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Research Paper

Seismic performance of mid-rise code-conforming X-braced steel frames

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ABSTRACT

Proximity and complexities inherent in the nonlinear dynamic analysis (NDA) impel engineers to lean towards nonlinear static analysis (NSA) in practice. This paper partly explores differences and similarities in responses obtained from these two simulation techniques. The scope of the study is narrowed down to ubiquitous regular code-conforming mid-rise steel X-braced frames. Different common lateral load patterns are coupled with NSA to study their interactive effects on responses. The modal nonlinear static analysis is also carried out, where deemed necessary. NDA using three different earthquake records are conducted as well, to validate NSA results. Results of nonlinear analyses are undertaken to compare provisions of seismic rehabilitation code with those of seismic design codes. Base shear, story drift, lateral displacement profile obtained for each model are independently thoroughly discussed. Eventually, some suggestions to improve design code requirements are accordingly provided, as applicable.

1 Introduction

Tehran, the capital city of Iran, is a metropolitan undergoing massive deal of construction at the current state of development. The city is located in one of the most seismic active regions of the world. It is sprawling horizontally more than rising vertically by the construction of mid-rise buildings, i.e., 4 to 12 story. Steel construction is as equally popular as reinforced concrete mainly owing to the availability of steel with reasonable prices and relative speed as well as ease of erection. As seismic load resisting system regards, the steel structure of a building is typically concentrically braced in one direction and incorporates with moment resisting connections in the direction perpendicular. Notwithstanding unsatisfactory performance of the system as delineated in different reconnaissance reports of major seismic catastrophes in the country, e.g.

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Bam Earthquake[1-4], as well as regardless of remarkable achievements to ameliorate seismic performance of concentric bracing system, e.g., see Lai and Mahin [5], this type of construction is quite famous.

Amongst all types of concentric bracings, X bracing is traditionally the most popular one. Moreover, there are many buildings which are built many years ago in this fashion, and now in need of rehabilitation to meet current minimum seismic requirements. Construction in Iran has to observe ‘National Building Regulations’ consisting of 22 parts prepared by ‘ministry of road and housing.’ The part 10 of regulations deals with steel construction [6]. This part jibes with AISC [7] for the most part, occasionally tweaked mainly to meet specific local requirements of the country. Part 6 of regulations [8] deals with the minimum loading of buildings. The seismic loading in this part is compiled mainly based on UBC 1997 [9]. The same code is also published by the Building and Housing Research Center (BHRC) of Iran renown among engineers as ‘Code no. 2800’ [10]. Noteworthy, Asgharian and Jalaefar explored the seismic behavior of concentric bracing designed in accordance with different versions of Code no. 2800 [11]. Moreover, the Management and Planning Organization of Iran published guidelines on the seismic upgrading of existing building which is the only available source throughout the country [12], which closely follows in FEMA 273 and FEMA 274 [13, 14] footsteps. In essence, seismic design and upgrading of steel structure using American or local Iranian codes lead to very same designs.

Additionally, nonlinear dynamic analysis (NDA) and nonlinear static analysis (NSA) is working out more and more their way into engineering communities due mainly to ever increasing dissemination of ‘performance-based design’ concept. Pros and cons of different versions of pushover analyses are discussed separately elsewhere in detail by Krawinkler and Seneviratna [15] and Chopra and Goel [16]. In this connection, we would also like to draw reader’s attention to the recent work of Landi et al. [17] presenting extensive research on the effectiveness of different pushover methods in handling nonlinear analysis of reinforced concrete frames. Furthermore, Karamanci and Lingos [18] discussed a comprehensive simulation approach to describe the inelastic seismic behavior of concentrically braced frames. Earlier work of Broderick et al. [19] also entails some analytical and experimental findings on the seismic behavior of concentrically braced frames. Previously, Tremblay’s work is of great significance to shed the light of the seismic performance of concentrically braced frames [20]. Shen et al. discussed the seismic behavior of the beam inserted by X bracing [21, 22].

Nevertheless, in line with the companion work of authors, in which a novel load pattern for pushover analysis of structures is proposed [23], this paper with the same principles discusses the results of a comparative study on simulation of ubiquitous ordinary code conforming X-braced mid-rise steel frames through NDA and NSA without any kind of irregularity. Collaboration between analytical results of two methods is discussed. The analytical results are also compared with requirements of the latest update of pertinent American as well as local Iranian model codes. This research concurrently seeks for an answer to a very fundamental yet crucial question of whether or not current seismic design requirements are resulting in a design passing the current seismic rehabilitation provisions.

2 Modeling and Analyses

This research is carried out relying on the analytical groundwork of the companion work, in which further details are available [23]. Nonlinear dynamic analysis (NDA) and nonlinear static analysis (NSA) are employed to study 3D models of regular mid-rise (i.e., 4-story, 6-story, 8-story, and 12-story) buildings. The buildings are designed in accordance with ASCE 7-10 and AISC recommendations using SAP 2000. They are designed in a way that they can pass the corresponding Iranian codes as well. The base shear (V) using Iranian earthquake code no. 2800 can be predicted as follows:

$$V = \frac{A.B.I}{R}W \quad (1)$$

where A is spectral acceleration for the building site, B is the response spectra of the region depending on the soil profile of the building site and the period of the building, I is risk factor of the building depending on the building importance, R is the response modification factor depending on the structural system, and W is the effective weight of the building. Similarly, the base shear can be obtained from IBC or ASCE 7-10 provisions as follows:

$$V = \frac{S_a I}{R}W \quad (2)$$

where I and R are the similar concepts as defined for Equation (1). S_a is the spectral acceleration of the region depending on the building period, soil type and geographical coordination of the building site. The base shear is required to be distributed over the height of the building using a similar relationship by ASCE 7-10, IBC 7-10, and Iranian no. 2800 code as follows:

$$F_i = \frac{W_i \cdot h_i^k}{\sum_{j=1}^n W_j \cdot h_j^k} V \quad (3)$$

W_i is the seismic weight of the i th floor, k is a coefficient depending on the period of the building, h_i is the height of the i th floor, and n is the number of the story of the building. Lumped plasticity model, also known as a phenomenological model, which takes the nonlinear behavior of frame elements using zero-length nonlinear springs at both ends of the member into account is used in accordance with ASCE 41 [24]. Fig.1 presents a general representation of a plastic hinge. The parameters are obtained from ASCE 41. Fig. 2 illustrates a 3D model of the 4-story building. Fig. 3 presents the analytical idealization of models. The structure is designed according to AISC LRFD-10 [7] and crosschecked according to Iranian steel design code (Part 10) [6]. Table 1 through Table 3 of Gholi Pour et al. [23] presents the design of archetypes under consideration. Steel material is DIN St 37 equivalent to ASTM A36 [25]. The lateral load resisting system of buildings is bracing in one direction and moment frame in the perpendicular direction. This system is one of the most common lateral resisting systems for mid-rise buildings to authors’ best knowledge. CSI SAP2000 software is employed to carry out nonlinear static and dynamic analyses [26].

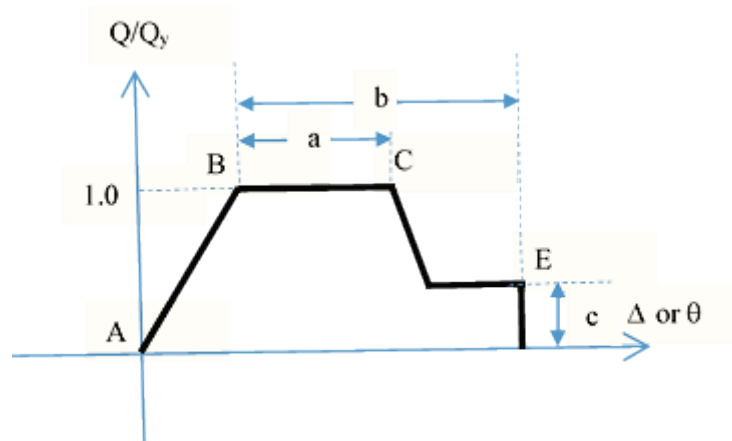


Fig. 1 – General representation of the ASCE 41 plastic hinge. Q is the generalized force which could be moment or force. Δ is displacement and θ is the rotation.

