



## Response Modification Factor of Chevron Braced Frame with Pall Friction Damper

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### A B S T R A C T

Response modification factors (R-factor) are used in current seismic building codes to reduce earthquake forces associated with seismic design level to determine force levels. In recent years, many authors have shown great interest in the development of seismic structural systems and several theoretical and experimental studies have been conducted to investigate performance of dampers, but little attention is given to the response modification factors of steel-braced frames equipped with friction damper. In this study the effect of pall friction damper, as an additional element to the structure, on the parameters of seismic behavior of steel braced frame is evaluated. Along with the pushover analysis, nonlinear dynamic analysis has been adopted to investigate the components of the behavior factor. For this purpose three steel frames of 5, 8 and 10 stories were analyzed without considering the existence of dampers, then each frame equipped with a friction damper of various slip loads-3, 8, 15, 20, 50 and 100 percentages of weight of the structure- is studied. The results show that behavior factor of chevron braced frame equipped with friction damper depends on its slip load and changing the R-factor can be an appropriate method to design steel frames equipped with friction damper.

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## 1. INTRODUCTION

For typical design of building against earthquake, resistant of the building is provided by stiffness, ductility and structural damping, so that it is possible that large amounts of energy is dissipated through localized damage or plastic hinges formed in the lateral resistant system. Performance of buildings can be increased by adding additional equipment called damper. Damper, as a tool for seismic retrofit or building in new construction, will dissipate major portion of input energy in predetermined parts, so damage to structure is reduced [1]. Systems of damper belong to the passive control category. Passive damper energy dissipation mechanisms in passive damping systems are based on friction, viscoelastic behavior, yielding of metals viscouse damper, TMD, TLD and etc. Viscoelastic dampers depend on velocity while friction and yield dampers are function of displacement. So far, lots of friction devices have been tested experimentally, e.i., Pall and Marsh [1], Aiken and Kelly [2], Grigorian and Popov [3], and many of them

have been implemented in buildings all over the world. The first friction damping system applied at the intersection of braces in an x-braced frame was introduced by Pall and used in a building in Canada. Then, this damper was used in diagonal and chevron braces namely Eaton building Palais Des Conngres of Montreal in 2000. Results of experiments carried out by Filliatrault [4] showed that this system can enhance the performance of buildings during earthquakes. Also, its high energy dissipation and damping can reduce displacement of the buildings compared to conventional rehabilitation. Patel and Gangid [5] investigated the influence of system parameters such as frequency ratio and mass ratio on the dynamic response of coupled structures. It has been observed that the frequency ratio has significant effect on the performance of the friction damper, whereas the effects of mass ratio are marginal. Also, C. L. Ng and Y. L. Xu [6] evaluated seismic response control of a building complex utilizing friction damper. They found that the concerned parameters influencing the design of optimal friction force and control efficiency include variety of earthquake excitation and differences in floor mass, story number as well as number of damper installed between the two

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buildings. Each seismic design code offers certain criteria to design structural systems. These criteria will consider type of system and its specific aspects, and are based on those variables that will determine how the system will behave. In most seismic design codes, lateral load is determined through equivalent static method on which behavior factor has a profound effect. In recent years, researches have been revealing the role of structural systems in R-factor assessment.

Akbari and Maher [7] investigated the response modification factors of steel-braced reinforced concrete framed dual systems. They found that the addition of steel  $x$  and knee braces increased the response modification factor significantly, and that the number of stories appeared to be the predominant variable. Balendar and Hung [8] found that the over-strength and the ductility factors were almost the same for inverted v-braced and split x-braced frames. They also observed that response modification factor decreased when the number of stories increased. Asgarian and Shokrgozar [9] evaluated the behavior factor of buckling restrained braced frames (BRBF) and found that the over-strength, ductility and behavior factor are decreased as the number of stories is increased. In this study the effect of friction damper on the parameters of seismic behavior of steel braced frame is evaluated and for this reason, pushover analysis, 2-D nonlinear time-history dynamic analyses was carried out using the computer program SAP2000 (Nonlinear version), developed by Computers and Structures Inc, Ibrahimbegovic and Wilson [10]. In contrast to the static method, dynamic analysis takes into account the effect of earthquakes on the structure, its dynamic properties and the effect of various vibration models.

