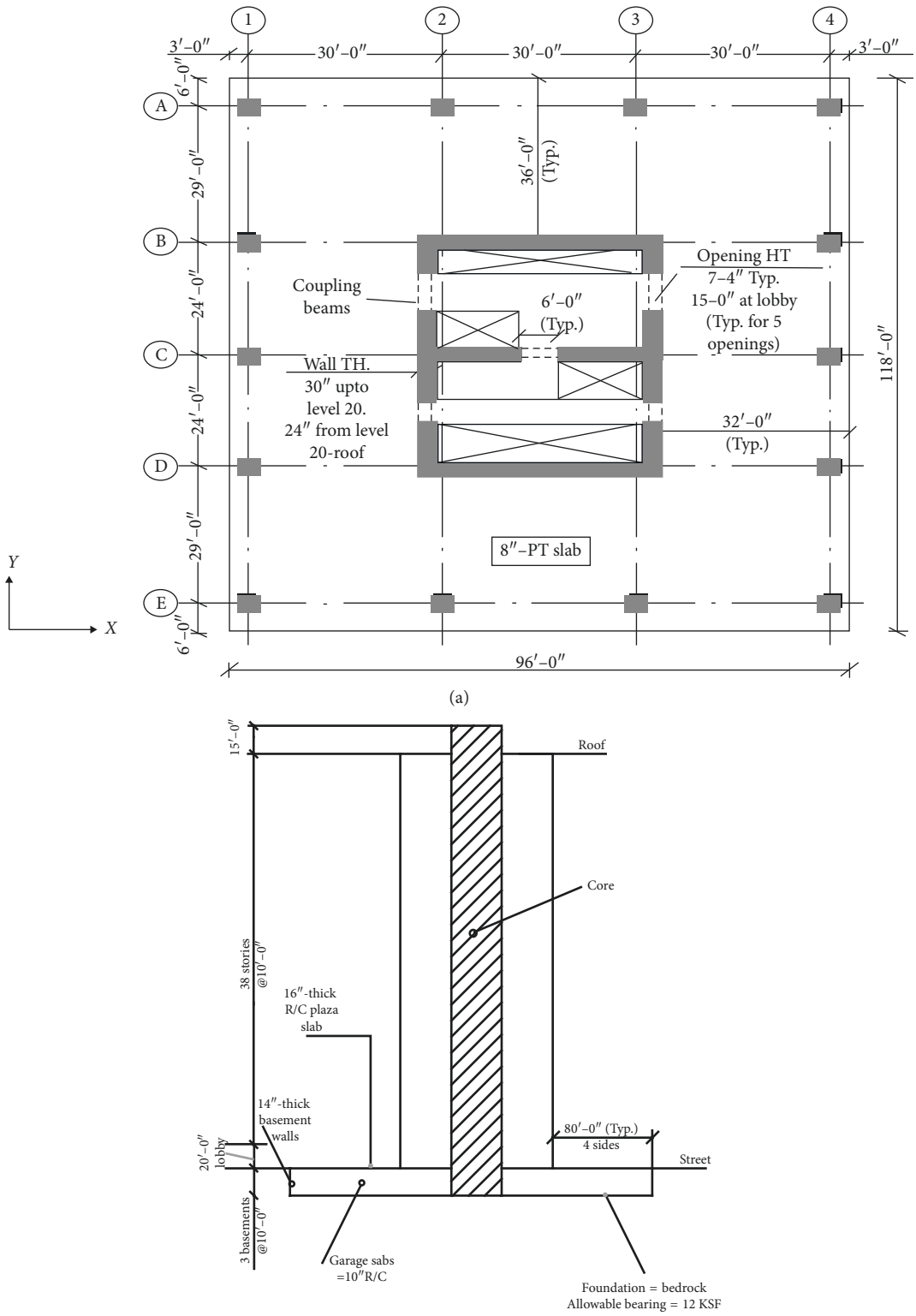

Figure 1: (a) Plan view of 40-story building [24, 25]; (b) elevation of 40-story building [24, 25].

correspondent to seismic zone 4 in the UBC-97 code. The soil below the foundation of building and in the surrounding region is considered as a stiff soil corresponding to soil type S_D in UBC-97. The seismic demands for the design of building were obtained from the design-based response spectrum in the UBC-97. As the seismic demands decrease with height, the thickness of the core wall and reinforcement were reduced along the height. The thickness of the reinforced concrete

central wall is kept 76.20 cm (30 inches) up to the story level 20 and 60.90 cm (24 inches) from story level 20 to the roof. The cross sections of the columns were also reduced along the height from 91.0 cm × 91.0 cm (36 inches × 36 inches) at the lower level to 60.90 cm × 60.90 cm (24 inches × 24 inches), 71.0 cm × 71.0 cm (28 inches × 28 inches) to 60.90 cm × 60.90 cm (24 inches × 24 inches) at the top level. The compressive strength of concrete was also reduced from 55 MPa (8000 psi) up to level 20 to 40 MPa (6000 psi) from story level 20 to the roof. The steel bars having a nominal yield strength of 415 MPa (60 ksi) were used. The core wall in the center of the building is a group of cantilever RC walls, having open spaces which are connected by coupling beams. The coupling beam dimensions were also reduced with the height of the core wall. The coupling beams have the dimensions of 76.0 cm × 152.0 cm (30 inches × 60 inches) above the lobby level opening, 76.20 cm × 81.28 cm (30 inches × 32 inches) at other floor level up to story level 20, and 60.90 cm × 81.20 cm (24 inches × 32 inches) from story level 20 to the roof.

3. Numerical Modelling of Case Study Building

As explained earlier, the current study focuses on the comparison of results from different simplified analysis procedures with the results from the time history analysis. To perform the nonlinear response history analysis (NLRHA), a nonlinear numerical model of the forty-story building has been developed in the Perform-3D version 4 [26] as previously created by Munir and Warnitchai [18]. The nonlinear model is a conventional fiber model, which is usually used for the modelling of high-rise RC core wall buildings. In this model, only the flexural behavior is considered nonlinear, whereas shear response is considered as linear elastic according to the principles of capacity design for the shear strength design. The core wall up to story level 5 has been modelled by nonlinear shear wall element in which each wall element consists of eight concrete and eight steel vertical fiber segments in each layer. The hysteretic curve used for steel fibers is a nondegrading bilinear type as shown in the Figure 2(a). The postyield stiffness of steel is assumed to be about 1.2 percent of initial stiffness. As the expected strength of the material is greater than the strength specified by the codes, the expected yield strength of steel rebar of grade 60 is supposed to be 485.0 MPa (70.20 ksi) which is greater than the nominal yield strength. Similarly, the compressive strength (f'_c) of concrete is assumed 1.3 times of the specified compressive strength. The concrete tensile strength is assumed to be ($7.5\sqrt{f'_c}$) psi as in UBC-97. The core wall above story level 5 is modelled by the elastic shear wall element segments as this portion is expected to remain in the elastic region. The stress-strain curve of Mander's model [27] for the unconfined and confined concrete is used for modelling the concrete fibers and is approximated by the trilinear envelope as shown in Figure 2(b). This is to account the confinement effect of postpeak strain ductility capacity and ultimate compressive strength by the transverse reinforcement in the core wall. In Perform-3D, the unloading stiffness of concrete in compression in the hysteretic curve is assumed to be approximately equal to the initial stiffness. However, the reloading stiffness can be modified, and it is set such that the stiffness degradation is directly proportional to the plastic strain. Columns and slabs are also modelled by elastic column and shell elements, respectively. As the plastic hinges are permitted at the end of coupling beams, the coupling beams are modelled by the elastic beam-column elements with the plastic hinges at both ends. The geometric nonlinear effects (P-delta effects) are also considered in the model. According to the Tall Building Initiatives (TBI) recommendations, a modal damping ratio of 2% is more reasonable for the high-rise building as compared to the traditionally assumed 5% damping ratio [28]. Therefore, the 2% damping ratio has been assigned to each translational mode. A small Rayleigh damping is also considered for stabilization of other higher modes. Further details about nonlinear modelling of the case study building can be found in the study of Munir and Warnitchai [18].

4. Seismic Demands by NLRHA and Simplified Analysis Procedures

The NLRHA procedure is one of the most precise methods to compute the seismic demands of buildings. Therefore, it will serve as a benchmark to check the accuracy of all simplified analysis procedures. Two levels of the earthquake are defined for the present case study, one is design basis earthquake (DBE) and the other is maximum considered earthquake (MCE). DBE has 10% probability of exceedance in 50 years. MCE can be described as the earthquake having a 2% probability of exceedance in 50 years. DBE is considered two-third of the MCE for the present study. A set of three ground motions is chosen for this study as shown in Table 1. These ground motions were chosen from PEER NGA and COSMOS database [29, 30], whose response spectra are similar with the target spectrum. Further information about ground motions selection, scaling, and matching can be found in the study of Munir and Warnitchai [18].

With nonlinear models of the building as discussed above, nonlinear response history analyses are executed for each of the ground motion for DBE and MCE levels applied in the X direction. The maximum shear force and the maximum bending moment at each story level of the building for each ground motion are determined and then plotted along the height of the building, but here the results are plotted only for HM ground motion as shown in the Figures 3(a) and 3(b). The plastic hinges are formed at the end portions of the coupling beams in the X direction and at the bottom region of the core wall as expected. The MCE shear demand at the base of the building is about 1.5 of the DBE level shear demand, while at the midheight, it is about 1.3 times the DBE level shear demand. Similarly, the MCE level bending moment at the base and midheight of the building is about 1.25 times of DBE-level earthquake. In the next section, seismic demands through simplified analyses procedures will be computed and will be compared with true seismic demands from the NLRHA procedure.

