International Journal of Civil & Structural Engineering – IJCSE
Volume 1 : Issue 1 *[ISSN 2372 – 3971]*
Publication Date : 09 January 2014

Evaluation of Overstrength Factor of Steel Moment Resisting Frames

Afshin Mellati, Mohammad Hadikhan Tehrani,
Yasha Haji Zeinali and Mehdi Banazadeh
Department of Civil Engineering
Amirkabir University of Technology
Tehran, Iran
afshin17886@aut.ac.ir

Farzin Paytam
Research Team Support Departement
Ashiansaz Farapay Construction Company
Shiraz, Iran
Paytam@ut.ac.ir

***Abstract*— Using inappropriate system overstrength factors may lead to weak seismic performance or uneconomic results. Regulations suggest the suitable system overstrength factors for various seismic force-resisting systems. This is while a lot of parameters might affect on this factor. In this paper, the effects of various parameters such as the type of the moment frame, Seismic Design Category (SDC) and height of the building on overstrength factors of steel Moment Resisting Frames (MRF) are investigated. Hence, 12 buildings with perimeter steel MRF are designed three-dimensionally based on ASCE/SEI 7-10, AISC-LRFD 360-05 and AISC 341-05 regulations. Then, 24 two-dimensional MRFs of these buildings are simulated in OpenSees software and by conducting the Nonlinear Static Analysis (NSA); pushover curves of these structures are obtained. Finally, having these curves, relevant overstrength factors are gained. The results demonstrate that although altering the mentioned parameters, changes the overstrength factors, but the regulation criteria are achieved well.**

***Keywords*— Nonlinear Static Analysis, Overstrength Factor, Pushover Curves, Steel Moment Resisting Frames**

I. Introduction

Generally, in a seismic force resisting system, while ductile elements experience inelastic deformation, force-controlled elements designed to remain elastic, sustain substantial seismic forces. Considering this effect, seismic codes use system overstrength factor to obtain realistic seismic forces in force-controlled elements through simplified elastic design seismic forces. Having the pushover curve for a structure, overstrength factor, Ω, is defined as the ratio of the maximum base shear resistance, Vy, to the design base shear, Vs (Fig. 1).

$$\Omega = Vy/Vs \qquad (1)$$

Many studies are performed on overstrength factors of various structures [1], [2]. In practical designs, system overstrength factor, Ω0, is used for a specific seismic force resisting system. The ASCE/SEI 7-10 standard [3], implements the system overstrength factor of 3 for all kind of steel MRFs. This is while, a lot of parameters affect on this factor. Therefore, in this study, using nonlinear static pushover

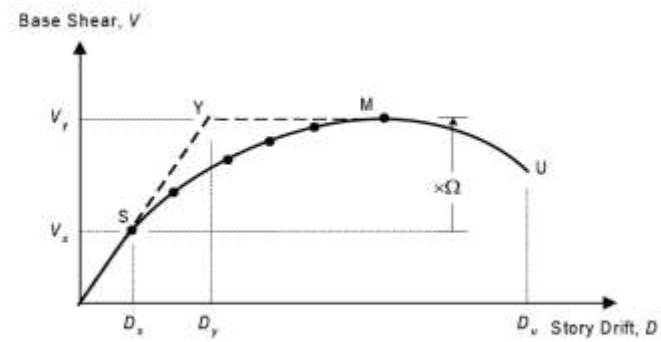

Figure 1. Pushover curve

analysis and application of FEMA P695 [4], the effects of height and different SDCs on overstrength factor of all kind of steel MRFs are assessed.

II. DESIGN AND ASSUMPTIONS

Utilizing ASCE/SEI 7-10 [3], AISC-LRFD 360-05 [5] and AISC 341-05 [6] regulations, 12 buildings are designed three-dimensionally. The design procedure is conducted based on simplified equivalent lateral forces method. The samples cover the design space with variation in SDCs (B and D), type of MRFs (ordinary, intermediate and special) and number of stories (4, 8, 12). The perimeter MRFs are applied as the seismic force-resisting system. The perimeter frames in the X directions consist of 2 bays and in Y directions, consist of 3 bays. The plan view of all buildings is the same and is depicted in Fig 2. All the buildings are located on hard soil with type of D. the other assumptions are:

- The height of the first story is 4m and the height of the others is 3.2m.
- The dead load is 5.88 KN/m2 over each floor and the live load is 1.92 KN/m2 on all floors and 0.96 KN/m2 on the roof.
- San Diego is chosen for SDC D with seismic parameters of SDS = 0.83g and SD1 = 0.48g and Denver is selected for SDC B with seismic parameters of SDS = 0.2g and SD1 = 0.097g.

