SEISMIC RESPONSE EVALUATION OF IRREGULAR HIGH RISE STRUSTURES BY MODAL PUSHOVER ANALYSIS

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ABSTRACT :
This paper investigates the accuracy of the modal pushover analysis to estimate the seismic performance of high rise buildings. The effects of structural irregularities in stiffness, strength, mass and combination of these factors are considered. In other words reliability of the modal pushover analysis (MPA) has been verified by defining a referenced regular structure for comparison between MPA and nonlinear dynamic analysis. Using two analysis method for vertically irregular and regular frames leads to the following results: (1) the mass irregularity conditions were found to have a negligible impact on the seismic performance of building, (2) The accuracy of modal pushover analysis to estimate the seismic performance increases when irregularities conducted at lower half stories, (3) Additionally the accuracy does not deteriorate when irregularities provides in the middle or upper story, and (4) Time saving and accuracy besides conceptual simplicity of MPA in brief leads to reliable estimation of seismic performance.

KEYWORDS:
Modal pushover analysis, Dynamic analysis, Multi-story frames, Vertical Irregularity

1. INTRODUCTION

Evaluation of earthquake induced demand has reviewed in most of recent studies conducted by civil engineers as a vital part of structural design. Researchers investigate the methods to simplify structural computation and have a good estimation of seismic deformation demands for multi degree of freedom (MDOF) structures. This importance emphasized when gathered by irregular factors frequently occurred at new buildings because of functional consideration.

Past analytical studies, like research efforts of Al-Ali and Krawinkler (1998), Chopra and Chintapakdee (2004) considered different kinds of structural irregularities. These irregularities considered at eight different positions of buildings height and also three separate and combined types of mass, stiffness and strength irregular cases. Al-Ali and Krawinkler performed nonlinear dynamic analysis of ten-story shear type building with various modification factors on referenced frame. Chopra and Chintapakdee at their study considered comprehensive range of structural height. Seismic response of vertically irregular frames of three to eighteen stories frame evaluated at their research effort and by introducing MPA procedure, they offers several attractive features. They believe that MPA procedure retains the conceptual simplicity and computational attractiveness of current pushover procedures with invariant force distribution- now common in structural engineering practice.

Comparing seismic response of structures based on FEMA force distribution and also MPA method with non linear dynamic history analysis, relatively clarified advantages and disadvantaged of these methods. To draw some general conclusion for MPA method, different kinds of structural height and also irregular cases investigated by MPA method and gathered by results of previous studies. Therefore, at this study concerning tall steel building, make it possible to more pronounced efficiency and deficiency of offered MPA method with other suggested methods of structural analysis. In other words, at this study, MPA method has implemented for high-rise structures more than previous studies of other researchers.

2. DESCRIPTION OF THE BUILDINGS

In the present study, one-bay, hypothetical sixteen-story steel moment resisting frame selected as reference frame. A story height, h, of 3.5 m was assigned at all floors. Hence, the structures with the height of 56 m studied herein are potentially active for inelastic seismic response, and they should demonstrate through nonlinear dynamic analysis. Since at this study investigation of MPA for high rise structures is the main purpose then we select our reference frame in accordance to previous research to determine accuracy of MPA for higher buildings.

The frame is designed according to Iranian code of practice for seismic resistant design of building, No 2800 version 3, with a preliminary linear dynamic analysis and corresponding design considerations. The design gravity loads, dead (D) and live (L) selected upon to 6th building code of Iran to be 500 kilograms per square meter at residential buildings and also normal direction span of 5m considered for hypothetical reference frame. Rigid framing construction was assumed with all members being rigid connected. To eliminate the over-strength effect, sections vary continuously at each two stories, Change from tube 50 cm at base level to tube 20 cm at roof. Tube shapes are square and are sized to meet load requirement. Beam sections are the same at all level, IPE500.

