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Assessment of current nonlinear static procedures on the estimation of torsional effects in low-rise frame buildings

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Abstract

The capability of current nonlinear static procedures in capturing torsional effects was evaluated by comparing their estimates to the "exact" results, obtained from nonlinear response history analyses under a ground motion ensemble scaled to 3 different hazard levels. Two different types of eccentricities were considered. In the first case, the mass center of the floor of a symmetric plan structure was shifted in one direction to create a uni-directional eccentricity and in the other case a structure with a plan asymmetry in both horizontal directions was used. Results of numerical analyses show that far-fault and near-fault ground motions have similar influences on the displacement demand of structures as far as torsional effects are concerned. The recent nonlinear static procedures proposed for asymmetric buildings were found to be effective in capturing torsional effects whereas the classical nonlinear static procedure developed originally for planar systems significantly underestimates torsional rotation demands in structures.

Keywords: Nonlinear static procedures; Asymmetric structures; Torsional effects; Seismic evaluation

1. Introduction

One of the crucial steps of performance based design procedures is the determination of the displacement demand of structures under the seismic actions defined by either acceleration time history or design response spectrum methods. The most accurate way to compute the displacement demands of a structure under a given seismic action is to carry out a nonlinear response history analysis (RHA) of the detailed three dimensional (3D) mathematical model of the structure. However, this is a tedious procedure and is accepted to be "unpractical" for design purposes by the earthquake engineering community. Hence, nonlinear static analysis procedures (NSP) [1–3] attract the attention of both practicing engineers and the research community in the sense that it is more practical and faster to implement [4–6]. Nevertheless, being an approximate method, nonlinear static analysis has certain shortcomings and researchers have put significant efforts into overcoming these shortcomings (e.g. [4,6–10]). However, most of the research on nonlinear static analysis procedures has been limited to planar structures.

Post-earthquake field surveys show that torsionally irregular buildings sustain more damage compared to regular ones. Irregularities in stiffness and mass tend to influence the capacity and demand. Plan irregularities cause non-uniform damage among the structural elements of the same story due to non-uniform displacement demands [11]. In this context, it is of significant importance to be able to capture the variations of displacement demands within the same story due to torsional effects for reliable damage assessment of structures with asymmetric plans. As stated in the work by Chopra and Goel [12], current practice is based on judgmental extension of NSP originally developed for planar analysis of buildings to 3D analysis of structures with plan irregularities, which appear inaccurate in capturing torsional effects. More recently, efforts have been made to address and overcome the deficiencies of nonlinear static analysis procedures in estimating the response of structures with plan irregularities [12–18]. This paper aims to evaluate the capability and applicability of two of these methods [12,14] in capturing the torsional effects that arise from plan irregularities. Of the current methods available, these

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two methods were chosen based on the fact they are extensions to well-established and widely used nonlinear static analysis procedures for planar buildings [6,19]. Results of the nonlinear response history analyses (RHA) of two structures with irregularities in plan were compared with the estimates obtained from the nonlinear static analysis procedures considered. An ensemble of 30 ground motions, 15 far-fault and 15 near-fault, scaled to three levels of seismic intensity was used in the numerical analyses. The comparisons were made only in terms of the torsional effects, and the advantages and shortcomings of the procedures in determination of the demand parameters that also exist for planar structures are not discussed. The reader is referred to various papers in literature that evaluate current nonlinear procedures using planar buildings [3,4,20,23,24].

2. Summary of the procedures

2.1. Extension of N2 method for the pushover-based seismic analysis of plan-asymmetric buildings

The original N2 Method was developed by Fajfar [19] for the nonlinear static analysis of planar structures and it combines the pushover analysis of multi-degree-of-freedom (MDOF) models with the response spectrum analysis of an equivalent single-degree-of freedom system. Fajfar and his co-workers worked on the seismic behavior of plan-asymmetric buildings and observed that the torsional effects are mostly pronounced in the elastic range and early stages of plastic behavior and tend to decrease with an increase in the plastic deformations [21,22]. Hence, the amplifications in the displacement demands due to torsional effects computed from elastic dynamic analysis can be used as a rough and conservative estimate both in the elastic and inelastic range. Based on this observation, Fajfar et al. [14] developed the extension of the N2 method to plan-asymmetric buildings. The fundamental steps of the procedure are summarized below:

1. Pushover analysis of the 3D mathematical model of the building is performed independently in two horizontal directions and the target roof displacement for each horizontal direction is computed using the N2 method [19].
2. A linear response spectrum analysis of the 3D model is carried out independently in two horizontal directions and the results are combined using the SRSS rule.
3. The correction factors to be applied to the relevant results of pushover analysis are determined. The correction factor is defined as the ratio between the normalized roof displacements obtained by elastic modal analysis and by pushover analysis. The normalized roof displacement is the ratio of the roof displacement at an arbitrary location to the roof displacement at the center of mass (CM). If the normalized roof displacement obtained from elastic response spectrum analysis is less than 1.0, it is taken as 1.0.
4. All the relevant quantities obtained by pushover analysis are multiplied with the appropriate correction factors. Examples for the relevant quantities may be the deformation demand of the ductile elements and the force demand of the brittle elements.

Fajfar et al. [14] reported that, this procedure yielded conservative results in estimating the torsional effects for the structures and ground motions they tested.

2.2. Modal pushover analysis procedure to estimate seismic demands for asymmetric-plan buildings

Modal pushover analysis (MPA) was developed by Chopra and Goel [6] to be able to ensure that the higher mode effects are reflected in the nonlinear static analysis procedures. Later, Goel and Chopra [23] and Chintanapakdee and Chopra [24] reported that MPA yields superior results compared to the classical single mode pushover analysis as far as the higher mode effects are concerned. Though MPA was originally developed using planar structures, Chopra and Goel [12] extended the concept to estimate seismic demands of plan-asymmetric buildings. The outline of this procedure is summarized below:

