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Comparison of Adaptive and non-Adaptive Nonlinear Static Analysis Methods for Estimating Seismic Demand

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ABSTRACT:

In this paper, we review several pushover analysis methods to illustrate the enhancements and refinements of the pushover analysis. In addition, how researcher could develop the procedures to overcome the disadvantages of the traditional pushover procedure. The traditional lateral analysis method has been developed to consider the contribution of higher modes as well as the change in the dynamic properties of structures after entering the nonlinearity and formation the hinges. We will study two structures of steel and others of reinforced concrete via the conventional method using first mode, modal pushover analysis (MPA), multi-mode adaptive displacement-based pushover (MADP), consecutive modal pushover (CMP) and non-adaptive displacement-based pushover (NADP) and then compare the results with nonlinear response history analysis to determine which of these methods is the most accurate. The result of the study shows that the MADP method gives the best estimate of the story drift than the rest of the methods compared to the nonlinear dynamic analysis, because it considers the higher modes effect and dynamic characteristics change of the model after entering the nonlinearity.

Key Words: Pushover Analysis, Modal Pushover Analysis, Adaptive Displacement-Based Pushover, Non-Adaptive Displacement-Based Pushover, Consecutive Modal Pushover.

المقارنة بين طرائق التحليل الستاتيكي اللاخطي التكيفية وغير التكيفية لتقدير الطلب الزلزالي

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الملخص:

سوف نستعرض في هذه المقالة العديد من طرائق تحليل الدفع الجانبي لتوضيح تحسينات هذه الطريقة وكيف استطاع الباحثون تطويرها للتغلب على عيوب طريقة الدفع الجانبي التقليدي .طُوِّرت طريقة تحليل الدفع الجانبي التقليدي لتأخذ بالاعتبار مساهمة الأنماط العليا وكذلك تغير الخواص الديناميكية للمنشأ بعد الدخول بالمرحلة اللاخطية وتشكل المفاصل اللدنة. تم في هذه المقالة دراسة مجموعتين من المنشآت المعدنية والبيتونية المسلحة بطرائق الدفع الجانبي – التقليدي باستخدام النمط الأول، والنمطي (MPA)، والنمطي المتتالي (CMP)، والدفع الجانبي التكيفي متعدد الأنماط المبني على الانتقال (MADP)، والدفع الجانبي غير التكيفي المبني على الانتقال (NADP)) ومقارنة النتائج مع طريقة التحليل اللاخطي للسجل الزمني (NL-RHA) لمعرفة الطريقة الأكثر دقة. أظهرت نتائج الدراسة أن طريقة MADP قد أعطت التقييم الأفضل للانزياح الطابقي النسبي مقارنة بطريقة التحليل الديناميكي اللاخطي وذلك لأنها تأخذ بالاعتبار مساهمة الأنماط العليا وتغير الخواص الديناميكية للمنشأ بعد الدخول بالمرحلة اللاخطية.

الكلمات المفتاحية: تحليل الدفع الجانبي، تحليل الدفع الجانبي النمطي، تحليل الدفع الجانبي النمطي المتتالي، تحليل الدفع الجانبي التكيفي المبني على الانتقال وتحليل الدفع الجانبي غير التكيفي المبني على الانتقال.

INTRODUCTION:

Earlier versions of pushover methods presented in different code provisions such as FEMA-356 (FEMA 356,2000), EuroCode-8 (CEN. Eurocode 8,2003) and ATC-40 (ATC-40,1996) , are limited to invariant load pattern (uniform, linear, or the first mode) until a predetermined target displacement is achieved or collapse occurs. In these methods, the influence of the contribution of the higher modes and the change of the dynamic properties of the structures after their entry into the nonlinearity on the loading pattern was not taken into consideration. Therefore, a lot of research was conducted to develope the method in order to avoid the previously mentioned defects.

Some methods considered the higher mode effects and neglected the effect of changing the dynamic characteristics of the structure during the analysis on the loading pattern (non-adaptive pushover analysis procedures), while other methods considered the two effects together (adaptive pushover analysis procedures).