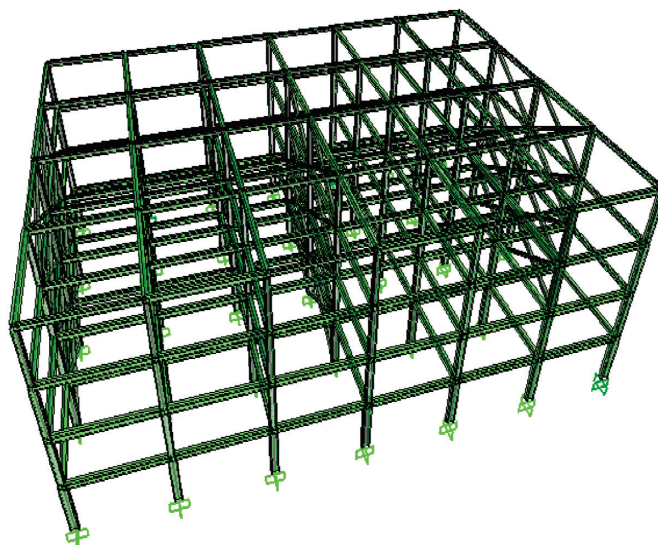


Fig. 2 – 3D Model of 4 story building.

Three different lateral load patterns, namely uniform (hereafter, U), equivalent lateral force (hereafter, ELF) and first mode deformed shape (hereafter, M1), is adopted for nonlinear static analyses as shown in Fig. 4. Pushover analysis load combinations are ‘1.1(Dead Load+ Live Load) + Lateral Load’ and ‘0.9Dead Load + Lateral Load.’ In addition, modal pushover analysis (MPA) is performed for 8 and 12 story buildings as well to account for the possibility of participation of higher modes in the response. For nonlinear dynamic analysis, models are analyzed by scaled time history of Northridge, California, Tabas, Iran and Naghan, Iran earthquakes provided by PEER website [27]. Northridge, California record is chosen because it is one of the most renowned ground motions. Tabas and Naghan earthquakes are chosen because they are the most damaging earthquakes happened in the recent history of Iran.

In the end, obtained displacements from two methods are compared with allowable displacements furnished by FEMA356 [28], ASCE 7-10 [29], and IBC2012 [30]. The first mode of vibration for each archetype is reported. The studied archetypes are mid-rise, therefore their seismic response is expected to be controlled mainly by the first mode of the vibration. However, as discussed earlier, modal pushover is conducted for 8- and 12-story archetypes.

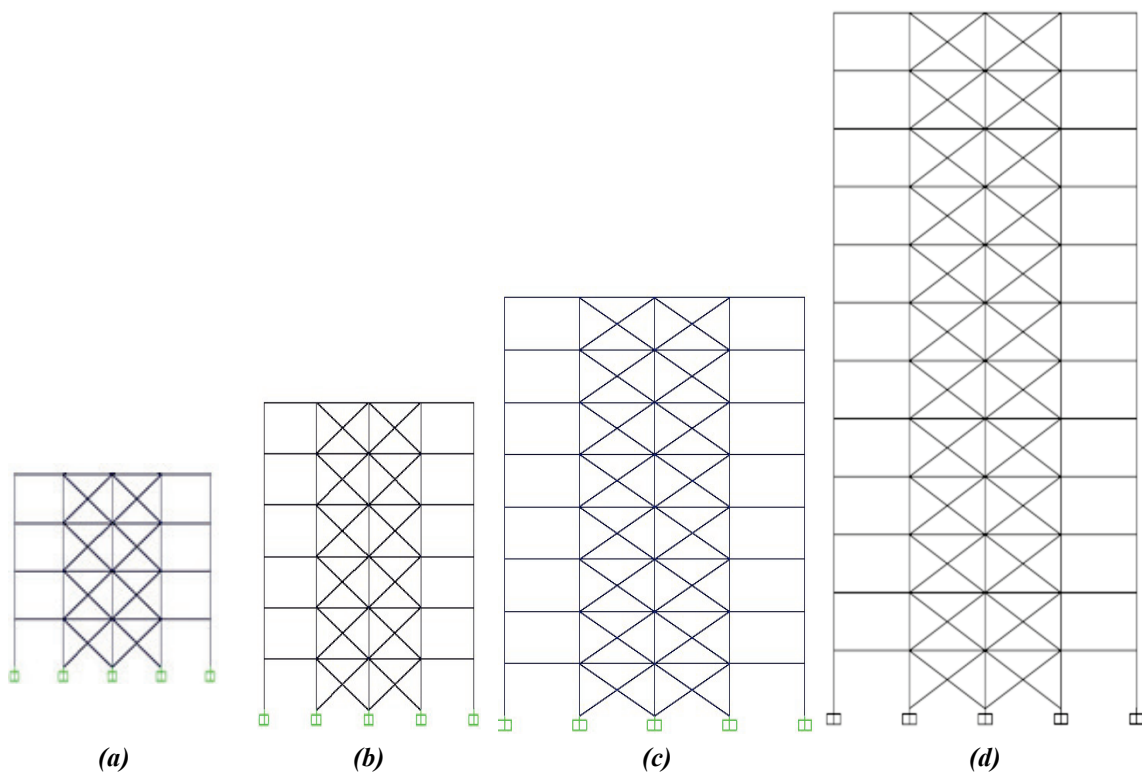


Fig. 3 – Elevation view of idealization of: (a) 4 story (b) 6 story (c) 8 story and (d) 12 story building

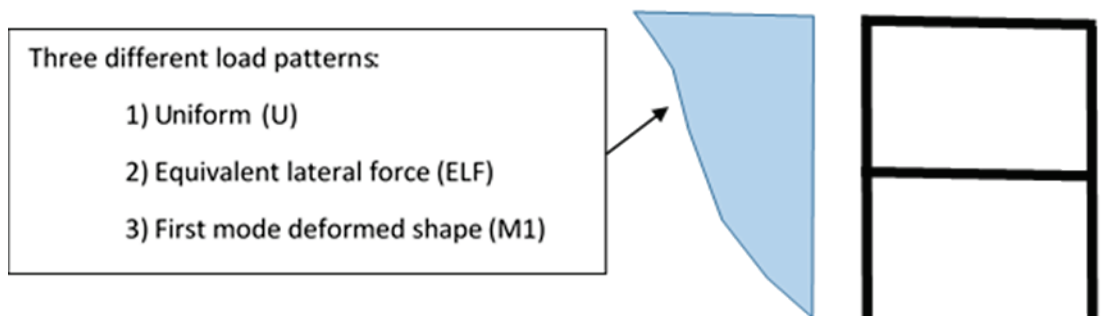


Fig. 4 – Schematic presentation of three load patterns undertaken for the current study.