**1. 1. Friction Damper** It is possible to use additional dampers in tall buildings in which base isolators cannot be used. Of all available methods to dissipate kinetic energy in buildings, the most widely adopted procedure is friction damper. Friction damper mechanism works based on friction between rigid bodies that clamped together with high strength steel bolts. Base metal selected for this type of damper is significant. Various materials were used for slippery surfaces, include the brake's layer on steel, steel on brass. As for bolted connections, a combination of graphite slider with bronze and other alloys can be employed on stainless steel. All friction dampers operate in the way having a part that is fixed and the other part that has to be dynamically slipping on it. Slip occurs at a certain level of force. These dampers are categorized into hysteretic dampers and have stable hysteretic loops and operate according to Columbus's friction law. Figure 1 shows some hysteresis models of friction dampers. Lee and Park [11]. As can be seen in each cycle, friction damper has higher energy dissipation in comparison to others.

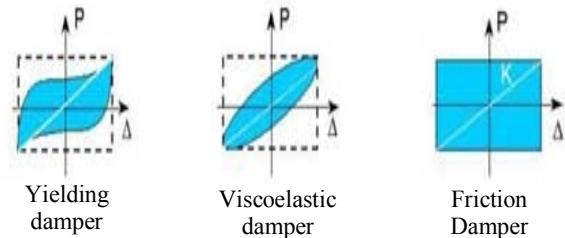


Figure 1. Hysteretic loops of dampers

It is noteworthy that the important thing about using friction dampers is how to determine the slip load in friction points that should be carefully determined by nonlinear analysis. This, should be considered carefully because the amount of force required to slip should not be so small that the friction damper starts to slip against the small forces such as wind and slight earthquake (slip load minimum), and not so large that does not slip during powerful earthquakes. This is because sliding in dampers should begin before yielding occurs in other structural members (maximum slip load). The slip load must be determined such that dissipated energy through friction action to be major portion of seismic energy. Cherry and Filliatrault [4] have been shown that variation of 20% in the optimum slip load, don't significantly alter the overall response of friction damped structure as well as optimal amount of force applied independent of the earthquake and the physical properties of frame.

**1. 2. Behavior Factor (R-factor)** Behavior factor is the ratio of the strength required to maintain structure elastic to the inelastic design strength of the structure. In other words, behavior factor is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. Figure 2 represents a typical base-shear versus roof displacement. The elastic force demand is expressed as  $V_e$ .

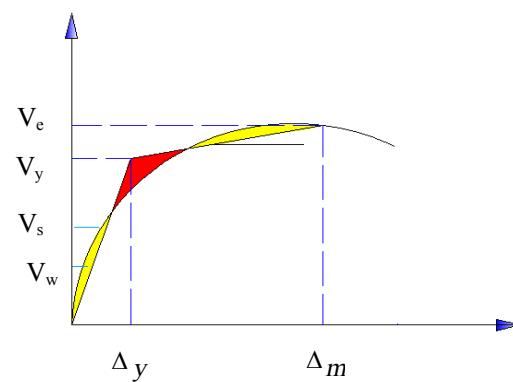


Figure 2. Capacity curve

With reference to Figure 2 in which the actual force-displacement response curve is idealized by a bilinear elastic-perfectly plastic response curve, using the Uang method, Uang [12], the parameters of behavior factor can be defined as Equation (1):

$$R_w = \frac{V_e}{V_w} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} \times \frac{V_s}{V_w} = R_\mu \cdot R_s \cdot Y \quad (1)$$

where,  $V_e$ ,  $V_y$ ,  $V_s$  and  $V_w$  correspond to the structure's elastic response strength, the idealized yield strength, the first significant yield strength and the allowable stress design strength, respectively. For structures designed using an ultimate strength method, the allowable stress factor,  $Y$ , becomes unity.

The structural ductility  $\mu$  is defined in terms of maximum structural drift and the displacement corresponding to the idealized yield strength as Equation (2):

$$\mu = \frac{\Delta_m}{\Delta_y} \quad (2)$$

The ductility reduction factor  $R_\mu$  was obtained using the structural ductility factor  $\mu$  by the procedure proposed by Nassar, Osterass and Krawinkler [13] as Equation (3, 4):

$$R_\mu = [C(\mu - 1) + 1]^{\frac{1}{C}} \quad (3)$$

$$C(T, a) = \frac{T^a}{1 + T^a} + \frac{b}{T} \quad (4)$$

where  $T$  is the main period of the structure, and the parameters  $a$  and  $b$ , depend on the strain hardening  $\alpha$ , and are shown in Table 1.

