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Evaluating Response Modification Factors of Concentrically Braced and Special Moment Steel Frames in Duplex Buildings

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Abstract

Response modification factor (R-factor) is one of the seismic design parameters to consider nonlinear performance of building structures during strong earthquake. Relying on this, many seismic design codes led to reduce earthquake loads imposed to the structure. The present paper tries to evaluate the R-factors of conventional concentric braced frames (CBFs) and special moment frames (MRFs) in duplex steel buildings with level difference in their floors. Since, the R-factor depends on ductility and over strength, the incremental nonlinear static analysis, push over analysis, has been performed on 4, 7 and 10 storey building models with three floor level differences and including CBFs and special MRFs systems in x and y directions of buildings respectively. The results showed that the R-factors for CBFs system in duplex buildings were higher than ones in conventional buildings without floor level differences while for MRFs system it was found that on 4 and 7 storey duplex buildings, the R-factors were decreased and with the increase in building height to 10 storey, they were increased compared to conventional models.

Keywords: Response Modification Factor, Duplex Building, Push-over Analysis, Special Moment Frames, Concentrically Braced Steel Frames

1. Introduction

Experiencing the effects of earthquakes on building structures indicates that they usually behave inelastically during moderate to severe earthquakes. Therefore, buildings can be designed for earthquake force much less than what is required in linear behavior. This reduction of design loads by seismic codes is through the application of response modification factor (R-factor). In fact, the R-factor reflects the capability of a structure to dissipate energy through inelastic behavior. Inelastic dynamic analysis, although yields accurate results, is time consuming and interpretation of its results demands high level of expertise. Researchers have long been interested in developing fast and efficient methods such as pushover analysis to simulate nonlinear behavior of structures due to earthquake loads. The idea of pushover analysis was first introduced in 1975 by Freeman for single degree of freedom (SDOF) systems. Then other researchers extended this method for multi-degree of freedom (MDOF) systems [1–4]. This method is also documented in the ATC 3-06 [5], ATC-19 [6] and ATC-34 [7] reports. In these documents the R-factor was calculated as the product of three factors: over strength factor, ductility factor and redundancy factor. In conventional pushover analysis, the shape of lateral load pattern is usually based on the first elastic mode of the structure and the higher mode effects are not accounted for. Moghadam and Tso [8] and later Chopra and Goel [9] introduced multi-mode methods to overcome this problem. Sasaki et al. [10], Satyarno et al. [11], Gupta and Kunnath [12] also proposed different ways of conforming the loading pattern with the structural stiffness.

Steel concentric braced frame (CBF) is one of the efficient and commonly used lateral load resisting systems, especially in the structures of high or moderate seismic regions [13]. The work lines of CBFs essentially intersect at points [14]. The steel braces improve the lateral strength and the stiffness by inelastic deformation during an earthquake that leads to seismic energy dissipation [15]. The behavioral or R-factors of CBFs have been the subjects of investigations by various researchers [13, 15-18].Osteraas [19] conducted a detailed study of reserve strength of three structural systems: distributed moment frames, perimeter moment frames, and concentric braced frames. They observed that the over strength factor of braced frames ranged between 2.8 and 2.2. Balendra and Huang [20] found that the R-factors decreased when the number of stories increased. For three, six and ten-storey braced frames, the response modification factor was found to vary from 8.5 to 3.5.

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Izadnia et al. [21] evaluated the R-factor for steel moment-resisting frames by different pushover analysis methods. In their analyses for each frame, eight different constant as well as adaptive lateral load patterns are used. They found that for three, six, and nine-storey moment-resisting frames, the average R-factor was about 5.5 and the maximum relative difference for R- factors was about 16% obtained by the methods of conventional and adaptive pushover analyses.
The present study focuses on the evaluation of over strength, ductility and response modification factors for 4, 7 and 10 storey building models which have CBFs and MRFs systems in x and y directions respectively. These models are studied in two types of conventional and duplex buildings where the duplex buildings have three level differences in their floors equal 0.8, 1.2 and 1.6 meters. For this purpose, the buildings designed according to standard 2800 code [22] and the tenth issue of the National Building Regulations [23] using the compression loading method for design of duplex structures. Then nonlinear static pushover analyses with two lateral load patterns, uniform and spectral, were carried out to obtain such behavior factors. Mentioned analyses are done using Sap2000 software.

