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Evaluation of Behavior Factor for Steel Moment-Resisting Frames

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Abstract**—**According to current seismic codes the structures are calculated using the capacity design procedure based on the concept of shear at the base depending on several parameters including behavior factor which is considered to be the most important parameter. The behavior factor allows designing the structure when it is at its ultimate limit state taking into account its energy dissipation through its plastic deformation. The aim of the present study is to assess the basic parameters on which is composed the behavior factor among them the reduction factor due to ductility, and those due to redundancy and the overstrength for steel moment-resisting frames of different heights and regular configuration. Analyses are conducted on these frames using the nonlinear static method where the effect of some parameters on the behavior factor, such as the number of stories and the number of spans, are taken into account. The results show that the behavior factor is rather sensitive to the variation of the number of stories and bays.

Keywords**—**Behavior, code, frame, ductility, overstrength, redundancy, plastic.

I. Introduction

THE main difficulty affecting the calculation of the temporal response of a building structure subject to seismic action represented by an accelerogram resides in its behavior beyond its elastic limit. The nonlinear dynamic analysis may seem the most complete method for calculating the response of a structure to deterministic excitement, but it requires firstly relatively heavy computation means and secondly it overcomes with difficulty overcomes the particular character of choosing a accelerogram. Consequently, modern seismic design codes recommend the use of the elastic response spectrum divided by an appropriate reduction factor taking into account the post-elastic behavior of the structure. In standards, this factor is commonly called "behavior factor," noted "R" in most codes [1], [2] and "q" in Eurocode 8 [3]. But, this approach is not totally safe because, under certain conditions, it can lead to behavior factors that are not fully adapted to the real behavior of the dissipative structure [4]-[6]. According to some researches [4], this phenomenon can be explained by taking into account of negligence of some forms of local ductility mainly at the base of columns of tall frames which are subjected in this case to very high compressive forces. This neglect can be dangerous even for steel structures which are considered a priori as highly dissipative.

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II. Notion Relating to the Behavior Factor in the Codes

In practice, the behavior factor is generally attributed so as inclusive in terms of materials and types of structures. The concept of the behavior factor was introduced for the first time by American Codes [7] and developed thereafter [8]-[10]. An appropriate definition of the behavior factor is based on a ductility-dependent component, R_μ, and a strength-dependent component, R_s. The strength factor, R_s, is the product of two factors: the redundancy factor R_ρ, and the overstrength factor, R_Ω. The ductility factor R_μ is deduced from the inelastic response spectrum; the redundancy factor, R_ρ, is intended to reflect the effects of the redundancy of the structure; and the resistance factor, R_Ω, represents a part of the capacity of global overstrength that holds the structure and depends on a number of sources such as design method, the nature of the earthquake and local construction practices [4]. Other studies have introduced another factor that takes into account the effect of viscous damping, noted R_ξ, [11], [12]. For comparison, the behavior factor, denoted q, in Eurocode 8 [13] is set, in the case of steel structures, by the product of the three factors mentioned above which are ductility factor, redundancy factor and overstrength factor. For steel structures, the value of the behavior factor depends on the ductility class of structure (DCL "low ductility," DCM "average ductility" and DCH "high ductility"). These values are indicated in Table I, given below. The ratio α_u/α_1 represents the ultimate plastic strength that divides the resistance corresponding to the formation of the first plastication of the structure and symbolizes the redundancy of the structure. This ratio can be estimated from capacity curve but its value should not exceed 1.6. Eurocode 8 also stipulates that in the absence of a detailed assessment, the ratio α_u/α_1 may take the following values: 1.1 for simple frame, 1.2 for multi-storey frame with single bay, and 1.3 for multi-storey frame with multi-bays. Unlike Americans and European codes, in the Algerian seismic code [14] the values of the behavior factor are generally given depending on the lateral load resistance system of the building structure with a single class of ductility and redundancy introduced implicitly through a factor called "quality factor" in which the regularity criterion is also considered but where overstrength is ignored. Furthermore, the components of behavior factor have been widely studied in the literature including the component, R_μ, has received the largest part [15]-[19].

According to Eurocode 8, regular structures located in areas of low seismicity, a behavior factor whose value is between 1.5 and 2.0 can be adopted without pass by the application of capacity design methods and without neglecting the presence of the effect of the ductility and overstrength.