1. The elastic natural frequencies, w_n, and mode shapes, ϕ_n, are determined.
2. For the nth mode, the base shear-roof displacement pushover curve is developed by using the force distribution $s_n^* = \begin{Bmatrix} m\phi_{xn} \\ m\phi_{yn} \\ I_p\phi_{\theta n} \end{Bmatrix}$ where m is a diagonal matrix with $m_j = m_{jj}$, lumped mass at the jth floor, I_p is a diagonal matrix with $I_{jj} = I_{oj}$, the polar moment of inertia of the jth floor diaphragm about a vertical axis through the CM, and ϕ_{in} is the mode shape vector of the nth mode in the direction i. Gravity loads are applied before pushover analysis and the lateral roof displacement due to gravity loads, u_{rg} is recorded.
3. Idealize the pushover curve as a bi-linear curve and convert the idealized pushover curve to the force–displacement relationship of an equivalent nth mode inelastic single-degree-of-freedom (SDOF) system.
4. Compute the peak deformation, D_n, of the nth mode inelastic SDOF system defined by the force-deformation relationship developed in Step 3 and damping ratio ξ_n using either non-linear response history analysis, inelastic design spectrum, or elastic spectrum along with the empirical equations for the ratio of inelastic to elastic displacement. Calculate the peak roof displacement, u_{rn}, associated with the nth mode inelastic SDOF system from $u_{rn} = \Gamma_n \phi_{rn} D_n$, where Γ_n is the mode participation factor for nth mode and ϕ_{rn} is the mode shape vector of the nth mode in the direction under consideration at the roof level.
5. From the pushover database, extract values of r_{n+g} due to combined effects of gravity and lateral loads.
6. Steps 2–5 are repeated for as many modes for sufficient accuracy.
7. Compute the dynamic response due to the nth mode: r_{n+g}-r_g where r_g is the contribution of gravity loads alone.
8. Determine the total response by combining gravity response and the peak modal responses using the complete quadratic combination (CQC) rule.

Chopra and Goel [12] recommended that the developed procedure be used to determine only the total floor

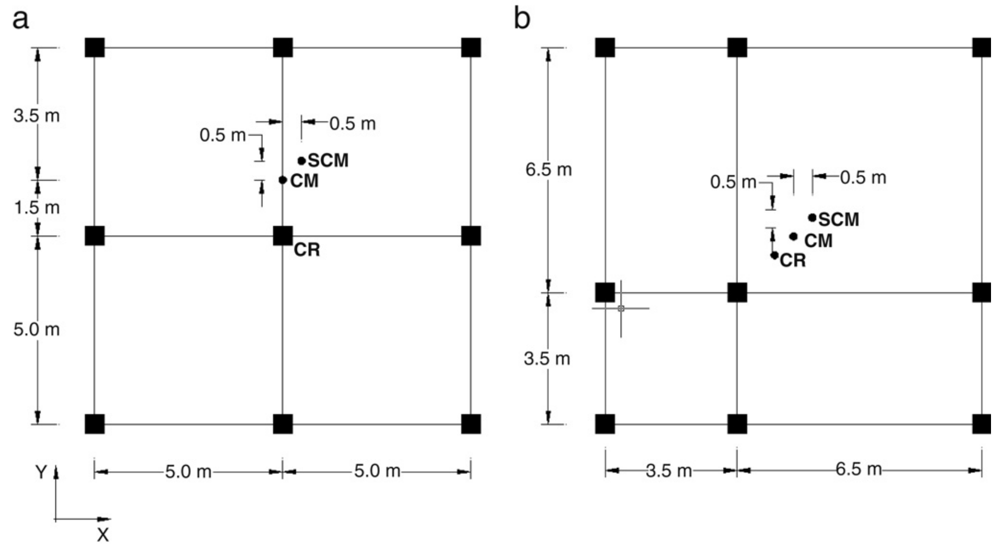

Fig. 1. Plan view of the (a) uni-directionally eccentric system (b) bi-directionally eccentric system.

displacements and story drifts at any location of the plan, and other responses including member plastic rotations for ductile members and element forces for brittle elements should not be computed using this procedure (they will be computed from the total story drifts using proper methods reported in the literature [23]). With regard to this comment, the response parameters considered in this study are limited to the inter-story drift ratio demand of all the stories along with the roof displacement demand at critical locations of the plan.

3. Structural systems

The primary cause of torsional effects is the eccentricity between the mass center and stiffness center in the plan. This eccentricity may occur in two separate cases: (1) symmetrical structural plan with a shift of mass center in one or both horizontal directions and (2) asymmetrical structural plan. Moreover, the eccentricity may be limited to one of the principal horizontal directions or may be in both horizontal directions. When the latter case is subjected to both components of the ground motion in two horizontal directions simultaneously, the most complex case of the torsional effects is achieved: a multistory building with bi-directional eccentricity subjected to bi-directional ground motions.

In this study, two different structural systems were considered to account for all the possible cases. The first system has a symmetrical structural plan with footprint of 10 m × 10 m. The center of mass was shifted in the Y direction by 1.5 m resulting in a uni-directional eccentricity of 15% of the plan dimension, Fig. 1(a). The second system was formed by altering the structural plan of the system and moving the center of mass to the center of the plan as shown in Fig. 1(b). The resulting system has a bi-directional eccentricity of 5% of the plan dimension in both directions. Henceforth, the first system is called a uni-directionally eccentric (UDE) system, while the second one is referred to as a bi-directionally eccentric system (BDE). Both systems are typical 3 story, moment resisting,

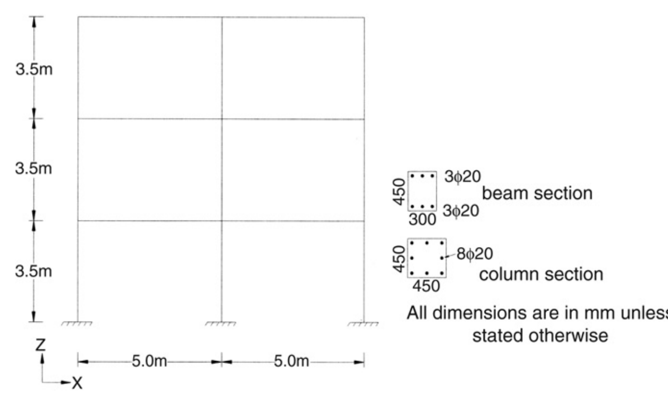

Fig. 2. Elevation view and the details of the member cross-section details for uni-directionally eccentric system.

reinforced concrete frames designed according to the provisions of IBC-2000 [25] for structures located in high seismicity zones. The story number is limited to 3 stories since the main purpose of this study is to evaluate the effectiveness of the current procedures in capturing the torsional effects; the higher-mode effects, which are mainly pronounced for medium and high-rise buildings that are also valid for planar structures are beyond the scope of this study. The reader is referred to several documents in literature (e.g. FEMA-440 [3], Gupta and Kunnath [4], Kalkan and Kunnath [20], Goel and Chopra [23], Chintanapakdee and Chopra [24]) that investigate the higher-mode effects that are also valid for planar buildings and the effectiveness of current nonlinear procedures in capturing these effects.