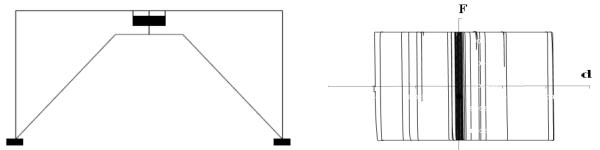
In this study we use  $\alpha=3\%$  and the value of  $a$ ,  $b$  are:  $a=0.975$  and  $b=0.36$ .

## 2. MODELING AND ASSUMPTIONS

Structural system selected in this study is steel chevron braced frame. The span and height of the frames are 5 and 3 meters, respectively. They are analyzed in 5, 8 and 10 story separately. Under the assumption that seismic responses in two perpendicular directions are independent, a two-dimensional plane frame model is used in all design analyses and seismic response simulations. All connections are considered to be rigid. After creating the model geometry of the model and loading it according to the Iranian Code of practice for Seismic Resistance Design of Building (2005), assuming the site to be in a seismic zone behavior factor of  $R=7$ , equivalent static analysis was used to analyze the model under seismic load distribution. In all of the models, the top story is 25% lighter than the others. Sections IPE, IPB and BOX are used for beams, columns and braces, respectively.

**TABLE 1.** Required parameters in Nassar and Krawinkler Equation [14]

$\alpha$	a	b
0.00	1.00	0.42
0.03	0.975	0.36
0.1	0.8	0.29



**Figure 3.** FD in Eccentric Brace with hysteretic loop[13]

**2. 1. Damper Modeling** Design of Pall friction damper is based on the amount of slip load dedicated to it. In fact, stiffness created in the structure equipped with Pall friction damper, depends on the slip load. In this study sliding loads are considered as varying percentages of structure weight so that we can form a clearer perception of the optimum load as well as making a more accurate assessment of the effect of various slip loads on the response of the structure. The weight of each building is defined according to equivalent static lateral loads for a normal building with importance factor equal to 1. For this purpose in each frame model, after designing the structure, the total weight can be obtained; then 3, 8, 15, 20, 50 and 100 percent of the weight were calculated. For all 3 frames values are distributed uniformly amongst dampers in each level. Also, damping ratio is taken 0.05 for the first few effective modes. For modeling of damper in software SAP2000, [10], the wen plastic element is used whose parameters can be adjusted to represent appropriate behavior of Columbus friction, Figure 3.

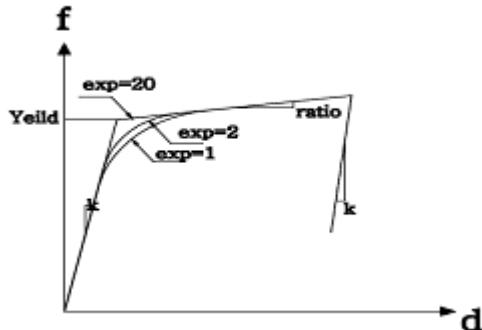
All internal deformations of wen plastic element are independent. The yielding at one degree of freedom does not affect the behavior of other deformations.

The nonlinear force-deformation relationship is given by Equation (5) [10]:

$$f = \text{ratio}(k).d + (1 - \text{ratio}) \text{yield}(z) \quad (5)$$

where  $k$  is the elastic stiffness coefficient ratio,  $\text{yield}$  is the yield force,  $\text{ratio}$  is the specified ratio of post-yield stiffness to elastic stiffness ( $k$ ), and  $z$  is an internal hysteretic variable. The initial value of  $z$  is zero, and it evolves according to the differential Equation (6) [10]:

$$\begin{aligned} \text{if } \dot{d} > 0 \Rightarrow \dot{z} &= \frac{k}{\text{yield}} d \left( 1 - |z|^{\text{exp}} \right) \\ \text{if } \dot{d} < 0 \Rightarrow \dot{z} &= \frac{k}{\text{yield}} \dot{d} \end{aligned} \quad (6)$$



**Figure 4.** Sharpness of yielding of Wen Plastic element in Sap 2000 to exp value [10]

where  $\text{exp}$  is an exponent greater than or equal to 1. Greater value of  $\text{exp}$  increases the sharpness of yielding as shown in Figure 4. The practical limit for  $\text{exp}$  is about 20, that can produce the hysteresis diagram for FD to be more realistic[10].