To draw some unified comparison, model assumptions are selected similar to pre-conceptions of reference regular frame in previous studies of Chopra and Goel (2001). The frame is designed according to the strong-column-weak-beam philosophy; therefore plastic hinges form only at beam ends and the base of the first story columns. However, other trends like column hinge model recommended at research of Krawinkler for hinge formation. At column hinge model higher ductility demand satisfied. Since in the column hinge model individual story mechanism will form, this type of model will provide an upper bound on the effects of irregularities on inelastic behavior. Based on presented model, awareness of this modeling assumption is critical in the interpretation of the results, because smaller effects of irregularities have to be expressed in most realistic cases since story mechanisms are usually not permitted in code designed structures [1]. Rayleigh damping is adopted with a same damping ratio for first and fourth mode shape of structures (5%). For calculation of fundamental vibration period at steel moment resisting frame Eqn.1.1. used based on seismic building code of Iran (2005).

$$T \approx 0.08H^{\frac{3}{4}} \tag{1.1}$$

Where H is the total height of the frame and T is period. T is 1.64 s. This value is smaller in comparison to stated formula of Chopra and Goel (2001), that defines the mean-plus or minus one standard deviation of measured periods of steel moment resisting frames.

For better understanding effect of irregularity, we deal exclusively with 2D frames and Irregularities exerted at the height of structures as shown in Fig.1. Thus, some of the current changes at plan related to torsion variation are not discussed at this study. A review of research on seismic behavior of irregular building structures, comprehensively, demonstrates an overview of the progress in research regarding seismic response of plan and vertically irregular building structures. The tendency to separate irregularity in plan and in elevation, simplify investigation of these effects for reference buildings.

This state of the art reports vertical irregularities, suggested by Krawinkler in mass, stiffness and strength distribution. By changing the mass, stiffness or strength distribution of the base case and keeping the same distribution as the base case, irregular cases produced. Most of the aforementioned studies were conducted using modification factors. Modification factors (MF) easily represents the amount of change for each parameter. For conventionally representation of each irregular case these letters introduced, Krawinkler and Al-Ali (1998):

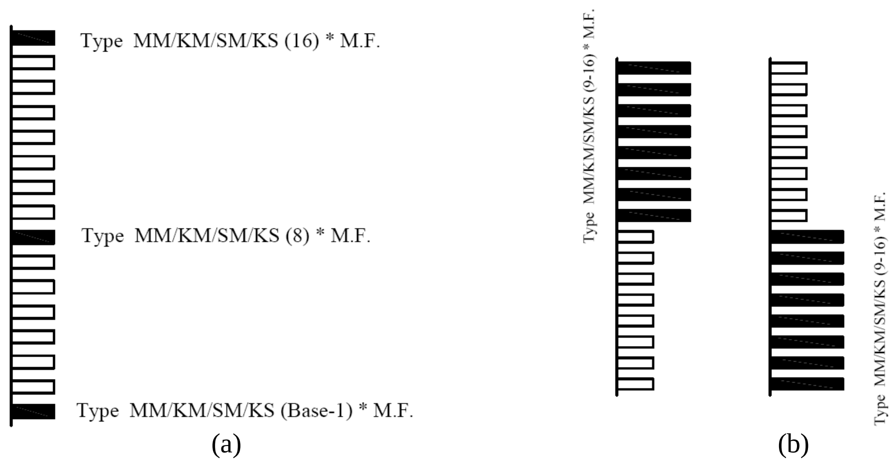

Figure 1 Irregular cases (a) Single story (b) Lower or upper story cases

MM: Cases with the mass irregularities
KM: Cases with the stiffness irregularities
SM: Cases with the strength irregularities
KS: Cases with combination of stiffness and strength irregularities
(n): n represents floors or stories with modification
M.F.: represents the amount of the modification factor.
For example, KM (8)*0.5 introduced a case with a soft story at mid-height. This case is created by modifying the stiffness of the eighth story of the base case by a factor of 0.5. Figure

In order to demonstrate the validity of statistical comparison at this study, each irregular case is created having the targeted mass, stiffness or strength distributions and a fundamental period value of 1.65 Sec. Therefore, with the aim of obtaining reliable comparison, base shear considered as a building indicator.