Some of the non-adaptive pushover methods are the modal pushover analysis (MPA) (Chopra et al.,2002), non-adaptive displacement-based pushover (NADP) (Amin et al.,2016) and the consecutive modal pushover (CMP) (Poursha et al., 2009), while the adaptive analysis methods include several methods as displacement-based adaptive pushover (DAP) (Antoniou et al.,2004), story shear-based adaptive pushover method(SSAP) (Shakeri et al.2010), adaptive force-based multimode pushover (AFMP) (Amin et al.,2018), and multi-mode adaptive displacement-based Pushover (MADP) (Jalilkhania et al., 2020).

(MPA) method ,which is a multi-run procedure, the structure is subjected to different load vectors (proportion to each mode) and the modal responses are combined with appropriate quadratic modal combination rules such as the square root of the sum of the squares (SRSS) and the complete quadratic Combination (CQC). The number of modes used in this method is determined so that the modal participation mass ratio is equal to or greater than 90 percent.

(NADP) method: the modal combination concept is applied to determine the displacement-based lateral force pattern. This method can be performing as the follows:

1. After developing the structural model with its linear and nonlinear properties, determine the natural frequencies,$\emptyset_n$, and mode shapes, Un, then normalize mode shapes and determine the target displacement
2. Calculate lateral load distributions by using following equation

$$D_1 = |\Gamma_1|\phi_n S_{d1}(\zeta_1, T_1) \quad (1)$$

$$D\_2 = d\_1 + d\_2$$
$$= |\Gamma\_1 \mid \phi\_n\ S\_d1\ (\zeta\_1, T\_1\ )$$
$$+ |\Gamma\_2 \mid \phi\_2\ S\_d2\ (\zeta\_2, T\_2\ ) \quad (2)$$

For two modes and

$$D\_3 = d\_1 + d\_2 + d\_3$$
$$= |\Gamma\_1 \mid \phi\_n\ S\_d1\ (\zeta\_1, T\_1\ )$$
$$+ |\Gamma\_2 \mid \phi\_2\ S\_d2\ (\zeta\_2, T\_2\ )$$
$$+\ |\Gamma\_3 \mid \phi\_3\ S\_d3\ (\zeta\_3, T\_3\ ) \quad (3\ )$$

for three modes.

Where d1, d2, d3, are the lateral displacement distributions for the first three modes and $S_{d1}, S_{d2}S_{d3}$, are the spectral displacements, T_1, T_2, T_3 are the vibration periods and $\zeta_1, \zeta_2, \zeta_3$, are the damping ratios of the first three modes for the earthquake ground motion.

3. Apply the gravity loads and then carry out the first pushover analysis using a force distribution according to Eq. (1) till the displacement at roof reaches the target displacement. The peak values of seismic responses of this step are denoted by r_1.
4. Perform another pushover analysis with the lateral force, which is determined using Eq. (2), till the displacement at roof is equal to the target displacement. The peak values of seismic responses of this step are denoted by r_2.
5. Only when the fundamental period of structure is 2.2 s or more, Carry out the third pushover analysis with the lateral load, which is obtained using Eq. (3), till the displacement at roof equals to the target displacement. The peak values of seismic responses of this step are denoted by r_3.
6. Envelop (r_1, r_2, r_3) as following:

$$r = \mathrm{Max}\{r\_1, r\_2\ \} \quad T < 2.2\ \ S \quad (4)$$
$$r = \mathrm{Max}\{r\_1, r\_2, r\_3\ \} \quad T \geq 2.2\ \ S\ (5)$$

(CMP) method: In this method, a set of consecutive modal pushover analyses is performed, where each stage starts from the end of the previous stage, that is, the initial state of the structure in each stage (deformations and stresses) is the same at the end of the previous stage . The following steps explain how to do this:

1. After developing an structural model with its linear and nonlinear properties, determine the natural frequencies,$\emptyset_n$, and mode shapes, Un, then normalize mode shapes and determine the target displacement δ_t .
2. Compute lateral load $s_n^* = m\emptyset_n$ for every stage.
3. Apply the gravity loads, then perform the first pushover analysis using a load pattern according to inverted triangular load pattern or uniform force distribution till the displacement at roof is equal to target displacement δ_t. The peak value of seismic response of this step is denoted by r1.
4. Perform the second pushover analysis using $s_1^* = m\emptyset_1$ as the lateral loads till the roof displacement increment equals to $u_{r1} = \beta_1\delta_t$ where $\beta_1 = \alpha_1$,Then continue analysis using the lateral load $s_2^* = m\emptyset_2$ till the roof displacement increment equals $u_{r2} = \beta_2\delta_t$ where $\beta_2 = 1 - \alpha_1$. The peak value of seismic response of this step is denoted by r2.
5. Perform the third pushover analysis using lateral loads $s_1^* = m\emptyset_1$,when the fundamental period equals to 2.2 s or more, till the roof displacement increment equals to $u_{r1} = \beta_1\delta_t$ where $\beta_1 = \alpha_1$, then continue the nonlinear static analysis using the incremental lateral load $s_2^* = m\emptyset_2$ till the roof displacement increment equals to $u_{r2} = \beta_2\delta_t$ where $\beta_2 = \alpha_2$. Thereafter, go on the pushover analysis using the lateral load $s_3^* = m\emptyset_3$ till the roof displacement increment reaches to $u_{r3} = \beta_3\delta_t$ where $\beta_3 = 1 - \alpha_1 - \alpha_2$. The peak value of seismic response of this step is denoted by r3.
6. Envelop (r1, r2, r3) as Eq. (4), (5).

(MADP) procedure, the pushover analysis starts with the lateral force distribution proportional to the elastic mode and when the first group of plastic hinges is formed, the lateral load pattern is changed to be proportional to story displacements of previous step. The lateral load distribution is modified continuously whenever a new set of plastic hinges is formed .The pushover analysis is continued until the target displacement is reached. SRSS or CQC rule can be used to get the final response. The following steps show how to apply the MADP method:

1. After developing an appropriate structural model with its linear and nonlinear properties, determine the natural frequencies,$\emptyset_n$, and mode shapes, Un, then normalize mode shapes and compute the target displacement δ_t .
2. Apply the gravity loads.
3. Carry out pushover analysis with lateral force distribution $s_n = m\emptyset_n$ until the first group of plastic hinge is formed then modify the lateral load pattern to $s_n = md_n$,where d_n is a displacement vector and normalize it.
4. Go on pushover analysis with the new lateral force distribution, till a set of new plastic hinge appears. Modify the lateral load pattern and continue the analysis till another new plastic hinges appear. Repeat that till the displacement at the roof is equal to the target-displacement.
5. Repeat Step 3 for many modes .The first three modes of vibration are adequate for most of buildings.
6. Combine the modal responses using square-root-of-sum-of-squares (SRSS) or complete quadratic combination (CQC) to determine the final structural responses.

Example structures:

1- concrete structures : The two models studied were two structures with a height of 12 and 20 stories, each of them was a concrete moment-resisting frame with three bays, the width of each bay was 6.1m and the story height was 3.96 m, except for the first story, which was 4.57m high. The considered dead load was 51.20 kN/m, while the considered live load was 14.64 kN/m .Dead load plus twenty percentage of live load were taken for seismic masses. The concrete compressive strength was 41.4 MPa for the columns and 34.5 MPa for beams. 460 MPa was assumed as the minimum yield strength of the reinforcement. Modeling the structures was done using OpenSees software where ‘one-component’ lumped plasticity elements were used to model the beams and columns. The plastic hinges at the element’s ends are modeled by nonlinear zero-length rotational springs with behavior as Ibarra–

Krawinkler (IK) deterioration model (Ibarra et al.,2005). (Amin et al.,2016)

2- **steel structures**: The two models studied were two structures with a height of 15 and 20 stories, each of them was a steel moment-resisting frame with three bays, the width of each bay was 5m and the story height was 3.2 m . The considered dead load was 32.5 kN/m , while the considered live load was 10 kN/m . Dead load plus twenty percentage of live load were considered to calculate seismic masses. Modeling the structures was done by OpenSees software using fiber modeling approach. The force- Beam-Column element, that uses distributed plasticity with length that equals to the section height at the member ends, was used to model columns and beams.

Ground motions:

Twenty 'far-field' ground motion records are selected from FEMA P695 document (FEMA-P695,2009) and used for nonlinear dynamic time-history analysis of the concrete example buildings. The dynamic features of the ground motions are displayed in Table 1. The ground motion records are scaled according to the procedure discussed in .(Reyes,2009)، however another twenty ground motion record are used for nonlinear dynamic time-history analysis of the steel example buildings. The dynamic features of the ground motions are displayed in Table 2. The records were scaled up to 0.7 g.