4.1. Equivalent Lateral Force Procedure. The ELF procedure in UBC-97 is still used in many developing countries for the

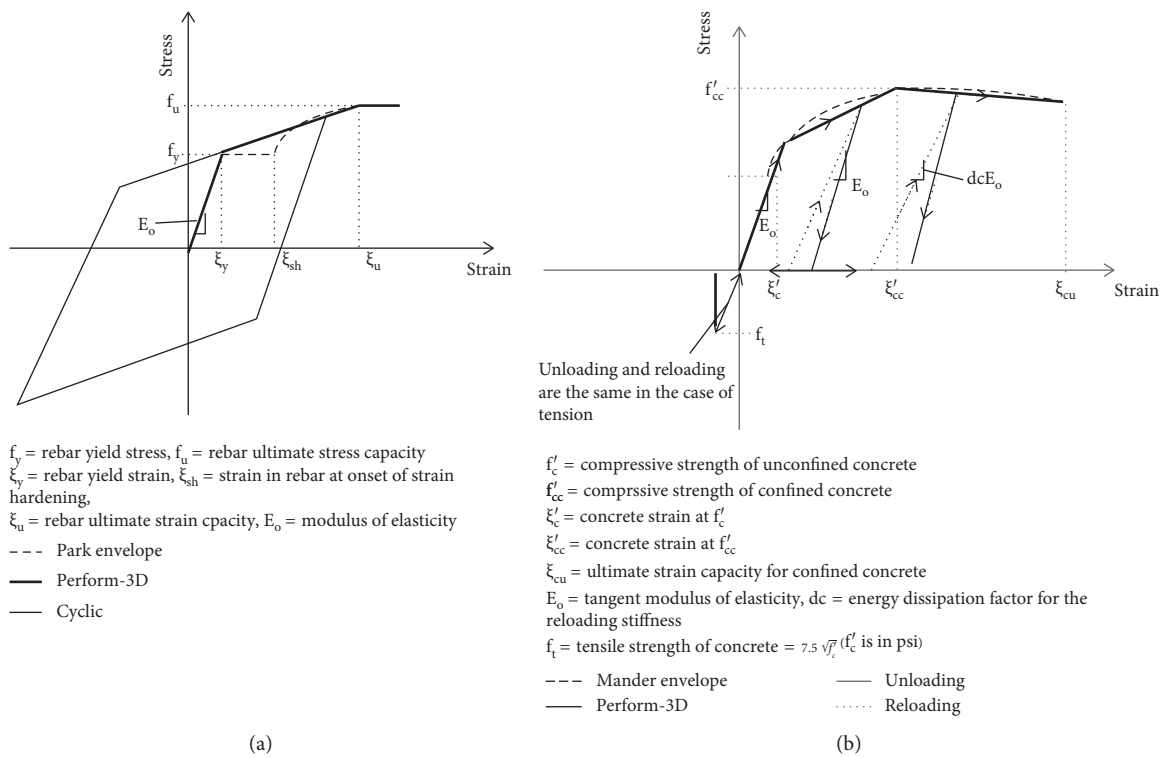

Figure 2: (a) Stress-strain curve of steel rebar [18]; (b) Mander's stress-strain model for concrete [18].

Table 1: Detail record of the three ground motions for the present study.

S. No.	Earthquake event	Years	Abbreviation	M_w	R (km)	PGA (g)	Duration (seconds)
1	Superstition Hills	1987	SH	6.5	11	0.30	22.30
2	Hector Mine	1999	HM	7.1	26	0.27	45.31
3	Loma Prieta	1989	LP	6.9	48	0.37	59.95

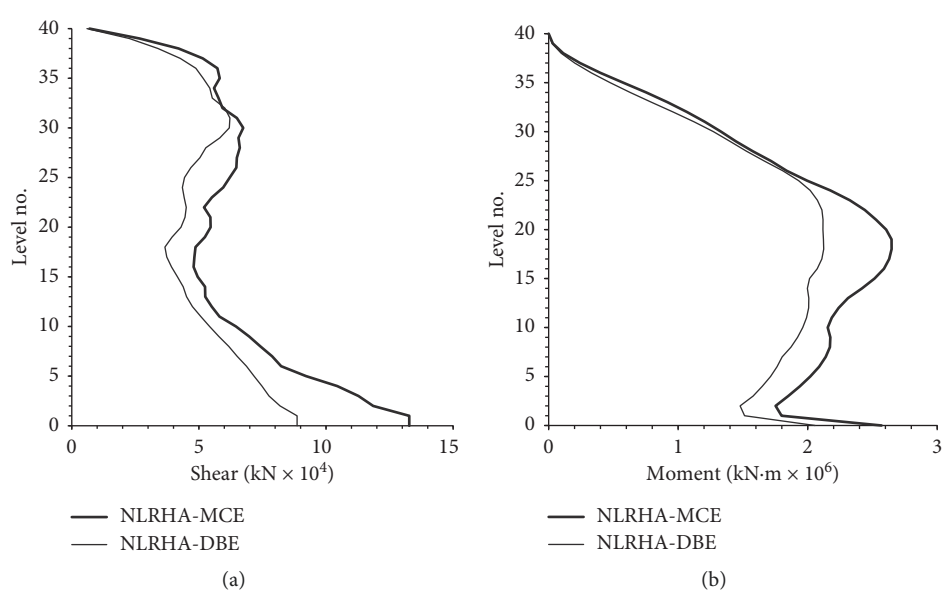

Figure 3: Building maximum (a) shear and (b) bending moment demands by the NLRHA procedure at DBE and MCE levels for ground motion HM.

design of building structures against lateral forces. As this method is developed on the fundamental mode structural responses of the building and on considering an extra force at top of the building for higher modes, it is therefore more suitable to be used for low-rise regular building in high seismic zones, all regular and irregular buildings in low-seismic zone, and irregular buildings less than five stories or 20 m (65 ft) in height. In accordance with above limitations of UBC-97, this procedure cannot be used for the present case study building which is a high-rise building located in the high seismicity zone. However, seismic demands by this method are computed only to show its underestimation for high-rise buildings because it is still used in some developing countries such as Pakistan. The ELF procedure in UBC-97 is adopted to compute the seismic demands for the present case study building as shown in Table 2.

The computed shear force and moment at each floor level are then compared with true shear force and true moment computed by the NLRHA procedure, which is considered the most accurate procedure for determining the seismic demands as shown in Figures 4(a) and 4(b). The comparison clearly shows that the true seismic demands are much higher than those computed by the ELF method. The true base shear is about 3 to 6 times higher than the base shear computed by the ELF procedure, while the midheight moment is about 1.85 to 3.5 times higher than the midheight moment of the ELF procedure. These results clearly show that the higher vibration modes significantly contribute to the total seismic demands.

4.2. Seismic Demands by RSA Procedure. The RSA procedure is a dynamic analysis procedure in which the elastic responses of the significant vibration modes are obtained from the DBE elastic response spectrum and then combined to obtain the total elastic responses of the structure either by SRSS or CQC modal combination rule. Total elastic responses are then reduced by response modification factor (R) to get the inelastic responses for the design purposes.

Traditionally, buildings are designed for the DBE but evaluated for MCE-level seismic intensity. Therefore, we will investigate the seismic demands for both levels of seismic intensity. Firstly, the DBE elastic response spectrum is defined by using the values of C_a and C_v, and damping ratio of 5%. This spectrum is then multiplied by the factor of 1.5 to obtain the MCE level spectrum for the 5% damping ratio. As a constant modal damping ratio of 2% is used in the NLRHA procedure, the same will be used here. 2% DBE and MCE spectrum are constructed by multiplying 5% DBE and 5% MCE spectrum with factor R_ζ. The value of R_ζ is computed by the equation $R_\zeta = (0.07/0.02 + \zeta)^{0.5}$, where ζ is the damping ratio.

The eigen value analysis method is performed to find the natural time periods, modal participating factors, mass participation factors, and mode shape factors for all the significant modes, and they are shown in Table 3. The sum of mass participating factors of the first five translational modes is 97.72% which is greater than 90% as recommended by the UBC-97 code. The elastic responses of the first five translational modes are determined from response spectrum for both DBE and MCE levels. The elastic modal responses for each case are combined by SRSS rule to obtain the total elastic responses and then divided by R factor to obtain the total inelastic responses. The total inelastic responses are then compared with the results of the NLRHA procedure as shown in Figures 5(a)–5(d). Both the shear force and bending moment at each story level for both DBE and MCE computed by the NLRHA procedure are much higher than those determined by the RSA procedure. The base shear of the NLRHA procedure is about 3 to 5 times higher than base shear of the RSA procedure. Similarly, the midheight bending moment determined by the NLRHA procedure is also about 3 to 5 times higher than that determined by the RSA procedure. These results evidently illustrate that the RSA procedure undervalues the seismic demands for high-rise core wall structures.

Table 2: Equivalent lateral force procedure.

Parameters	Value
Seismic zone factor (zone 4)	$Z = 0.4$
Type of soil	S_D
Seismic acceleration coefficient	$C_a = 0.44$
Seismic velocity coefficient	$C_v = 0.64$
Response modification factor	$R = 5.5$
Importance factor	$I = 1.0$
Natural time period of the first mode (by method A)	$T = 2.91$ sec
Total seismic dead load	$W = 4.0 \times 10^5$ kN (89750 kips)
Equivalent static lateral load procedure	
Design base shear $V = (C_v IW)/(RT)$	$V = 0.039\ W = 1.60 \times 10^4$ kN (3600 kips)
Minimum design base shear $V = 0.11 C_a IW$	$V = 0.046\ W = 1.92 \times 10^4$ kN (4300 kips)
Minimum design base shear $V = 0.8 Z N_v I/R$	$V = 0.06\ W = 2.31 \times 10^4$ kN (5200 kips) (governed)

4.3. Seismic Demands by MMSP. Priestley et al. [19] modified the modal response spectrum analysis procedure by assuming that ductility primarily acts in the 1st mode to limit the 1st mode responses while assuming the higher modes to be in the elastic range. This means that 1st mode responses are independent of the ground motion intensity, whereas the higher modes responses are proportional to intensity. The total response obtained is an approximation to the true response. The responses of the higher modes will be modified to little extent due to 1st mode ductility, but they cannot increase the base moment which will be anchored by the base plastic hinge moment capacity.

The seismic demands by MMSP are also computed for the present case study building and then evaluated with the true seismic demands of the NLRHA procedure for ground motion HM as shown in Figures 5(a)–5(d). The MMSP underestimates the shear force at the midheight of the building while overestimate at the base shear. However, the overall shear force at each floor level matches well with the true shear force computed by the NLRHA procedure as compared to the RSA procedure. Similarly, the bending

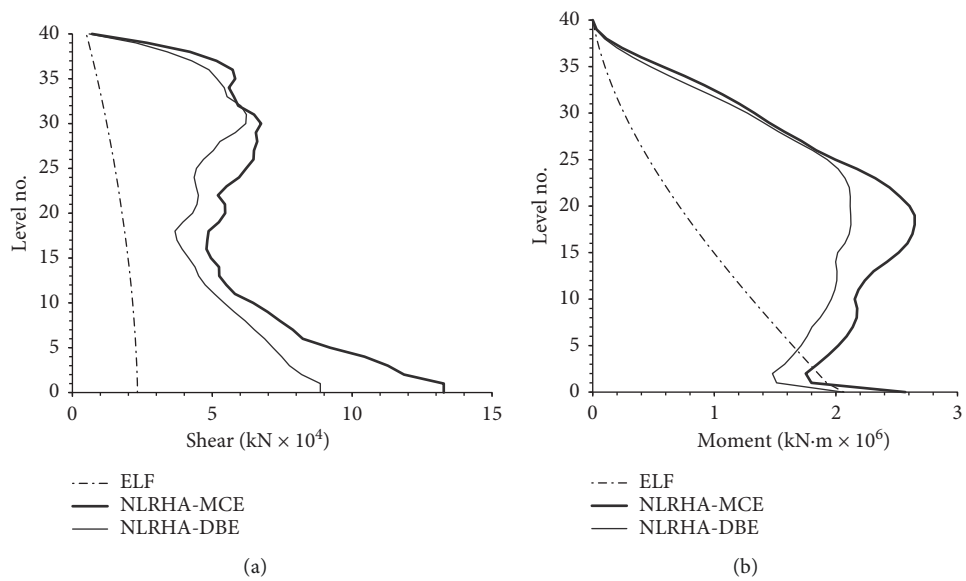

Figure 4: Comparison of (a) shear and (b) bending moment demands of ELF and NLRHA procedures for ground motion HM.