**2.2. Nonlinear Analysis** After the linear analysis and design of members, plastic hinges according to seismic rehabilitation of formation code were assigned to points with high possibility of formation of plastic joints. Flexural hinge (M3), shear hinge (V2) were assigned both ends and middle of beams respectively. Axial hinge (P) was assigned to the middle of brace and flexural-axial hinge (P-M3) was assigned to both ends of column.

**2.2.1. Nonlinear Static Analysis** In the beginning, models are analyzed under gravity load ( $\text{DL}+0.2\text{LL}$ ) and then a certain lateral load distribution. Lateral load distribution along the height of the structure is Equation (7, 8):

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^N W_j h_j^k} V \quad (7)$$

$$k = \begin{cases} 1 & T \leq 0.5 \\ 0.75 + 0.5T & 0.5 < T \leq 2.5 \\ 2 & T \geq 2.5 \end{cases} \quad (8)$$

where:

$W_i$  and  $h_i$ : are weight and height of the  $i_{th}$  storey.

$k$ : power that varies in different codes.

$N$ : number of classes.

In this study, the maximum relative displacement of stories is determined as a criterion to determine the level of structure damage according to regulations of 2800 code:

- 1) For frames with dominant period of less than 0.7sec:  $\delta_m < 0.025H$

- 2) For frames with dominant period of more than 0.7 sec:  $\delta_m < 0.02H$

where,  $H$  is the height of stories and  $\delta_m$  is the maximum relation displacement of stories in nonlinear analysis.

After pushover analysis obtaining the base shear-roof displacement curve, the curve is idealized with two lines. It should be noted that according to the seismic rehabilitation the area under the real capacity curve and the idealized curve should be the same and intersect each other at  $0.6V_y$ . For this purpose, a program named macro was written in excel software into which the values calculated from pushover analysis can be entered along with the acceptable percentage of error, and then the idealized capacity curve is given.

Based on idealized capacity curve, seismic properties of the structure such as ductility, overstrength and behavior factor can be calculated. The obtained behavior factor is compared to its initial value and if there is convergence error of more than 3% of the new R value, the structure must be analyzed and redesigned. The iterations go on until the obtained behavior factor in the new step is almost the same as the value in the previous step.

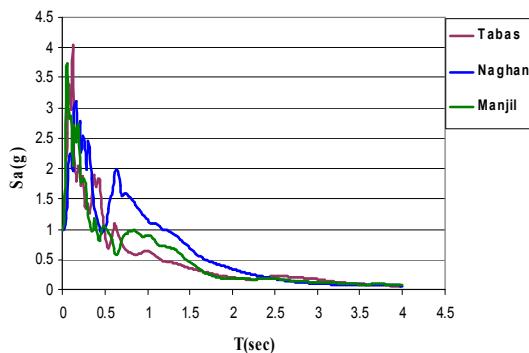
**2.2.2. Time History Analysis** In this study in order to investigate the actual behavior of the frames and ensure the accuracy and precision of the results in the static analysis, all models are analyzed under records of Manjil, Naghan and Tabas. In each model the scale factor of records is changed such that relative displacement of stories reaches the mentioned criteria in part (2.2.1).

The maximum nonlinear base shear of this time history analysis is the inelastic base shear of the structure; to gain the base shear related to the first plastic hinge formation in structure  $V_s$ , the pushover analysis was carried out. It means that the linear ultimate limit of structure in nonlinear static analysis and nonlinear dynamic analysis has been considered to be the same [9].

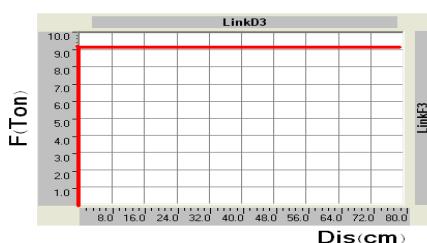
The seismic properties of the records and the response spectra are shown in Table 2 Figure 5, respectively. The soil is of type 3.

