Model And Cyclic Pushover Analysis For Seismic Over Performance Assessment Of Buckling Confined Braced Steel Frame

Mr.Mohammed Parvez Affani[1], Mrs.K. Vandana[2], Mrs. Pooja Satale[3]

1,2,3Assistant Professor
1,2,3 Department of Civil Engineering,
1,2,3 Global Institute of Engineering & Technology, Moinabad, Rangareddy Dist., Telangana State.

Abstract :
Buckling-constrained Braces (BRBs) are showed to have nearly the equal yielding stress and remaining strength below tension and compression. The BRBs can undergo fully-reversed axial yield cycles without lack of stiffness and electricity, whose seismic energy dissipation ability is superior. based on modal pushover analysis, the have an effect on of better vibration modes of Buckling-confined Braced steel frame turned into taken into consideration. as compared to non-linear static manner, the results of modal pushover evaluation agree better with that of nonlinear response history analyses. based totally on cyclic pushover evaluation, the hysteretic conduct of Buckling-confined Braced metallic frame (BRBSF) changed into researched. After installed with BRBs, the energy dissipation of BRBSF is finished by way of the hysteretic deformation of BRBs, the seismic responses of the structure may be ubstantially reduced and seismic overall performance could be advanced.
key phrases: Buckling-restricted Braces; Buckling-restrained Braced metal frame; modal pushover evaluation; cyclic pushover evaluation; seismic overall performance

1. INTODUCTION

Steel braces are used as an economic means of providing lateral stiffness to a steel structure. However, the energy dissipation capacity of a steel braced structure subjected to earthquake loads is limited due to the buckling of braces, which show unsymmetrical hysteretic behavior in tension and compression, and exhibit substantial strength deterioration when loaded monotonically in compression or cyclically. If buckling of a steel brace is restrained and the same strength is ensured both in tension and compression, the energy absorption of the brace will be markedly increased and the hysteretic property will be good. So the Buckling- Restrained Brace (BRB) is proposed. The capacity of resisting earthquake loads and energy dissipation of Buckling- Restrained Braced Frame (BRBF) is better than the frame with the installation of steel braces [1-3].

Recently, Modal Pushover Analysis (MPA) has been developed to improve conventional pushover procedures by including higher mode contributions to seismic demands [4]. This MPA procedure offers several attractive features. Developed herein is an improved pushover analysis procedure based on structural dynamics theory, which retains the conceptual simplicity and computational attractiveness of current procedures with invariant force distribution common in structural engineering practice. In this MPA, the seismic demand due to individual terms in the modal expansion of the effective earthquake forces is determined by a pushover analysis using the inertia force distribution for each mode. Combining these 'modal' demands due to the first two or three terms of the expansion provides an estimate of the total seismic demand on inelastic systems [5].

The accuracy of MPA have been evaluated for a wide range of structural systems and ground motions to identify the conditions under which it is applicable for seismic evaluation of structures. To this end, it has been applied to code- esigned buildings [6], and generic frames [7] designed according to the static force distribution specified in the International Building Code (IBC) [8]. By studying the bias and dispersion of this approximate procedure, MPA has been shown to be accurate enough in estimating seismic demands for the seismic evaluation of many buildings.

This Cyclic Pushover Analysis (CPA) procedure is that the structure is loaded horizontally and quasi-statically under force or displacement control, the loading history consisted of stepwise increasing force or displacement cycles. The hysteretic behavior and energy dissipation capacity of structures can be researched with CPA, to study whether which exhibit substantial strength and stiffness deterioration. The objectives of this investigation are as follows: (1) To study the seismic demands of Bechmark BRBF, to evaluate the accuracy of MPA in estimating seismic demands and document the bias and dispersion of the ratio of the seismic demands on BRBF determined by MPA procedure to their ''exact'' values computed by nonlinear Response History Analysis (RHA), and (2) To study whether the good hysteretic behavior and energy dissipation capacity of BRBs reflect on BRBF, the hysteretic behavior of BRBF is researched by CPA.

