THE DETERMINATION OF MODIFICATION FACTOR FOR OUTRIGGER BRACED STRUCTURES USING TIME HISTORY ANALYSIS

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ABSTRACT

According to developments in construction of highrise buildings both in Iran and developing countries, more concentration and studies on seismic behaviour of these types of structures are required. Due to a large variety of highrise buildings, current research is carried out on a special type of above mentioned buildings, which is composed of a steel braced welded structure equipped with outrigger bracings both on roof and mid height levels. Outrigger braced highrise structures are assumed to be a basic solution for overturning problem by outbreaking the total outer structure of highrise buildings. This system is much more effective than the conventional braced frame structures for buildings ranged from 40 to 60 stories high. First the computational method for modification factor and related effective parameters are briefly described in this research. Then a total of 8 two dimensional frames (ranging from 20 to 60 stories high) equipped with outrigger bracings of various types both on roof and mid height levels are modelled. Then the assumed finite element models are analysed and designed according to Iranian 2800 seismic code taking into account the site specifications and S_a=0.25g , S_a=0.35g spectral acceleration levels. Afterwards, by using accelerograms recorded on soil types 1,2,3 & 4 (due to 2800 code), and after scaling them to S_a=0.25g & S_a=0.35g, a total of 1280 linear and nonlinear Time History analyses are carried out on the above mentioned 2D models, using Sap 2000 ver. 16.1.1 finite element software. By performing a “Modal Push-over Analysis” on each model and by using achieved results of above mentioned analyses, the ductility and overstrength reduction factors are computed for each model and related records. Finally, the modification factors are calculated for both “Ultimate state” and “Working stress” design methods respectively.

INTRODUCTION

Previous experience of earthquakes illustrates that many types of structures behave nonlinearly during a severe earthquake. So a huge amount of input energy is mainly dissipated through the form of damping and hysteresis. According to this, the structures are usually designed for much lower lateral forces than those demanded by aseismic design codes in elastic range. The aseismic behaviour analysis and accurate design of structures for severe earthquakes are mainly carried out using Nonlinear Time history Analysis method (NTHA). Using the NTHA method for analysis of somehow simple structures in consulting engineer’s offices is not appropriate enough, due to the complexity and time taking behaviour of the method. So according to simplicity and popularity of structural linear analysis techniques, they are mainly proposed in most aseismic design codes using the reduced lateral forces meanwhile. The seismic linear force for structural design purposes is achieved from linear earthquake spectra. The computed lateral force from the spectra is decreased by the means of a reduction factor or modification factor, according to ductility, damping, overstrength and so on. This research is carried out to compute the modification factor of “outrigger braced structures”. A central core, composed of braced frames or shear walls is included in these types of highrise structures as shown in fig.1. When the structure is subjected to lateral loads, the planar rotation of core is limited by compression-tension functioning of the outside columns by the use of outriggers, as shown in fig1. According to the height of outrigger braces, the overall lateral stiffness of the structure is increased, causing the lateral deformation decrease to a great extent. This method is a proper solution for overturning control for highrise buildings ranging from 40 to 60 stories high. It is also a proper solution to construct a highrise building without any additional fee due to height. The technique of outrigger braced structures is a high efficient one, comparable with framed tube system, capable to reduce the lateral deformation as much as 25%~30%.