2.1 4-Story Building Analysis Results

Results obtained from an analytical study on 4-story model could be summarized as follows:

- Period of the first mode is predicted as $T_1 = 0.34$ Sec.
- For both static and dynamic methods, the lateral deformed shape of models is similar to the first mode deformed shape as shown on Fig. 5. Lateral displacements obtained from pushover analysis are in the same range of those of dynamic analysis under Tabas and Northridge earthquakes with the highest deviation of 9%. However, the model exhibits stiffer behavior under Naghan earthquake. Fig. 5 shows that pushover method is quite conservative in the prediction of model response.
- Comparison of maximum displacements shows that IBC2012, ASCE7 2010 and Iranian earthquake code no. 2800 set relatively high displacement level for X type braced steel frames which are far beyond FEMA's immediate occupancy (IO), Life safety (LS), and Collapse Prevention (CP) performance levels.
- The maximum base shear (V_s) of nonlinear is $V_{s,NDA} = 1250.0$ Ton for dynamic, and $V_{s,NSA} = 1340.0$ Ton for static analysis as shown in Fig. 6. Hence, NSA Base shear conservatively predicts NDA results.
- The maximum drift of stories occurs in first story level for both methods according to Fig. 7. Drifts of pushover using uniform, equivalent lateral force and first mode lateral load patterns are near maximum drift of nonlinear dynamic analysis method.
- Plastic hinge formation patterns predicted by two methods show firstly, the formation of the plastic hinge in compression bracing at first and second story, next plastic hinge at tensional bracings, and finally formation of the plastic hinge in compression column near bracing.
- The compressive force developed in bracing plastic hinge at 'life safety' performance level is 68%, 62%, 65% of story base shear for equivalent lateral load, the first mode and uniform pushover methods, respectively. The compressive force of plastic hinge in bracing is 61% of story base shear for nonlinear dynamic analysis at 'life safety' performance level.
- FEMA356 limitation on the application of lateral load patterns is satisfied with 'first mode mass participation' predicted around 83%.