2. Response modification factors

Response modification factor (R-factor), was mostly determined based on field observations of buildings performance against previous earthquakes until the early 1980s that the researchers sought to identify the factors effective in determination of R-factor and analyze them, but this should be done in a way that would satisfy both main and basic features of the objective. These features include:
1.Become controllable by creating enough stiffness and resistance of internal forces of the structure and displacements due to earthquakes, and would prevent major damages and losses to the members in slight earthquakes.
2.Preventing structural dismantle in severe earthquakes by providing adequate plasticity to the structure.
ATC-19 [6] proposed a simplified procedure to estimate the R-factors, in which the response modification factor, R, is calculated as the product of the three factors that profoundly influence the seismic response of structures:

$$R = R_s R_\mu R_r \qquad (1)$$

where R_s is the overstrength factor to account for the observation that the maximum lateral strength of a structure generally exceeds its design strength. R_μ is a ductility factor which is a measure of the global nonlinear response of a structure and R_r is redundancy factor to quality the improved reliability of seismic framing systems constructed with multiple lines of strength.
Fig. 1 shows a diagram that represents the overall performance of a building. This figure represents the relation between the base-shear and displacement of a structure, which can be developed by a nonlinear static analysis.

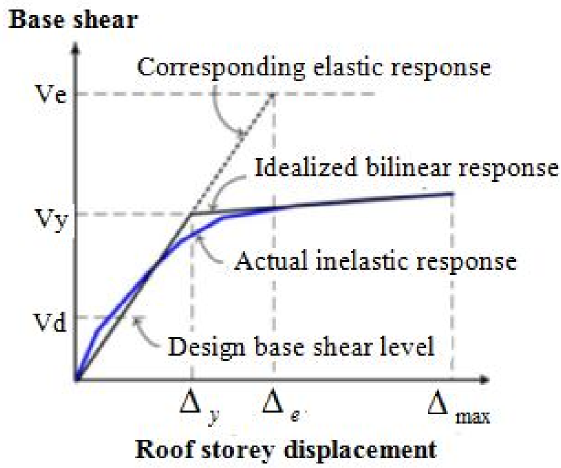


Fig. 1. Lateral load–roof displacement relationship of a structure.

The overstrength factor R_s, ductility factor R_μ and redundancy factor R_r are defined as follows:

$$R_s = V_e / V_y \qquad (2)$$

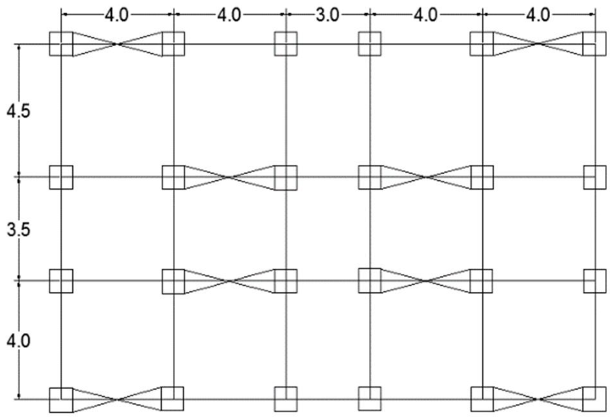
$$R_\mu = V_y / V_d$$

$$R_r = V_y / V_w$$

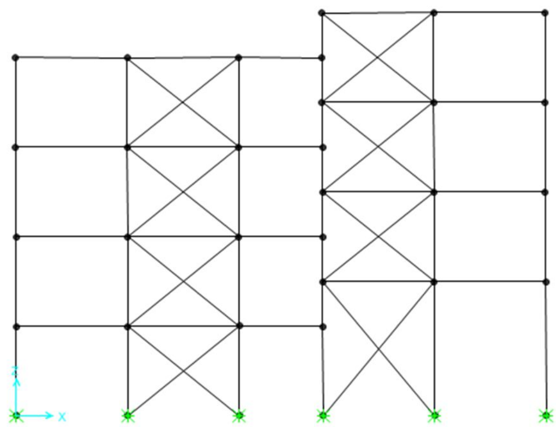
where V_d is the design base shear, V_e is the maximum seismic demand for elastic response, V_y is the base shear corresponding to the maximum inelastic displacement and V_w is the allowable base shear.
It should be noted that R-factor introduced by the Iran earthquake regulations [22] the earthquake is permitted based on design by stress method in which the R_r Factor is called the R-factor in the stress mode of surrender limit or R-factor in stress mode of permitted limit.