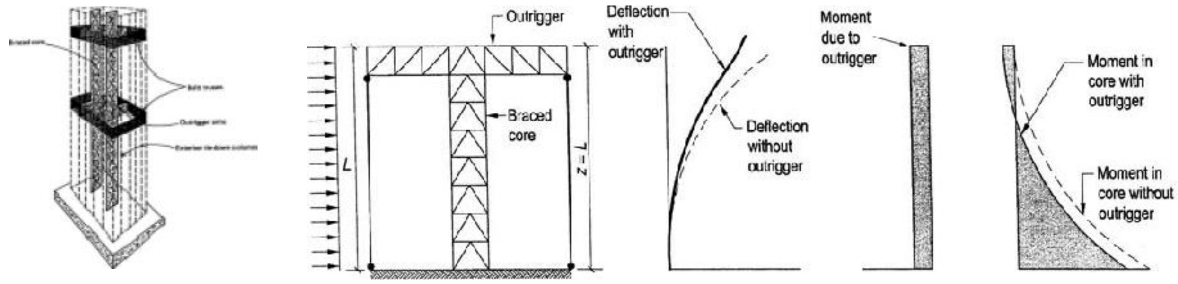

Figure 1: Highrise outrigger Braced Structure Schematic view

MODIFICATION FACTOR THEORITICAL BASES

As mentioned previously, according to energy absorption characteristics of structures, a reduction factor R is used in seismic codes worldwide. The full concept of this factor is shown in Fig 2. Real behaviour of the structure is estimated equal to a bilinear relation in which C_y and Δ_y indicate the yielding force and yielding displacement respectively. If the structure behaves linearly during an earthquake, the maximum resulted seismic force will be equal to C_{eu}. But in reality, due to energy absorption in nonlinear range, the applied force would not exceed the yielding force Cy. In this case, the related displacement would be equal to Δ_{max}. In fig 2, Δ_s is the displacement according to the first plastic hinge formation in any of the structural elements. According to fig 2, it could be seen that the Base Shear Factor C_u is decreased in two stages:

a- Reduction from C_{eu} to C_y due to structural ductility

b- Reduction from C_y to C_s due to structural overstrength, which is an additional strength that the structure demonstrates after transmission from elastic stage to the plastic one. The above mentioned reduction factors could be summarized as below:

$$R_\mu = C_{eu} / C_y \text{ Due to Ductility} \tag{1}$$

$$R_s = C_y / C_s \quad \text{Due to Overstrength} \tag{2}$$

$$R = R_\mu . R_s \text{ Modification Factor} \tag{3}$$

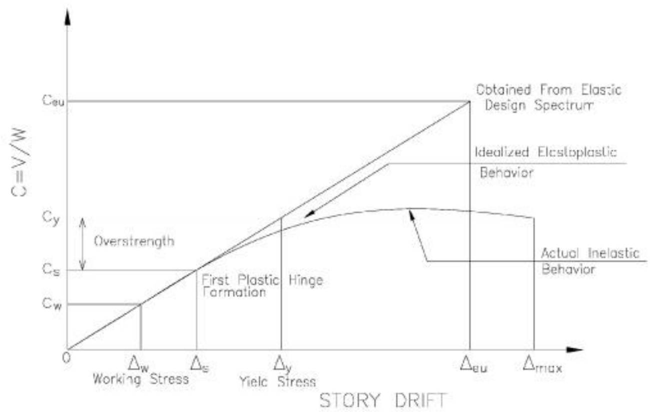

Figure 2: General seismic response of structures

Reduction factor due to ductility R_μ, indicates the energy absorption capacity due to structural hysteresis behaviour. If the hysteresis behaviour is non-deteriorative, then the structure will preserve its strength for a longer time. To calculate the ductility reduction factor for a specific record, resulted base shear from a linear analysis of the structure is divided by the resulted base shear from nonlinear analysis of the same structure. Carried out researches demonstrate that R_μ is strongly sensitive to ductility ratio μ and structural vibration period T.

The Overstrength factor R_s is another important factor which could not be obtained easily. Analytical and tentative methods should be used to obtain R_s. The role of R_s factor is much more important in the case of intensive earthquakes and its value is based on material properties, lateral load bearing system, geometry of the structure and the structural details. So it could be seen that this value is particular for each structure. Practical method to find R_s is based on a static push-over analysis. The R_s factor shown in equation (2) is not real and additional corrections should be applied to obtain the real value of R_s:

$$R_s = R_{s0} F_1 F_2 . F_n \tag{4}$$

In equation (5), F_1 indicates the difference between nominal and real statically yielding strength and for steel structures F_1 is equal with 1.05. F_2 is another factor which indicates the increasing rate for yielding stress due to strain effect during an earthquake and is equal to 1.10. The remained factors could be computed due to trustable information; otherwise it should be estimated equal with 1.0. The

Above mentioned technique is for seismic codes which are based on ultimate strength method. For Working Stress design codes, the modification factor's value is as follows:

$$Y = C_s / C_w \tag{5}$$

$$R_w = R_u . R_s . Y \tag{6}$$

The value of Y ranges 1.4~1.5.

COMPUTATIONAL MODELS

According to above mentioned descriptions, 8 two dimensional outrigger braced frames, containing 20,30,35,40,45,50,55 & 60 stories which are equipped with 1,2,2,2,3,3,3 & 3 outriggers respectively, height to width ratio, to fulfil the highrise structure requirements. The loading area for all models is assumed to be 2.5 square-meters, due to their location as exterior frames. All connections are assumed to be welded rigid, including column to base connections. Steel used for all structural elements is of St-37 grade steel demonstrating complete elasto plastic behaviour.