TABLE I
BEHAVIOR FACTORS FOR STEEL FRAMES ACCORDING TO EC8

Type of structure	Classed	q
non-dissipative	DCL	1.5 to 2.0
Frame	DCM	4.0
	DCH	$5\alpha_u/\alpha_1$

III. EVALUATION OF THE BEHAVIOR FACTOR

Several methods have been proposed for determining the behavior factor [20]-[28]. These methods have been widely recalled in past and recent work [4], [28]. In the case of this present study, the evaluation of the behavior factor is made by the use of the nonlinear static method (MSNL) on steel structures including ductility, redundancy and overstrength factors. By this method the overall behavior of the structure is described by the capacity response curve obtained under the effect of an increasing monotonic lateral loading. Loading, obtained from the provisions of the code, is inverted triangular shape with the shape of the first vibration mode. The capacity response curve is then idealized by bilinear curve of type perfect elastic-plastic. These two curves are illustrated in Fig. 1.

From Fig. 1, the behavior factor can be written as [4]:

$$R= V_e/V_d = (V_e /V_u).(V_u/V_y).(V_y/V_d) = R_\mu.R_\rho.R_\Omega$$

where V_e, V_u, V_i and V_d indicate respectively the maximum yield strength, ultimate plastic strength, yield strength and design strength calculated from the code. A comparison with Eurocode 8 leads to:

$$R = q = q_o.\ \alpha_u/\alpha_1 \text{ with: } q_o = R_\rho.R_\Omega \text{ and } R_\rho = \alpha_u/\alpha_1$$

where; q_o is the basic value of the behavior factor.

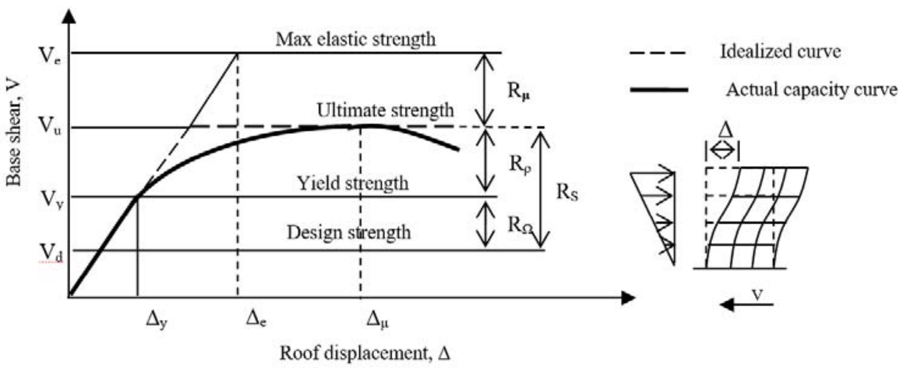


Fig. 1 Relationship between base shear and roof displacement

TABLE II
CHARACTERISTICS OF STEEL MOMENT FRAMES (WITH 3 AND WITH 6 BAYS)

Frame with:	**Column: HEA** (N° of storey)	**Beam: IPE** (N° of storey)
3 stories	240(1 to 3)	330(1 to 3)
6 stories	280(1 to 4) + 260(5 to 6)	360(1 to 4) + 330(5 to 6)
9 stories	340(1 to 5) + 320(6 to 7) + 300(8 to 9)	400(1 to 4) + 360(5 to 7) + 330(8 to 9)
12 stories	400(1 to 5) + 360(6 to 7) + 340(8 to 12)	450(1 to 4) + 400(5 to 7) + 360(8 to 10) + 330(11 to 12)

IV. NONLINEAR STATIC ANALYSIS

A. Choice of Structures

Height structures are assessed in this study. They are designed according to the requirements of Eurocode 8 and Eurocode 3 [29]. The seismic design load is defined on the basis of a soil type A, damping coefficient ξ = 5%, with a maximum ground acceleration γ = 0.25g and behavior factor q = 4.0. The inter-storey height for the 1st floor is 3.5 m and 3.0 m for the upper floors. Each bay has a length of 5.0 m. The steel frames, of S275JR quality, have a limit elastic resistance estimated to 275 MPa. It will be assumed that the mass can be concentrated only to the levels of the stories. The characteristics of each element of the frames and their general geometry are shown respectively in Table II and Fig. 2. Only frames with three bays are shown in Fig. 2.