**TABLE 2.** Seismic properties of the records

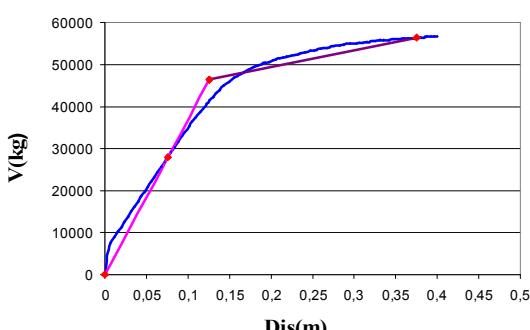
Station	Year	PGA(g)	Duration(sec)
Tabas	1978	0.846	32.82
Mangil	1990	0.41	46
Naghan	1977	0.872	22.29



**Figure 5.** Variation of spectral acceleration with period of structures



**Figure 6.** The behavior of Columbus friction in pushover analysis



**Figure 7.** Idealized capacity curve of 5-story model vs. original curve with 8% slip load

### 3. RESULTS AND DISCUSSION

**3. 1. Nonlinear Static Analysis** In order to show the nonlinear static analysis procedure, a five story frame equipped Pall friction damper with slip load of 8% weight of structure as an example is described. After

nonlinear static analysis, in order to verify the precision of modeling and accuracy of performance of dampers in SAP2000 software, and also considering same behavior of links at all stories of a frame, as example, skeleton curve of the link at the 3<sup>rd</sup> story after pushover analysis is shown in Figure 6. It can be seen that the behavior of plastic element of wen, like the behavior of Columbus friction, occurs in the ¼ of the loop.

In Figure 7 the capacity curve obtained from the pushover analysis is idealized by two lines according to the seismic rehabilitation code. It can be observed that by the aid of the Figure 7 the key points of Table 3 can be detected. The parameter of behavior factor can be determined. Using the Yang method, parameter resulting from nonlinear behavior of 5 story chevron braced framed equipped with friction damper with slip load of 8% and R=7 are listed in Table 4.

As it can be seen, the obtained behavior factor is 5.11 which is 27 percent different from the initial value. With several try and errors the structure is designed and analyzed under nonlinear static analysis again until the obtained R in the final stage is less than 3 percent different from the last value. The final seismic parameters for this structure are shown in Table 5. Also  $R_{S2} = R_{SI} \times \gamma$  is known as lateral capacity of the structure.

The results of pushover analysis of the frames are depicted in Figures 8 to 10.

Also, according to the curves, when the structure is under incremental loading before the members enter the plastic zone damper acts and slides and causes an initial stiffness in the structure.

Using the idealized curve, key points of the structure resulting from pushover analysis including the strength of the structure when the first plastic hinge is formed,  $V_s$ , and yielding strength,  $V_y$ , are determined and the seismic parameters are then obtained. In order to better evaluate the obtained coefficients, the components of R-factor are shown in Figures 11 to 14. It should be noted that the horizontal axis is the percentage of the weight of the structure which is slip load and the last point (number of 110) represents chevron frame. Also, the lateral capacity of the structure is the result of overstrength multiplied by allowable stress factor.

**TABLE 3.** The key points of 5-story model with 8% slip load

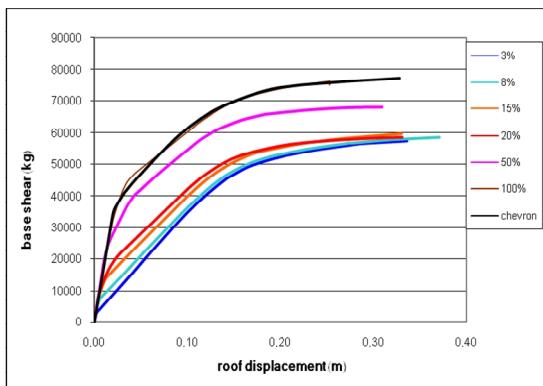
Linear analysis (R=7)		First plastic hinge		Yielding point		Ultimate point	
$V_d$ (ton)	$\delta_d$ (cm)	$V_s$ (ton)	$\delta_s$ (cm)	$V_y$ (ton)	$\delta_y$ (cm)	$V_t$ (ton)	$\delta_t$ (cm)
28.49	7.6	35.34	10.11	46.55	12.6	56.39	37.5

**TABLE 4.** The components of behavior factor of 5-story model with 8% slip load

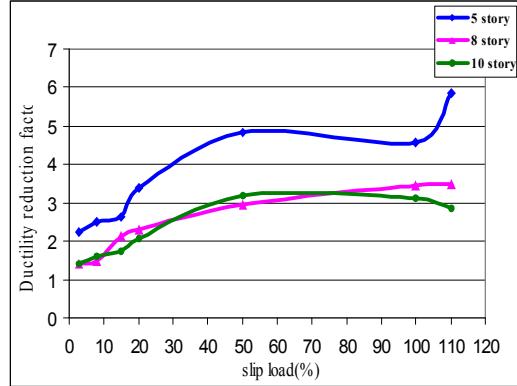
STORY	$\mu$	$R\mu$	$Rs$	$1Rs$	$\gamma$	$2Rs$	$RW$
5	2.98	2.72	1.32	1.52	1.24	1.88	5.11