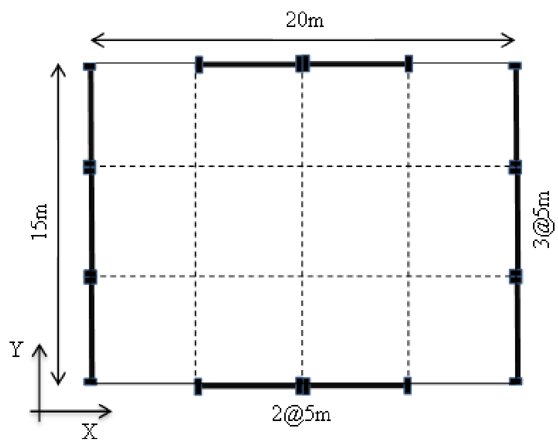

Figure 2. Plan view of the buildings

- ASTM A36 steel is used for design. Box sections are used for column sections and beam sections are chosen from European sections (IPE).

Table 1 shows the properties of the steel MRFs used in this study.

TABLE I. STEEL MOMENT RESISTING FRAMES PROPERTIES

Model No.	Type of MRF	No. of Stories	Direction	SDC	Fundamental Period T1 (sec)
1	SMF	4	Y	D	1.46
2	SMF	4	X	D	1.35
3	SMF	4	Y	B	3.32
4	SMF	4	X	B	3.18
5	IMF	4	Y	B	2.79
6	IMF	4	X	B	2.64
7	OMF	4	Y	B	2.69
8	OMF	4	X	B	2.44
9	SMF	8	Y	D	2.11
10	SMF	8	X	D	1.93
11	SMF	8	Y	B	3.59
12	SMF	8	X	B	3.64
13	IMF	8	Y	B	3.64
14	IMF	8	X	B	3.38
15	OMF	8	Y	B	3.79
16	OMF	8	X	B	3.77
17	SMF	12	Y	D	2.67
18	SMF	12	X	D	2.59
19	SMF	12	Y	B	4.46
20	SMF	12	X	B	4.26
21	IMF	12	Y	B	4.45
22	IMF	12	X	B	4.27
23	OMF	12	Y	B	4.90
24	OMF	12	X	B	4.78

III. STRUCTURAL MODELING

Each frame is modeled in OpenSees software for conducting the nonlinear static pushover analysis. Components are modeled by the elastic elements and the nonlinear behavior of them is assigned by plastic hinges located at the end regions of the elements. Fig. 3 shows the model for two-story and one-bay frame. As it is seen in this figure the P-Delta effects of interior frames are considered by means of leaning column connected to the MRF through truss elements. The leaning column is loaded at each level with load combination of 1.05D+1.5L related gravity loads of half of the structure.

The Panel zone is modeled utilizing 8 rigid elements connected to hinges at three corners (Fig. 4). Two rotational springs are added at fourth corner representing the shear force – shear distortion behavior using trilinear model [7].

The modified Ibara-Krawinkeler monotonic backbone curve is used for modeling nonlinear behavior of beam and column hinges (Fig. 5). The beams backbone curves parameters are in accordance with ATC-72 [8] for non-RBS connections.