Fig. 2 illustrates the elevation view of one of the frames of the uni-directionally eccentric system along with the cross-sectional properties of the structural elements. *nonlinearBeamColumn* element of OpenSEES [26] with 4 integration points along with a fiber section was used to model all the structural elements. Mander model [27] was used to compute the properties of the confined concrete and longitudinal reinforcement was modeled using a bilinear

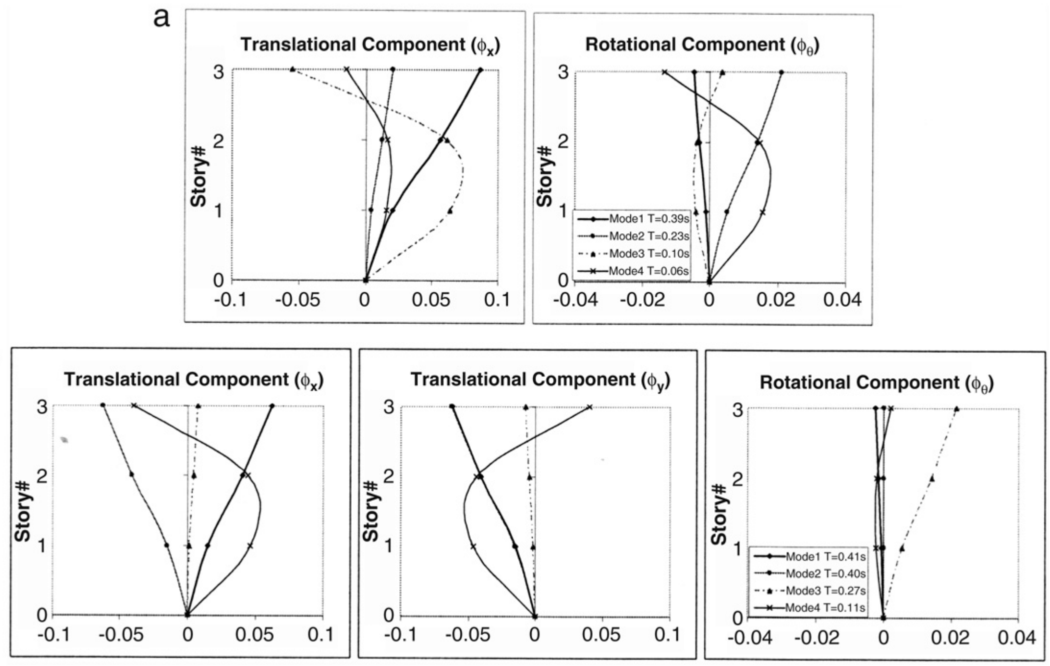

Fig. 3. Natural periods and elastic mode shapes of (a) uni-directionally eccentric (b) bi-directionally eccentric system.

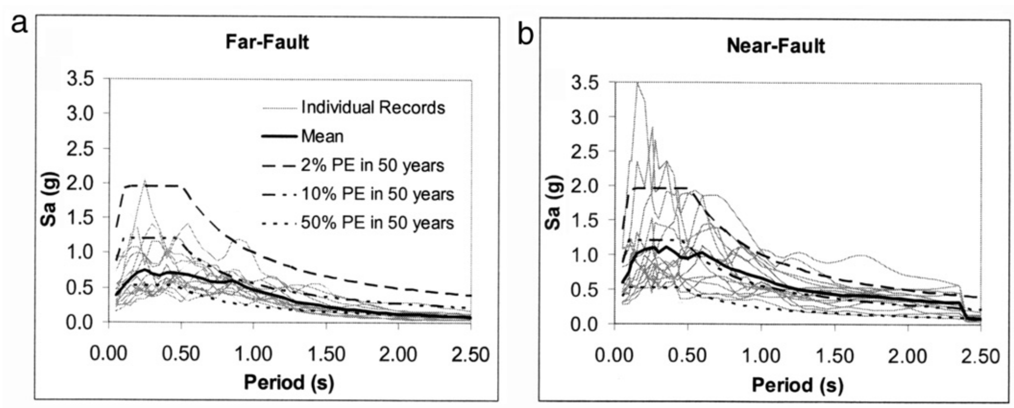

Fig. 4. 5% elastic response for (a) far-fault (b) near-fault ground motions used in the study.

stress–strain curve with 2% strain hardening. By using fiber sections, the behavior of columns under biaxial bending could be considered in determining the overall building response. Moreover, use of the fiber section enables the utilization of the biaxial moment-axial force interaction at each step of the loading, which is of significant importance for systems susceptible to torsional effects due to considerable variation in axial force demands on the columns in a story. The soil-foundation system of the structure was assumed to be infinitely rigid compared to the structural members (fixed base assumption) and the soil-structure effects were neglected.

Gravity loads, equal to the sum of the dead loads and 30% of the live loads, were applied as equivalent distributed load to the beams of the buildings. Gravity load analysis preceded all the nonlinear response analyses carried out in this study. P-delta effects due to gravity loads were considered in the nonlinear analyses.

Elastic free vibration analyses were carried out to determine the dynamic properties of the buildings. Fig. 3 depicts the elastic periods and modes of vibration for both uni-directionally eccentric and bi-directionally eccentric systems.