3. EARTHQUAKE EXCITATIONS

To investigate the accuracy of different methods to predict the seismic response of moment resisting high-rise steel frames, a set of large-magnitude-small-distance records (LMSR) considered. These ground motions were obtained from pacific earthquake engineering research center database, with the aim of using diverse records and also in some cases in accordance to ground motion listed in Chopra and Chintapakdee (2004), to comply results of this study with previous studies. In addition to previous records at location of California, Tabas record, considered for seismic zone of Iran. This ground motion record has peak ground acceleration (PGA) equal to 0.852g, with magnitude 7.4, and known as strong motion in comparison to previous records. Varying the range of records at this study, make it possible to emphasize effects of distributed earthquakes and also investigate the accuracy of MPA method for case of Tabas defined as extremely strong record. Constant-ductility

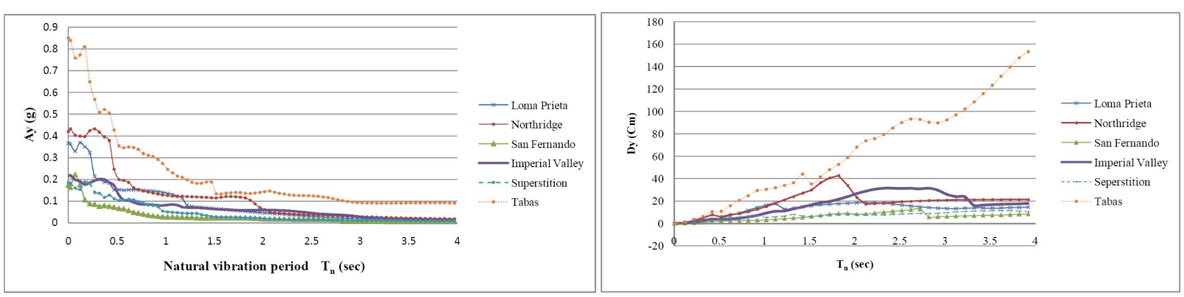

Figure 2 Pseudo-acceleration spectra and deformation spectra of larger-magnitude-small-distance records set of ground motions. Damping ratio=5 %, strain hardening ratio α=3%, μ=6

In general, the buildings are assumed to be located on a soil type 2 of 2800 building code of Iran equivalent to type S_B of the UBC 1997, in a seismically active area, zone 4 with PGA of 0.35g. Scale factor for each record acquired in accordance to proposed method of 2800 Standard code.

4. FEMA NONLINEAR STATIC PROCEDURE

Based on FEMA-356, the Nonlinear Static Procedure, NSP, is generally more reliable approach to characterizing the performance of structure than linear procedure. However, it is not exact, and cannot accurately account for changes in dynamic response as the structure degrades in stiffness or account for higher mode effects. When the NSP is utilized on a structure that has significant higher mode response, the Linear Dynamic Analysis (LDP) is also employed to verify the adequacy of the design, FEMA 356 (2000).

In the NSP, or pushover analysis, monotically increasing lateral forces are applied to control point of structure, until the predetermined target displacement occurred at structure. Normally control node defined at roof level. Target displacement could be introduced as probable value of earthquake induced demand. When we consider a target value for roof displacement, it should be adaptive with structural capacity under design earthquakes and also, it should demonstrates structural withstanding due to reliable limit of deformation (displacement). Because of correlation of nonlinear static methods and target displacement, determination of target displacement is as important as performing analysis.

Consideration of target roof displacement can be done in two ways. First, we use value of nonlinear response history analysis, as obtained form time history analysis of structure. This value is known as exact value of structural response due to earthquake excitation and hence it makes the most reliable answers. Other way for calculation of target displacement at roof level has been suggested in nonlinear static procedure of FEMA-356. The target displacement, δ_t at each level shall be calculated in accordance with Eqn. 4.1., and as specified below.

$$d_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4p^2} g \tag{4.1}$$

C_0 = Modification factor to relate spectral displacement and expected maximum elastic displacement at the roof level; C_1 = Modification factor to relate expected maximum inelastic displacement to displacement calculated for linear elastic response; C_2 = Modification factor to represent the effect of stiffness degradation, strength deterioration and pinched hysteretic shape on maximum displacement response; C_3 = Modification factor to represent increased displacement due to dynamic $P-\Delta$ effects; T_e = Effective fundamental period of the building in the direction under consideration calculated using the secant stiffness at a base shear force equal to 60% of the yield force; S_a = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g, FEMA-356 (2000).