3. Design of model structures

To evaluate the overstrength factors, ductility factors, redundancy factors and the response modification factors of braced frames, 4,7 and 10 storey CBFs and special moment frames with the bay length varied as 3, 3.5, 4, and 4.5 meters were designed using the ‘Allowable Stress Design’ [22]. The story height of every model structure was fixed to 3.2 meter and the floor level differences in duplex buildings, split level, are considered 0.8, 1.2 and 1.6 meters. Fig. 2(a) shows the plan of the prototype structure and the braces are located in the different bay of the frames according to Figs. 2(a) and (b). The constitutive model of bracing, beams and columns are also presented in Fig. (3). The elasticity modulus, poison ratio and yielding stress of steel material are considered, 2.1*106 (kg/cm2), 0.3, 2400(kg/cm2) in calculations.



(a) Plan



(b) Brace configuration.

Fig. 2. Configuration of model structures.

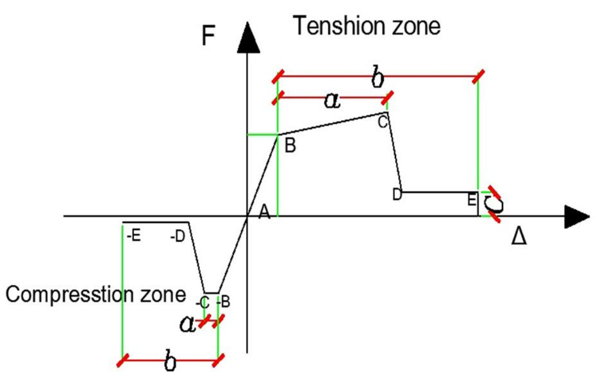


Fig. 3. Constitutive model of Bracing, beams and columns

The dead and live loads of 5 and 2 kN/m^2, respectively, were used for gravity load, and the earthquake design base shear was determined based on the third edition of Standard 84-2800[22], using the following parameters: Seismic Use Group II, soil type II, and the response modification factors = 6.0 for CBF and 10.0 for Special MRF. ST37 steel was used for every structural member. The braces were designed to resist lateral seismic loads in direction x, and the beam–column joints were assumed to be pinned. The structural design was carried out using the program code UBC97-ASD [24].The structural members selected for the seven-storey model structures with split level 0.8 meter are listed in Tables 1 and 2.

In the seismic provisions for structural steel buildings the slenderness ratios of compression members (columns) are limited as follows:

$$\frac{KL}{r} \leq 200 \tag{3}$$

where r is the radius of gyration and KL is the effective length.

Table 1
Sectional properties for beams of seven-story model structures with split level 0.8 m

a) Concentric Braced Frames		
Span length (m)	Story	beam
3	1	2IPE180+2PLF160*10
	2	2IPE160+2PLF140*10
	3	2IPE140+2PLF120*10
	4	2IPE140+2PLF120*10
	5	2IPE140+2PLF120*10
	6	2IPE140+2PLF120*10
	7	2IPE120+2PLF100*10
4	1	2IPE200+2PLF180*10
	2	2IPE180+2PLF160*10
	3	2IPE180+2PLF160*10
	4	2IPE180+2PLF160*10
	5	2IPE180+2PLF160*10
	6	2IPE180+2PLF160*10
	7	2IPE160+2PLF140*10
b) special MRF		
3.5	1	2IPE200+2PLF180*10
	2	2IPE180+2PLF160*10
	3	2IPE180+2PLF160*10
	4	2IPE180+2PLF160*10
	5	2IPE160+2PLF140*10
	6	2IPE140+2PLF120*10
	7	IPE160
4	1	2IPE200+2PLF180*10
	2	2IPE200+2PLF180*10
	3	2IPE180+2PLF160*10
	4	2IPE180+2PLF160*10
	5	2IPE160+2PLF140*10
	6	2IPE140+2PLF120*10
	7	IPE160
4.5	1	2IPE200+2PLF180*10
	2	2IPE200+2PLF180*10
	3	2IPE180+2PLF160*10
	4	2IPE180+2PLF160*10
	5	2IPE160+2PLF140*10
	6	2IPE140+2PLF120*10
	7	IPE160