4. Ground motion ensemble

A ground motion ensemble comprising of two horizontal components of 30 ground motions with a moment magnitude ranging from 5.8 to 7.6 were used in the non-linear response history analyses (RHA). 15 of the selected ground motions can be considered as far-fault ground motions recorded at sites that are further than 15 km from the causative fault, whereas the remaining are near-fault ground motions with forward directional effects, which were recorded within 15 km of the fault. Table 1 summarizes the properties of the ground motions used in the study. Fig. 4 depicts the 5% elastic response spectra of the components of the unscaled ground motions that were applied in X-direction. Both uni-directionally and bi-directionally eccentric systems were subjected simultaneously to both components of the ground motions in two orthogonal

Table 1
List of ground motions used in the study

Eq#	Type[a]	Earthquake	Magnitude (Mw)	Station name	Distance (km)	Site Class[b]	Direction X		Direction Y	
							Component	PGA (g)	Component	PGA (g)
1	FF	Chi-Chi Taiwan 1999	7.6	TCU045	24.06	B	TCU045-N	0.512	TCU045-W	0.474
2	FF	Northridge 1994	6.7	24278 Castain - Old Ridge Route	22.6	B	ORR360	0.514	ORR090	0.568
3	FF	Victoria Mexico 1980	6.4	6604 Cerro Prieto	34.8	B	CPE045	0.621	CPE315	0.587
4	FF	Northridge 1994	6.7	90014 Beverly Hills - 12520 Mulhol	20.8	B	MU2035	0.617	MU2125	0.444
5	FF	Northridge 1994	6.7	24303 LA - Holywood Stor FF	25.5	C	HOL360	0.358	HOL090	0.231
6	FF	Northridge 1994	6.7	90091 LA - Satunr St	30	C	STN110	0.439	STN020	0.474
7	FF	San Fernando 1971	6.6	24278 Castain - Old Ridge Route	24.9	B	ORR291	0.268	ORR-021	0.324
8	FF	Imperial Valley 1979	6.5	6604 Cerro Prieto	26.5	B	H-CPE237	0.157	H-CPE147	0.169
9	FF	Northridge 1994	6.7	90018 Hollywood-Willoughby Ave	25.7	B	WIL180	0.245	WIL090	0.136
10	FF	Westmorland 1981	5.8	5051 Parachute Test Site	24.1	B	PTS315	0.155	PTS225	0.242
11	FF	Whittier Narrows 1987	6.0	90084 Lakewood-Del Amo Blvd	20.9	C	A-DEL000	0.277	A-DEL090	0.178
12	FF	Imperial Valley 1979	6.5	6621 Chihuahua	28.7	C	H-CHI282	0.254	H-CHI012	0.270
13	FF	Loma Prieta 1989	7.0	1028 Hollister City Hall	28.2	C	HCH090	0.247	HCH0180	0.215
14	FF	Landers 1992	7.3	22074 Yermo Fire Station	24.9	C	YER270	0.245	YER360	0.152
15	FF	Loma Prieta 1989	7.0	1678 Golden Gate Bridge	85.1	B	GGB270	0.233	GGB360	0.123
16	NF	Northridge-1994	6.7	Jensen Filt. Plant	6.2	C	JEN022	0.424	JEN292	0.593
17	NF	Kobe-1995	6.9	KJMA	0.6	B	KJM000	0.821	KJM090	0.599
18	NF	Loma Prieta-1989	7.0	Lexington Dam	6.3	B	LEX090	0.409	LEX000	0.442
19	NF	Northridge-1994	6.7	Sepulveda Va. Hospital	9.5	C	SPV360	0.939	SPV270	0.753
20	NF	Tabas-1978	7.4	Tabas	3.0	C	TAB-TR	0.852	TAB-LN	0.836
21	NF	Imperial-Valley 1979	6.5	Holtville Post Office	8.8	C	HVP-225	0.253	HVP-315	0.221
22	NF	Duzce	7.2	Bolu (Bol)	6.1	C	BOL090	0.822	BOL000	0.728
23	NF	Northridge-1994	6.7	Newhall Pico Canyon	7.1	C	WPI316	0.325	WPI046	0.455
24	NF	Superstition Hills - 1997	6.4	Parachute Test Site	0.7	C	PTS225	0.455	PTS315	0.377
25	NF	Imperial-Valley 1979	6.5	EC Meloland Overpass	3.1	C	EMO270	0.296	EMO000	0.314
26	NF	Erzincan 1992	6.7	Erzincan	2.0	B	ERZ-EW	0.496	ERZ-NS	0.515
27	NF	Landers 1992	7.3	Joshua Tree Fire Stn.	10.0	C	JOS000	0.270	JOS090	0.284
28	NF	Cape Mendocino-1992	7.1	Petrolia, General Store	15.0	B	PET090	0.662	PET000	0.590
29	NF	Imperial-Valley-1979	6.5	El Centro Array #4	8.3	C	E04230	0.356	E04140	0.485
30	NF	Imperial-Valley-1979	6.5	El Centro Array #6	3.5	C	E06230	0.438	E06140	0.410

[a] FF: Far-Fault; NF: Near-Fault.
[b] According to USGS classification; $V_s = 360$–750 m/s for site class B and $V_s = 180$–360 m/s for site class C.

horizontal directions. Although the component of the ground motion in the Y-direction does not have any influence on the torsional behavior of the uni-directionally eccentric system in the elastic range, the yielding of certain elements may result in incidental eccentricities along the X-direction of the uni-directionally eccentric building which may cause the Y component of ground motions to create additional torsional effects. To be able to take this effect into account, uni-directionally eccentric systems were also subjected to both components of the ground motions.

The components of the ground motions in the X-direction were scaled to match the design response spectrum of FEMA-356 [2] for site class B at the fundamental period of each structure for three different hazard levels: 50%, 10% and 2% probability of exceedance in 50 years for San Francisco. The target spectrum for each hazard level is presented in Fig. 4 along with the unscaled response spectrum of each ground motion. The component of the ground motions applied in the Y direction was scaled with the scale factor computed for the X component of the ground motion. Spectral acceleration values at periods of 0.2 s and 1.0 s for the corresponding hazard levels were taken from the 2002 USGS Probabilistic Uniform Hazard Response Spectra [28].