In addition to target displacement determination, lateral load distribution through the height of structure is a very important factor for evaluation of structural response due to earthquake excitation. One of the primary assumptions of nonlinear static analysis procedures is that the behavior of a structure with multi degrees of freedom (MDOF) subject to seismic ground motion can be estimated from the response of an oscillator with a single degree of freedom (SDOF). In order to generate the SDOF model, the engineer generates a global force-deformation relationship for the structure by subjecting a MDOF model to a predetermined lateral load vectors. This relationship is then converted to an equivalent SDOF representation to estimate the maximum global displacement of the model using displacement modification or equivalent linearization techniques FEMA-440 (2005).

Development of a pushover curve by nonlinear static procedure (NSP) in FEMA-356 is followed by monotonically increasing lateral forces with a specified height-wise distribution. There are a number of options for the form of the load vector used to generate the SDOF model of a structure. Indeed, Generation of pushover curve or capacity curve, representing the relationship between the applied lateral forces and the global displacement at the roof level or some of other control point, is proportional to selection of each load options each one depend to the mass at that level and acceleration determination from a specific shape vector assumption, FEMA 356, 440 (2000), (2005).

Uniform, Equivalent lateral forces (ELF) and First mode are the current load vectors recommended by FEMA. Results of FEMA analyses for floor displacements, story drifts and hinge plastic rotations have shown in Fig. 3. for Tabas record.

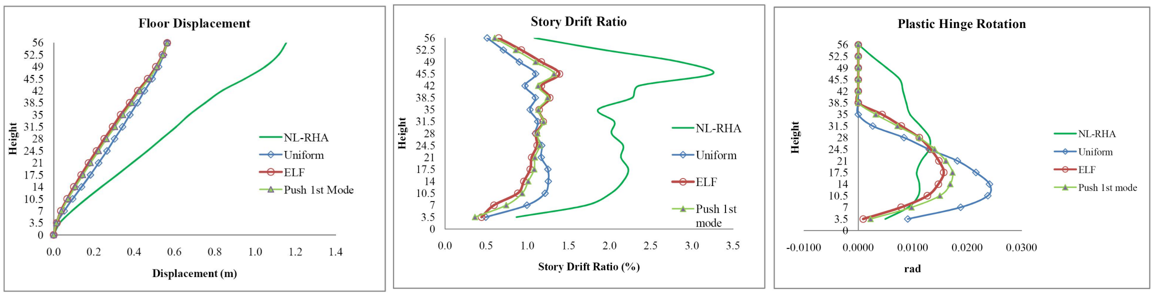

Figure 3 Tabas (a) Roof displacements (b) Story drifts (c) Hinge rotations

5. MODAL PUSHOVER ANALYSIS

MPA algorithm for determination of structural response is performed like recommended procedure of Chopra and Goel (2001). It is apparent and logical that the use of multiple mode pushover techniques (MPA) should produces generally better estimates of inter-story drift than single load vectors. Although higher modes typically contribute little to displacement, multiple mode pushover analyses may be useful for identifying cases in which displacement responses are dominated by a higher mode FEMA 440 (2005).

The results of modal pushover analyses procedure considering the response due to Tabas excitation have demonstrated In Fig. 4. In order to explain the validity of the modal pushover analyses procedure to predict the seismic responses (floor displacements, story drift ratio, hinge plastic rotation), results of Figs. 3 and 4 compared in section 6. Even though, seismic responses for each solution can be considered as an indicator of structural behavior, but the differences between the various approaches determine the level of answer's dispersion. The COV values for displacement demands are not large enough to cause significant differences in the values between the simple mean and COV and the median of floor displacement and story drift, but accuracy decreased for joint rotations. For the base case structure and the ground motions used in this study the mean value of the parameter α_1 is 1.45 and its COV is 0.068.