Table 3: Modal properties of the case study building.

Mode shape	Natural time period (sec)	Mass participating factors	Modal participating factors
1	3.84	66.08	1.6048
2	0.745	20.29	−0.9252
3	0.3026	06.75	0.5355
4	0.1724	03.06	−0.3653

moment at each floor level computed by the MMSP procedure as compared to the RSA procedure is in good agreement with bending moment of the NLRHA procedure for the upper half height of the building, while it overestimates the bending moment at the lower half height. The overestimation for the lower half height is noticeable in the case of MCE.

4.4. Seismic Demands by MPA Procedure. The modal pushover analysis (MPA) procedure was developed by Chopra and Goel. In this procedure, the target displacements of each significant mode are first computed from the equivalent SDOF system which is subjected to the specified ground motion, and then the structure is pushed laterally to each target displacement. The various seismic responses are then determined for each mode at the target displacement from standard pushover analysis and then combined by a modal combination rule such as SRSS or CQC rule to obtain the total seismic responses of the structure. Using the MPA procedure, seismic demands are determined for both DBE and MCE levels and then compared with seismic demands from the NLRHA procedure. Figures 6(a)–6(d) shows the comparison of shear and moment demands at each story level. The comparison shows that the shear demands computed by the MPA procedure for both DBE and MCE levels match approximately with those obtained by the NLRHA procedure, except from story level 26 to story level 33 where the discrepancy is noticeable. However, it underestimates the bending moment at the middle and overestimates at the base of the core wall. Despite relatively accurate results, it is important to mention here that, as opposed to previously shown simplified procedures, the MPA procedure requires nonlinear modelling of the structure.

4.5. Seismic Demands by MMPA Procedure. Goel and Chopra [22] modified the MPA procedure by considering that higher vibration modes of the building having usually very limited inelastic action to remain in the elastic region. This is called modified modal pushover analysis (MMPA) procedure. The need of pushover analysis for higher modes is also eliminated. This procedure is limited only to systems having a moderate damping ratio. This is because the increase in seismic demands of higher modes are unacceptably very large for lightly damped systems, while for modest damped systems, the increase in the seismic demand is small and acceptable. For the present case study building, the higher mode target displacements are determined from the equivalent elastic SDOF systems and modal seismic demands are then obtained from these target displacements and combined by the modal combination rule to determine the total seismic demands. These seismic demands are compared with true seismic demands of the NLRHA procedure. Figures 6(a)–6(d) show the comparison of shear force and bending moment of the MMPA procedure with the results of the NLRHA procedure at each story level of the building. The comparison shows that shear forces at each floor level computed by the MMPA procedure for both DBE- and MCE-level ground motion match well over the upper half of the building with true shear forces of the NLRHA, while it overestimates from the base to midheight of the building.

Various simplified procedures presented in this section shows that the ELF procedure, RSA procedure, and MMSP are relatively less accurate in predicting the seismic demands

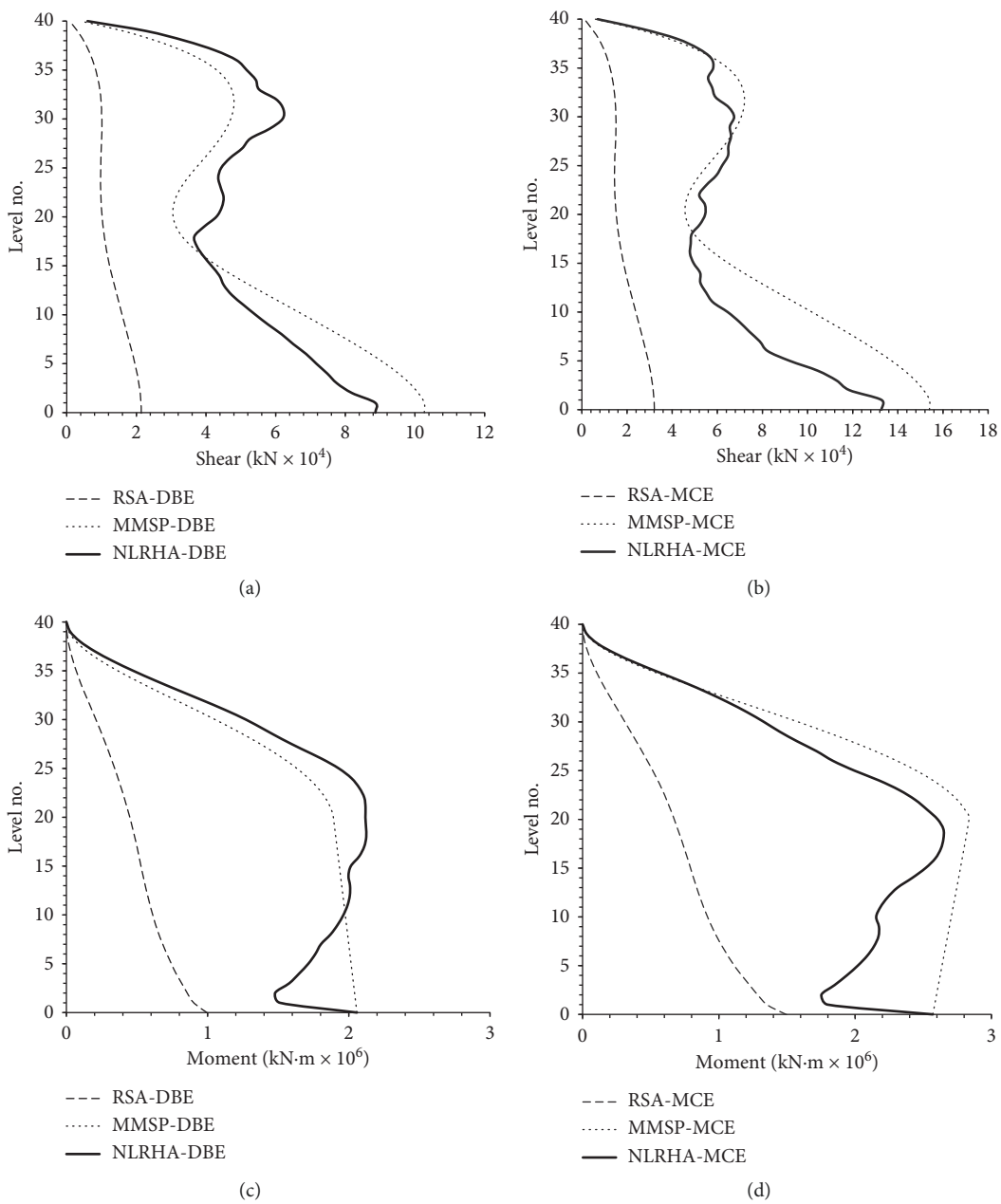

Figure 5: Evaluation of seismic demands by the RSA procedure and MMSP with the NLRHA procedure for ground motion HM: (a) shear at DBE level; (b) shear at MCE level; (c) bending moment at DBE level; (d) bending moment at MCE level.

of the case study building. Although MPA and MMPA procedures show better results, these procedures require nonlinear modelling. To further explore the results from different procedures, a modal decomposition technique is used in the current study to separate the total true seismic demands from NLRHA to their respective modal demands. This allows the mode by mode comparison of true and predicted seismic demands of various procedures and hence can give better understanding of the shortcomings in different procedures.

For this purpose, a modal decomposition method called the uncoupled modal response history analysis (UMRHA) procedure has been adopted in the next section to explore and understand the discrepancies in different simplified analysis procedures.

4.6. UMRHA Procedure. The UMRHA procedure has been developed by Chopra and Goel. Initially, this procedure was used to calculate the target drift of the SDOF system which is then used in the MPA procedure to determine the modal seismic responses. However, this procedure is used in the recent past by many researchers to comprehend the complex seismic responses of buildings by disintegrating the total seismic response into modal responses. The UMRHA procedure has also been recently used by Tahir et al. as a

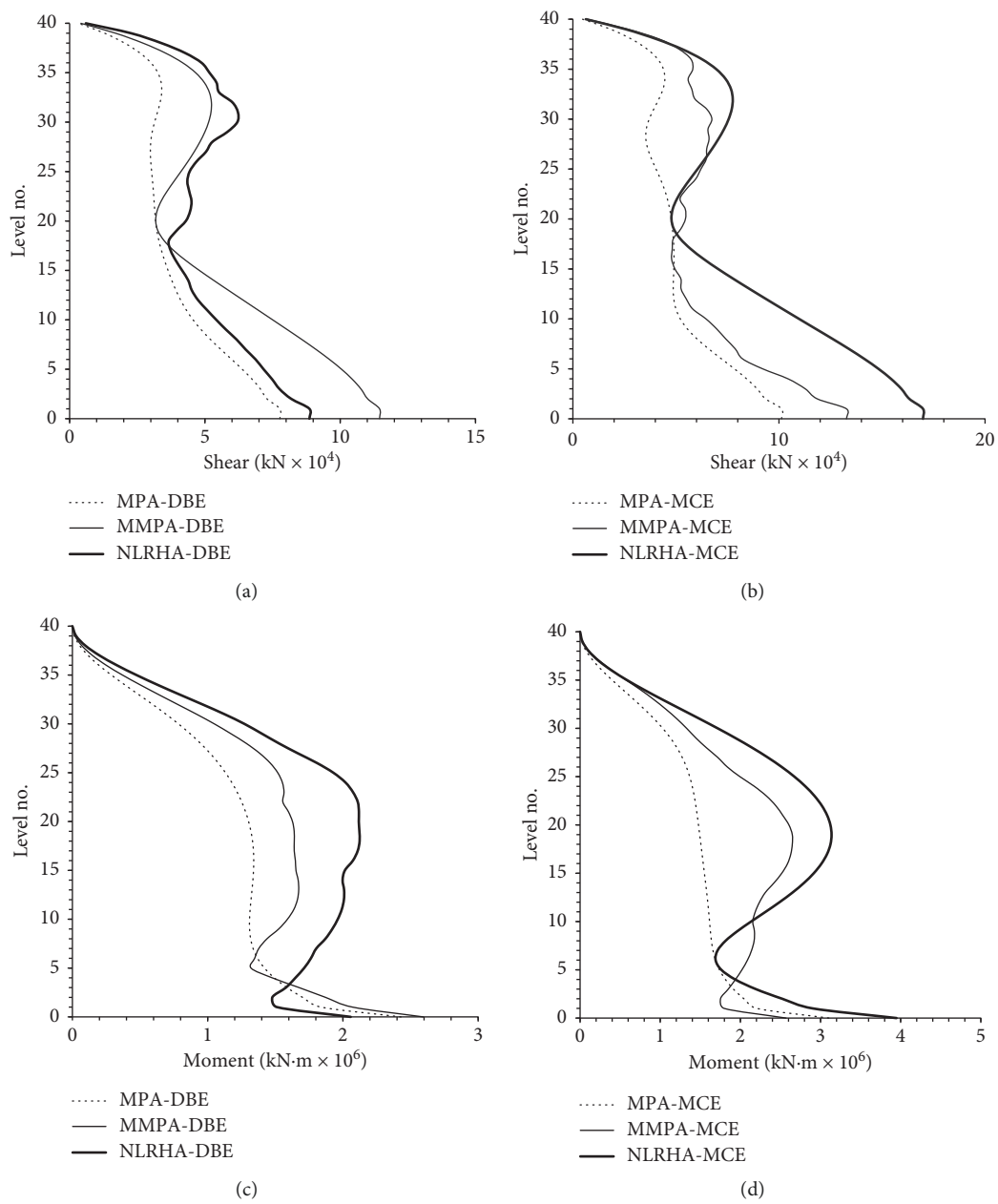

Figure 6: Evaluation seismic demands by MPA and MMPA with the NLRHA procedure for ground motion HM: (a) shear at DBE level; (b) shear at MCE level; (c) bending moment at DBE level; (d) bending moment at MCE level.

simplified analysis procedure to accurately determine the seismic demands of high-rise buildings [3]. UMRHA takes less time as compared to the NLRHA procedure.