5. Results of the numerical analyses

5.1. Nonlinear static analysis procedures

The capacity curves obtained from the pushover analysis of the uni-directionally eccentric system under invariant force

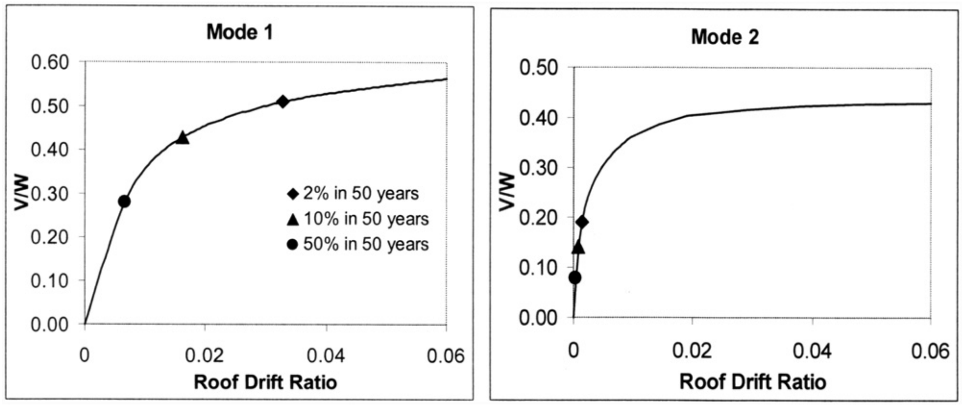

Fig. 5. Capacity curves of the uni-directionally eccentric system obtained under an invariant force distribution proportional to the first and second mode shapes.

distributions proportional to the first and second mode shapes are presented in Fig. 5. The capacity curve of each mode was then converted to the force-deformation relationship of an equivalent single-degree-of-freedom (SDOF) system. Nonlinear time history analyses of each equivalent SDOF under each ground motion scaled to desired hazard level were carried out to compute the target displacement. For the modified N2 method, the pushover analysis under the first mode force distribution was used while the results of first four mode pushover analyses were combined for the MPA procedure. Depicted also in Fig. 5 are the mean target displacements computed for each mode and hazard level using nonlinear dynamic analysis of equivalent SDOF procedure. In addition to the N2 and MPA procedures, two different versions of first mode pushover analyses were also applied to the structures. The only difference between the two versions of the first mode pushover analysis is the point of application of the lateral loads. In the first one, the lateral load was applied at the mass center of the floors (CM in Fig. 1) whereas in the second version the point of application was the shifted center of mass (SCM in Fig. 1) which was shifted by 5% of the plan dimension in each direction as per the current code provisions that is intended to account for the accidental eccentricities. IBC-2000 [25] requires amplification of the accidental eccentricity value for torsionally flexible structures by a dynamic amplification of torsion factor, A_x. The A_x value was computed to be 0.98 and 0.84 for the uni-directionally eccentric and bi-directionally eccentric frames, respectively; hence it was taken as 1.0 for both frames according to the recommendations of IBC-2000 [25]. From this point on, the first version will be named as the first mode pushover procedure and the second version will be named as the code procedure. Here it may be useful to repeat that, although named as the code pushover procedure, the lateral load pattern applied in this procedure is the first mode load pattern, not the code lateral load pattern.

5.2. Response history analyses

The results of all the numerical analyses are presented in terms of the demand parameter, investigated at different locations of the plan, normalized by the corresponding parameter at the mass center. Inherent in this presentation is the assumption that the nonlinear static procedures can perfectly estimate the corresponding demand parameter at the mass center of each story. Although it is well known that none of the current nonlinear procedures is capable of perfectly estimating the displacement demand at the mass center, this assumption was made to eliminate the variation and bias rising from the nonlinear static procedures developed for planar structures, since the objective of this article is to evaluate the effectiveness of the nonlinear static procedures in capturing torsional effects only.

Records of ground motions recorded within 10–15 km of causative faults have provided evidence that near-fault ground may be characterized by a large, long-period pulse. These types of ground motions may generate high structural demands. Hall et al. [29], Alavi and Krawinkler [30] and Kalkan and Kunnath [31] showed that high velocity pulses can generate severe inelastic demands on frame structures compared to the far-fault ground motions, which are characterized by relatively longer durations. These studies were limited to two-dimensional (2D) frame structures which could not provide any information on the effect of near-fault ground motions on the torsional response.

Hence, the results of the RHA were examined first to identify the difference between the structural response under near-fault and far-fault ground motions in terms of torsional effects. Fig. 6 (a) shows that the torsional effects on the roof displacement are virtually identical in case of far-fault ground motions and near-fault ground motions for uni-axially eccentric systems and for all the three hazard levels considered. The slight discrepancy between the mean results of far-fault and near-fault ground motions, in terms of torsional effects observed at the torsionally stiff side, is not large enough to draw a firm conclusion regarding the effect of near-fault and far-fault ground motions on the torsional response of uni-directionally eccentric systems. This slight discrepancy may be attributed to record-to-record variability of the seismic response of structures as well as to the idealization and assumptions made during the modeling phase. For bi-directionally eccentric systems, even this slight difference between far-fault and near-fault ground motions disappears, Fig. 6(b). Based on this observation and the results of the nonlinear RHA carried out in this study, it can be stated that there is no significant discrepancy between the torsional