Table(1) Earthquake ground motions used in concrete structures study. [10]

NO.	Earthquake	Year	Magnitude	Station name	Component	PGV (cm/s)	PGA (g)
1	Imperial Valley	1979	6.9	Chihuahua	H-CHI282	30.1	0.25
2	Imperial Valley	1979	6.9	Chihuahua	H-CHI012	24.9	0.27
3	Northridge	1994	6.7	Hollywood Stor FF	HOL090	18.3	0.23
4	San Fernando	1971	6.6	Lake Hughes #1	L01021	17.3	0.15
5	San Fernando	1971	6.6	Hollywood Stor Lot	PEL090	18.9	0.21
6	Superstition Hills(B)	1987	6.6	Plaster City	B-PLS135	20.6	0.19
7	Superstition Hills(B)	1987	6.6	Calipatria Fire Station	B-CAL315	14.6	0.25
8	Cape Mendocino	1992	7.1	Rio Dell Overpass	RIO270	43.9	0.39
9	Cape Mendocino	1992	7.1	Rio Dell Overpass	RIO360	42.1	0.55
10	Whittier Narrows	1987	5.7	Beverly Hills	A-MUL279	10.3	0.13
11	Imperial Valley	1979	6.9	El Centro Array #12	H-E12140	17.6	0.14
12	Loma Prieta	1989	7.1	Agnews State Hospital	AGW090	17.6	0.16
13	Imperial Valley	1979	6.9	Cucapah	H-QKP085	36.3	0.31
14	Loma Prieta	1989	7.1	Sunnyvale - Colton Ave.	SVL270	37.3	0.21
15	Imperial Valley	1979	6.9	El Centro Array #13	H-E13140	14.7	0.12
16	Imperial Valley	1979	6.9	Westmorland Fire Station	F-WSM360	4.7	0.09
17	Loma Prieta	1989	7.1	Sunnyvale - Colton Ave	SVL360	36.0	0.21
18	Imperial Valley	1979	6.9	El Centro Array #13	H-E13230	13.0	0.14
19	Imperial Valley	1979	6.9	Westmorland Fire Station	F-WSM-UP	2.0	0.12
20	Tabas	1978	7.4	Boshrooyeh	BOS-L1	13.7	0.11

(Jalilkhania et al.,2020)

Table(2) Earthquake ground motions used in steel structures study.

No.	Earthquake	Date	Station	Magnitude	Component	Fault distance (km)	PGA (g)	PGV (cm/s)	PGD (cm)
1	Duzce, Turkey	11/12/1999	Lamont 1061	7.1	E	15.60	0.13	13.69	8.20
2	Hollister	1/26/1986	SAGO South - surface	Ml (5.5)	295	-	0.09	9.27	1.70
3	Imperial Valley	10/15/1979	Parachute Test Site	6.5	315	14.20	0.20	16.06	9.97
4	Imperial Valley	10/15/1979	Cerro Prieto	6.5	147	26.50	0.17	11.58	4.24
5	Imperial Valley	10/15/1979	Superstition Mtn Camera	6.5	135	26.00	0.20	8.78	2.78
6	Kern County	7/21/1952	Taft Lincoln School	7.4	111	41.00	0.18	17.48	8.84
7	Livermore	1/24/1980	CSUH	5.8	146	31.00	0.07	4.11	0.75
8	Loma Prieta	10/18/1989	Anderson Dam	6.9	250	21.40	0.24	20.28	7.69
9	Loma Prieta	10/18/1989	Coyote Lake Dam	6.9	285	22.30	0.18	22.63	13.18
10	Loma Prieta	10/18/1989	Hayward-BART Sta	6.9	220	58.90	0.16	15.06	3.74
11	Morgan Hill	4/24/1984	Corralitos	6.2	310	22.7	0.109	10.788	2.133
12	N. Palm Springs	7/8/1986	Cranston Forest Station	6	315	35.30	0.17	11.70	1.15
13	Northridge	1/17/1994	Featherly Park	6.7	0	84.20	0.10	7.64	0.81
14	Northridge	1/17/1994	LA-Baldwin Hills	6.7	90	31.70	0.24	14.85	6.22
15	Northridge	1/17/1994	Inglewood-Union Oil	6.7	90	44.70	0.10	10.25	3.21
16	Northridge	1/17/1994	LA-Chalon Rd	6.7	70	23.70	0.23	16.59	3.39
17	San Fernando	2/9/1971	Palmdale Fire Station	6.6	210	25.40	0.15	8.09	1.87
18	Trinidad, California	11/8/1980	Rio Dell Overpass FF	Ms (7.2)	270	-	0.15	8.48	3.25
19	Victoria, Mexico	6/9/1980	Cerro Prieto	Ms (6.4)	45	-	0.62	31.58	13.08
20	Westmorland	4/26/1981	Parachute Test Site	5.8	225	-	0.24	39.23	26.89