The equation of motion for a multidegree of freedom (MDOF) system subjected to a horizontal ground motion in the nonlinear range is

$$M\ddot{u}(t) + C\dot{u}(t) + f_s(u, \dot{u}) = -M\iota\ddot{u}_g(t), \quad (1)$$

where C and M are the damping constants and story lumped mass matrices, $u(t)$ denotes the displacement vector consists of N lateral floor displacements, f_s denotes the lateral resisting force vector, and ι denotes the influence vector of the MDOF system.

The floor displacement for Equation (1) can be shown as sum of the modal responses:

$$u(t) = \sum_{i=1}^{N} \phi_i q_i(t), \quad (2)$$

where $q_i(t)$ is the modal coordinate of the ith mode and ϕ_i is the mode shape vector of the ith natural vibration mode within the elastic range. The UMRHA procedure assumes that Equation (2) is approximately valid for buildings in the nonlinear range.

$M\iota\ddot{u}_g(t)$ is the effective earthquake force and its spatial distribution is defined as $s = M\iota$ and can be described as a sum of modal inertial forces distribution:

$$-M\iota\ddot{u}_g(t) = -s\ddot{u}_g(t) = \sum_{i=1}^{N} -s_i\ddot{u}_g(t) = \sum_{i=1}^{N} -\Gamma_i M\phi_i\ddot{u}_g(t), \tag{3}$$

where $\Gamma_i = \phi_i^T M\iota/\phi_i^T M\phi$.

The seismic response of the structure against i^{th} modal inertial force vector is expressed by

$$M\ddot{u}(t) + C\dot{u}(t) + f_s(u, \dot{u}) = -s_i\ddot{u}_g(t). \tag{4}$$

The response vector $u(t)$ in Equation (4) can be substituted by $\phi_i q_i(t)$. Premultiplying Equation (4) by ϕ_{iT}, we obtain:

$$M_i\ddot{q}_i + 2\zeta M_i\dot{q}_i + F_s(q_i, \dot{q}_i) = -\Gamma_i M_i\ddot{u}_g(t), \tag{5}$$

where $M = \phi_i^T M\phi_i$ and ω_i and ζ_i denote the angular frequency of vibration and damping ratio of the ith mode, respectively. The lateral resisting force $F_{si} = \phi_i^T f_s(u = \phi_i q_i, \dot{u} = \phi_i\dot{q}_i)$, and it is therefore a nonlinear function of q_i and $\dot{q}_i$.

A new modal coordinate $D_i(t)$ is introduced and is given by

$$\dot{q}_i(t) = \Gamma_i D_i(t). \tag{6}$$

Equation (5) can be written as

$$D_i + 2\zeta_i\omega_i\dot{D}_i + \frac{F_{si}(D_i, \dot{D}_i)}{L_i} = \ddot{u}_g(t), \tag{7}$$

where $L_i = M_i\Gamma_i$.

Equation (7) is the governing equation of motion for inelastic SDOF systems in the standard form. The inelastic response time history of D_i can be determined from Equation (7) if the nonlinear function $F_{\text{si}}(D_i, \dot{D}_i)$ is known. Chopra and Goel showed that $F_{\text{si}}(D_i, \dot{D}_i)$ can be accurately defined by a simple bilinear hysteretic curve for steel moment-resisting frame building. The bilinear hysteretic curve was obtained by standard pushover analysis using the ith modal inertial force distribution $s_i = M\phi_i$. Due to strength and stiffness degradation under cyclic loading in reinforced concrete (RC) special moment-resisting frame buildings, the bilinear hysteric curve does not accurately describe the true nonlinear force-deformation relationship of the structures. Therefore, Bobadilla and Chopra [31] proposed a cyclic pushover analysis to obtain the nonlinear curve of structures where their modal hysteretic behavior is not known.

The ith mode's cyclic pushover analysis can be performed by applying a force vector with the ith modal inertial force pattern s_i^* to the structure together with gravity loads. The lateral force vector is altered in magnitude and then reversed, while the gravity loads remain in its position, to create cyclic responses with gradually increasing amplitude. With above force distribution, the lateral displacement response and the other seismic responses are considered to be dominated by those of the ith mode, and therefore, the roof displacement (u_i^r) and D_i are approximately given by

$$D_i = \frac{u_i^r}{\Gamma_i\phi_i^r}, \tag{8}$$

where ϕ_i^r is the value of mode shape at the roof level.

The base shear V_{bi} and resisting force F_{si} with the ith mode inertial force distribution pattern is given by the following relationship:

$$\frac{F_{\text{si}}}{L_i} = \frac{V_{\text{bi}}}{\Gamma_i L_i}. \tag{9}$$

The cyclic pushover results are first expressed in the form of the cyclic base shear (V_{bi})-roof displacement (u_i^r) relationship and then presented in the $F_{\text{si}} - D_i$ relationship using Equations (8) and (9) and is shown in Figures 7(a) and 7(b). Against a low-displacement demand, the structure behaves in a linear elastic manner and the gradient of the force-deformation curve represents the initial linear elastic stiffness. As soon as the roof displacement increases against an increasing load, the building goes into inelastic range and the tension cracks appear in RC walls and columns resulting in softening of the overall structural stiffness as shown in the Figure 7(a). During the unloading phase, the cracks in the RC walls and columns tend to close under the action of high gravity force and low reinforcement ratio. As the cracks fully close, the structure almost completely restores its initial stiffness, resulting in a minimal residual drift. This causes the structure to exhibit a bilinear elastic or flag-shaped hysteresis behavior. Similar behavior has been observed by Adebar et al. [32] during experimental testing of a large-scale high-rise wall model simulating the first mode lateral load pattern. To model this flag-shaped behavior obtained from pushover analysis in Perform-3D, Ruaumoko-2D program [33] is used which has a readily available flag-shaped hysteretic model. The comparison of actual and idealized hysteretic behaviors is shown in Figure 7(b).

Equations (8) and (9) are then used to compute the deformation and force related modal responses of the MDOF system.

The symbol $R_i(t)$ is generally used to represent each of deformation-related or force-related responses of the ith vibration mode. These individual responses are then combined in the time domain to obtain the total response $R(t)$ of the structure as follows:

$$R(t) = \sum_{i=1}^{n} R_i(t), \tag{10}$$

where n is the number of vibration modes.

4.6.1. UMRHA Results and Discussion. The UMRHA procedure is used to determine the seismic demands of the 40-story building to each of the three ground motions, both for the DBE and MCE levels. The modal seismic response time histories are first determined and then combined in the time domain for each of the floor level using Equation (10) to obtain the total seismic response time history of the building. The modal response time histories are determined for the first four translational vibration modes since the contribution from other higher modes is very low. The total seismic response time history at each floor obtained from the UMRHA procedure for each of the ground motion is

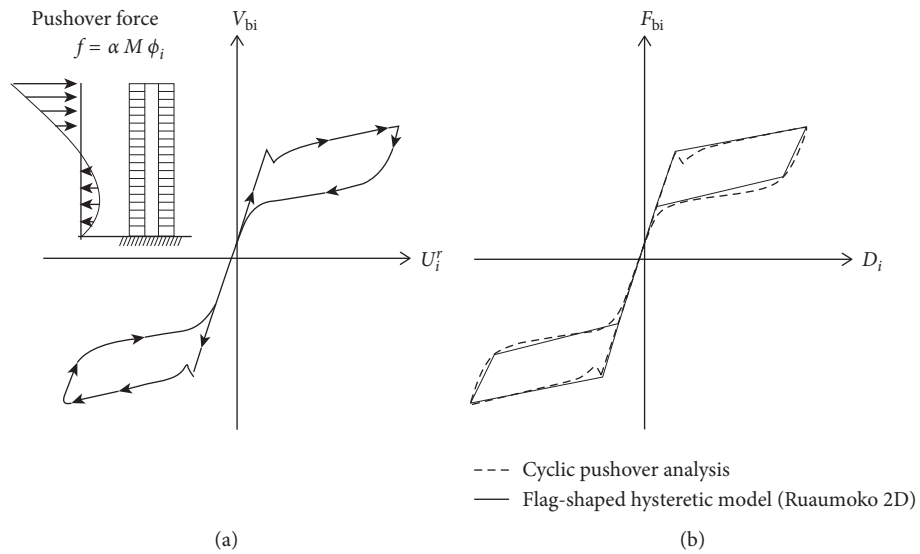

Figure 7: Hysteretic behavior of the ith mode inelastic SDOF system: (a) cyclic pushover curve; (b) F_{si}-D_i relationship.

expected to match with those determined from the NLRHA procedure. The maximum responses throughout the total time history for each floor is obtained and then compared with maximum responses from the NLRHA for HM ground motion as shown in Figures 8(a)–8(d). The comparisons show that the shear forces and the bending moments obtained from the UMRHA procedure for each ground motion matches well with those of the NLRHA procedure.

The modal shear forces and bending moments for the ground motion HM at MCE level is shown in the Figures 9(a)–9(d). Figures 9(a) and 9(b) illustrate that the shear demand is dominated by the 2nd and 3rd modes, while Figures 9(c) and 9(d) show that the moment demand is controlled by the first three vibration modes. The 1st mode attains its highest shear and moment demand at the base decreasing along the height, while the higher modes attain maximum and minimum values of shear force and bending moment at certain heights. The modal decomposition results of the shear demand show that, from the base to level 6, shear force is dominated by the second and third vibration modes, while, at level 13, it is dominated by the first two modes and the fourth mode. The modal decomposition results of the moment demand show that, at level 7, moment demand is controlled by 1st mode while higher modes moment demand at this level is approximately equal to zero. On the contrary, the moment demand at story level 20 is dominated by the second mode.