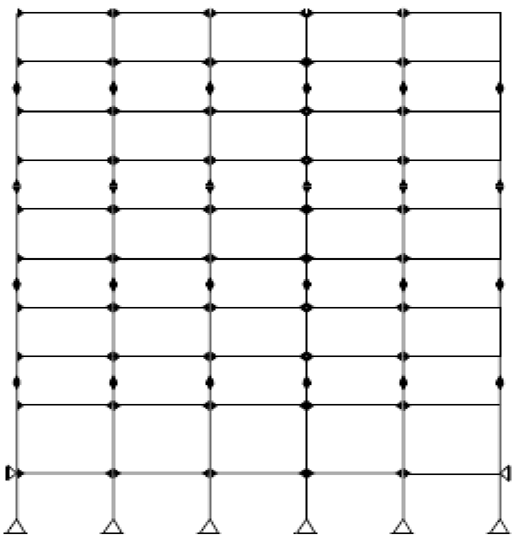
2. STRUCTURAL SYSTEM

The 9-story structure used for this benchmark study was designed for the SAC Phase III Steel Project. Although not actually constructed, the structure meets seismic code and represents a typical mid-rise building designed for the Los Angeles, California region. This building was chosen because it will also serve as a benchmark structure for SAC studies and thus will provide a wider basis for comparison of the results from the present study. The Los Angeles nine-story (LA 9-story) structure is 45.53 m by 45.73 m in plan, and 37.19 m in elevation. The bays are 9.15 m on center, in both directions, with five bays in the north-south (N-S) direction and five bays in the east-west (E-W) direction. The building's lateral load-resisting system is comprised of steel perimeter moment-resisting frames (MRFs). The columns are 345 Mpa steel. The columns of the MRF are wide-flange.

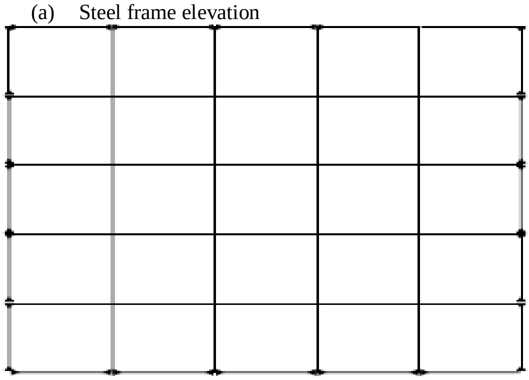
The levels of the 9-story building are numbered with respect to the first story, located at the ground (first) level (see Fig. 1). The 10th level is denoted the roof. The building has an additional one basement levels. The level directly below the ground level is the first basement (B-1). Typical floor-to-floor heights (for analysis purposes measured from center-of-beam to center-of-beam) are 3.65m. The floor-to-floor heights for the basement level are 3.65 m and for the first floor is 5.49 m .

The column lines employ three-tier construction, *i.e.* monolithic column pieces are connected every three levels beginning with the second story. The column bases are modeled as pinned (at the B-1 level) and secured to the ground. Concrete foundation walls and surrounding soil are assumed to restrain the structure at the first floor from horizontal displacement. In accordance with common practice, the floor system, which provides diaphragm action, is assumed to be rigid in the horizontal plane. The floor system is comprised of 248 Mpa steel wide-flange beams acting compositely with the floor slab. The inertial effects of each level are assumed to be carried evenly by the floor diaphragm to each perimeter MRF, hence each frame resists one half of the seismic mass associated with the entire structure. The seismic mass of the structure is due to various components of the structure, including the steel framing, floor slabs, ceiling/flooring, mechanical/electrical, partitions, roofing and a penthouse located on the roof.

The seismic mass of the first story is 1010t, for the second story to 8th level is 989t, and for the 9th story is 1070t. The seismic mass of the entire structure is 8041t. This benchmark study will focus on an in-plane (2-D) analysis of one-half of the entire structure. The frame being considered in this study is one of the N-S MRFs. The height to width ratio for the N-S frame is 0.82:1. The Bechmark MRF are installed with BRBs, the layout of BRBs in Bechmark MRF is depicted in Fig. 2. The BRBs are 235 Mpa steel, in ANSYS which are modeled by LINK8 element, columns and beams are modeled by BEAM189 element, and floor mass is modeled by MASS21 element. The elastic modulus after yielding is 1 percent of which before yielding for all steel. The finite element model in ANSYS of N-S BRBSF is also depicted in Fig. 2.



(a) Steel frame elevation



(b) Standard floor plan

NOTES

Beams (248 Mpa):
1st – 3rd level W36x160;
4th – 7th level W36x135;
8th level W30x99;
9th level W27x84;
Roof W24x68.

Columns (345 Mpa):
Column sizes change at splices
Corner columns and interior columns the same,
B1-1st level W14×500; 3rd W14×455;
5th W14×370; 7th W14×283;
9th W14×257

Restraints:
Columns pinned at base;
Structure laterally restrained at 1st level.

Splices:
Denoted with
are at 1.83 m (6 ft) w.r.t. beam-to-column joint

Connections:
◀▶ indicates a moment resisting connection.
—— indicates a pinned connection.