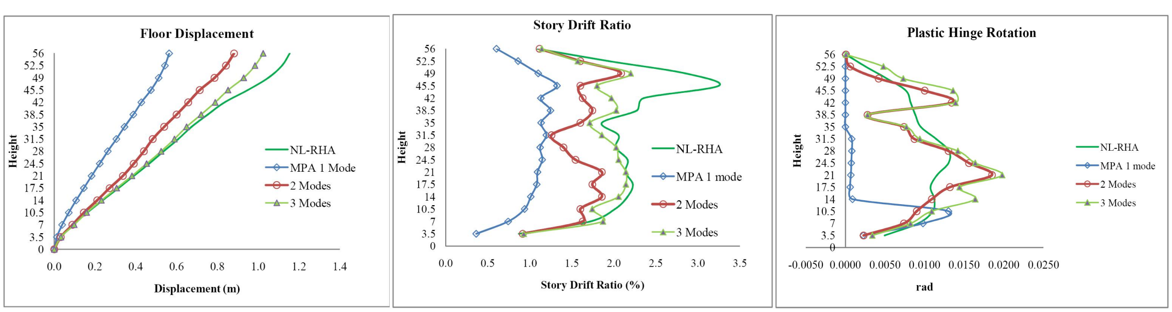

Figure 4 Tabas (a) Roof displacements (b) Story drifts (c) Hinge plastic rotations
Determined by MPA analysis with variable number of modes

6. COMPARATIVE EVALUATION OF EARTHQUAKE INDUCED DEMANDS BETWEEN FEMA AND MPA PROCEDURE

Comparative evaluation of the accuracy of MPA and pushover analysis using force distribution specified in FEMA-356 demonstrates that MPA is almost always more accurate in estimating story drifts than all of the FEMA force distribution , Chintapakdee and Chopra (2004) and Khoshnoudian and Mohammadi (2008). Based on structural dynamics theory, the MPA procedure retains the conceptual simplicity and computational attractiveness of the standard pushover procedures with invariant lateral force distribution over the height of structure, Chopra and Goel (2001). Because higher-mode pushover analyses are similar to the first-mode analysis, MPA is conceptually no more difficult than procedures now standard in structural engineering practice. Because pushover analyses for the first two or three modal force distributions are typically sufficient in MPA, it requires computational effort that is comparable to the FEMA-356 Without additional conceptual complexity or computational effort.

The ability of nonlinear static procedures to predict the maximum *roof displacement* caused by the ground motion is generally reliable and using MPA method emphasized this accuracy through the height of structure, especially at roof level.

Story drifts by MPA method, show more accurate and reliable answers in comparison to FEMA force distribution. Deficiency of FEMA evaluation, clearly show us importance of higher mode contributions for structural responses. None of the FEMA distributions leads to drifts that are close to the exact results from NL-RHA. “ELF” method reveals smaller value near the base level and greater value at the top stories. Contrary, The “Uniform distribution underestimate story responses (drift) at the top and overestimate them in bottom stories. Structural response by nonlinear case of “First mode shape” may be more accurate in accordance to NL-RHA.

Accuracy of nonlinear static procedures for determination of *Hinge plastic rotations* is not suitable. Both MPA and FEMA methods estimate these values in a limited level of satisfaction. Hinge rotations contain unacceptably large errors. MPA procedure gives estimates better than all the FEMA force distributions, but it is still inaccurate with errors reaching 125% in some cases.

7. CONCLUSION

The MPA procedure seems to produce results that are somewhat more reliable than those obtained from single load vectors in FEMA. However, it is readily apparent that the accuracy of these depends upon the parameter of interest (e.g., drift, plastic hinge rotation, …), the characteristics of the structure and the details of the specific procedure. It is also possible that future development of the basic MPA procedure may improve predictions further. On the other hand, MPA procedures are fundamentally limited, as are NSPs more generally. Form a broader perspective, it is important to develop practical versions of nonlinear dynamic response history analyses of detailed and, perhaps, simplified MDOF models, Khoshnoudian and Mohammadi (2008).