The UMRHA procedure has been confirmed in the above section to be reasonably accurate, and hence the modal results can be analyzed to draw conclusions about the simplified analysis procedures. Reason behind the underestimation of seismic demands by simplified analysis procedure can be comprehended from the modal decomposition results of the UMRHA procedure. The mode wise seismic demands comparison of all simplified procedures with that of the UMRHA procedure have been presented in the Figures 10(a)–10(d) and 11(a)–11(d) for ground motion HM at MCE level. The comparison in Figure 10(a) shows that the inelastic shear forces at each story level of the 1st mode computed by the RSA procedure matches well with 1st mode shear forces from the UMRHA procedure, while Figures 10(b)–10(d) shows that the inelastic shear forces at each floor of the higher modes computed by the RSA procedure are much lower than that of the UMRHA procedure. Similarly, the bending moment at each story level of the 1st mode determined by the RSA procedure is in good agreement with that of the UMRHA procedure. The bending moment at each story level of the higher modes computed by the UMRHA procedure is much higher than that of the RSA procedure.

Figures 10(a)–10(d) and 11(a)–11(d) also show the comparison of shear force and bending moment computed by the MPA procedure with the UMRHA procedure. The MPA procedure's first mode shear forces at each story level matches well with that of the UMRHA procedure. The 2nd mode shear forces of the MPA procedure are also in good agreement. The other higher mode shear forces are also in good correspondence with the UMRHA procedure. Similarly, the bending moment at each story level computed by the MPA procedure is close with that computed by the UMRHA procedure. The higher mode seismic demands are in good agreement due to the use of nonlinear hysteric behavior in seismic demand computation. The overall results of the MPA procedure are lower due to modal combination rule. Also, it is important to note that the UMRHA procedure also showed some underestimation.

The shear force and bending moment at each floor computed by the MMPA procedure for first, third, and fourth mode is the same with those of the MPA procedure and match well with those of the UMRHA procedure, but the second mode shear force and bending moment are much higher than those of the UMRHA procedure. This is due to the fact that the MMPA procedure considered the second mode to be elastic, but UMRHA results are showing yielding in the second mode.

The MMSP procedure's shear force and bending moment at each story level for the 1st mode is the same with

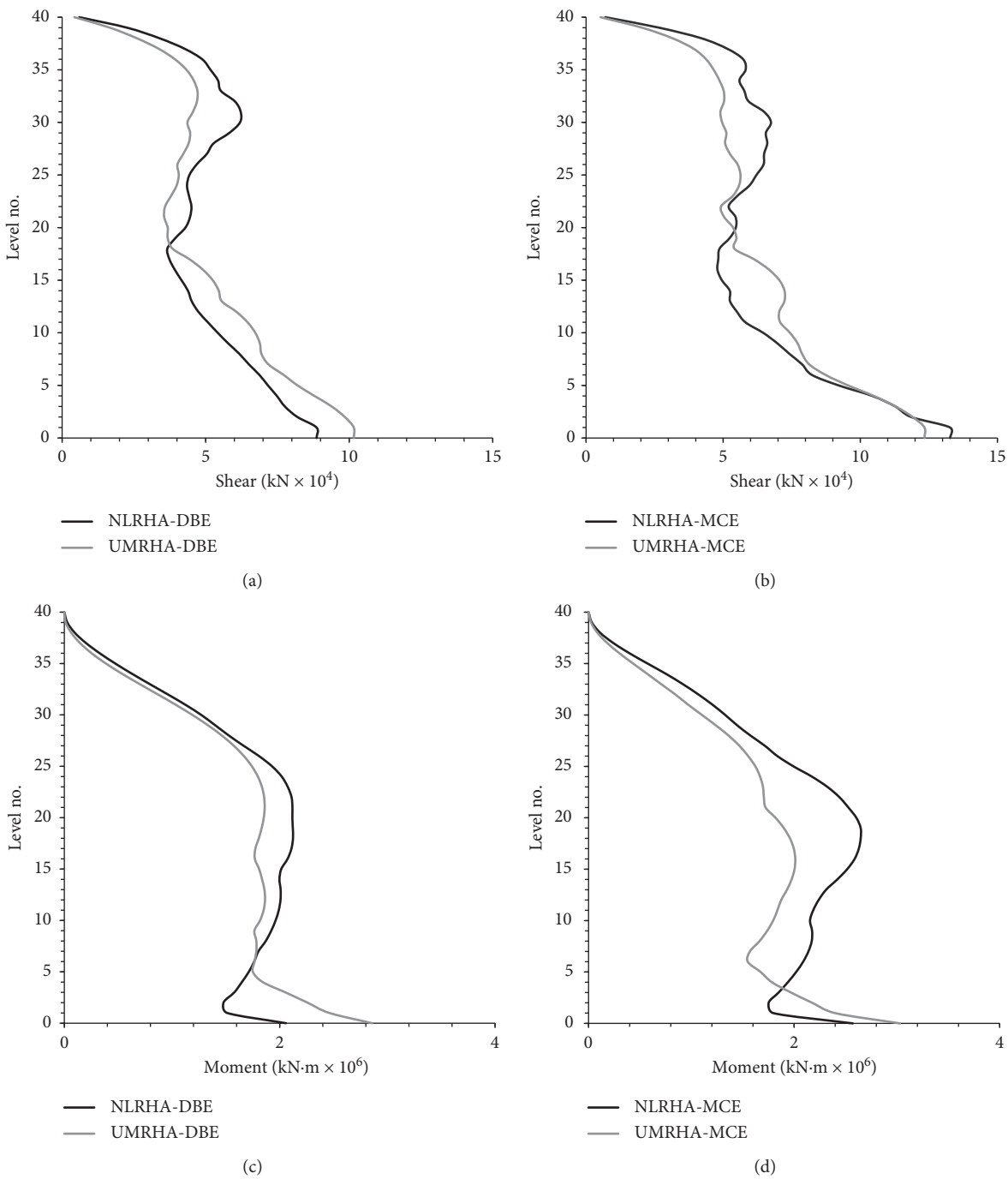

Figure 8: Comparison of peak shear and peak moment demands between UMRHA and NLRHA procedures for both DBE and MCE levels for ground motion HM: (a) shear at DBE level; (b) shear at MCE level; (c) bending moment at DBE level; (d) bending moment at MCE level.

those of the RSA procedure and is in good correspondence with those of the UMRHA procedure. The 2nd mode shear force and bending moment of the MMSP procedure is much higher than those of the UMRHA procedure. The 3rd and 4th mode shear force and bending moment at each floor level of the MMSP procedure matches well with those of the UMRHA procedure as it remains in the elastic region in both procedures. From the above discussion, it is concluded that the main source of error in determining the seismic demand through simplified analysis procedures lies in the 2nd mode seismic demand.

To further explore the reason of inaccuracies in simplified analysis procedures, the force-deformation relations of first four modes for ground motion HM at MCE level, as illustrated in the Figures 12(a) and 12(d), are considered. The hysteretic behavior for the first and second modes is flag shaped, while it is straight line for other higher modes which shows that, for the first two modes, the building goes into the nonlinear range as the base moment demand exceeds the flexural yield capacity. When the yielding occurs at the base and the deformation continues, then the base moment

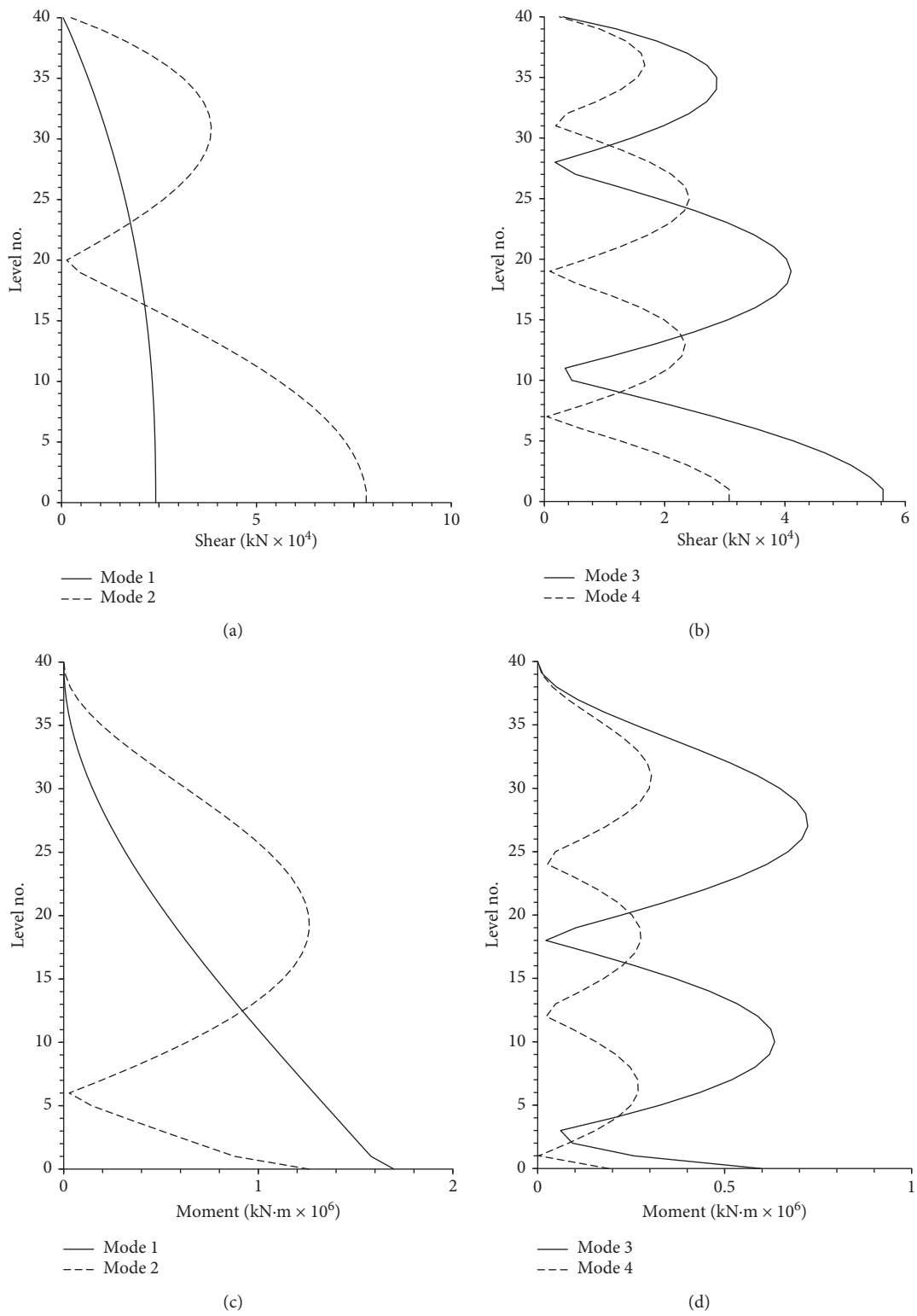

Figure 9: Modal peak shear and moment demands from the UMRHA procedure for MCE level ground motion. Modal peak shear of (a) 1st and 2nd modes; (b) 3rd and 4th modes. Modal peak moment of (c) 1st and 2nd modes; (d) 3rd and 4th modes.