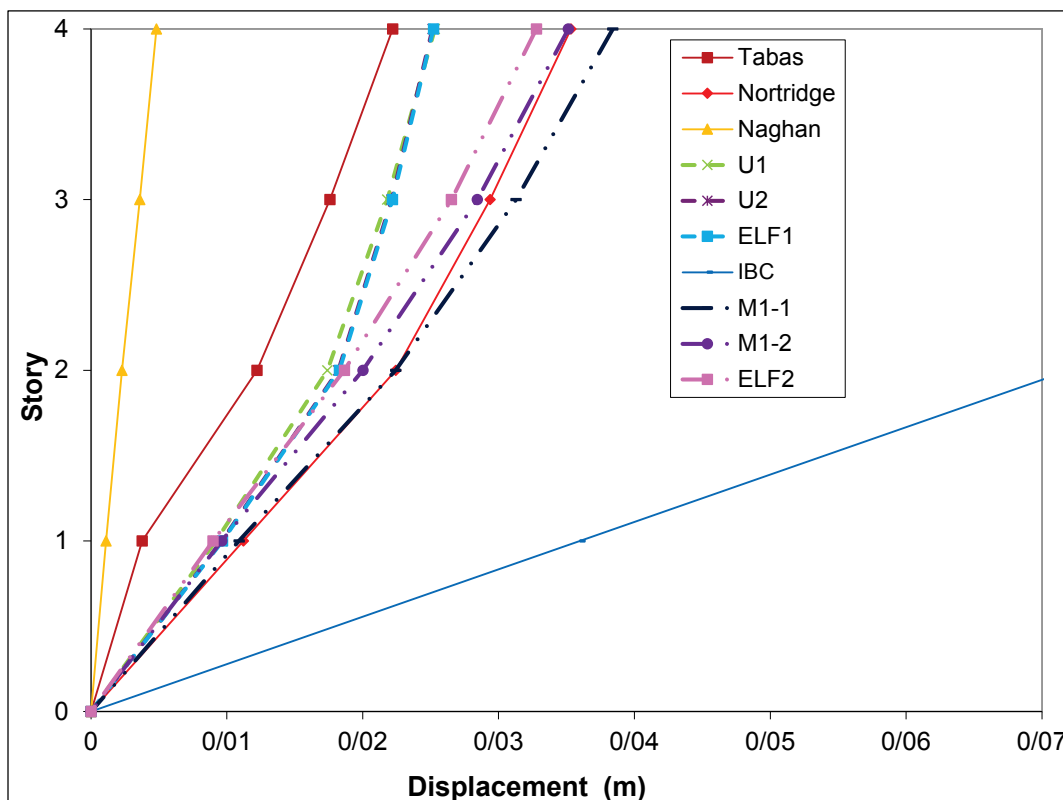


Fig. 5 – Story displacement profile over 4-story building height

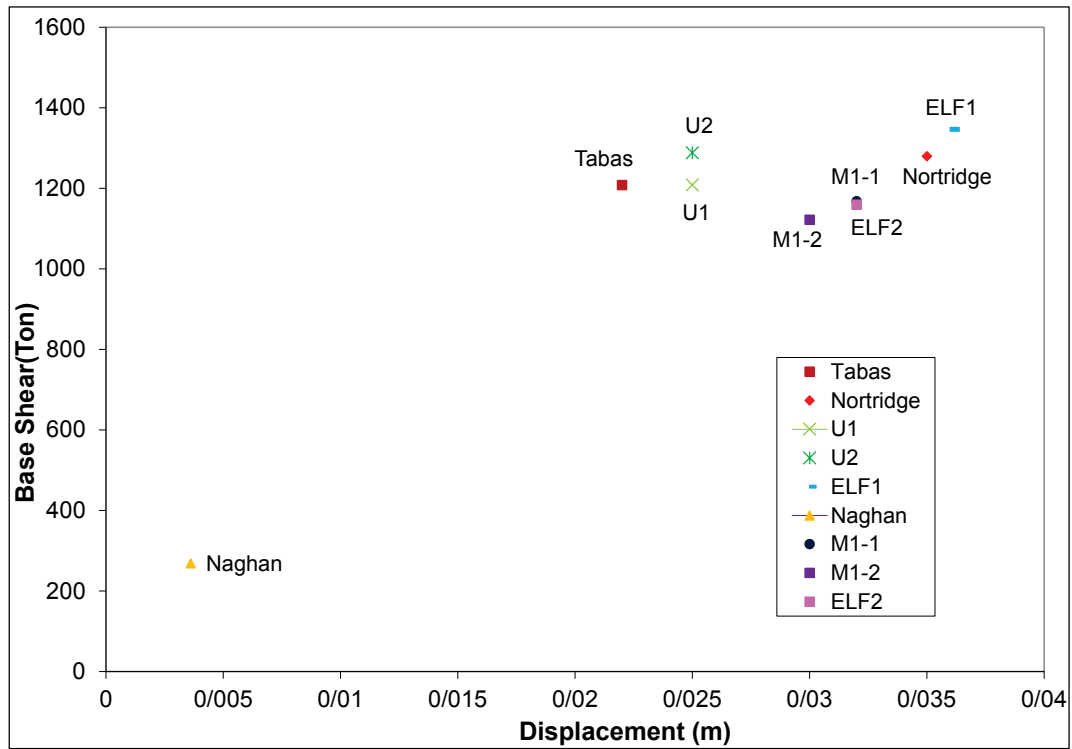


Fig. 6 – Maximum base shear versus displacement of 4-story building

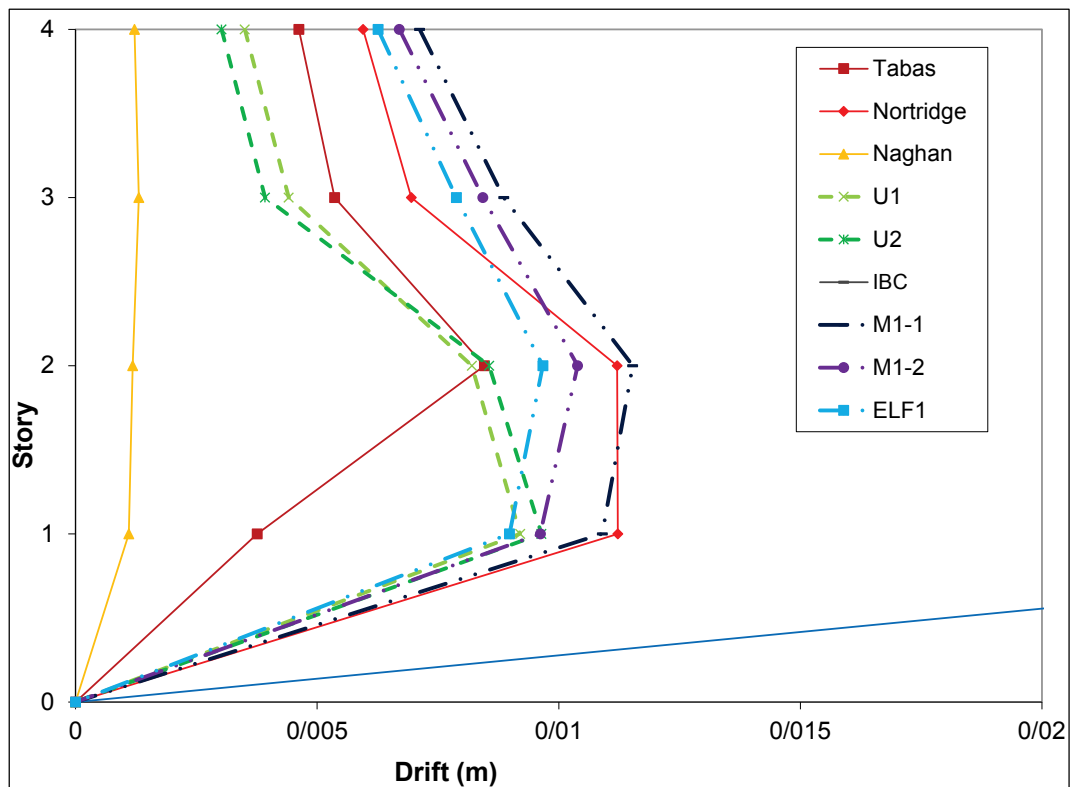


Fig. 7 – Maximum story drift of 4-story building

2.2 6-Story Building Analysis Results

Results obtained from an analytical study on 6-story models could be summarized as follows:

- Period of the first mode is predicted as $T_1 = 0.545$ Sec.
- For both static and dynamic methods, the lateral deformed shape of models is similar to the first mode deformed shape as shown in Fig. 8. Lateral displacements obtained from pushover analysis are in the same range of those of dynamic analysis under Tabas and Northridge earthquakes with the highest deviation of 15%. However, models exhibited stiffer behavior under Naghan earthquake.
- Maximum base shear (V_s) of nonlinear is $V_{s,NDA} = 1399.0$ Ton For dynamic, and $V_{s,NSA} = 1420.0$ Ton for static analysis as compared on Fig. 9. Hence, NSA base shear conservatively estimates NDA results.
- Comparing results with IBC2012 and Iranian earthquake code, it is evident that mentioned codes enforce conservatively high displacement performance criteria for this type of buildings. FEMA performance assessment indicates that the target displacement of roof provisions for ‘life safety’ and ‘damage prevention’ performance levels, is much less stringent than IBC2012, ASCE7-10 and Iranian earthquake code no. 2800 criteria.
- Maximum story drift in NSA occurred in Story 1, and then 2, but in NDA method maximum drift took place on story 2 (Fig. 10). Differences of drifts of NSA using uniform, equivalent lateral force and first mode lateral load patterns with the maximum result of NDA method are more significant than those of 4-story building as described in the previous section.
- Plastic hinge formation patterns show that for both methods, after formation of the plastic hinge in compression bracing, plastic hinge at tensional bracing and compression column near bracing will form. In pushover analysis method formation of plastic hinges starts from story 1, then story 2 and other upper stories. However, plastic hinge forms in story levels 2, 1, 4 using dynamic analysis method.
- Force corresponding to plastic hinge in compression bracing in ‘life safety’ performance level is 75%, 69% and 68% of story base shear for equivalent lateral force, the first mode, and uniform pushover methods, respectively. This force comes to 65% of base shear of the story using nonlinear dynamic analysis method in ‘life safety’ performance level.
- Limitation of FEMA on lateral load patterns application is met sine mass participation in the first mode is around 71%.