Discussion of the results:

The previous models (concrete and steel) were studied according to the traditional pushover method using first mode, non-adaptive pushover methods (MPA, NADP), adaptive pushover methods (CMP, MADP) and non linear dynamic time history in order to make comparisons. The seismic response (story drift ratio) for the studied models via pushover analysis and their errors are shown in the following figures.

Fig. 1 shows that MADP method gave an acceptable estimate of the story drift compared to the nonlinear time history analysis for all stories, while the MPA method gave a better estimate of the studied seismic response in the upper two-thirds than in the lower third. The results of the traditional method using the first mode were less accurate on the upper floors than on the lower ones, while the results of the NADP method were close to the results of the traditional method, and therefore the effect of higher modes did not appear, except for some floors. The results of CMP method in 12 stories model were better than in the 20-floor model, as the effect of the higher patterns did not appear.

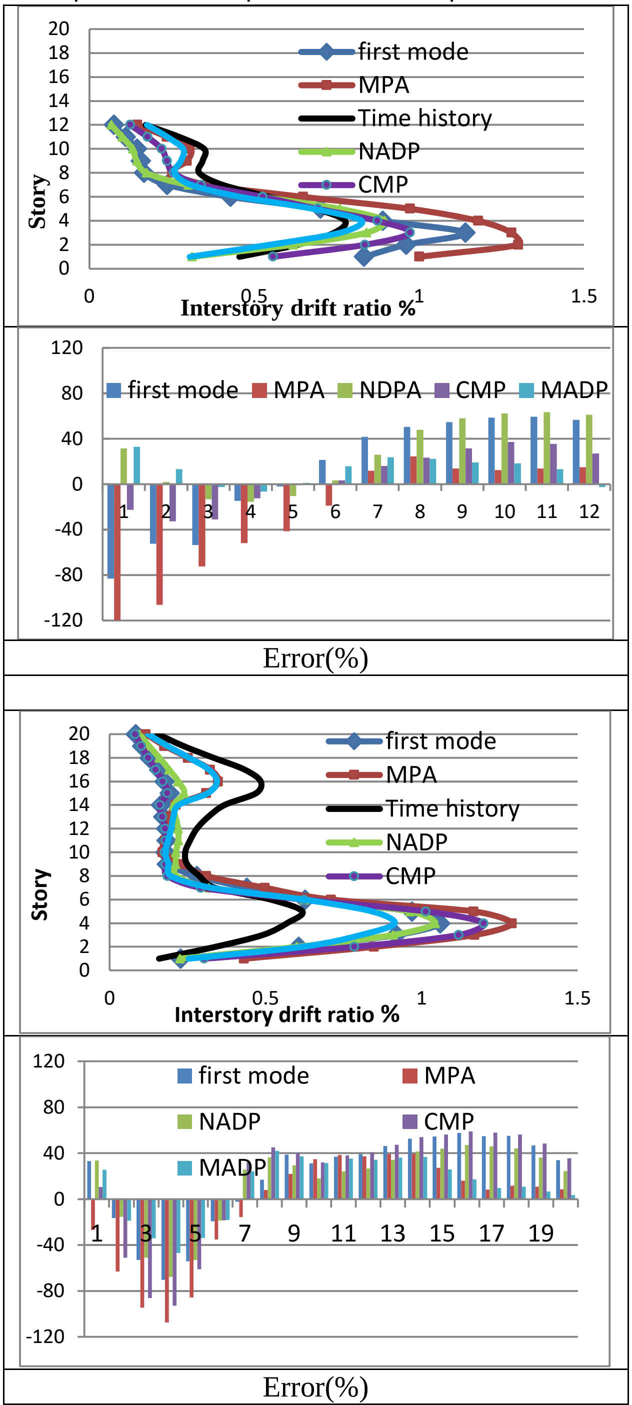

Fig (1) The story drift ratios due to the traditional pushover, MADP, NADP, CMP and MPA procedures and there errors compared to the exact nonlinear time history method for concrete models.