B. Nonlinear Static Analysis

The four structures are subjected to horizontal loads distributed along their height according to the shape of the first vibration mode whose intensity increases incrementally until a mechanism is formed, while the gravity loads remain constant. It is performed using the SAP2000 software/version 14 [30] and his aim is to construct the push-over curve for the capacity response curve of each structure which will be deducted the values of ductility, redundancy, overstrength factors and finally those of the behavior factor. In this manner we can have, on the basis of comparisons, a general idea about the evolution of the latter according to the number of stories and the number of bays. In total eight frames of 3, 6, 9 and 12 stories with 3 and 6 bays each are considered.

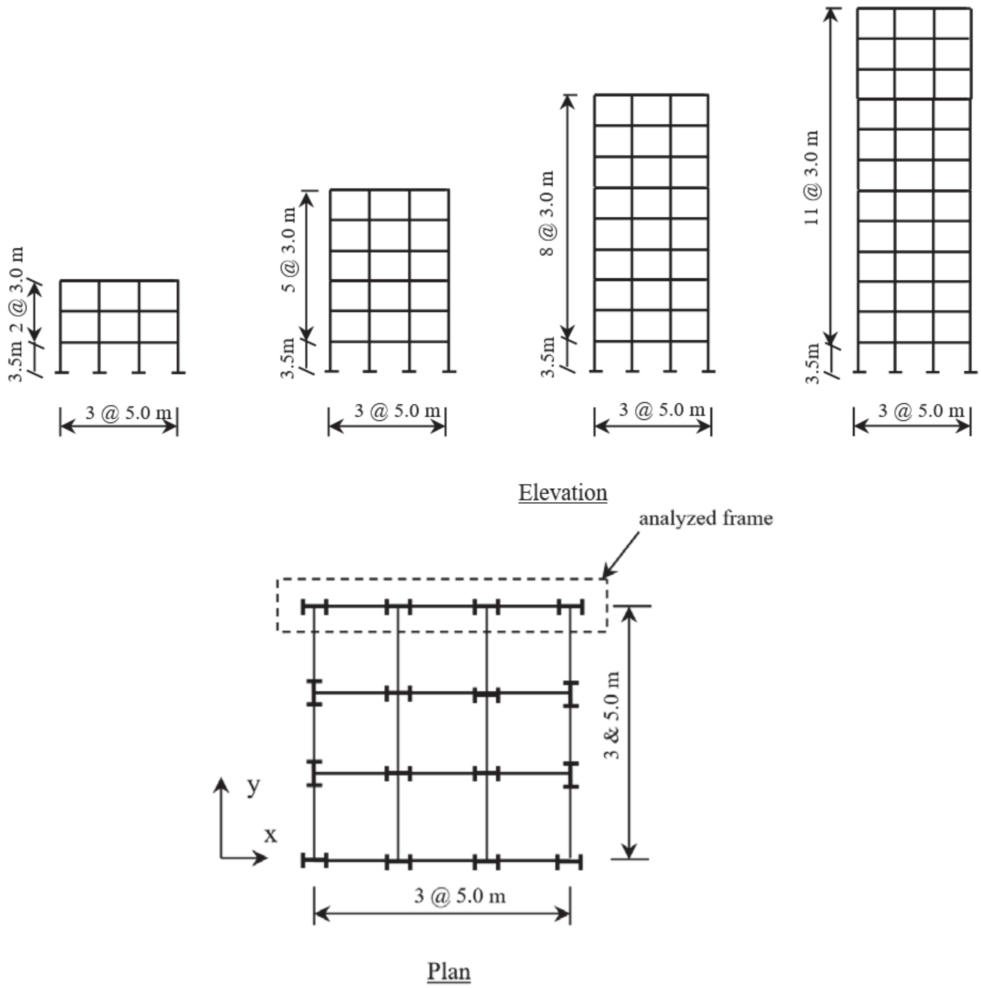


Fig. 2 Schematic representation of the frames

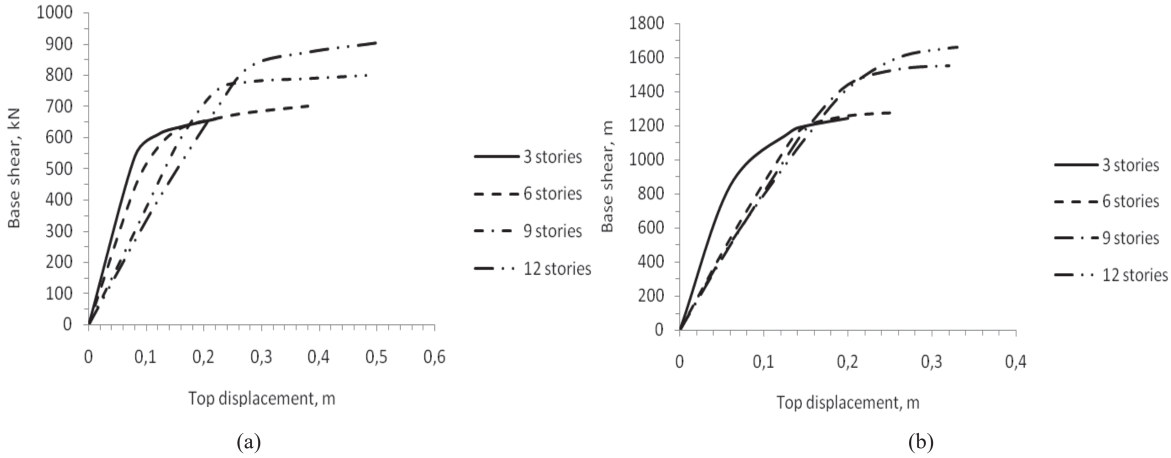


Fig. 3 Pushover curves of analyzed frames: (a) 3 bays, (b) 6 bays

V. Analysis Results

The complete relations between the base shear and the roof displacement of the eight analyzed steel moment-resisting frames are shown in Fig. 3. The corresponding values plasticity to different factors are given in Tables III and IV. The histograms of Fig. 4 show the evolution of different resistance factors and behavior factor according to the number of stories and bays. From these results, it emerges the following conclusions:

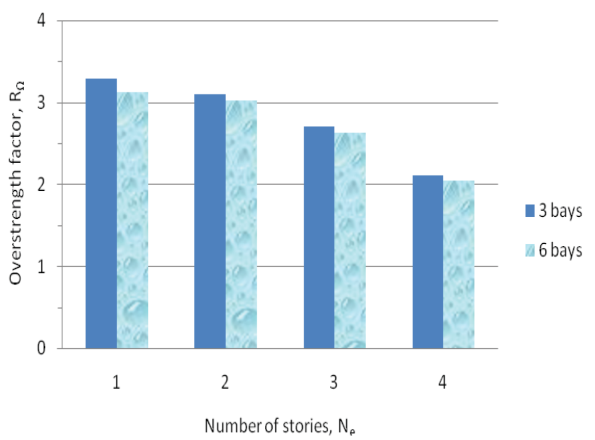
- Overstrength, ductility, redundancy and behavior factors show a reduction when the number of stories increases. Same observation also, but less important when the number of bay increases.
- The average value of R_ρ related to the redundancy factor is 1.46 and 1.45 for frames with 3 and 6 bays respectively. These values are slightly higher than that provided by EC8: $\alpha_u/\alpha_1 = 1.3$.
- Values of behavior factors, R, obtained for the frames of 3, 6 and 9 stories and with 3 bays and those of the frames of 3 and 6 stories with 6 bays are greater than those of EC8. By against, for frames of 12 stories, with 3 bays and the frames of 9 and 12 stories, with 6 bays, are they are lower than that of the EC8.

TABLE III
Values of Various Parameters for the Pushover Curve

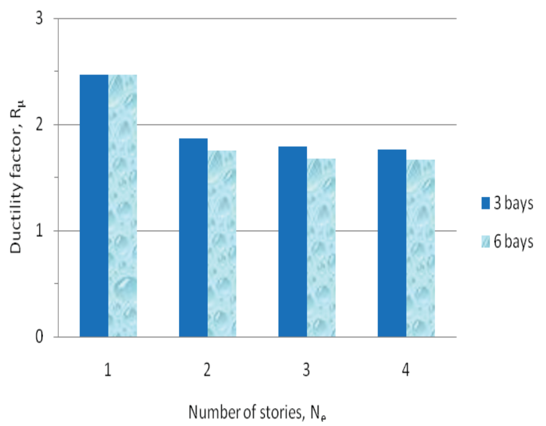
	Frames with 3 bays					Frames with 6 bays				
Number of stories	**V_u (kN)**	**V_y (kN)**	**V_d (kN)**	**Δ_{max} (m)**	**Δ_y (m)**	**V_u (kN)**	**V_y (kN)**	**V_d (kN)**	**Δ_{max} (m)**	**Δ_y (m)**
3	627.50	407.40	124.01	0.160	0.060	1184.07	755.10	241.0	0.160	0.060
6	658.81	456.45	147.10	0.196	0.105	1242.07	877.68	290.89	0.182	0.104
9	781.89	557.62	205.70	0.258	0.144	1495.50	1081.36	410.75	0.239	0.142
12	836.76	574.83	272.84	0.297	0.169	1597.44	1118.20	545.46	0.276	0.164