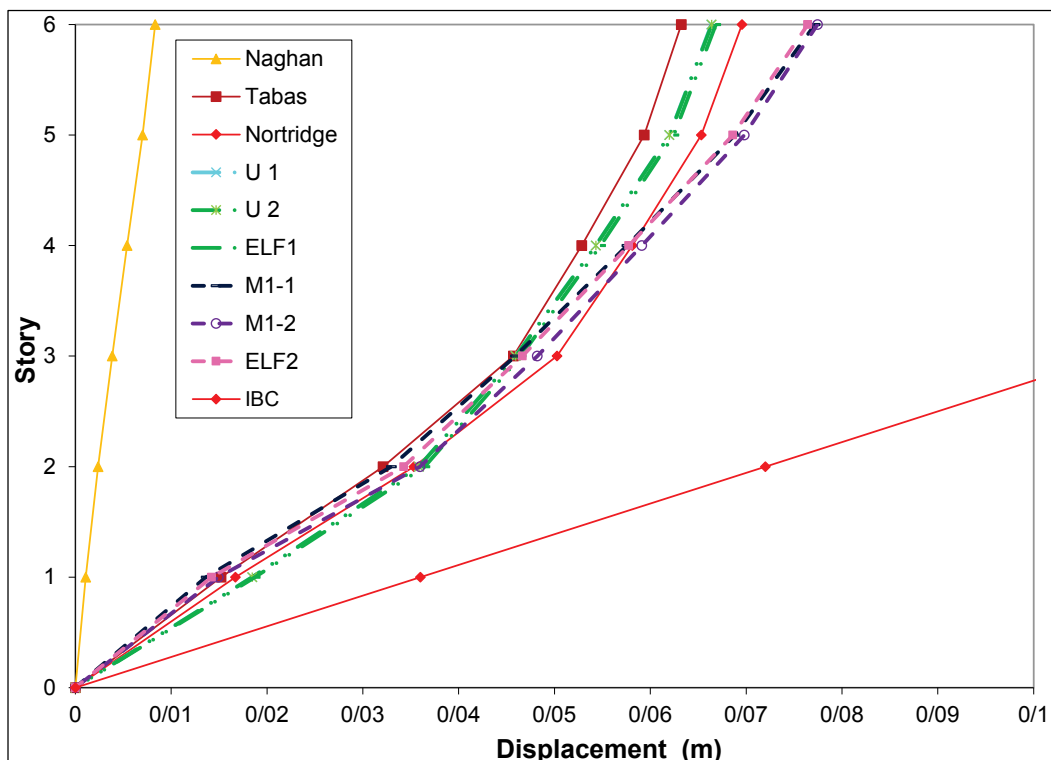


Fig. 8 – Story displacement profile over 6-story building height

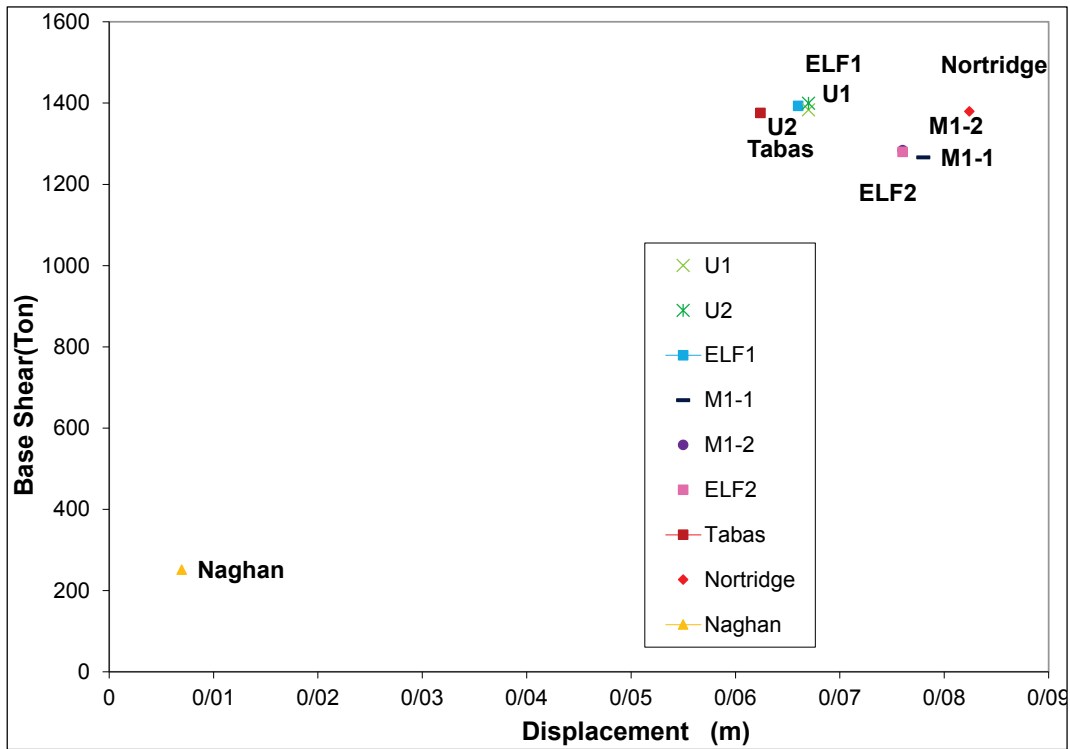


Fig. 9– Maximum base shear versus displacement of 6-story building

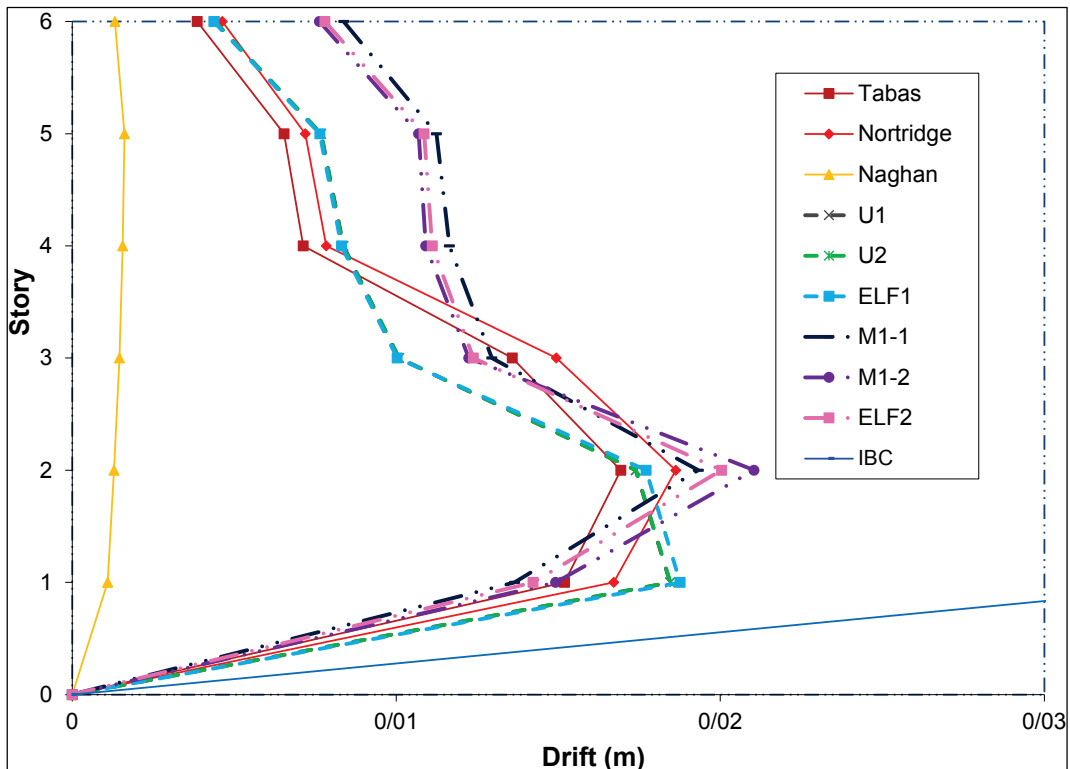


Fig. 10 – Maximum story drift of 6-story building

2.3 8-Story Building Analysis Results

Results obtained from an analytical study on 8-story models could be summarized as follows:

- Period of the first mode is predicted as $T_1=0.72$ Sec.

- Displacement distribution pattern over building height for NSA method is similar to mode-1 deformed shape, but the deformed shape of NDA method does not agree with the mode-1 deformed shape. Consequently, these two methods yield different displacement distribution patterns as shown in Fig. 11. Displacement profile developed by two methods differ mainly between story 3 and 4. To improve consistency between NDA and NSA results, modal pushover is employed to study static nonlinear behavior. The difference between two methods decreased from 38% to 8% incorporating two modes with 87% mass participation.
- Maximum base shear (V_s) of nonlinear is $V_{s,NDA}=2400.0$ Ton. For dynamic, $V_{s,NSA}=1700.0$ Ton. for pushover analysis and modal pushover $V_{s,MPA}=2600.0$ Ton. as shown in Fig. 12. Hence, MPA base shear conservatively could be used instead of NDA results.
- Comparing IBC2012, ASCE7-10 and Iranian earthquake code no. 2800, it is evident that mentioned codes enforce conservatively relatively high displacement performance criteria for this building. FEMA assessment indicates that target displacement of the roof is less than predictions of IBC2012 and Iranian earthquake code no. 2800 in ‘life safety’ and ‘damage prevention’ performance levels.
- NDA and NSA using uniform, equivalent lateral force and first mode lateral load patterns result in different maximum story drift as shown in Fig. 13. Modal pushover (two modes and mass participation of 87%) also fails to capture Northridge, Tabas and Naghan earthquake drift pattern completely.
- Observations on plastic hinge formation patterns are as follows: In NDA, first plastic hinges form in the fourth story and followed by the fifth and sixth story. In NSA using uniform, equivalent lateral force and first mode load patterns, first plastic hinges form in the first story and followed by the upper stories. In modal pushover, first plastic hinges form at the fourth story and followed by the fifth and sixth story.
- Force corresponding to plastic hinge in compression bracing is 73%, 70% and 68% of story base shear pertinent to equivalent lateral force, the first mode and uniform load pattern, respectively, at ‘life safety’ performance level. The plastic hinge of compression bracing is 68% and 69% of base shear of the story using MPA and NDA method at ‘life safety’ performance level.
- Limitations of FEMA356 on the application of lateral load patterns are not satisfied except for MPA.