Figure1 9-story Bechmark MRF

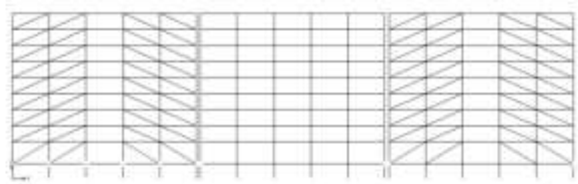


Figure 2 Finite element model of 9-storey BRBSF

3. CALIBRATION OF FINITE ELEMENT MODEL

The first five natural frequencies of Bechmark MRF are get by Finite Element (FE) analysis in ANSYS, the comparison of the results in ANSYS with the datas in papers of ASCE is listed table 1. The dispersion of the results is very little, so the final "corrected" FE model in ANSYS was refined and calibrated to match the identified structural natural periods. It was found that a FE model in ANSYS can be calibrated to give a good prediction of earthquake response.

Table 1 Comparison of natural frequency

Mode	1	2	3	4	5
Paper	0.443	1.18	2.05	3.09	4.27
ANSYS	0.435	1.14	1.99	3.06	4.18

4. MODAL PUSHOVER ANALYSIS

Estimating seismic demands requires explicit consideration of inelastic behavior of the structure. While non-linear response history analysis (RHA) is the most rigorous procedure to compute seismic demands, while RHA quire longer computational time and encounter convergent difficulties. Current civil engineering practice prefers to use the non-linear static procedure (NSP) or pushover analysis. The seismic demands are computed by non-linear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a predetermined target displacement is reached [9, 10]. Both the force distribution and target displacement are based on the assumption that the response is controlled by the fundamental mode and that the mode shape remains unchanged after the structure yields. Obviously, after the structure yields, both assumptions are approximate, satisfactory predictions of seismic demands are mostly restricted to low- and medium-rise structures provided the inelastic action is distributed throughout the height of the structure [11, 12]. The fact that MPA is able to estimate the response of buildings responding well into the inelastic range to a similar degree of accuracy indicates that this procedure is accurate enough for practical application in building retrofit and design. The MPA procedure was developed based on structural dynamics theory that includes the contribution of several modes of vibration [4]. This procedure was further refined and systematically evaluated [6] using six buildings, each analyzed for 20 ground motions. It was found that with sufficient number of "modes" included, the height-wise distribution of story drifts estimated by MPA is generally similar to trends noted from nonlinear response history analysis (RHA).
To estimate the seismic demands, the contribution of the first three 'modes' were included in analysis of the BRBSF. The combined values of roof displacements were computed including one, two, or three "modal" pairs. Combining these peak modal responses by an appropriate modal combination rule (e.g. SRSS, CQC rule) leads to the MPA procedure. Table 2 shows the floor displacements for the building by MPA and NSP together with the exact value determined by nonlinear RHA of the system. For the comparison purpose, four recorded ground acceleration time histories were used. These four earthquakes include four types filed seismic acceleration records, the peak values of acceleration are 620gal, 140gal, respectively.

Table 2 Comparison of results based on several methods

Types of Acceleration	Analysis Methods	Roof Displacement/m	Bias
Acceleration Records in Field I 620gal/140gal	RHA	0.1025/0.02275	
	NSP	0.0774/0.0175	24.4% / 23.1%
	MPA	0.0865/0.0195	15.6% / 14.3%
Acceleration Records in Field II 620gal/140gal	RHA	0.246/0.083	
	NSP	0.199/0.0608	19.1% / 26.1%
	MPA	0.2403/0.0558	2.3% / 32.7%
Acceleration Records in Field III 620gal/140gal	RHA	0.321/0.07324	
	NSP	0.2904/0.0545	9.5% / 25.5%
	MPA	0.3403/0.0776	6% / 6%
Acceleration Records in Field IV 140gal	RHA	0.083	
	NSP	0.1611	15.2%
	MPA	0.1779	6.3%

Table 2 shows the bias in these estimates relative to the exact response from non-linear RHA. The bias in the MPA results for two or three modes included are generally significantly smaller than in NSP only considering of the first one "mode". The first 'mode' alone is inadequate, especially in estimating the displacements. Significant improvement is achieved by including response contributions due to the second and third'mode'. The higher "modal" pairs contribute significantly to the seismic demands for the selected systems and MPA is able to capture these effects. With sufficient number of "modal" pairs included, the height-wise distribution of peak roof displacements estimated by MPA is generally similar to the "exact" results from nonlinear RHA, and much superior to the first "modal" pair result. However, because MPA is an approximate method, it does not match the "exact" demands determining by nonlinear RHA. Instead MPA has the goal of estimating seismic demands to a useful degree of accuracy for practical application with the advantage of much less effort than required for nonlinear RHA.