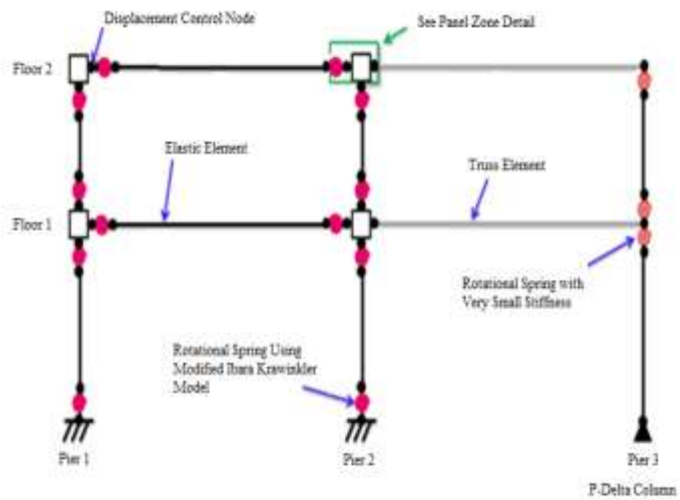

Figure 3. Structural model of a two-story, one-bay frame

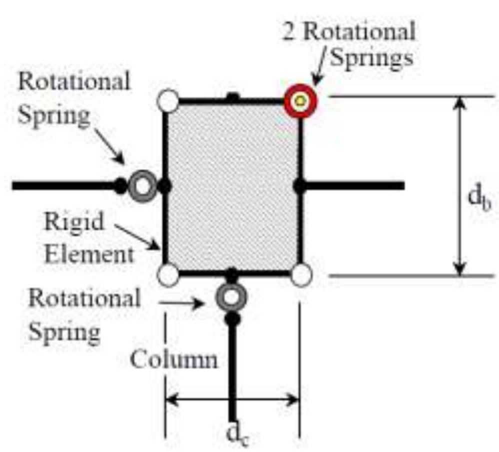

Figure 4. Panel zone detail [7]

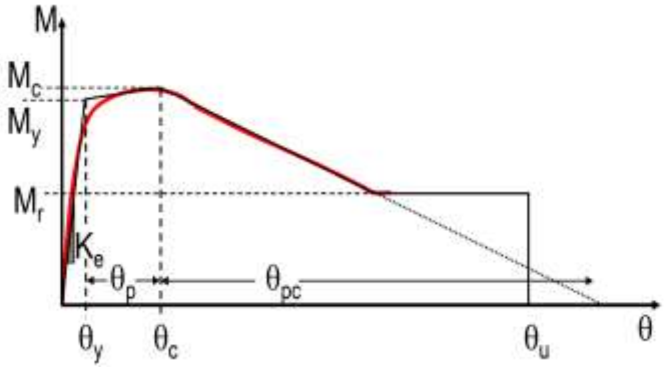

Figure 5. Parameters of the monotonic backbone curves [9]

The columns backbone curves parameters are based on the equations derived by Lignos [10]. The effect of composite slab is not considered due to lack of information of available to model this effect with accuracy [9].

IV. Nonlinear Static Pushover Analysis and Results

A. *Pushover Curves*

Nonlinear static pushover analyses are performed with OpenSees program according to section 6.3 of FEMA P695 [4]. Fig. 6-11 map the pushover curves for MRFs located in SDC B.