Slabs used in all models are considered of composite steel-reinforced concrete type, bearing a live load equal with 200kg/m2.

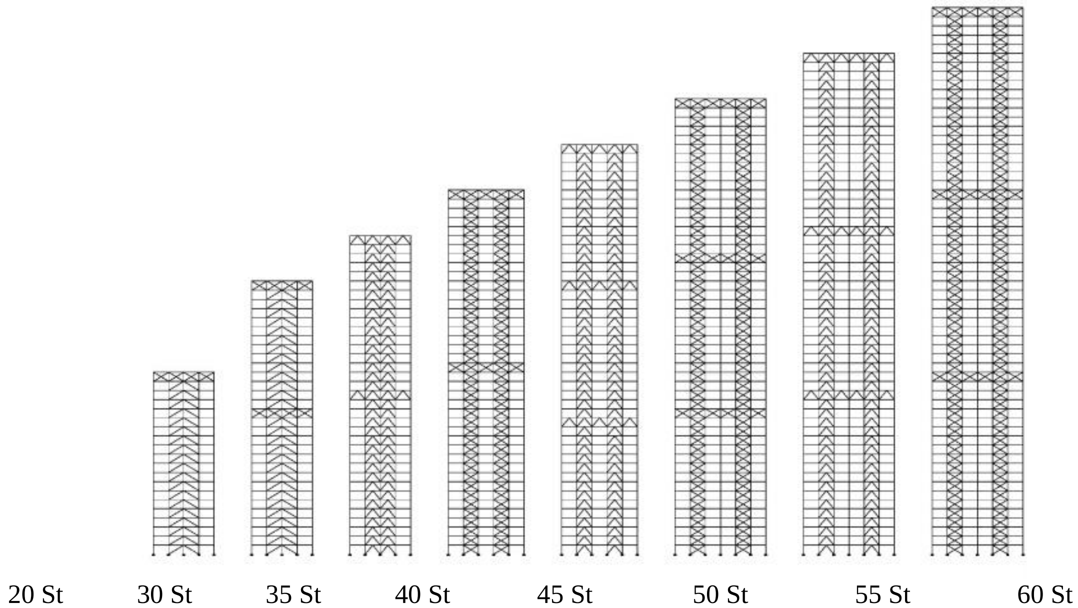

Figure 3: Finite element computational models

The lateral load bearing system is considered to be special moment resisting frame + concentric bracings. In design process, the requirements of lateral displacement and interstory drift limitations due to 2800 Iranian code are satisfied. Column sections are considered of steel plate box sections. For beams and bracings, IPB sections are used for analysis and design purpose. All computational models are then analysed and designed for two levels of S_a=0.25g & S_a=0.35g spectral acceleration, considering soil types 1, 2, 3 &4 of 2800 Iranian code. Sap2000 ver.16.1.1 software is used for analysis and design purposes, taking into account P-Δ effects. Then the final sections for structural elements are obtained according to AISC-ASD 89 code, considering special moment resisting frame (SMRF) coordinates. Finally, the Modal Push-over analysis is performed and the base shear required to form the first plastic hinge is calculated for all 48 frames as indicated in Table 1:

Table 1: Base Shear Force to form first Plastic Hinge (ton)

Spectral Acceleration & Soil Types		20 St	30 St	35 St	40 St	45 St	50 St	55 St	60 St
Soil Type 1	0.25g	71.8	53.6	48.9	83.1	113.6	97.0	128.1	157.0
	0.35g	72.5	69.6	52.6	76.8	124.3	141.4	176.0	158.6
Soil Type 2	0.25g	71.2	53.9	78.8	71.0	114.7	107.4	128.1	156.9
	0.35g	78.1	86.5	60.7	111.2	138.0	139.0	176.1	158.4
Soil Type 3	0.25g	78.0	71.9	58.1	111.0	136.6	141.8	176.2	158.8
	0.35g	82.1	121.1	92.3	165.0	173.9	174.6	188.4	250.5
Soil Type 4	0.25g	83.1	121.8	94.9	174.5	174.2	182.8	285.8	260.0
	0.35g	116.0	129.2	165.9	173.0	270.7	257.4	309.2	278.7