5. CYCLIC PUSHOVER ANALYSIS

One reversal load pattern is proposed for the pushover analysis of BRBSF, that is Cyclic Pushover Analysis (CPA). The reversal loading is considered as an earthquake event, and the hysteretic behavior indicate the seismic performance to resist the earthquake load and dissipate seismic capacity of BRBSF.
Based on the push-over analysis and reversal load pattern analysis for the structure, the hysteretic curve is depicted in Fig. 3, Fig.3 shows the BRBSF shows stable hysteretic behavior without pinching and strength, stiffness degradation and the energy dissipation is large. It is seen that the BRBs are able to develop the steel strength to its full capacity and show dramatic strain-hardening behavior, good hysteretic behavior and energy dissipation capacity of BRBs reflect on BRBSF. At the same time, the energy dissipation is completed by the hysteretic deformation of BRBs, the seismic responses of the structures will be greatly reduced and seismic performance will be improved.

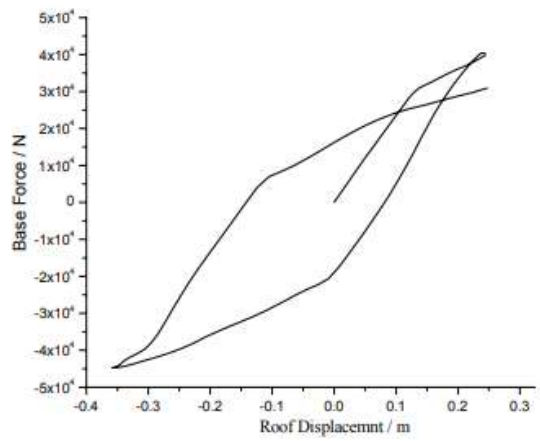


Figure 3 The Cyclic Pushover Analysis curve of BRBSF

6. CONCLUSIONS

It is an evident that the MPA and CPA procedure is an improved tool for estimating seismic demands on buildings. Using the MPA and CPA procedure, the structural behavior can be examined during seismic loading. RHA were computed to measure the bias and dispersion of MPA estimates, which indicate the dispersion values of the ratio of roof displacement demands determined by MPA considering of the first three "mode" is smaller than NSP only considering of the first one "mode". MPA provides adequate predictions of peak roof displacement of BRBSF when higher mode contributions are significant. It was also shown that NSP based on invariant load vectors cannot capture the changes to the dynamic modes resulting from inelastic action. CPA is a good method to evaluate the hysteretic behavior of a structure, the BRBSF shows stable good hysteretic behavior and energy dissipation capacity because of the installation of BRBs, the seismic performance of which is markedly improved simultaneously. 7. ACKNOWLEDGEMENTS The support of National Science

Foundation of China through projects (Grant No. 90715021, 50678057, 50108005), Nation `Eleventh Five` Technology Sustain Foundation and Heilongjiang Science Foundation (Grant No. ZJG0701) is greatly appreciated. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of these Foundation of China.

REFERENCES

1 Qiang Xie. (2005). State of the art of buckling-restrained braces in Asia. Journal of Constructional Steel Research 61:6, 727–748.

2 R. Sabelli, S. Mahin, C. Chang. (2003). Seismic demands on steel braced frame buildings with buckling-restrained Braces. Engineering Structures 25:5, 655–666.

3 Larry A. Fahnestock, James M. Ricles and Richard Sause. (2007). Experimental Evaluation of a Large-Scale Buckling-Restrained Braced Frame. Journal of structural engineering 133:9, 1205–1214.

4 Chopra, A. K., and Goel, R. K. (2002). A modal pushover analysis procedure for estimating seismic demands for buildings. Earthquake Engineering and Structural Dynamics 31:3, 561–582.

5 Chatpan Chintanapakdee1 and Anil K. (2004). Chopra. Seismic Response of Vertically Irregular Frames: Response History and Modal Pushover Analyses. Journal of Structural Engineering 130:8, 1177-1185.

6 Goel, R. K., and Chopra, A. K. (2004). Evaluation of modal and FEMA pushover analyses: SAC buildings. Earthquake Spectra 20:1, 225–254.

7 Chintanapakdee, C., and Chopra, A. K. (2002). Evaluation of modal pushover analysis using generic frames. Engineering and Structural Dynamics 32:3, 417–442.

8 International Code Council. (2000). International Building Code, Falls Church, Va.

9 Erol Kalkan, Sashi K. Kunnath. (2007). Assessment of current nonlinear static procedures for seismic evaluation of buildings. Engineering Structures 29:3, 305-316.

10 He Liu, Feifei Bai, Jesse L. Gobeli. (2006). FEA Modeling and Modal Pushover Analysis of a 14-story Office Building in Anchorace, Alaska. Structures, Redistribution subject to ASCE license or copyright; see http://www.ascelibrary.org/

11 Gupta A, Krawinkler H. (1999). Seismic demands for performance evaluation of steel moment resisting frame structures (SAC Task 5.4.3).