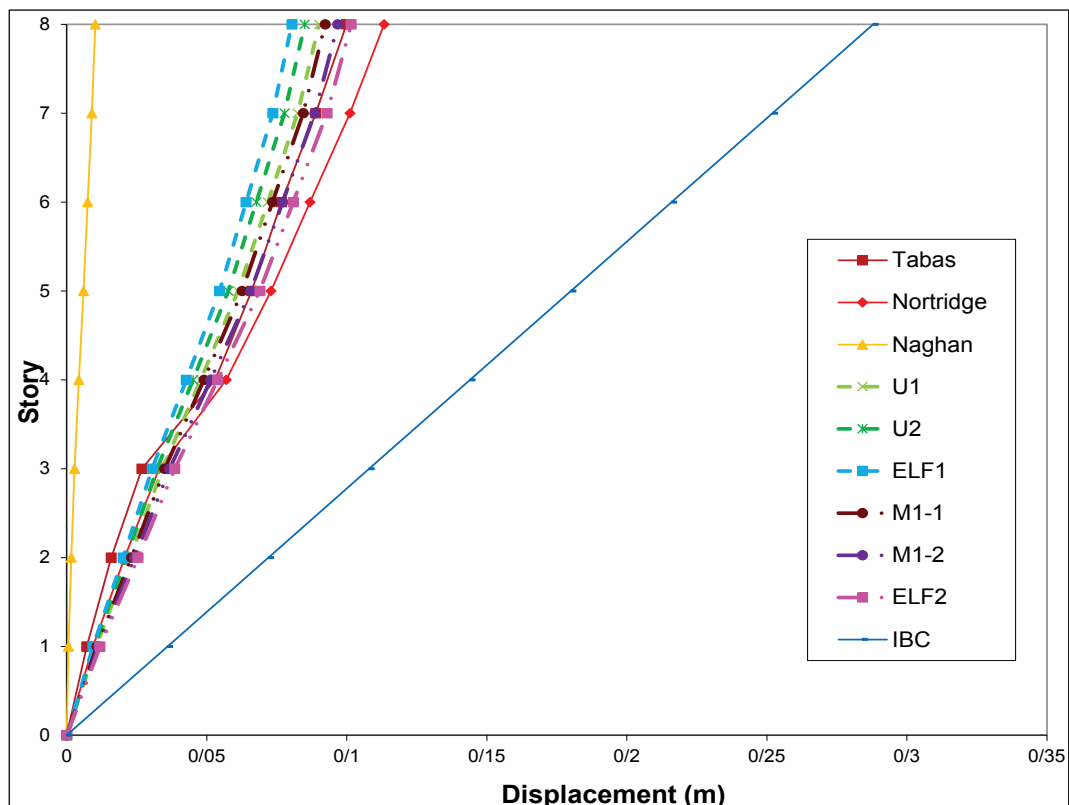


Fig. 11 – Story displacement profile over 8-story building height

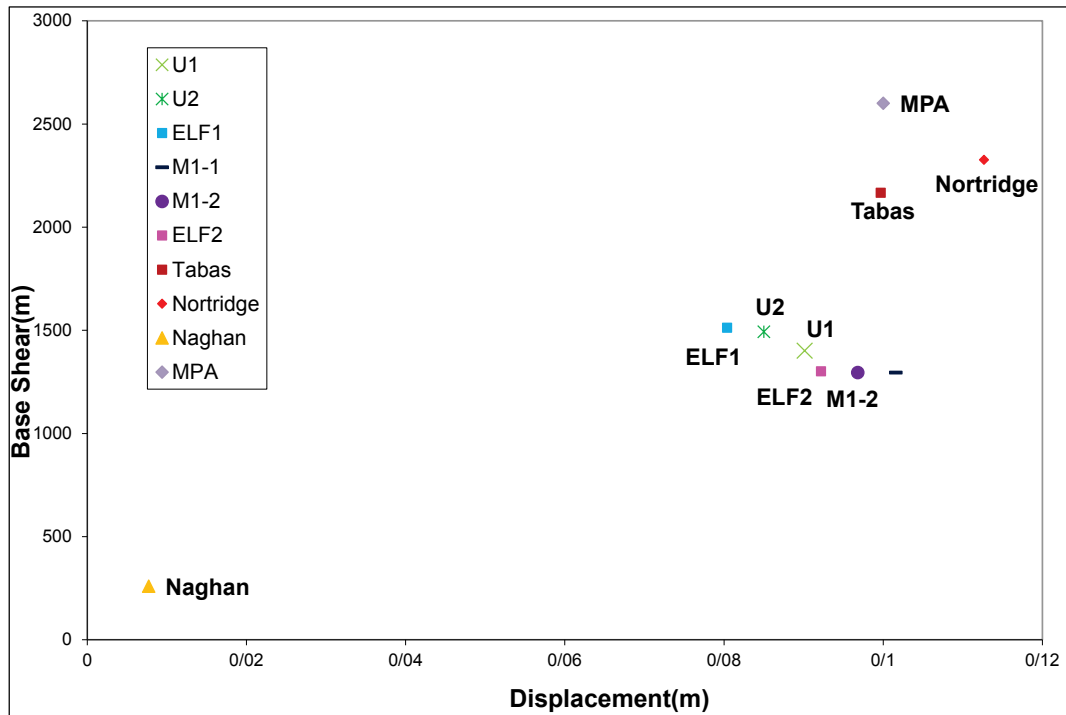


Fig. 12 – Maximum base shear versus displacement of 8-story building

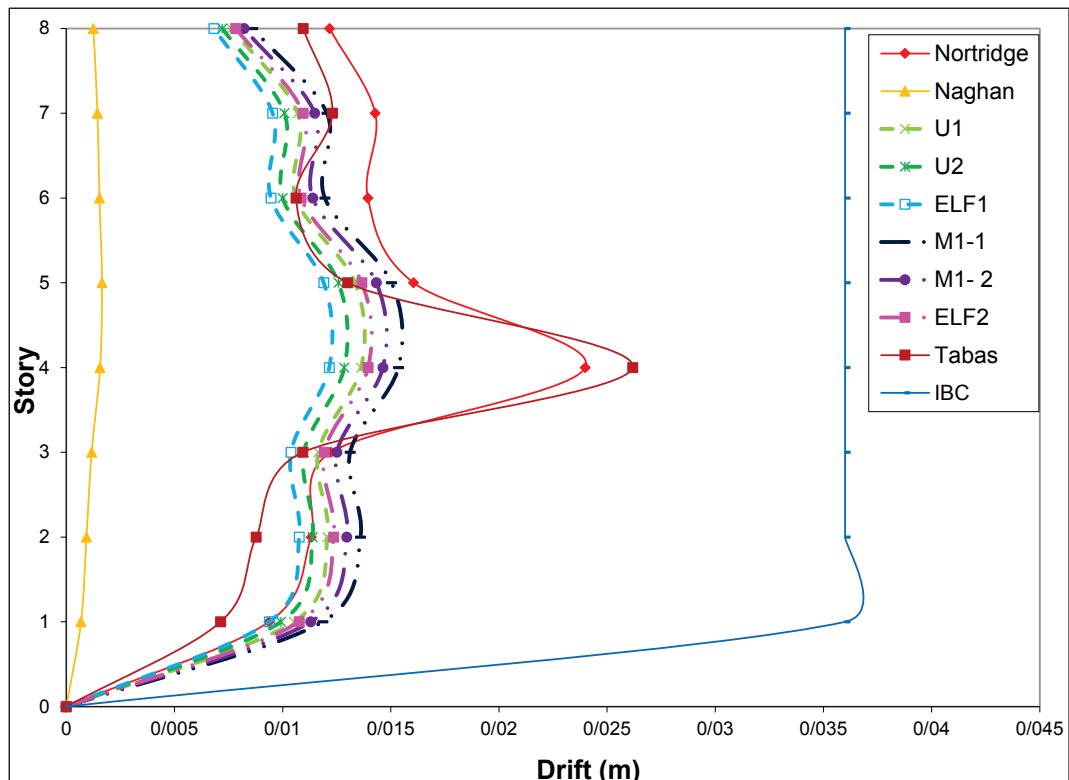


Fig. 13 – Maximum story drift of 8-story building

2.4 12-Story Building Analysis Results

Results obtained from an analytical study on 12-story models could be summarized as follows:

- Period of the first mode is estimated as $T_1=1.089$ Sec.