**TABLE 5.** The final value of the components of behavior factor of 5-story model with 8% slip load

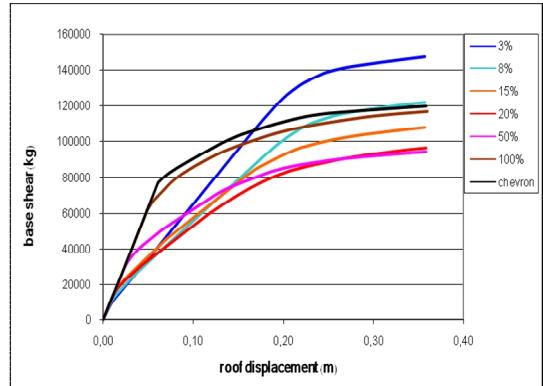
STORY	$\mu$	$R\mu$	$Rs$	$1Rs$	$\gamma$	$2Rs$	$RW$
5	2.83	2.61	1.36	1.56	1.27	1.98	5.17



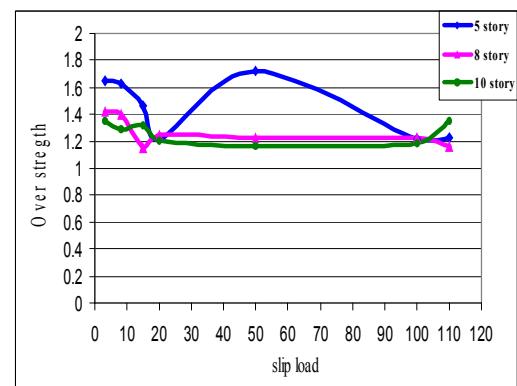
**Figure 8.** The capacity curves of 5-story frames



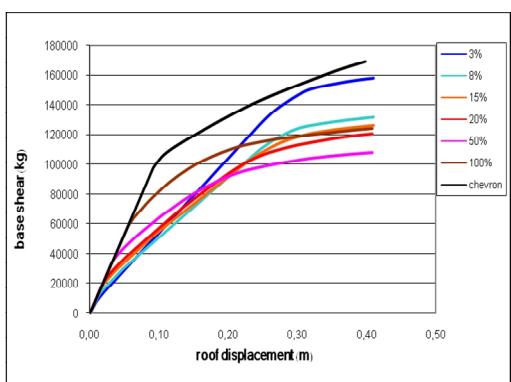
**Figure 11.** Ductility reduction factor



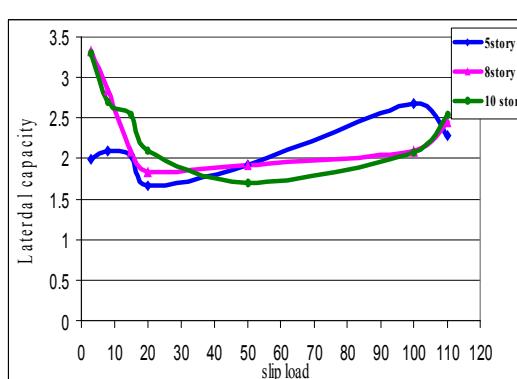
**Figure 9.** The capacity curves of 8-story frames