The Push-over Diagrams of all models are illustrated in figures 4 to 11:

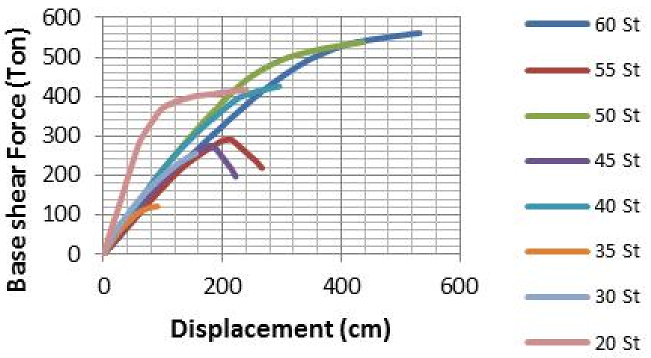

Figure 4: Soil Type 1 & S_a=.25g

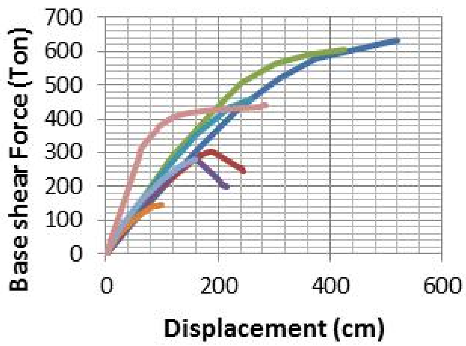

Figure 5: Soil Type 1 & S_a=.35g

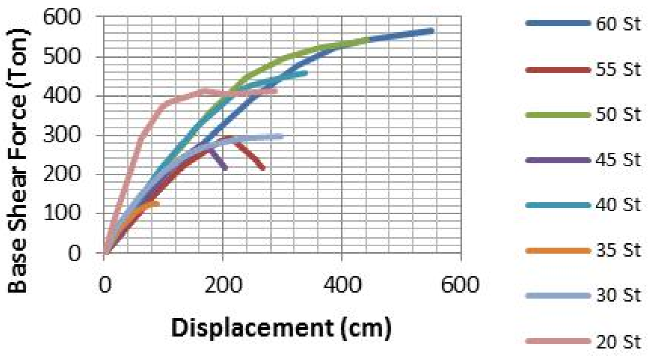

Figure 6: Soil Type 2 & S_a=.25g

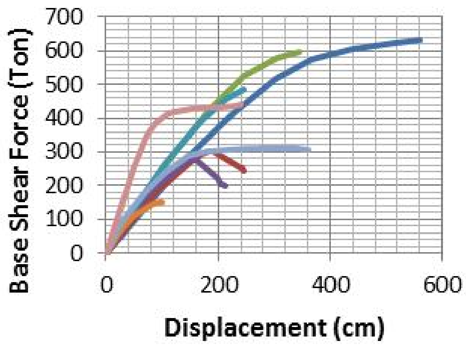

Figure 7: Soil Type 2 & S_a=.35g

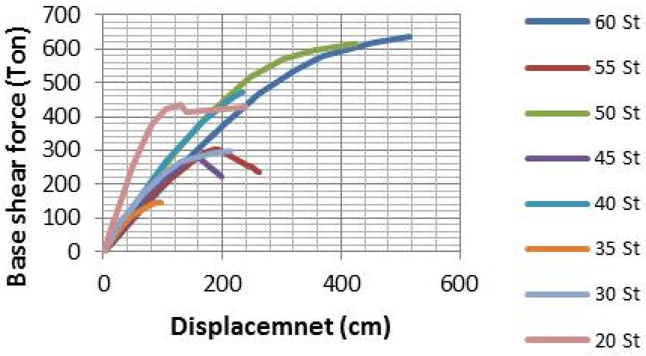

Figure 8: Soil Type 3 & S_a=.25g

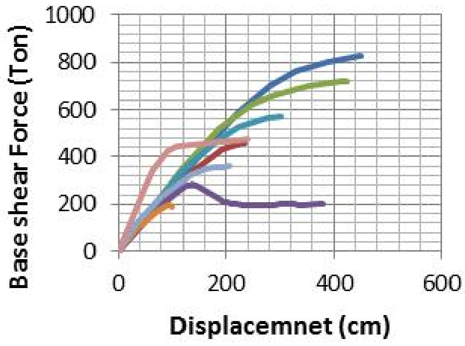

Figure 9: Soil Type 3 & S_a=.35g

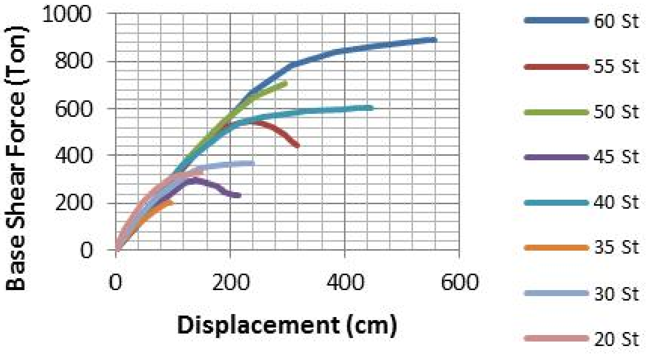

Figure 10: Soil Type 4 & S_a=.25g

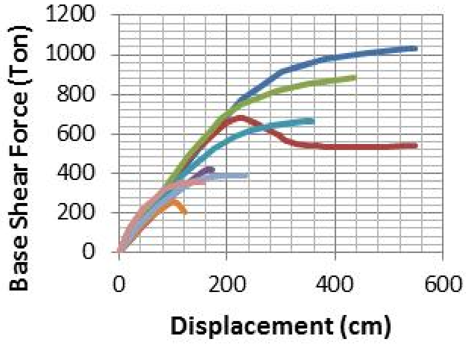

Figure 11: Soil Type 3 &S_a=.35g

LINEAR AND NONLINEAR TIME HISTORY ANALYSES

In order to perform the time history analyses, 10 accelerograms of earthquakes recorded on each soil type (1, 2, 3 &4, according to 2800 Iranian code) were selected. Then each record was scaled to spectral accelerations of S_a=0.25g & S_a=0.35g separately due to related response spectrum in 2800 Iranian code. Selected records are of the earthquakes listed in Table 2.

Table 2-Charachteristics of used Earthquakes

Event	Year	Mag.	Mechanism
Ducze-Turkey	1999	7.14	Strike-Slip
Chi-Chi-Taiwan	1999	7.62	Reverse-Oblique
Irpinia-Italy	1980	6.9	Normal
Tabas-Iran	1978	7.35	Reverse
Denali-Alaska	2002	7.9	Strike-Slip
Loma Prieta	1989	6.93	Reverse-Oblique
Kocaeli-Turkey	1999	7.51	Strike-Slip
Northridge	1994	6.69	Reverse
San Fernando	1971	6.61	Reverse