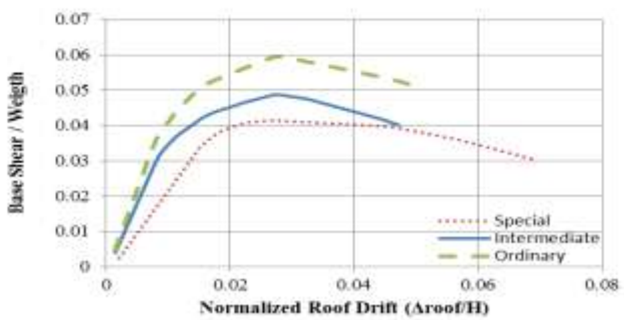

Figure 6. Pushover curves of 4-story frames - X direction

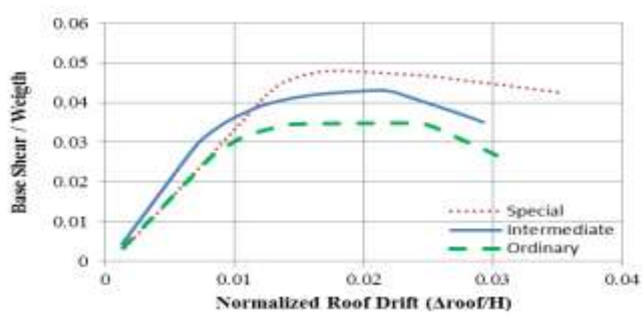

Figure 7. Pushover curves of 8-story frames - X direction

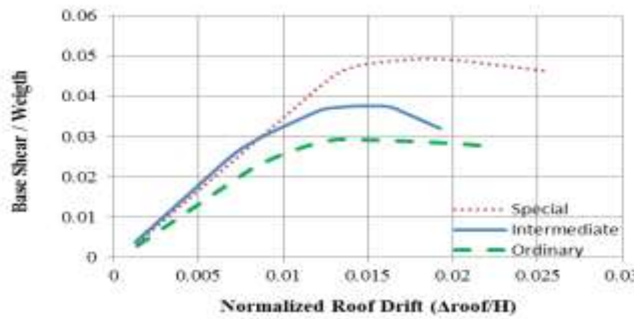

Figure 8. Pushover curves of 12-story frames - X direction

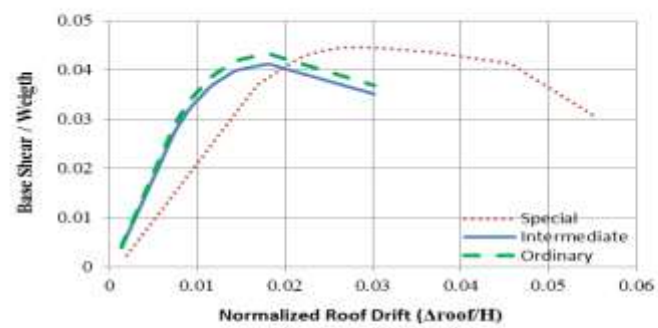

Figure 9. Pushover curves of 4-story frames - Y direction

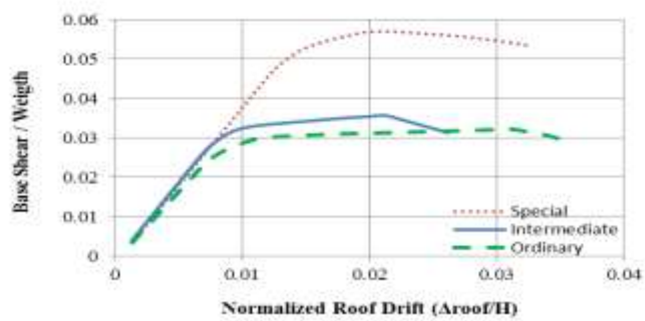

Figure 10. Pushover curves of 8-story frames - Y direction

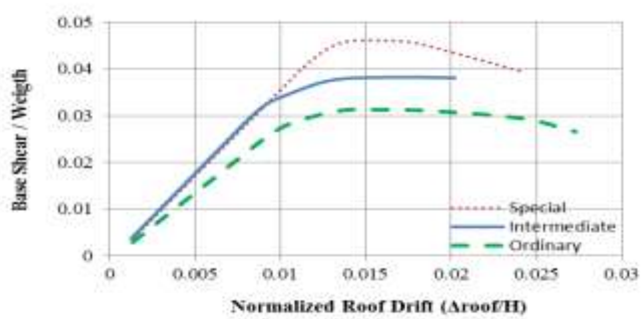

Figure 11. Pushover curves of 12-story frames - Y direction

As it is seen in these figures, generally, in low-rise MRFs with low bays, OMFs show higher resistant than IMFs and SMFs (Fig. 6). As the height of the buildings increases, this order would be reversed and this is the SMF that shows the highest resistant. This is due to design criteria. In low-rise MRFs, most member sizes are controlled by strength requirements directly related to response modification factor (R). In accordance with [3], design base shear coefficient (Cs) and R-factor have an inverse relationship. SMF, IMF and OMF have an R-factor of 8, 4.5 and 3.5 respectively [3]. Hence, in low-rise buildings with low bays, OMF, IMF and SMF have the highest design base shears respectively and have the greater strength in the same order subsequently. By increasing the building height, most member sizes are controlled by stiffness requirements (drift limitations and P-Delta conditions) and SMF, IMF and OMF have the highest resistant respectively.

Increasing the number of bays, SMFs could show more resistant than OMFs and IMFs in low-rise buildings (Fig. 9). In SMFs, sizes of interior columns are usually controlled by beam sizes because of strong column – weak beam criterion. Therefore, interior column sizes of SMFs are usually larger than interior column sizes of IMFs and OMFs. Augmenting the number of bays, increases the number of interior columns too. Thus it is possible for a low-rise SMF that has greater resistant than the other types of MRFs.

B. *Overstrength Factors*

Having the normalized design base shear and normalized maximum base shear, it is possible to calculate overstrength factors using (1). Table II demonstrates these factors for the MRFs.