- Lateral displacement distribution pattern over height for pushover analysis method (using uniform, equivalent lateral force and first mode load patterns) is not in agreement with Tabas and Northridge earthquakes analysis results as shown in Fig. 14. For example, the maximum displacement of the roof in Northridge time history analysis is 1.3 times greater than pushover method results. However, the maximum displacement of the roof is 1.1 times greater than the magnitude of that of NDA using MPA (3 modes with 91% mass participation).
- Maximum base shear (V_s) of nonlinear dynamic analysis method and pushover performance level base shear are: $V_{s,NDA}=3400$ Ton., $V_{s,NSA}=3010$ Ton. And $V_{s,MPA}=3550$ Ton as shown in Fig. 15. V_s , MPA exceeds V_s , NDA which means conservatism of MPA method in regard to NDA in terms of base shear.
- Comparing the results with IBC2012 and Iranian earthquake code no. 2800 requirements show that these codes entail conservatively high displacement levels for this building genre. FEMA seismic assessment indicates that in ‘life safety’ and ‘damage prevention’ performance levels, target displacement of the roof is less than IBC2012 and Iranian earthquake code criteria.
- The maximum drift of stories as shown in Fig. 16 occurs in the eighth story level for NDA method. For NSA using uniform, equivalent lateral force and first mode the maximum drift of stories occurs in the first story level. For MPA using three modes that have the mass participation of 91%, maximum drift occurs at the eighth story.
- Observations on plastic hinge formation patterns are abridged as follows: First plastic hinge occurs in the eighth story and followed by the fifth and sixth story in NDA method. Plastic hinges form first in the fourth story, followed by the fifth and sixth stories for NSA method using uniform, equivalent static load, and first mode load pattern. Plastic hinges form firstly at eighth and followed by the fifth and sixth stories using MPA.
- Force corresponding to plastic hinge in compression bracing at ‘life safety’ performance level is 77%, 74% and 69% of story base shear for equivalent lateral force, the first mode and uniform load lateral load pattern, respectively. The plastic hinge of compression bracing to reach ‘life safety’ performance level requires 69% of base shear of the story using nonlinear dynamic analysis method.
- Limitations of FEMA356 on the application of lateral load patterns are not met except for MPA (three modes with 91% mass participation).