increases but at much lower rate due to low postyield stiffness as compared to the initial stiffness. Therefore, the base moment will saturate around the flexural yielding strength and consequently the shear force and moment of each floor level will also saturate for that mode. For the first mode, the seismic responses from the UMRHA procedure are therefore equal to the 1st mode seismic demands obtained by the RSA procedure. The slight difference between the 1st mode responses is due to the actual material strength used in the nonlinear model. The 1st mode seismic demands

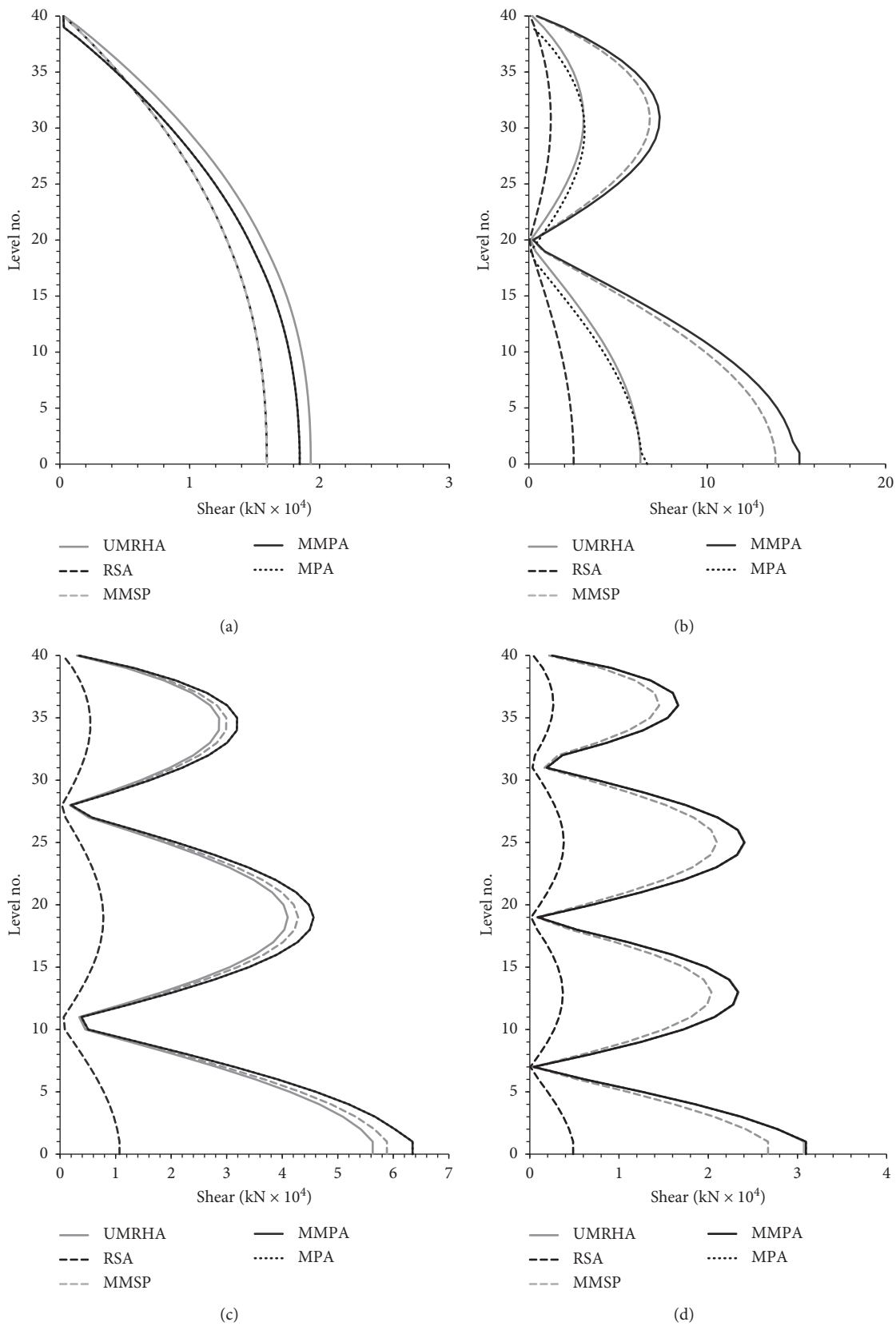

Figure 10: Comparison of modal shear MCE level demands of RSA, MMSP, MPA, and MMPA with that of the UMRHA procedure: (a) first mode; (b) second mode; (c) third mode; (d) fourth mode.

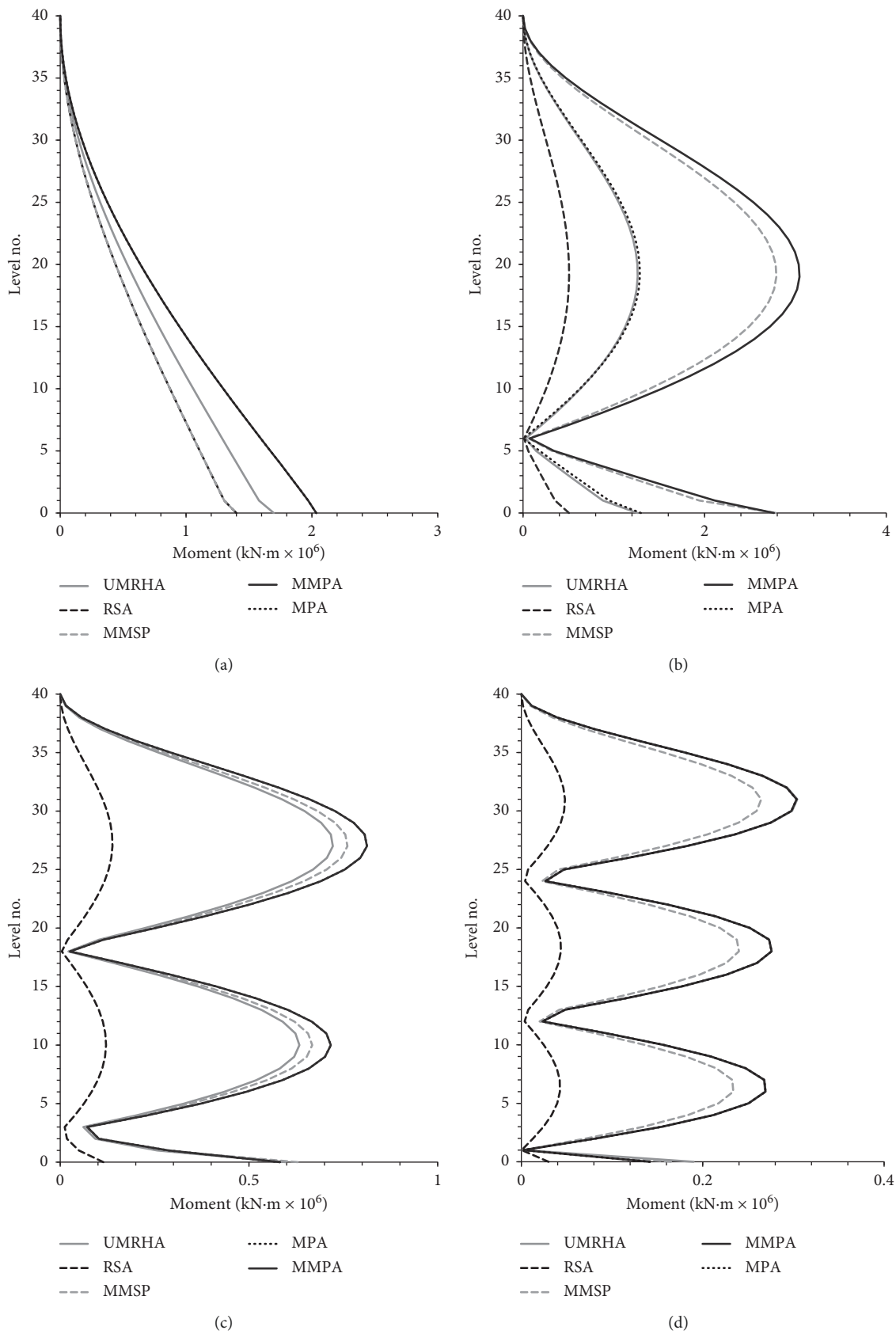

Figure 11: Comparison of modal moment MCE level demands of RSA, MMSP, MPA, and MMPA with that of the UMRHA procedure: (a) first mode; (b) second mode; (c) third mode; (d) fourth mode.