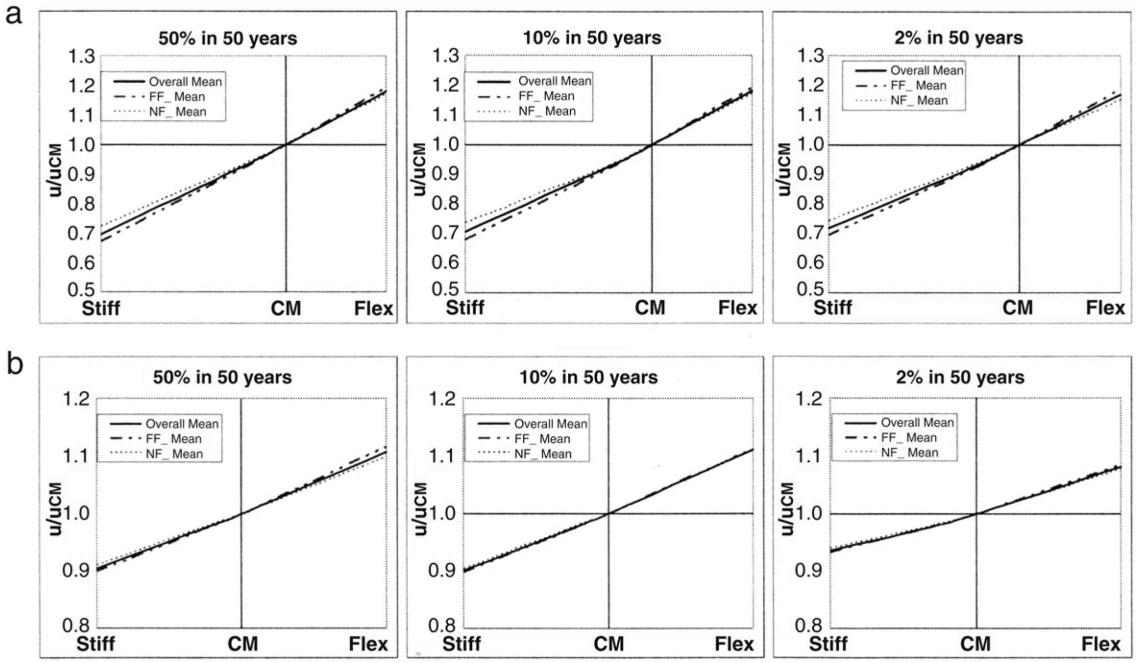

Fig. 6. Effect of ground motion type on torsional response for (a) uni-directionally eccentric system for different hazard levels.

response of structures subjected to near-fault ground motions and the torsional response under far-fault ground motions. Hence, the comparison of the non-linear static procedures with the nonlinear RHA was carried out for the ensemble of all 30 ground motions without discriminating near-fault and far-fault ground motions.

Fig. 7 presents torsional effects for the uni-directionally eccentric system estimated by the modified N2 method [14], MPA [12], elastic response spectrum analysis (RSA), first mode pushover procedure (1st mode) and code pushover procedure, together with the mean (50 percentile) and mean plus standard deviation (84 percentile) values obtained from nonlinear RHA for three different hazard levels and four response parameters: roof drift ratio and interstory drift ratio for all three stories. For all the hazard levels and response parameters, the torsional rotations estimated by first mode pushover procedure are very low compared to the "exact" results obtained from nonlinear RHA resulting in the underestimation of displacement demands in the torsionally flexible side. However, this underestimation in the torsional rotations by the first mode procedure results in conservative demand estimates on the torsionally stiff side. For the uni-directionally eccentric structure, application of the first mode load pattern at the shifted mass center to account for accidental eccentricities (code pushover procedure) results in an improvement in the estimation of torsional effects for all three hazard levels, Fig. 7. However, the code procedure still underestimates the torsional rotations for all demand parameters resulting in unconservative displacement estimates for the torsionally flexible side. On the torsionally flexible side, the elastic response spectrum analysis method coincides with the modified N2 method, due to the definition of the modified N2 method. RSA yields very similar results to the nonlinear RHA in terms of torsional rotations for 50% in 50 years events. However, for higher hazard levels, particularly for 2% in 50 years events, RSA overestimates the torsional rotations resulting in conservative demand estimates for the torsionally flexible side and unconservative estimates for torsionally stiff side. The torsional rotations tend to decrease as the hazard level increases. With an increase in the hazard level, the inelasticity level in the building increases which results in the variation of significant parameters that affect the torsional response of the buildings such as the periods, mode shapes and the eccentricities between the mass center and the center of stiffness. Previously, it was reported that the torsional rotations decrease with an increase in the inelastic displacements [21, 22]. Accordingly, the increase in the hazard level results in a decrease in the torsional rotations observed. Thus, RSA, being an elastic procedure, tends to overestimate the torsional rotations of the structures that undergo significant levels of inelastic deformation.

To eliminate the possible underestimation of displacement demands on the torsionally stiff side, the modified N2 procedure assumes that the displacement demands on the torsionally stiff side are equal to those of the mass center resulting in conservative estimates for all cases.

Modal pushover analysis (MPA) overestimates and underestimates the torsional rotations for 50% in 50 years and 2% in 50 years events, respectively. For the 10% in 50 years event, the estimation of torsional rotation by MPA is consistent with the nonlinear RHA. Accordingly, MPA estimates of the displacement demands for this event are consistent with the nonlinear RHA results for both torsionally flexible and torsionally stiff sides. MPA yields conservative and unconservative demand estimates for the 50% in 50 years and 2% in 50 years events on the torsionally flexible side, respectively, whereas on the