Landers	1992	7.28	Strike-Slip
Hector Mine	1999	7.13	Strike-Slip
Cape Mendocino	1992	7.01	Reverse

Then the scaled records were applied to the computational models separately due to the soil type and spectral acceleration for which the selected model was analysed and designed. For linear Time history analyses, the “Modal Extension Method of Earthquake forces” technique was used. Nonlinear Time history analyses for all models were completed using Newmark – β method. Due to structural characteristics and estimated welded connections, the damping ratio for linear analyses was determined equal to 0.02 for all mode shapes. For nonlinear analyses the Rayleigh damping was used, determining damping ratio equal to 0.05 for first two modes of vibration. By using the analyses results, overstrength factor $\mathbf{\Omega_0}$, corrected overstrength factor R_s, reduction factor due to ductility R_μ, and finally the modification factor for ultimate strength design level R and for working stress design level R_w, were computed for each record as illustrated in Table 3. Whole results for all models and records are summarized in Table 4.

Table 3- Calculated R factor for 40St Model, for soil Type 3 & S_a=0.35g

	Ω_0	R_s	R_μ	R	R_w
1	2.40	2.77	1.37	3.80	5.47
2	3.00	3.46	1.26	4.36	6.28
3	1.82	2.10	1.07	2.26	3.25
4	1.65	1.91	1.35	2.57	3.70
5	2.72	3.14	1.24	3.89	5.60
6	2.30	2.66	1.49	3.96	5.70
7	2.70	3.12	1.34	4.18	6.02
8	2.33	2.70	1.49	4.02	5.79
9	1.33	1.53	1.77	2.71	3.90
10	1.46	1.69	1.21	2.04	2.94