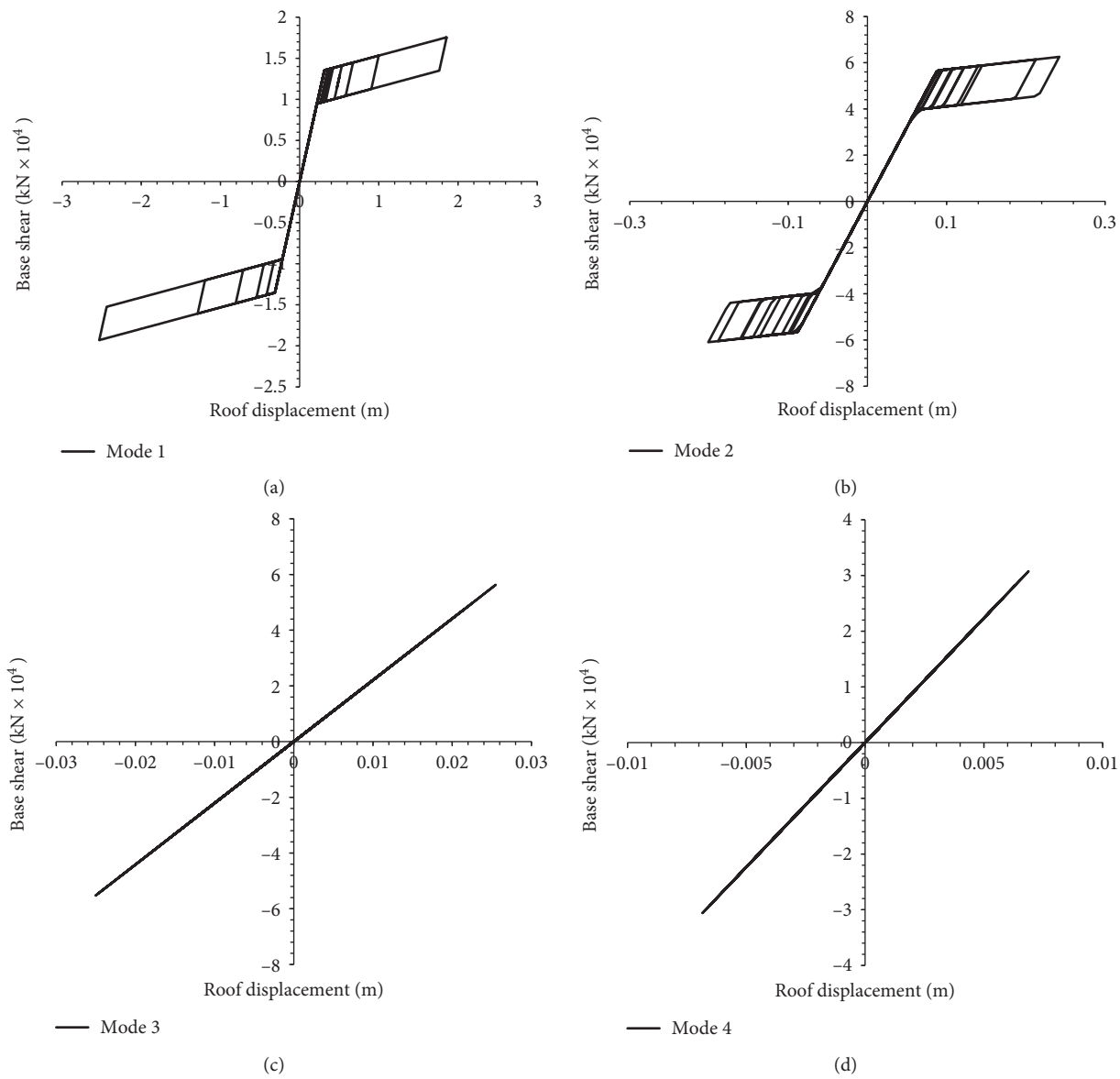

Figure 12: Hysteretic behavior of first four translational modes: (a) first mode; (b) second mode; (c) third mode; (d) fourth mode.

of the MMSP procedure are also the same as those of the RSA procedure as these are reduced by the same R factor. The MPA and MMPA procedures use the nonlinear force-deformation relationship for computing the 1st mode seismic demand, and therefore, it matches well with 1st mode seismic demand of the UMRHA procedure. The hysteretic behavior of the 2nd mode also shows yielding but it is less extensive as compared to that of the 1st mode yielding. The force-deformation relationship of the 2nd mode is also flag-shaped self-centering as shown in the Figure 12(b). The seismic demands of the 2nd mode are also limited at the level when the base moment of the 2nd mode reaches flexural yielding strength. However, the RSA procedure assumes the same R factor for second and higher modes which UMRHA results have shown to be inaccurate and the 2nd mode is shown to exhibit a much less nonlinear behavior producing a smaller R value for this mode. The MMSP's 2nd mode seismic demands are the elastic demands of the RSA procedure and are much higher than the seismic demands from the UMRHA procedure. Only the MPA 2nd mode seismic demands match with the seismic demands of the UMRHA procedure as it incorporates the nonlinear behavior into seismic demand computation. The 2nd mode demands from MMPA are the elastic demands and are much greater than the seismic demands from the UMRHA procedure. The hysteretic behavior of the third and fourth mode are illustrated in Figures 12(c) and 12(d), and it shows that these higher modes remain in the elastic region and there is no yielding occurring in these modes to reduce or limit their seismic demands. Consequently, the 3rd and 4th mode seismic demands from UMRHA are much higher than the seismic demands of the RSA procedure and equal to the elastic seismic demands of the RSA procedure. The MMSP 3rd and 4th mode demands match well with those of the

UMRHA procedure. The MPA and MMPA procedure 3rd and 4th mode seismic demands are equal to the UMRHA procedure. If these modes seismic demands were higher than yield moment at the base, then the MMSP and MMPA procedure seismic demands will be inaccurate, while MPA procedure will accurately estimate the seismic demands because it uses the actual nonlinear behavior of the building. The above discussion suggests that the response modification factor (R) from the RSA procedure is suitable only for the 1st mode, while a smaller value of force reduction factor should be used for the 2nd mode, and the higher modes should not be reduced by any force reduction factor as these remain in the elastic region. Although the modal seismic demands of the MPA procedure match well with those of the UMRHA procedure, the total seismic demands of the MPA procedure differ. This is due to the use of the modal combination rule for determining the total seismic demands from modal seismic demands in the MPA procedure in contrast to the addition of modal time histories to get the total response in the UMRHA procedure. The above discussion suggests that the current simplified procedures cannot be reliably used for the prediction of seismic demands of high-rise core wall structures. Therefore, an effort is made in the next section to further explore the reasons of inaccuracies and to come up with a simplified yet effective procedure with improved accuracy.

5. Proposed Modifications in Simplified Analysis Procedure

Based on the reasons of inaccuracies in simplified analysis procedures discussed in the previous section, we can suggest some modifications in simplified analyses procedure to improve their accuracy. As the MPA and MMPA procedures require nonlinear modelling, these are not used in the current section. The RSA procedure, on the contrary, does not require nonlinear modelling, is easy to use and most practicing engineers are aware of this method and hence is used in this section.

The RSA procedure assumes that inelastic demands of each mode can be calculated by dividing their elastic seismic demands with a constant response reduction factor but as the aforementioned discussion shows only the 1st mode seismic demands reduce by the code-based response reduction factor while higher modes do not reduce by the same factor. Therefore, each modal elastic demand computed by the RSA procedure should be divided by separate response modification factor and then combined by SRSS combination rule to obtain the total seismic demand of the high-rise building. To calculate the response modification factor for each mode, a rather simple method is adopted. The elastic modal overturning moment at the base for each mode is divided by the yield base moment capacity/design base moment capacity of the building. A value equal to or less than unity means the absence of yielding in that mode, and hence, the elastic seismic demand for that particular mode is used. The value of the force reduction factor obtained for the 1st mode of DBE level ground motion HM is 4.3 which is smaller than the code-based value of 5.5, while it is 6.4 for MCE level ground motion which is greater than the code-based value. The force reduction factor for the 2nd mode is 1.53 for DBE level, while it is 2.30 for MCE level ground motion. These values for 2nd mode are much smaller than the code-based values used in the RSA procedure. The higher mode elastic modal moments at the base are smaller than the yield moment capacity at the base. Therefore, the force reduction factor for these higher modes is unity. After obtaining the value of each modal force reduction factor, the elastic modal shear forces and bending moments at each floor computed by the RSA procedure are divided by these force reduction factors and then combined by the SRSS combination rule to get the total seismic demands.

The seismic demands obtained by this modified RSA procedure are then compared with seismic demands of RSA, MPA, and NLRHA procedures for ground motion HM for both DBE and MCE levels as shown in Figures 13(a)–13(d). The results show that the seismic demands computed by this modified method are much higher and accurate than those of the RSA procedure. It is important to note here that this modified RSA procedure does not require nonlinear modelling, and the accuracy is comparable to that of the MPA procedure which requires nonlinear modelling and pushover analysis. However, still this procedure is underestimating the seismic demands and hence needs to be improved. As discussed earlier in UMRHA results, the modal pushover behavior shows yielding at the base against both 1st and 2nd mode seismic demands, while higher modes remain in the linear elastic range. However, in a real structure with the coupling of different modes, such a situation seems unrealistic. As a structure yields at base against the 1st mode forces, it is physically not possible for the higher mode seismic demands to cause another yielding at the same point. However, higher modes can cause yielding at the midheight where the seismic demands from the 2nd mode can exceed the midheight moment capacity, but as the case study building is modelled as elastic above the base, a yielding against higher modes above the base is not possible. This implies that the MMPA procedure and MMSP should be able to accurately estimate the seismic demands as these procedures consider yielding in only first mode. However, as discussed earlier, these procedures also did not accurately estimate the seismic demands of the case study building. The first reason behind this is that when the 1st mode yields and plastic hinge forms at the base, the stiffness at the base reduces, and due to the reduction in the stiffness, the higher mode natural time period increases. When the natural time period increases, the spectra show a decrease in the pseudoacceleration due to which the 2nd mode seismic demands decrease. The second reason behind the inaccuracy of the MMSP and MMPA procedure is that the when the 1st mode yields at the base, then the pattern of mode shapes also changes due to change in the boundary condition while these procedures are based on the elastic mode shape patterns.

The modified RSA procedure accurately estimates the seismic demands of the case study building compared to the RSA and matches well with MPA procedure, but still it underestimates the seismic demand compared to UMRHA and NLRHA procedures. Qureshi [34] and Pennucci et al. [35] showed that when the 1st mode yields at the base, then

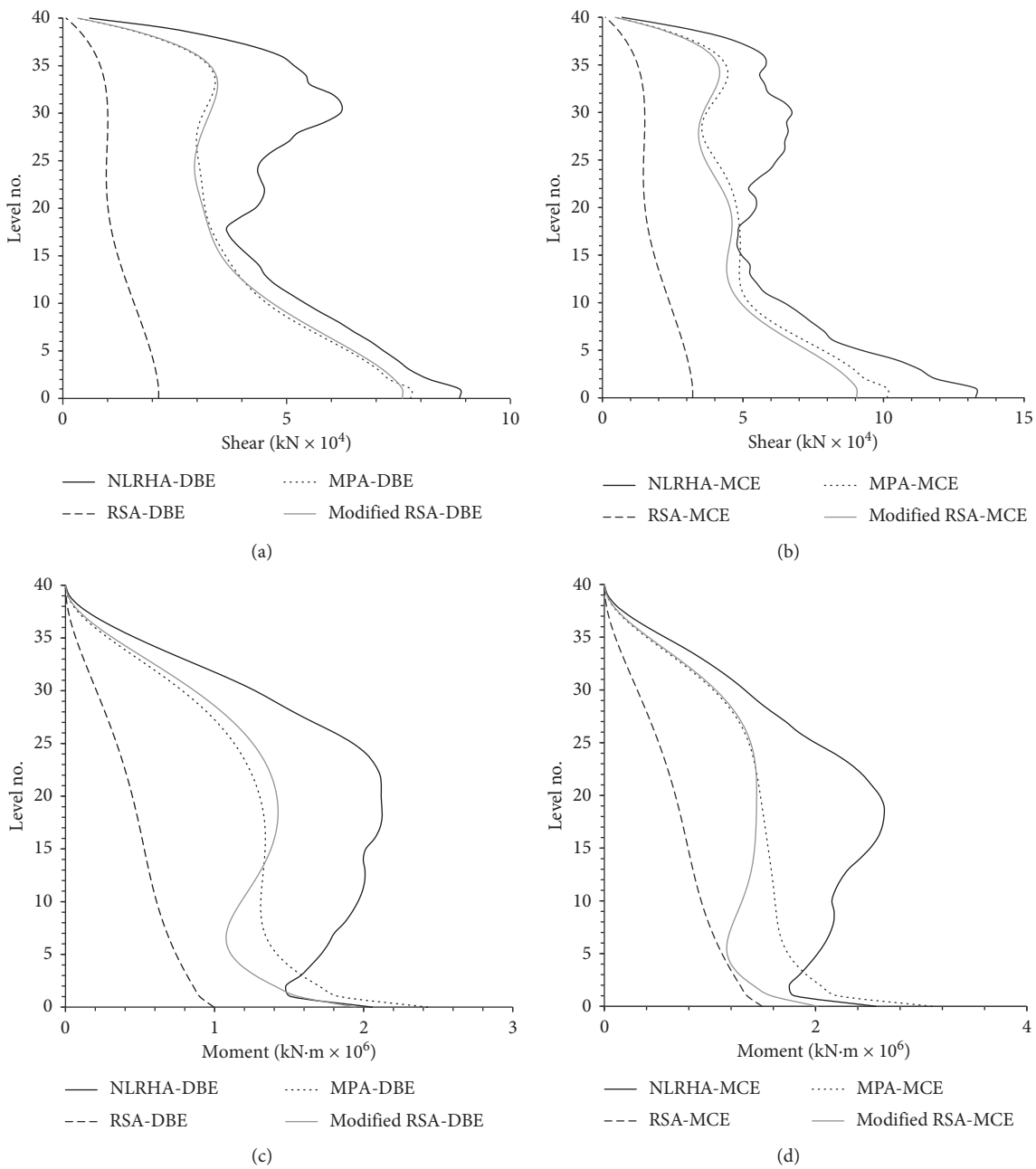

Figure 13: Evaluation of shear and moment demands of the modified RSA procedure with RSA, MPA, and NLRHA procedures for ground motion HM for both DBE and MCE levels: (a) shear at DBE level; (b) shear at MCE level; (c) bending moment at DBE level; (d) bending moment at MCE level.