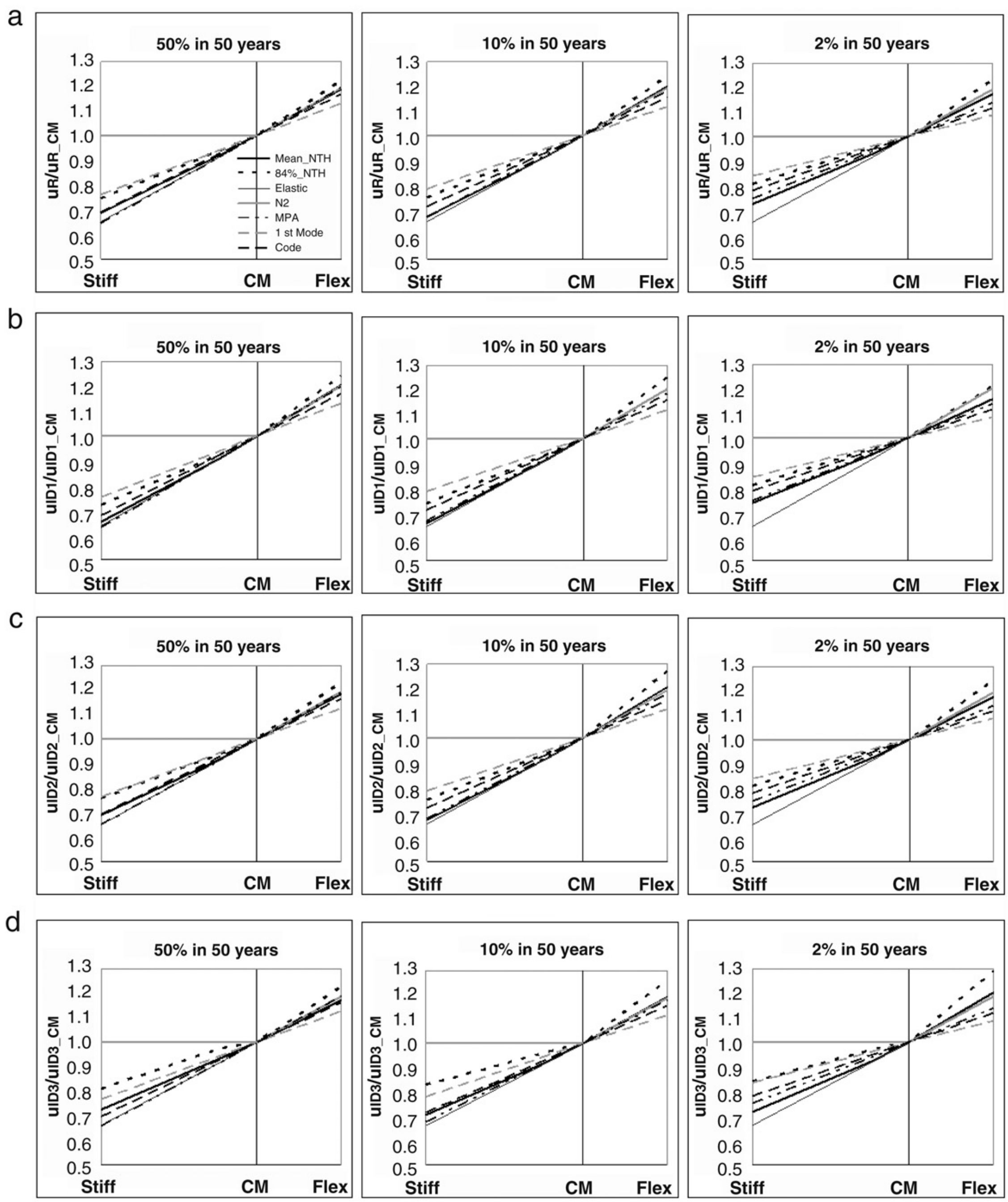

Fig. 7. Comparison of NSP with the nonlinear RHA for the uni-directionally eccentric system at (a) roof displacement (b) 1st story IDR (c) 2nd story IDR and (d) 3rd story IDR levels.

torsionally stiff side it yields unconservative and conservative displacement demands for these two events respectively.

For the bi-directionally eccentric system, results of the nonlinear RHA show that the torsional effects tend to decrease with the increase in plastic deformations viz., torsional effects are more pronounced in case of 50% in 50 years than 2% in 50 years event, Fig. 8. Fig. 8 also shows that the first mode pushover procedure underestimates the torsional rotations for this system and the displacement demands on the torsionally flexible side for all hazard levels. For the torsionally stiff side, the first mode procedure yields conservative estimates for 50% in 50 years event. For the 2% in 50 years event the first mode procedure results are much closer to the mean of the nonlinear RHA results.

Fig. 8 shows that applying the first mode pushover procedure at the shifted mass center (SCM) significantly improves the torsional rotation estimates for all the demand parameters considered for the bi-directionally eccentric system. For all the hazard levels, the torsional rotations estimated by the code pushover procedure exceed the mean torsional rotations obtained from nonlinear RHA resulting in conservative displacement estimates in the torsionally flexible side. Here, it