Table 2
Sectional properties for columns and braces of seven-story model structures with split level 0.8 m

Story	Column	Brace
1	TUBO320*320*17.5	2UPN120_D10
2	TUBO240*240*16	2UPN100_D10
3	TUBO220*220*16	2UPN100_D10
4	TUBO220*220*16	2UPN80_D10
5	TUBO200*200*12.5	2UPN80_D10
6	TUBO18*180*12.5	2UPN65_D10
7	TUBO180*180*12.5	2UPN65_D10

4. Non-linear static analysis of model studied structures

Studied models for the study of nonlinear performance of duplex structures, three 4, 7 and 10-storey steel structure buildings with normal constructional frame system with coaxial steel bracing in the x-direction and special moment frame in the y-direction in duplex and normal modes have been considered. It should be noted that, each of the duplex models has been analyzed and designed in three different modes with levels difference of 0.8, 1.2 and 1.6 meters in x-direction frames (braced), according to 2800 standard code [22] and the tenth issue of the National Building Regulations [23] and through the loading method of compression type by sap-2000v17 software. The height of floors in normal structure (non-duplex) is 3.2 meters and in duplex structures varies depending on the level difference.

Given that duplex structures are considered among irregular structures in height and the designed models have been brought under three-dimensional nonlinear static analysis (3D Pushover) according to the regulations of seismic rehabilitation instructions of existing buildings (Publication 360) [25], so that impact of tension would be applied to the models in real form, and in order to better comparison of results, three ordinary 4, 7 and 10-storey models were also analyzed and designed.

Progressive non-linear static analysis is an effective way to determine overstrength and formability of the existing structures as well as recognition of the manner of destruction mechanisms of structures. In this method the lateral load patterns are gradually increased from zero so that the building would pass the stage of linear behavior and would proceed in the non-linear behavior to that extent that the structure would go destroyed and its stiffness would reach zero [26].

Since the non-linear static analysis with increasing load pattern has recently been introduced in new regulations such as FEMA and ATC, in this paper, the relative displacement of floors corresponding to the performance level of life safety, have been considered according to the FEMA356 publication [14]. According to this Regulations, the maximum relative displacement of the floors (total relative displacement of transient and steady-state) corresponding to the life safety performance level have been considered 2% for braced frames and 5.3% for special moment frames. Therefore, with these presuppositions, R-factor of all models will be analyzed under the two uniform and spectral lateral load patterns for braced and special moment frames in nonlinear static analysis (pushover) method and the results will be examined.

5. Results

In Figs. 4 to 9, the capacity curves for normal 4, 7 and 10 storey models (with no level difference) and duplex for braced frames and moment frames have been shown. Figs. 10 and 11 show the location of inelastic deformation and plastic hinges of the seven storey building with split level 1.2 meter for special MRF and CBF systems.

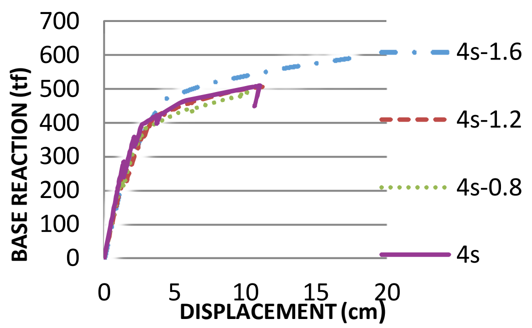


Fig. 4. Capacity Curve of braced frames for normal and duplex 4-storey buildings with different level

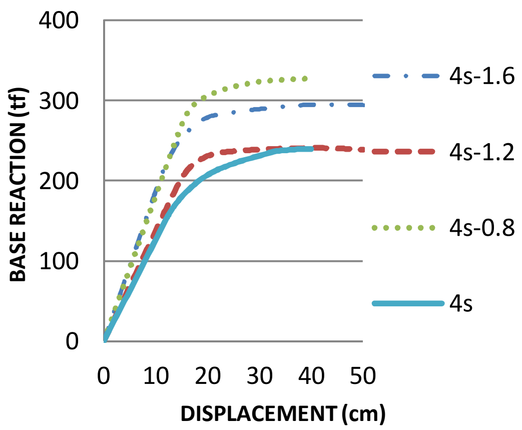


Fig. 5. Capacity Curve of moment frames for normal and duplex 4-storey buildings with different level differences