Previous studies show that MPA errors come from the following assumptions and approximations: (1) the coupling among modal coordinates $q_n(t)$ arising from yielding of the system is neglected; (2) the superposition of responses to $P_{eff,n}(t)$ (n=1,2…N) according to SRSS relation is strictly valid only for linearly elastic systems; and (3) the $F_{sn}/L_n - D_n$ relation is approximated by a bilinear curve to facilitate solution of Displacement Equation for SDF system. Although several approximation are inherent in this MPA procedure, when specialized for linearly elastic systems it is identical to the RHA procedure, Chopra and Goel (2001), Khoshnoudian and Mohammadi (2008).

Based on structural dynamics theory, the MPA procedure retains the conceptual simplicity and computational attractiveness of the standard pushover procedures with invariant lateral force distribution. Because higher-mode pushover analyses are similar to the first-mode analysis, MPA is conceptually no more difficult than procedures now standard in structural engineering practice. Because pushover analyses for the first two or three modal force distributions are typically sufficient in MPA, it requires computational effort that is comparable to the FEMA-356 Without additional conceptual complexity or computational effort; MPA estimates seismic demands much more accurately than FEMA-356 procedures, as demonstrated by a comparison of Figs. 4 and 5. However, MPA is an approximate method that cannot be expected to always provide seismic demand estimates close to the "exact" results from nonlinear RHA procedure. As a case in point, accuracy of MPA for plastic hinge rotation determination decreases especially for higher degree of excitation and these kinds of errors are not acceptable at practical structural dynamics.

The effects of mass irregularities, stiffness irregularities, and strength irregularities are evaluated for seismic demands. Vertical Mass irregularities have known to be in smaller degree of attention due to change at upper stories. Effects of vertical irregularities generally increased when irregularity conducted to base or lower stories. Strength irregularities based on case SM1 and SM (1-8) by modification factors of 2 seems to have more intense degradation on accuracy of MPA method. In addition, roof displacements are not very sensitive to the presence of either mass or stiffness irregularities, Krawinkler and Al-Ali (1998). In general, strength irregularities have much higher effects on the seismic response than mass or stiffness magnitudes. Cases with more intense correlation to structural irregularities have shown in Fig. 5.

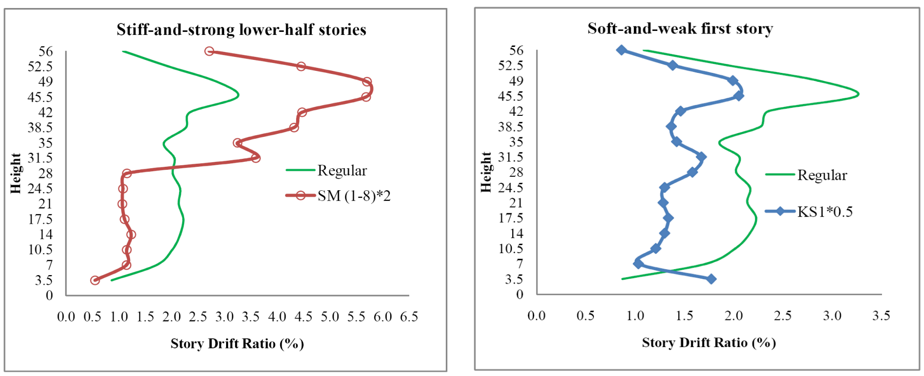

Figure 6 Story drift demands determined by nonlinear RHA for "regular" and "irregular" frame
(a) KS1*0.5, (b) SM (1-8)*2

As a final remark, we like to mention that the advantages of using MPA procedure for determination of structural response leads to computationally reliable answers in comparison to current non-linear procedures. Although, MPA preciseness decreases for high rise building and particularly first mode solution of MPA method is not acceptable for structures with higher mode contribution; but, simplicity and similarity of next modes calculation, make this method to be modified easier in higher levels of accuracy. Time saving, besides conceptual simplicity of MPA method makes it as reliable procedure for estimation of seismic performance at high rise steel structures.

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