**Figure 12.** Over strength factor



**Figure 10.** The capacity curves of 10-story frames



**Figure 13.** Lateral capacity factor

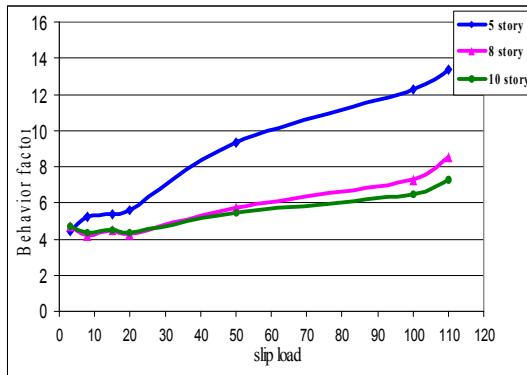


Figure 14. Behavior factor

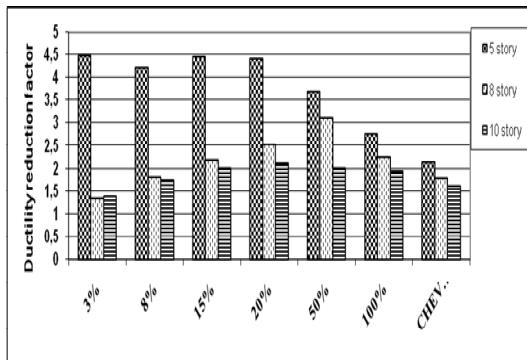


Figure 15. Ductility reduction factor from dynamic analysis

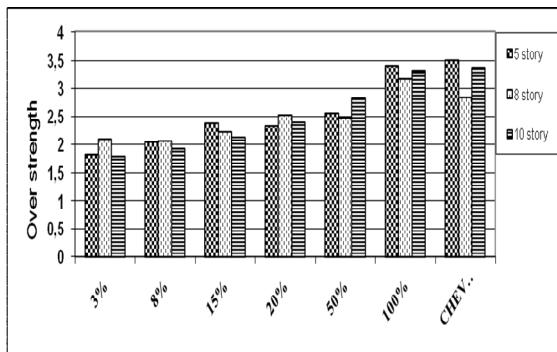


Figure 16. Over strength factor from dynamic analysis

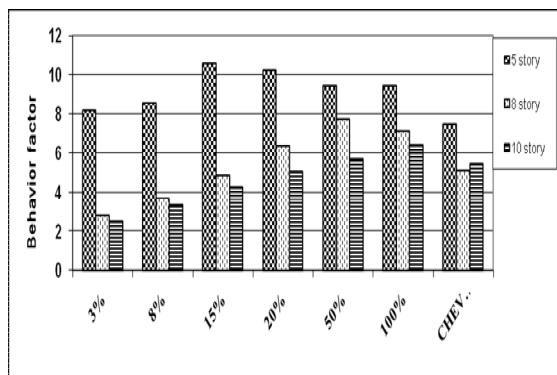


Figure 17. Behavior factor from dynamic analysis

According to the curves, the ductility of the structure in levels of 8 and 10 storey increases as slip load of friction dampers rises, but in the 5-storey model ductility increases until slip load reaches 50%, then it decreases. This is because of probable errors in idealizing pushover curve of the 5-storey frame. The error might have occurred because slip load of 50% is neither large enough to concentrate hinge formation on braces, nor so small that frame components acts before the dampers.

The over-strength of 8 and 10 storey models are not different from each other, but the lateral capacity of the mentioned structures decreases with increas in slip load until 50% and then increases. This is because of the effect of allowable stress factor. Considering the changes of ductility reduction factor and lateral capacity of the structure that have unlike trends, the variations of the R-factor against slip loads have a slight slope. Of course it is considered that the process of change of the parameters in 5- story is different from 8 and 10 story that is because of different performance of dampers in short buildings.

### 3. 2. Nonlinear Dynamic Analysis

Considering the intensity, duration and frequency content of different records, the seismic response of the structure will be different. The claim is substantiated by results of dynamic analyses under records of Tabas, Naghan and Manjil. The results show that R-factor depends on the parameters of the structures such as dominant period and also the property of the record. Therefore, using dynamic analysis the elastic resistance and plastic strength of the structure are determined and ductility reduction factor can be determined directly ( $R_d = V_e / V_y$ ), Chopra (1999).

Figures 15 to 17 show the changes in seismic parameters of the structures of 5, 8 and 10 stories with different slip loads. Ductility reduction factor of 5-story model is 4.25 at 8% slip load and reaches its maximum value of 4.5 at slip load of 15% after which it decreases reaching its lowest value for chevron-braced frame. Also, in all models the over-strength has slight rise as the slip load increases. Considering the effect of height and slip load of chevron braced frames equipped with friction damper, the dynamic behavior factor increases until a certain slip load and then decreases. It indicates that the frames have a better performance at a certain slip load known as optimum slip load [15]. These varations in R factor are not in accordance with nonlinear static analyses results. Therefore, static

analyses has not been found a suitable method for analyses structures equipped with damper.

#### 4. CONCLUSION

In this study the possibility of the presence of friction damper in chevron braced frame and its effect on the seismic parameters of the structure such as ductility reduction factor, over-strength and the behavior factor of the structure were evaluated. To do so the mentioned models with different slip loads in three different heights (5, 8 and 10 stories) are analyzed under nonlinear static and dynamic analysis (21 frames). This section presents the results of the investigation.