TABLE IV
Values of Different Resistance and Behavior Factors

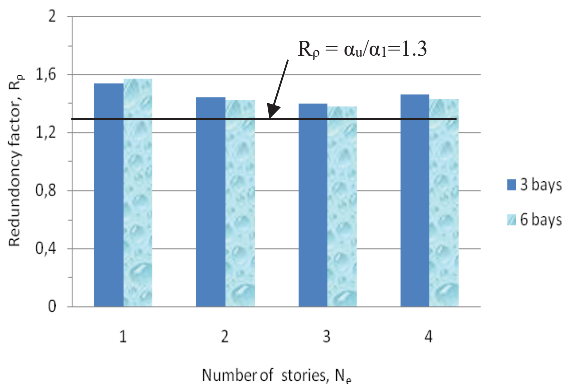
Number of stories	**Frames with 3 bays**				**Frames with 6 bays**			
	R_μ	**R_ρ**	**R_Ω**	**R**	**R_μ**	**R_ρ**	**R_Ω**	**R**
3	2.47	1.54	3.29	12.51	2.47	1.57	3.13	12.14
6	1.87	1.44	3.10	8.35	1.75	1.42	3.02	7.51
9	1.79	1.40	2.71	6,79	1.68	1.38	2.63	6.10
12	1.76	1.46	2.11	5,42	1.67	1.43	2.05	4.89



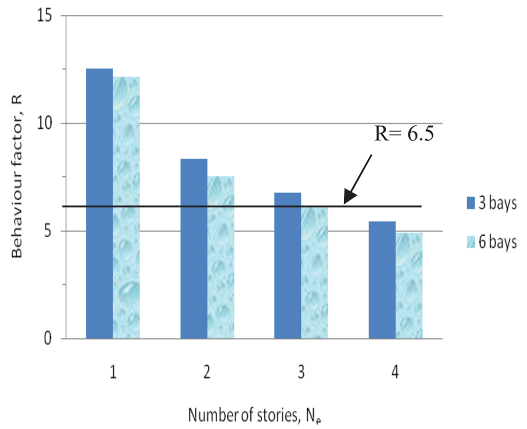
(a)



(b)



(c)



(d)

Fig. 4 Relationship between "resistance factors" and "number of stories and bays

VI. Conclusions

We can conclude that in the case of the structures studied, the use of the nonlinear static method gives a general idea of nonlinear behavior. It should be underlined that the behavior factor and its resistant components decrease when the number of stories and bays increase. Even finding for the behavior factor. However, an average of 6.5 is obtained for the behavior factor.