Table 4 - Calculated R Factors

		20 St		30 St		35 St		40 St	
		25g	35g	25g	35g	25g	35g	25g	35g
Soil Type 1	1	3.07	3.92	5.46	5.72	4.14	5.45	2.52	4.00
	2	2.80	4.64	4.15	4.60	4.44	5.78	3.84	5.98
	3	3.76	4.69	5.47	5.45	6.92	9.36	3.87	5.81
	4	2.97	4.06	2.90	3.08	5.92	7.20	3.85	5.94
	5	1.79	2.98	4.93	5.62	3.48	5.35	2.74	3.62
	6	3.33	4.51	5.33	5.32	5.73	7.68	3.30	4.67
	7	3.73	6.09	3.18	3.26	6.40	8.82	3.32	4.83
	8	3.18	4.62	5.08	5.58	4.78	6.00	4.04	6.15
	9	2.37	3.65	2.88	3.30	5.27	6.73	4.27	7.47
	10	4.29	6.62	5.02	4.82	6.46	8.40	3.76	6.38
Soil Type 2	1	2.83	3.45	3.86	3.90	5.20	5.56	4.34	3.70
	2	2.50	3.71	3.65	3.22	5.81	6.00	4.73	3.87
	3	3.31	4.82	5.10	4.35	6.65	8.11	6.30	7.11
	4	2.88	3.89	5.20	4.09	5.07	6.12	3.49	3.27
	5	3.35	4.47	5.36	4.64	7.43	7.97	5.50	5.33
	6	3.05	3.45	4.03	3.80	5.44	6.22	5.03	5.00
	7	2.73	4.16	3.29	3.14	5.66	6.46	5.94	5.79
	8	2.64	2.58	5.33	4.09	4.57	5.78	2.49	2.21
	9	3.66	4.86	3.56	2.46	4.18	4.76	3.56	3.32
	10	3.98	5.77	4.02	3.86	6.38	6.77	5.18	4.98
Soil Type 3	1	3.04	4.88	3.25	2.87	5.31	4.53	3.23	3.80
	2	3.67	5.09	5.44	4.29	6.86	6.04	4.34	4.36
	3	3.32	4.60	5.01	3.92	5.48	5.74	2.42	2.26
	4	3.00	4.68	4.64	4.16	5.53	4.06	2.62	2.57
	5	2.54	3.09	5.98	4.32	8.24	6.89	3.54	3.89
	6	4.09	6.34	3.96	2.90	6.19	5.36	3.78	3.96
	7	4.42	8.13	6.18	3.95	6.86	6.03	4.02	4.18
	8	4.15	4.63	3.72	3.96	6.61	6.32	3.49	4.00
	9	2.09	2.53	3.79	3.07	5.98	5.03	2.73	2.71
	10	3.41	4.47	3.12	2.79	3.17	4.96	3.28	2.04
Soil Type 4	1	2.54	2.93	2.03	3.01	2.84	2.20	2.20	3.29
	2	5.47	4.66	3.22	4.29	4.63	3.90	3.29	4.77
	3	4.84	5.28	2.69	3.65	4.00	3.05	1.49	2.13
	4	3.30	3.72	2.29	3.02	3.61	3.43	1.96	2.65
	5	4.00	4.45	2.85	4.09	3.98	3.42	2.52	4.04
	6	9.16	7.78	3.80	4.97	5.42	4.98	3.48	4.36
	7	3.12	3.21	1.76	2.38	2.56	2.47	1.74	2.14
	8	6.08	6.47	2.04	3.02	3.73	3.26	3.64	5.64
	9	4.16	3.53	2.28	3.29	4.43	3.07	2.20	3.33
	10	4.09	4.53	2.38	2.94	4.54	4.28	2.00	3.65