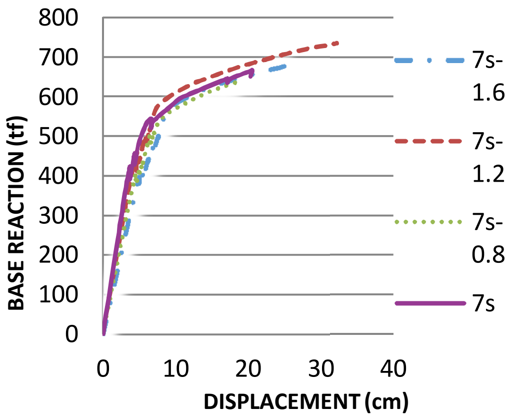


Fig.6. Capacity Curve of braced frames for normal and duplex 7-storey buildings with different level differences

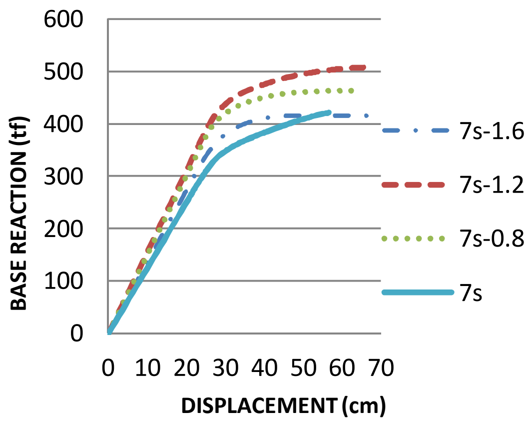


Fig.7. Capacity Curve of moment frames for normal and duplex 7-storey buildings with different level differences

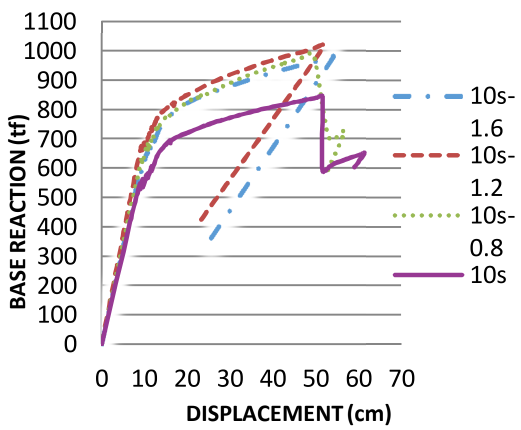


Fig. 8. Capacity Curve of braced frames for normal and duplex 10-storey buildings with different level differences

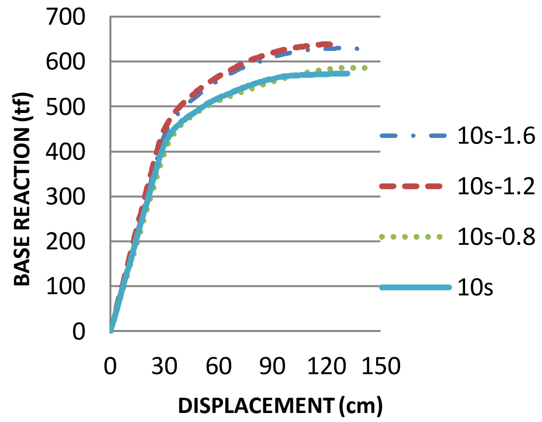


Fig.9. Capacity Curve of moment frames for normal and duplex 10-storey buildings with different level differences