References

[1] Uniform building code, vol.2, Structural Engineering Design provisions. Whittier, CA, 1997.
[2] Règlement parasismique Algérien, RPA99/version 2003, CGS, 2003.
[3] Eurocode 8, Design of structures for earthquake resistance, Part 1: general rules, seismic actions and rules for buildings. European Committe for standardization, CEN, EN 1993, 2005.
[4] Ferraioli M., Lavino A. and Mandara A., Behaviour factor of code-designed steel moment-resisting frames, International journal of steel structures, vol. 14, N° 2, pp. 243-254, 2014.
[5] Elghazouli, Y., Assesment of European seismic design procedures for steel framed structures, Bulletin of earthquake engineering, 8, pp. 65-89, 2010.
[6] Ferraioli, M., Avossa, A. M., Progessive collapse of seismic resistant multistory frame buildings, Proc. 3rd International Symposium on Life-Cycle Civil Engineering, IALCCE, pp. 2048-2055, 2012.
[7] ATC3-06, Tentative provisions for the development of seismic regulations for buildings. Applied Technology Council, Redwood City, CA, 1978.
[8] ATC-19, Structural response modification factors, Applied Technology Council, Redwood City, CA, pp. 5-32, 1995.
[9] ATC-34, A critical review of current approaches to earthquake-resistant design, Applied Technology Council, Redwood City, CA, 1995.
[10] ATC-63, Quantification of building seismic performance factors, Applied Technology Council, Redwood City, CA, pp. 6-31, 2008.
[11] Uang, C.-M., Bertero, V. V. Earthquake simulation tests and associated studies of a 0.3-scale model of a six-story concentrically braced steel structure. Rep. No. UCB/EERC-86/10, University of California, Berkeley, Calif., 1986.
[12] Whittaker, A. S., Uang, C.-M., and Bertero, V. V., Earthquake simulation tests and associated studies of a 0.3-scale model of a sixstory eccentrically braced steel structure. Rep. No. UCB/EERC-87/02, University of California, Berkeley, Calif., 1987.
[13] EN 1998-1, Eurocode 8: design of structures for earthquake resistance, Part 1: general rules, seismic actions and rules for buildings, European Committee for Standardisation, CEN., 2004.
[14] RPA99/version 2003, Règles parasismiques Algériennes, Document Technique Règlementaire, D.T.C., B.C. 2.48, Alger, 2003.
[15] Newmark, N. M., Hall, W. J., Earthquake spectra and design. EERI Monograph series. Oakland, 1982.
[16] Nassar, A. A., Krawincler, H., Seismic demands for SDOF and MDOF systems. Report N° 95, The John A. Blume Earthquake Engineering Center, Stanford, University, California, USA, 1991.
[17] Miranda, E, Evaluation of site-dependent inelastic seismic design spectra, Journal of Structural Engineering, 115 (5), pp. 1319-1338, 1993.
[18] Lam, N., Wilson, J., Hutchinson, G., The ductility reduction factor in the seismic design of buildings. Earthquake Engineering and structural Dynamics, vol.27, pp. 749-769, 1998.
[19] Ordaz, M., Perez-Rocha, L. E., Estimation of strength-reduction factors for elasto-plastic systems: a new approach. Earthquake Engineering and structural Dynamics, vol.27, pp. 889-901, 1998.
[20] Newmark, N. M., Hall, W. J., Seismic design criteria for nuclear reactor facilities. Report No. 46, Building Practices for disaster Mitigation, National Bureau of Standards, U.S. Department of Commerce, pp. 209-236, 1973a.
[21] Newmark, N. M., Hall, W. J., Procedures and criteria for earthquake resistant design. 1973b. National Bureau of Standards, 45, pp. 94-103.
[22] Ballio, G., ECCS approach for seismic design of steel structures against earthquakes. Proc. ECCS-IABSE Symposium, Luxembourg, 1985.
[23] Giuffré, A., Giannini, R., La risposta non lineare delle strutture in cement armato. Fondamenti di Ingegneria sismica, Bologna (in Italian), 1983.
[24] Krawinkler, H., Nassar, A. A., Seismic design based on ductility and cumulative damage demands and capacities. Nonlinear seismic analysis of RC buildings, Edited by P. Fajfar and H. Krawinkler, eds. Elsevier Science, New York, pp. 23-40, 1992.
[25] Como, M., Lanni, G., Elementi di costruzioni antisismiche. Ed. Cremonese, Roma, 1979.
[26] Kato, B., Akiyama, H., Energy concentration pf multi-storey buildings, Proc. 7th World Conference on Earthquake Engineering, Istanbul, 1980.
[27] Ballio, G., Castiglioni, C. A., An approach to the seismic design of steel structures based on cumulative damage criteria., Earthquake Engineering and structural Dynamics, 23, pp. 969-986, 1994.
[28] Castiglioni, C. A., Zambrano, A., Determination of the behavior factor of steel moment-resisting frames by a damage accumulation approach, Journal of constructional Saserch, 66, pp. 723-735, 2010.
[29] CEN. EN 1993-1-2. Eurocode 3: design of steel structures. Part 1.1: general rules for buildings. European committee for standardization Brussels; 2005.
[30] SAP2000. Linear and nonlinear static and dynamic analysis of three-dimensional structures. Advanced Version 14.0. Analysis Ref. Manual, Computer and Structures, Berkeley, CA., 2010.