		45 St		50 St		55 St		60 St	
		25g	35g	25g	35g	25g	35g	25g	35g
Soil Type 1	1	25g	35g	3.28	2.37	1.66	2.21	1.37	2.00
	2	1.43	1.80	4.20	4.17	2.45	3.21	2.05	3.58
	3	3.28	4.49	3.90	3.78	3.04	3.18	2.50	4.03
	4	2.74	4.15	2.98	2.61	1.53	1.73	1.47	2.00
	5	2.15	2.98	2.98	3.44	1.88	1.99	1.86	2.14
	6	2.68	3.36	2.70	3.00	2.63	2.74	2.14	3.34
	7	2.68	3.45	3.04	3.15	2.19	2.52	1.30	2.09
	8	2.08	2.69	3.93	4.77	1.85	2.22	1.57	2.16
	9	3.96	4.77	5.24	5.46	4.27	3.71	4.74	5.29
	10	2.90	3.87	4.53	3.81	3.22	3.75	2.44	4.05
Soil Type 2	1	2.96	3.47	3.66	3.15	3.98	4.05	3.66	4.77
	2	2.21	2.52	4.75	5.03	4.62	3.66	4.65	5.83
	3	3.70	4.80	4.69	5.85	2.30	3.16	2.10	3.07
	4	4.06	4.45	3.37	3.59	2.75	2.76	2.10	3.09
	5	2.10	2.37	4.34	4.72	4.21	4.18	4.21	5.26
	6	3.39	4.03	4.08	5.00	3.18	3.39	2.68	3.76
	7	3.42	4.21	5.21	6.14	4.71	4.09	5.24	5.87
	8	3.22	3.80	2.48	2.60	2.62	2.79	2.01	3.12
	9	1.59	2.02	1.93	2.26	1.76	1.59	1.93	2.23
	10	1.45	2.06	4.35	4.41	5.25	4.53	4.72	5.98
Soil Type 3	1	3.12	4.18	4.14	6.56	3.17	4.43	4.85	3.86
	2	4.07	4.09	3.41	4.58	2.28	3.83	2.21	3.02
	3	3.28	3.87	2.82	2.81	1.74	2.72	2.07	2.14
	4	1.96	2.41	1.50	1.83	1.38	1.80	2.00	2.07
	5	1.61	2.34	2.67	3.52	2.17	2.74	2.39	2.19
	6	2.43	2.93	4.52	5.15	3.58	4.46	3.79	4.27
	7	3.26	3.56	3.39	4.54	1.77	2.88	2.35	3.32
	8	3.34	4.45	5.22	6.14	5.57	6.86	5.37	6.85
	9	3.50	3.78	1.98	2.56	3.42	2.09	3.94	2.45
	10	1.76	2.55	3.16	3.60	2.60	4.41	2.80	3.66
Soil Type 4	1	2.00	2.03	2.05	2.43	1.36	1.99	1.55	2.39
	2	1.78	2.54	3.07	2.17	2.22	3.09	2.94	3.03
	3	2.30	1.76	3.04	3.58	1.82	2.86	2.23	3.20
	4	1.82	1.83	2.20	2.30	2.22	2.22	2.71	3.07
	5	1.91	2.26	2.25	2.32	2.06	2.38	2.37	2.89
	6	4.06	3.56	3.31	5.02	3.22	3.55	4.10	4.72
	7	2.06	2.70	2.80	2.53	1.27	2.39	1.32	2.50
	8	2.36	3.71	2.59	2.46	1.74	2.80	2.15	2.94
	9	2.59	2.25	2.54	2.69	1.39	2.72	1.95	2.52
	10	2.03	1.78	2.69	2.67	1.63	2.62	1.60	3.20

By computing the average of the values in Table 4, the modification factors for ultimate strength and working stress levels are as follows respectively:

$$R = 3.71 \tag{7}$$

$$R_w = 5.34 \tag{8}$$

CONCLUSIONS

By using the outrigger braced frames in highrise steel structures, the interstory drift significantly decreases, resulting reduce in the total steel weight used in the structure simultaneously. Outrigger braced structures cause the ductility factor, overstrength factor and finally the modification factor to get increased in comparison with conventionally braced structures of similar width and height. Due to resulted Push-over diagrams, it could be observed that the outrigger braced structures (X-braced), demonstrate excellent hysteresis behaviour, without any decrease of strength and stiffness in nonlinear range. They are able to dissipate the input seismic energy more desirably than the similar conventionally braced frames. On the contrary, by using Λ shaped bracings in outrigger braced structures, non-desirable hysteresis behaviour occurs, resulting decrease in strength and stiffness, especially in the range of 35 ~ 55 stories high. Carried out linear and nonlinear analyses of outrigger braced frames show that the modification factor could be increased due to indeterminacy degree for these types of structures (caused by added spans or using outrigger trusses). Finally one can observe that for highrise structures with long periods of vibration, the modification factor decreases. Estimating similar modification factors for highrise structures with those of moderate or lowrise ones

Could cause the safety factor to get decreased, resulting an underestimated structural design.

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