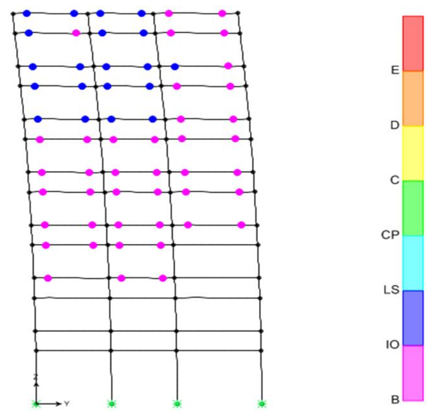


Fig.10. Location of plastic hinges for seven-storey building with 1.2 split level in special MRF system

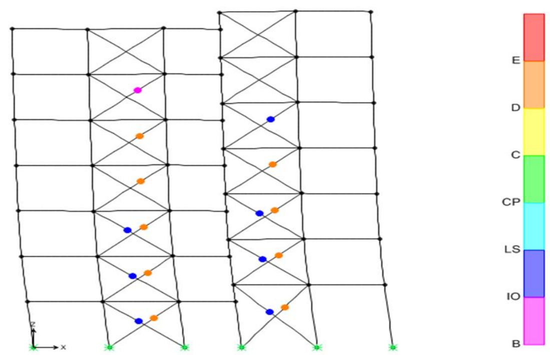


Fig.11. Location of plastic hinges for seven-storey building with 1.2 split level in CBF system

As noted above, for calculating the R-factor, the R_μ and R_s and R_r factors are obtained and presented in tables 3 to 6. In these tables all models are introduced using storey numbers and floor difference levels (split levels) such as 4S-0.8 for 4-storey building with 0.8 split level.

Table 3
R-factor of CBFs in normal and duplex structures under uniform load pattern

Model	V_S (tonf)	V_y (tonf)	V_w (tonf)	R_s	R_μ	R_r	R
4S	145.113	166.25	58.57	1.145	1.488	2.477	4.22
4S-0.8	210	273.67	61.65	1.3	1.19	3.4	5.27
4S-1.2	162.67	214.31	61.37	1.317	1.338	2.65	4.67
4S-1.6	181.33	232.33	61	1.281	1.415	2.972	5.38
7S	271.7	320.24	87.25	1.178	1.23	3.114	4.51
7S-0.8	360.92	400.56	89.87	1.1	1.183	4.016	5.22
7S-1.2	327.18	395.9	89.36	1.21	1.182	3.661	5.23
7S-1.6	276.51	355	88	1.283	1.286	3.142	5.18
10S	351.65	417.2	107.37	1.186	1.315	3.275	5.1
10S-0.8	344.28	415.96	111.75	1.2	1.336	3.08	4.94
10S-1.2	357.42	437.6	112.91	1.224	1.376	3.165	5.33
10S-1.6	352	437.92	109.97	1.244	1.351	3.2	5.38

Table 4
R-factor of CBFs in normal and duplex structures under spectral load pattern

Model	V_S (tonf)	V_y (tonf)	V_w (tonf)	R_s	R_μ	R_r	R
4S	136.86	165.65	58.57	1.21	1.578	2.336	4.46
4S-0.8	168.72	208.96	61.65	1.283	1.3	2.736	4.56
4S-1.2	146.54	185.1	61.37	1.263	1.446	2.387	4.36
4S-1.6	169.19	219.5	61	1.297	1.488	2.773	5.35
7S	211.43	258.53	87.25	1.222	1.356	2.423	4
7S-0.8	273.1	347.56	89.87	1.272	1.264	3.038	4.88
7S-1.2	229.68	315	89.36	1.371	1.334	2.57	4.7
7S-1.6	252.89	317.89	88	1.257	1.325	2.873	4.78
10S	299	366	107.37	1.224	1.392	2.784	4.74
10S-0.8	291	350.2	111.75	1.2	1.362	2.604	4.25
10S-1.2	304.75	374.72	112.91	1.229	1.443	2.699	4.78
10S-1.6	319	395.8	109.97	1.24	1.435	2.9	5.16

Table 5
R-factor of MRFs in normal and duplex structures under uniform load pattern