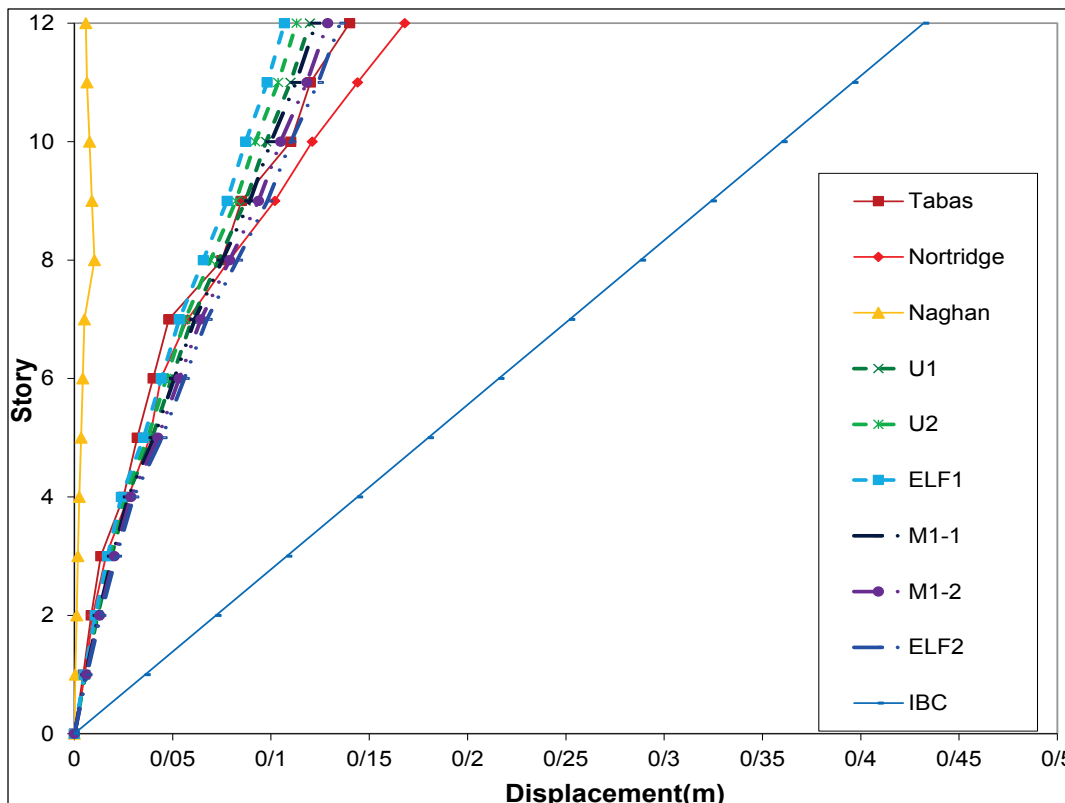


Fig. 14 – Story displacement profile over 12-story building height

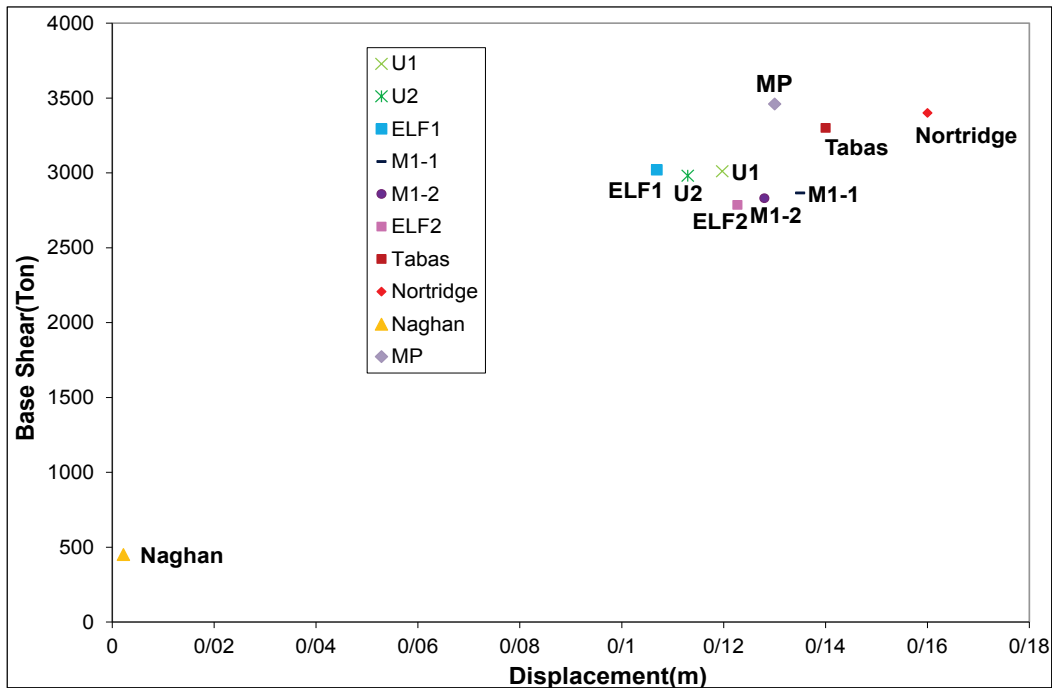


Fig. 15 – Maximum base shear versus displacement of 12-story building

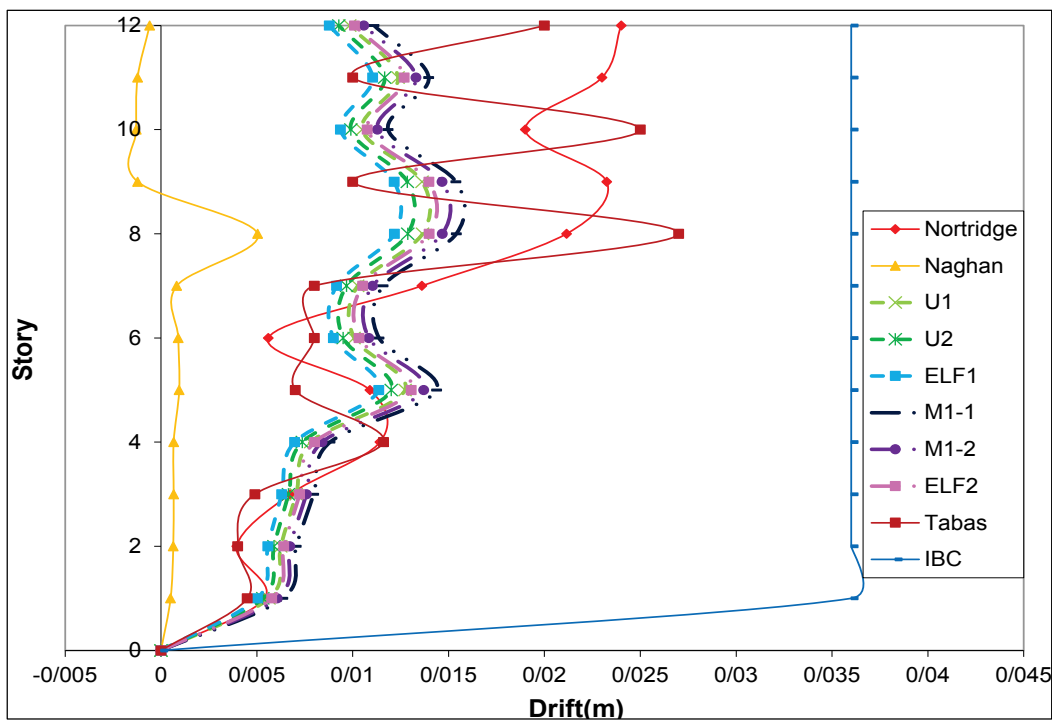


Fig. 16 – Maximum story drift of 12-story building

3 Comparison of results with IBC model code

In this section, the maximum ‘roof displacements’ of AISC-LRFD10 code conforming X-braced buildings as described earlier are compared to FEMA 356 and IBC2012 requirements. Fig.17 presents a comparison between maximum roof displacements obtained from NSA, MPA, and NDA and those of FEMA ‘life safety’ performance level. Life safety is one of the fourth performance objectives defined by FEMA 273 or ASCE 41, which is roughly equivalent to the target objective of the prescriptive design codes, e.g., ASCE 7-10 and Iranian code no. 2800. It could be observed that NSA, NDA, and MP results are consistent for 4- and 6-story buildings. It could also be concluded that performance level of AISC-LRFD10 code

conforming buildings is near to FEMA's 'life safety' level. Fig.18 shows maximum roof displacements through NSA, MP, NDA and those of FEMA 'life safety' performance level along with IBC2012 displacement level. It could be perceived that IBC2012 and Iranian earthquake code no. 2800 exhibits conservatively high-performance level for X-type braced steel frames.

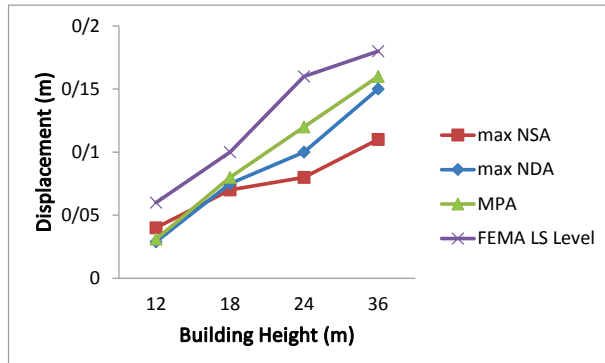


Fig. 17 – Roof Displacements

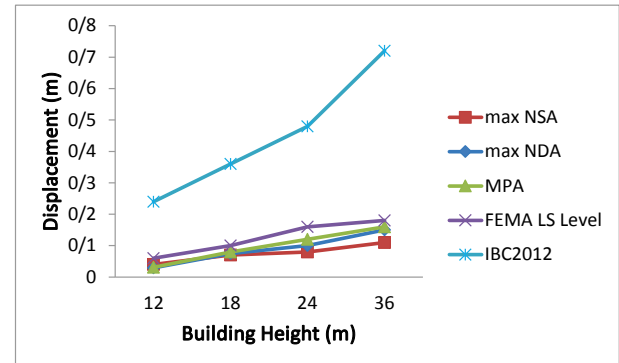


Fig. 18 – Roof Displacements comparison IBC2012

4 Conclusion

The following conclusions could be drawn as a result of the current course of study:

Consistency observed between NSA and NDA results for X type braced steel frames up to 6 stories proves that all FEMA 356 load patterns can be used for vulnerability assessment of buildings with such a system as long as their T1 is less than 0.545 Sec. and their mass participation is greater than 71%.

NSA using ELF exhibits the highest agreement with NDA results. These conditions can guarantee the conservation of NSA, with the first mode deflection pattern, in the prediction of the nonlinear dynamic behavior of the structure. In this regard, FEMA and Iranian rehabilitation code no. 360 stipulate only 75% limitation on mass participation to allow for incorporation of the first mode and equivalent lateral force patterns into modeling. However, the current study suggest T1 shall also be limited in addition to minimum 75% mass participation in the first mode for these X type braced structures.

If X type braced steel frames pose T1 greater than 0.545 Sec. and mass participation less than 71%, then results of NSA using uniform, equivalent lateral force and first mode load patterns could differ significantly from NDA results in an unconservative manner. MPA method (with modes that have a minimum of 87% mass participation) could yield more exact and reliable results for these types of structures while NSA is mostly unable to capture dynamic response of models.

Seismic rehabilitation model codes, such as Iranian rehabilitation code no. 360, do not set forth any criteria concerning incorporation of uniform load pattern. Critical deviations from dynamic responses are observed in the current course of study from NDA for 8 and 12 story archetypes using this method. Hence, it is proposed that mass participation percentage limit be also imposed on the application of this procedure similar to that of ELF and M1 load patterns. Comparing analytical nonlinear displacements with IBC2012 and ACSE7 2010 displacement limits reveals that the magnitude of allowable displacement according to these codes are conservatively high for X-type braced steel frames. These codes present a general criterion for buildings' displacements which are more stringent from any performance level defined by FEMA356 such as IO, LS, and CP. To formalize an allowable displacement criterion for design codes (like IBC and ASCE7), it is proposed equations to be sought relating the displacement to the first mode period, i.e., $\delta = \alpha T1^p$. δ shall be limited to the relevant FEMA's performance level target displacement. This limitation could lead to the more realistic estimation of allowable displacements of relatively rigid systems such as X type braced steel frames.

It is observed that the axial compressive force of bracing obtained from NSA runs, between 62% and 77% of story base shear at 'life safety' performance level. However, the same force runs between 61% and 69% of story base shear in NDA of the same model. Regardless of the number of stories, pushover proves to be conservative in terms of prediction of plastic axial hinges in X type bracings. Comparing these results with AISC-LRFD and Iranian design steel code provisions on the design of compression bracing stipulating 50 to 70% of story base shear shows that NDA results relative to those of NSA is closer to AISC-LRFD and Iranian design steel code 2013 provisions on compression bracings design.

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