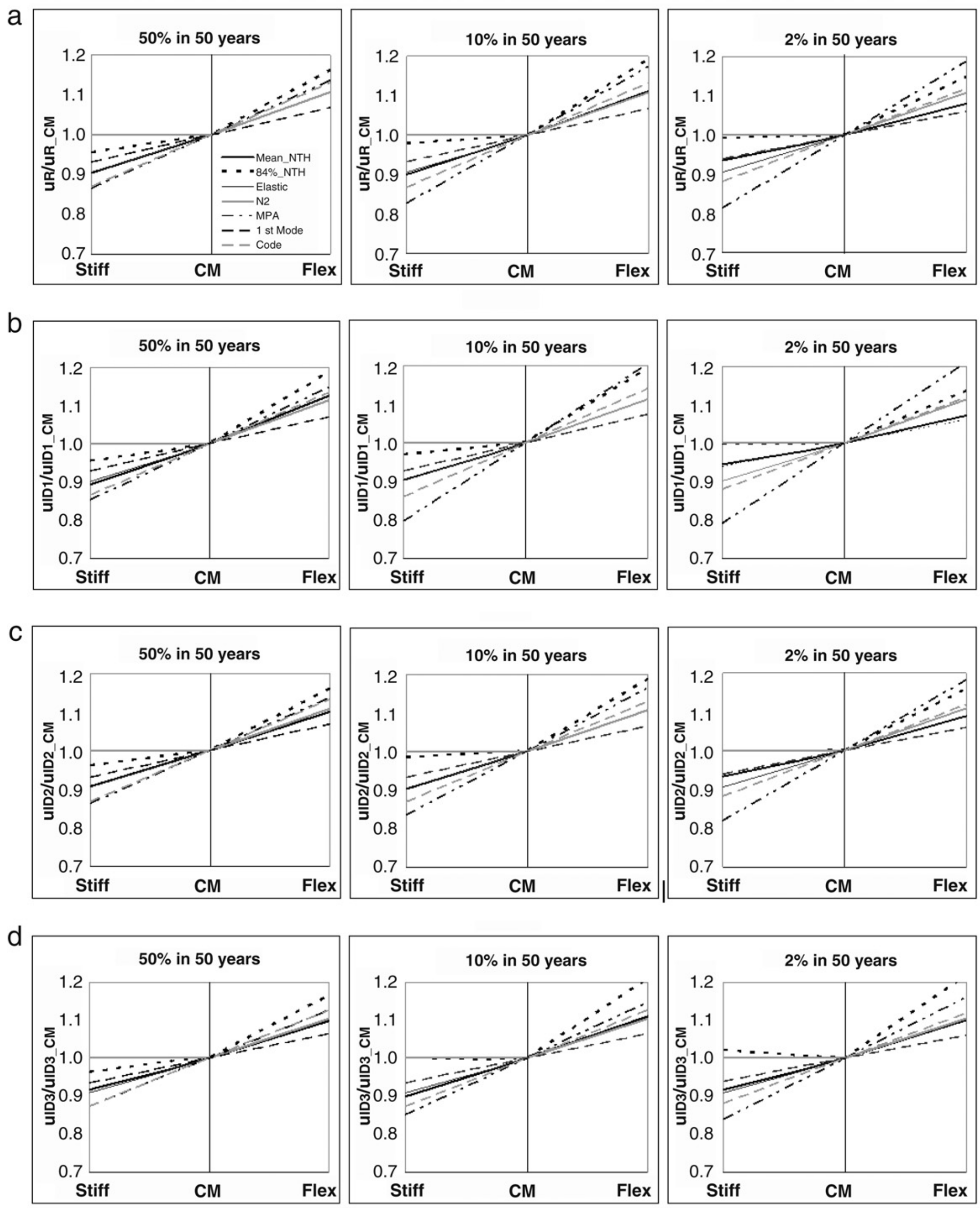

Fig. 8. Comparison of NSP with the nonlinear RHA for the uni-directionally eccentric system at (a) roof displacement (b) 1st story IDR (c) 2nd story IDR and (d) 3rd story IDR levels.

is worth recalling that the code pushover procedure, although it was an improvement compared to the first mode pushover procedure as long as torsional effects were considered, yielded unconservative torsional rotation demands for the uni-directionally eccentric systems. For the bi-directionally eccentric system, however, the code pushover procedure has been found to be effective in capturing the torsional effects. The main reason for this difference might be the eccentricity level of the two systems. The distance between the mass center and the stiffness center of the uni-directionally system was specified to be 1.5 m resulting in an eccentricity of 15% of the plan dimension of the building, whereas the eccentricity level of the bi-directionally eccentric system was limited to 5% of the plan dimension in both directions. As a result, the torsional effects are more pronounced for the uni-directionally eccentric system than the bi-directionally eccentric system, Figs. 7 and 8. It can be stated that the additional torsional rotations obtained by applying the lateral load at the shifted mass center instead of the mass center can be sufficient to capture the torsional rotations created due to eccentricities in the order of 5% of the plan dimension. However, when the eccentricity between the mass center and the stiffness center in a floor reaches the order of 15% of the plan dimension, these additional rotations may be insufficient in capturing the torsional rotations obtained from nonlinear RHA.

For the 50% in 50 years event, RSA estimates for the torsional rotation are consistent with the "exact results" obtained from nonlinear RHA. However, for the 10% in 50 years and particularly for 2% in 50 years events, RSA overestimates the torsional rotations resulting in conservative and unconservative displacement demand estimates on torsionally flexible and torsionally stiff sides, respectively.

The modified N2 method, although dependent on the torsional rotation estimations of the RSA, yields conservative results on both torsionally flexible and stiff sides (Fig. 8), since it eliminates the unconservative estimates on the torsionally stiff side by equating the displacement demands on this side to those of the mass center in case the RSA estimates for this side are smaller than those for the mass center. For all the demand parameters and hazard levels considered, MPA overestimates the torsional rotations, Fig. 8. On the torsionally flexible side, this overestimation is beneficial and results in conservative estimates of displacement demands. However, on the torsionally stiff side, the overestimation of torsional rotations results in unconservative estimates of displacement demands.

6. Conclusion

The effectiveness of two recently proposed nonlinear static procedures for plan asymmetric structures in capturing torsional effects were evaluated. The "exact" results obtained from nonlinear RHA under a ground motion ensemble of 30 records scaled for 3 different hazard levels were compared with the predictions of the modified N2 and MPA procedures. Several assumptions and idealizations were made throughout the modeling phase of the study such as fixed-base assumption. Under these assumptions, the following conclusions were delineated as a result of the numeric analyses.

- Far-fault and near-fault ground motions were observed to create similar torsional demands. However, the ground motion ensemble and number of structures investigated in this study is not sufficient to draw firm conclusions. A more detailed study using a larger ensemble of ground motions is needed to evaluate the torsional response of asymmetric structures under far-fault and near-fault ground motions.
- The first mode pushover procedure, where the lateral forces are applied at the mass center significantly underestimates the torsional rotations resulting in an underestimation of the displacement demands on the torsionally flexible side for both uni-directionally eccentric and bi- directionally eccentric systems. For the uni-directionally eccentric system, the underestimation of torsional rotations results in conservative displacement demand estimates for the torsionally stiff side.
- When the first mode load pattern is applied at the shifted mass center to account for the accidental eccentricities, as per the requirements of the seismic codes, torsional rotation estimations significantly increase compared to the case where the load pattern is applied at the mass center. For the bi-directionally eccentric system where the eccentricity between the mass center and the center of stiffness is 5% of the plan dimension in both directions, the improvement in the torsional rotation estimations is sufficient to provide satisfactory estimates of the mean torsional rotation demands obtained from the nonlinear RHA. However, for the uni-directionally eccentric system, where the eccentricity between the mass center and center of stiffness is 15% of the plan dimension, the code pushover procedure, despite the improvement compared to the first mode pushover procedure, underestimates torsional rotations.
- Elastic response spectrum analysis (RSA) generally yields very consistent results with the nonlinear RHA in terms of torsional rotations. Hence, the modified N2 method [14], which uses results of RSA to determine the torsional effects, gives satisfactory estimates for the displacement demands on the torsionally flexible side for both of the systems considered. On the torsionally stiff side, the results of RSA shows that the displacement demands are de-amplified compared to the mass center. The modified N2 method assumes the displacement demands on this side are equal to those at the mass center. This assumption results in conservative estimates of the displacement demands on the torsionally stiff side.
- Modal pushover analysis (MPA) for asymmetric structures gives conservative estimates of torsional rotations for both uni-directionally and bi-directionally eccentric systems except for the 2% in 50 years hazard level for uni-directionally eccentric system. On the torsionally flexible side, the overestimation of torsional rotations proves to be beneficial and results in conservative displacement demand estimates. However, on the torsionally stiff side, the MPA procedure yields unconservative displacement demands due to the overestimation of torsional rotations. Provided that a "safety mechanism" similar to the one employed by the modified N2 method to prevent underestimation of displacement demands on the torsionally stiff side is utilized, MPA is a very effective procedure in capturing the torsional effects.

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