TABLE II. STEEL MOMENT RESISTING FRAMES OVERSTRENGTH FACTORS

Model No.	Type of MRF	No. of Stories	Dir.	SDC	$C_s = V_{design} / W$	Vmax / W	Ω
1	SMF	4	Y	D	0.041	0.250	6.07
2	SMF	4	X	D	0.045	0.228	5.07
3	SMF	4	Y	B	0.01	0.044	4.40
4	SMF	4	X	B	0.01	0.041	4.13
5	IMF	4	Y	B	0.01	0.041	4.11
6	IMF	4	X	B	0.01	0.048	4.80
7	OMF	4	Y	B	0.01	0.043	4.30
8	OMF	4	X	B	0.01	0.059	5.90
9	SMF	8	Y	D	0.036	0.187	5.19
10	SMF	8	X	D	0.036	0.186	5.17
11	SMF	8	Y	B	0.01	0.057	5.70
12	SMF	8	X	B	0.01	0.048	4.80
13	IMF	8	Y	B	0.01	0.036	3.57
14	IMF	8	X	B	0.01	0.043	4.30
15	OMF	8	Y	B	0.01	0.032	3.20
16	OMF	8	X	B	0.01	0.034	3.40
17	SMF	12	Y	D	0.036	0.156	4.33
18	SMF	12	X	D	0.036	0.176	4.89
19	SMF	12	Y	B	0.01	0.046	4.60
20	SMF	12	X	B	0.01	0.049	4.90
21	IMF	12	Y	B	0.01	0.038	3.82
22	IMF	12	X	B	0.01	0.037	3.70
23	OMF	12	Y	B	0.01	0.031	3.10
24	OMF	12	X	B	0.01	0.029	2.90

The ASCE/SEI 7-10 standard [3], truncates the value of Cs. This value for all the MRFs located on SDC B studied in this paper is limited to lower bound. Hence, the overstrength factors of these MRFs follow the same order of their resistance as mentioned in section IV-A.

As it is seen in Table II, overstrengh factors are in the range of 2.9 to 6.07 with the average of 4.43. By comparison of these factors with system overstrength factor suggested by ASCE/SEI 7-10 ($\Omega 0 = 3$) [3], it is understood that except an OMF (model No. = 24), the rest of MRFs have higher overstrength factors. Therefore, the regulations criterion is gained well.

The overstrength factors do not follow a regular pattern. For OMFs and IMFs, this factor usually decreases by increasing the number of stories. Irregularity is higher in SMFs. The reasons for irregularity are numerous including different design criteria. Low-rise frame members are controlled by strength criterion while taller frames are designed in accordance with drift criterion. In low-seismic areas, lower members are controlled based on P-Delta effect. These different criteria play an important role in seismic performance of the structures.

Acknowledgment

The authors would like to acknowledge the Ashiansaz Farapay construction company for substantial contribution to this research and publication.

References

[1] T. Balenra, “Overstrength and ductility factors for steel frames designed according to BS 5950,” Structural Engineering. J., ASCE, vol. 129, no. 8, pp. 1019-1035, Aug. 2003.

[2] J. Kim and H. Choi, “Response modification factors of chevron-braced frames,” Engineering Structures. J., vol. 27, pp. 285-300, Dec. 2004.

[3] ASCE/SEI 7-10, “Minimum design loads for buildings and other structures,” American Society of Civil Engineering, Reston, Virginia, 2010.

[4] FEMA P695, “Quantification of building seismic performance factors,” Federal Emergency Management Agency, Washington, D.C., 2009.

[5] ANSI/AISC 360-05, “Specification for structural steel buildings,” American Institute of Steel Construction, Chicago, Illinois, 2005.

[6] ANSI/AISC 341-05, “Seismic provisions for structural steel buildings,” American Institute of Steel Construction, Chicago, Illinois, 2005.

[7] A. Gupta, H. Krawinkler, “Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures,” John A. Blume Earthquake Engrg. Ctr. Rep. No. 132, Dept. of Civ. And Envir. Engrg., Stanford University, Stanford, Calif, pp. 49-51, 1999.

[8] PEER/ATC-72-1, “Modeling and acceptance criteria for seismic design and analysis of tall buildings,” Applied Technology Council, Redwood City, C.A., 2005.

[9] F. Zareian, D.G. Lignos and H. Krawinkler, “Evaluation of seismic collapse performance of steel special moment resisting frames using FEMA P695 (ATC-63) methodology,” in Proc. Structural Congress, Structural Engineering Institute of the ASCE, Orlando, 2010, pp. 1275-1286.

[10] D.G. Lignos, “Sideway Collapse Of Deteriorating Structural Systems Under Seismic Excitations,” PHD Dissertation, Dept. of Civ. And Envir. Engrg., Stanford University, Stanford, Calif, pp. 131-140, 2008.