Model	V_S (tonf)	V_y (tonf)	V_w (tonf)	R_s	R_μ	R_r	R
4S	220.93	378.95	102.98	1.715	3.24	2.145	11.92
4S-0.8	170.61	359.76	113.33	2.1	3.521	1.5	11
4S-1.2	184.84	389.1	112.81	2.1	3.154	1.638	10.85
4S-1.6	266.84	409.98	113.87	1.536	2.8	2.343	10
7S	324.3	506	200.83	1.56	2.815	1.614	7
7S-0.8	296.67	508.72	203.49	1.714	2.573	1.457	6.42
7S-1.2	348.75	533.12	204.5	1.528	2.474	1.881	7.11
7S-1.6	283.6	530.85	199.82	1.871	2.387	1.419	6.33
10S	419.8	621.96	238.61	1.481	2.125	1.759	5.53
10S-0.8	466.28	718.63	253.98	1.541	2.01	1.835	5.68
10S-1.2	495.52	734.92	256.62	1.483	2.1	1.930	6
10S-1.6	438.45	715.54	255.74	1.631	2.05	1.714	5.73

Table 6
R-factor of MRFs in normal and duplex structures under spectral load pattern

Model	V_S (tonf)	V_y (tonf)	V_w (tonf)	R_s	R_μ	R_r	R
4S	200.69	357.4	102.98	1.78	3.215	1.948	11.15
4S-0.8	142.47	344	113.33	2.414	2.584	1.257	7.84
4S-1.2	166.2	340	112.81	2.04	2.54	1.473	7.63
4S-1.6	201.72	340.65	113.87	1.688	2.545	1.771	7.6
7S	285.56	432.67	200.83	1.515	2.275	1.421	4.89
7S-0.8	273.41	418.2	203.49	1.529	2	1.343	4.1
7S-1.2	224.75	425.19	204.5	1.891	2.04	1.099	4.24
7S-1.6	229.82	419	199.82	1.823	2.03	1.5	5.55
10S	270.98	496.25	238.61	1.831	1.915	1.135	3.98
10S-0.8	282.47	558.42	253.28	1.976	1.846	1.115	4
10S-1.2	301.65	564.63	256.62	1.871	1.86	1.175	4
10S-1.6	312.74	557.1	255.74	1.781	1.814	1.222	3.95

6- Conclusions

1. The response modification factor of steel braced frames under uniform load pattern in 4-storey normal and duplex models was obtained about 4.2-5.4, in 7-storey models about 4.5-5.2 and in 10-storey models 4.9-5.4. Also, R-factor of steel braced frames under spectral load pattern in 4-storey normal and duplex models was achieved between 4.3-5.35, in 7-storey models between 4-4.9 and in 10-storey models was achieved 4.2-5.2. While, the suggested factor of Iran standard 2800 for normal models of steel braced frame is 6, hence, the base shear that regulation gives for design is less and in other words, it is down-handed.
2. R-factor of MRFs under uniform load pattern in 4-storey normal and duplex models was obtained between 10-12, in 7-storey normal and duplex models between 6.3-7.1 and in 10-storey models has obtained about 5.5 to 6. Also, R-factor of special steel moment frames under spectral load pattern in 4-storey normal and duplex models was achieved between 7.5-11.1, in 7-storey normal and duplex models between 4.5-5 and in 10-storey models was achieved about 3.95-4. While, the suggested factor of Iran standard 2800 for normal models of special steel moment frames is 10, hence, it can be concluded that the factor proposed by Iran standard 2800 applies to normal models of Special MRFs in short buildings and it is down-handed for high-rise buildings.
3. By increasing the height of structure, R-factor of structures regarding BRFs under both spectral and uniform patterns has decreased almost to a rate of 2.5%, and in the case of special MRFs under uniform pattern to 52% and under spectral load pattern has been decreased to 60%.
4. R-factor of each 4, 7 and 10-storey duplex steel braced frames under uniform load pattern are 8%, 4% and 5% and under spectral load pattern are 10%, 7% and 5% higher than models without level difference, respectively.
5. R-factor of special MRFs under uniform load pattern regarding 4 and 7-storey duplex models has decreased to the arte of 11% and 9% towards the models without level difference, respectively. But, it is increased in 10-storey duplex models. Also, R-factor of special moment frames under spectral load pattern regarding 4 and 7-storey duplex models has decreased to the rate of 31% and 5% respectively, and has not changed much in 10-storey models.
6. In most models, among differences of studied levels, the model with 0.8 meter of level difference has a smaller R-factor, hence, it is recommended to avoid making level difference with small ratios of dh/h in structural design as much as possible.

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