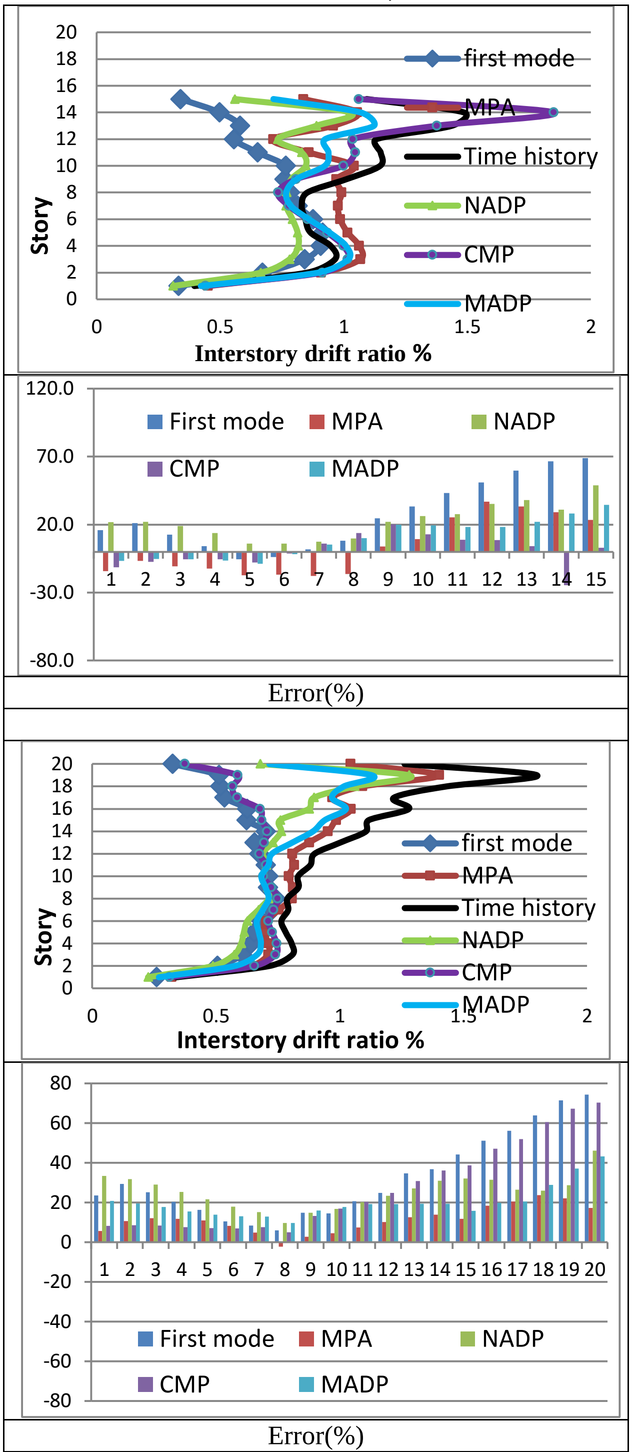

Fig (2) The story drift ratios due to the traditional pushover, MADP, NADP, CMP and MPA procedures and there errors compared to the exact nonlinear time history method for steel models .

Fig. 2 shows that for steel structures, the MPA method gave an acceptable estimate of the story drift compared with the nonlinear time history analysis for all floors, while the MADP and NADP methods gave similar results, but the MADP gave more accurate results than the NADP for all floors .Traditional method with first mode underestimated the story drift in the upper floors. The results of CMP method in 15 stories model were better than in the 20-floor model.

Conclusions:

1. For concrete and steel models, the MADP gave the best estimate of the story drift than the rest of the methods compared to the nonlinear time history dynamic analysis, because it considers the effect of higher modes and the change of dynamic characteristics of the model after entering the nonlinearity.
2. MPA method overestimated the story drift in the lower floors in the most cases, while it gave acceptable results for upper stories.
3. The results of CMP method were acceptable in models (12, 15) stories, however, it was not acceptable in 20 stories especially in upper stories for concrete and steel models.
4. NADP method was effective for steel model ,while it was not for concrete models.

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