plastic hinge forms which changes the mode shape pattern and modal responses. The modal periods also lengthen when the 1st mode yields. Therefore, the use of separate R factor for higher modes is not a feasible solution in the current case. In fact, the lengthened modal periods should be used for determination of seismic demands for higher modes.

To calculate the reduced modal stiffness, Sullivan used secant stiffness to find out the lengthened time periods and eventually used them to calculate the nonlinear modal responses of higher modes. Irshad [34] used postyield and secant and tangent stiffness to estimate the reduced modal stiffnesses. Secant stiffness was found out to give relatively accurate results. However, that study used the direct displacement based design (DDBD) for the design procedure which means that the total deformation demand was known at the start, and hence, it can be used to calculate the secant stiffness by joining the origin with the maximum deformation point on the force-deformation curve. In the current study, deformation demand is not readily available; however, the force-deformation hysteresis behavior of the structure against ground motions calculated with the UMRHA procedure can be used to calculate the maximum deformation to be used for the secant stiffness. This deformation value can also be calculated by using the natural

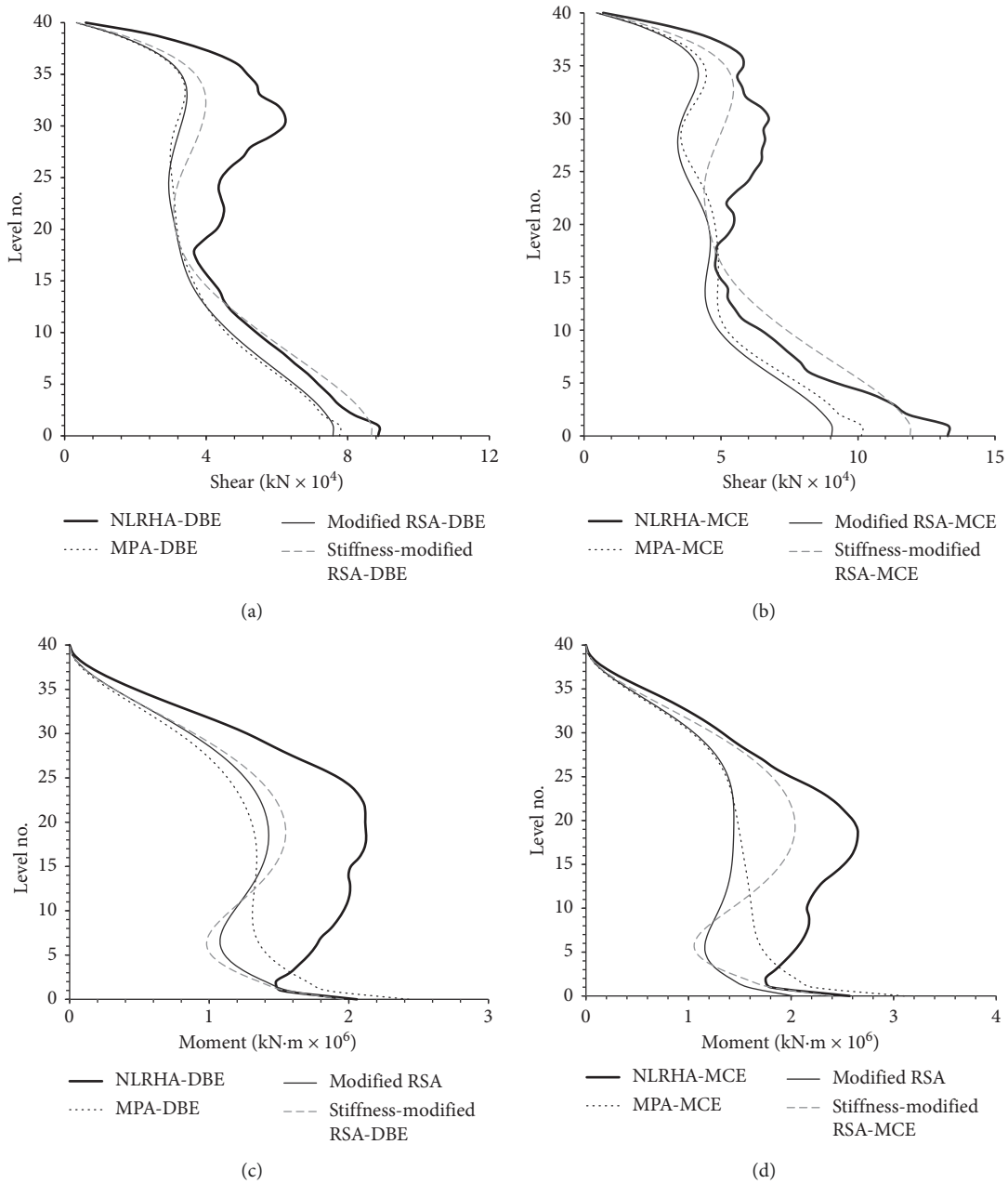

Figure 14: Evaluation of shear and moment demands of stiffness-modified RSA procedure with Modified RSA, MPA, and NLRHA procedures for ground motion HM for both DBE and MCE levels: (a) shear at DBE level; (b) shear at MCE level; (c) bending moment at DBE level; (d) bending moment at MCE level.

time period of the structure using design displacement spectra by assuming that the equal displacement assumption holds which states that the linear and nonlinear deformations in a structure are approximately equal. It is important to mention here that the yielding does not only lengthen the time periods but also changes the shape of the inertial modal force patterns. However, this change in the modal shape pattern is ignored in the current study for simplicity.

The seismic demands of higher modes are obtained by using the secant stiffness for both DBE and MCE levels and joining the 1st mode inelastic or design demands with higher modes seismic demands with elongated period. Figures 14(a)–14(d) shows that both the shear force and bending moment results from the abovementioned formulation (stiffness-modified RSA procedure) are more accurate as compared to the modified RSA and MPA procedures. The limitation of this new procedure is that the modal secant stiffness cannot be obtained if the elastic modelling is considered. Hence, future work should focus on the calculation of deformation demand to obtain the secant stiffness. One possible solution to be considered is to assume the postyield stiffness of the structure as a percentage of initial stiffness which is known. Then calculate the maximum deformation demand from the displacement spectrum by relying on the equal displacement assumption. Now the secant stiffness can easily be calculated by joining

origin with maximum deformation point, and all of this can be done without any nonlinear modelling and pushover analysis.

6. Conclusions

The current study is focused on the comparison of a number of simplified analysis procedures based on their relative accuracy and ease of application to a real structure. For this purpose, a 40-story RC core wall building was selected to compute the nonlinear seismic demands using the various simplified analysis procedures and NLRHA procedure. Comparison of individual simplified analysis procedures with the NLRHA procedure shows that ELF and RSA procedures highly underestimate the seismic demands while the MMSP and MMPA procedures overestimate the seismic demands. Only the MPA procedure seismic demands are comparable with the NLRHA procedure. To further explore the seismic responses and understand the reason of inaccuracies of simplified analysis procedures, a modal decomposition method is employed to decompose the total responses into model responses. The modal decomposition results show that actual modal seismic demands of the 1st mode reduced by the same code-based R factor used in the RSA procedure and MMSP procedure and matches well with the 1st mode seismic demands of MPA and MMPA procedures. The modal seismic demands of the 2nd mode of RSA procedure does not reduce by the code-based response reduction factor. MMSP and MMPA procedure seismic demands of the 2nd mode were higher than the actual seismic demands. Only the 2nd mode seismic demands of the MPA procedure matched with true seismic demands of the 2nd mode as the MPA procedure considers the actual nonlinear behavior in the model. The actual modal seismic demands of the 3rd and 4th mode are much higher than the modal seismic demands of the RSA procedure due to the use of R factor for all modes and match well with modal seismic demands of MMSP, MPA, and MMPA procedures as these higher modes were found to remain in linear elastic range. The reason behind the inaccuracy in simplified analysis procedures lie in the computation of the 2nd mode seismic demands. However, the modal seismic demands of MPA procedures for all modes match well with actual modal seismic demands but still the total seismic demands are inaccurate due to the use of modal combination rule and use of elastic mode shapes in computing the modal demands. In the current study, a modified RSA procedure was suggested to compute the seismic demands by using separate force reduction factors for each mode to reduce the elastic demands into nonlinear seismic demands. The results of this procedure, based on linear elastic modelling and without any need for pushover analysis, are comparable with those of the MPA procedure for the present case study building, but still it underestimates the seismic demands. To further improve the results of this modified RSA procedure, nonlinear modal properties are calculated by using secant stiffness. These modified modal properties are then used to calculate the higher mode seismic responses and are then combined with 1st mode seismic demands based on R factor. The proposed procedure is found to be simple to use with a higher level of accuracy.

Abbreviations

NLRHA: Nonlinear response history analysis
RC: Reinforced concrete
UMRHA: Uncoupled modal response history analysis
ELF: Equivalent lateral force
RSA: Response spectrum analysis
MMSP: Modified modal superposition procedure
WCD: Weighted capacity design
MPA: Modal pushover analysis
NSP: Nonlinear static procedure
MMPA: Modified modal pushover analysis
SDOF: Single degree of freedom
MDOF: Multidegree of freedom
MCE: Maximum considered earthquake
DBE: Design basis earthquake
TBI: Tall Building Initiatives
SRSS: Square root of the sum of square method
CQC: Complete quadratic combination
R: Force reduction factor.

Data Availability

The data of this research article are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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