According to the pushover analysis:

1. Increasing the slip load of the friction damper, the displacement  $\delta_m$  decreases and the capacity curves get close to the chevron curve, such that in 100% slip load the capacity curves comply with those of the chevron.
2. In all frames, the ductility reduction factor increases and the lateral capacity decreases as the slip load increased to 50%, which results in slight increases in R-factor.

Results of dynamic analysis show:

3. With increase in the height of the frame, the behavior factor reduces, but in nonlinear static analysis this trend can be seen for slip loads of more than 50%.
4. In dynamic analysis as the slip load increases, the over-strength of the structure increases consistently while ductility increases and then decreases.
5. Considering average results of all records, it can be said that: the behavior factor of 5, 8 and 10 story models at optimum slip load reaches its highest value of 10.55, 7.71, 6.45.
6. Finally, the results show that variations of the static analyses results are not in accordance with dynamic analyses results for R factor

#### 5. REFERENCES

1. Pall, A. S., Marsh, C., "Response of friction damped braced frames". *Journal of Structural Engineering*, Vol. 108, (1982), 1313-1323.
2. Aiken, I., Kelly, S., "Earthquake simulator testing and analytical studies of two energy absorbing systems for multi-storey structures". Report No. UBC/EERC-90/03, (1990), EERC, Berekely.
3. Grigorian, C. E., Yang, T. S., and Popov, E. P., "Slotted bolted connection energy dissipaters", *Earthquake Spectra*, Vol. 9, No. 3, (1993), 491-504.
4. Filliatrault, A., and Cherry, S., "Performance evaluation of friction damped steel frames under EQ. load"; Engineering Labratory Report, Department of Civil Engineering, University of British Columbia, Vancouver, Canada, (1985).
5. Patel, C. C., and Jangid, R. S., "Dynamic response of adjacent structures connected by friction damper", *Earthquakes and Structures*, Vol. 2, No. 2, (2011), 149-169.
6. Ng, C. L. and Xu, Y. L., "Seismic response control of a building complex utilizing passive friction damper: Experimental investigation", *Earthquake Engineering & Structural Dynamics*, Vol. 35, No. 6, (2006), 657-677.
7. Maher, M. R., and Akbari, R.; "Seismic Behavior Factor, R, for Steel X-Braced and Knee- Braced RC Buildings"; *Engineering Structures*; Vol. 25, No. 15, (2003), 1505-1513.
8. Balendra, T., and Huang, X., "Over strength and ductility factors for steel frames designed according to BS 5950"; *Journal of Structural Engineering, ASCE*; Vol. 129, No. 8, (2003), 1019-1035.
9. Asgarian, B. and H. Shokrgozar, "BRBF response modification factor", *Journal of Constructional Steel Research*, Vol. 65, No. 2, (2009), 290-298.
10. Ibrahimbegovic, A. and Wilson, E., "Simple numerical algorithms for the mode superposition analysis of linear structural systems with non-proportional damping", *Computers & Structures*, Vol. 33, No. 2, (1989), 523-531.
11. Lee, S. K., Park, J. H., Moon, B. W., Min, K. W., Lee, S. H., and Kim, J., "Design of a bracing-friction damper system for seismic retrofitting", *Smart Structures and Systems*, Vol. 4, No. 5, (2008), 685-696.
12. Uang, C., "Seismic force reduction and displacement amplification factors", in 10th World Conference on Earthquake Engineering, Madrid. Vol. 10, (1992), 5875-5880.
13. Vaseghi Amiri, J., Navayinia B., and Navaei, "Evaluation of performance of eccentric braced frame with friction damper", *Structural Engineering and Mechanics*, Vol. 39, No. 5, (2011), 717-732.
14. Nassar, A. A., Osterass, J. D. and Krawinkler, H., "Seismic design based on strength and ductility demands," Process Tenth world Conference on Earthquake Engineering, Vol. 10, (1992), 5861-5866.
15. Esmaeiltabar, P., Vaseghi, J., "Investigate of the behavior factor of the chevron steel braced frame equiped with pall friction damper", Thesis, Master of Science, Noshirvani University of Technology, Babol, Iran, (2011).

## Response Modification Factor of Chevron Braced Frame with Pall Friction Damper

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در آینه نامه‌های لرزه‌ای کنونی از ضریب رفتار جهت تعیین نیروهای طرح لرزه‌ای استفاده می‌شود. در سالهای اخیر، محققان مطالعات تئوریک و آزمایشگاهی گسترده‌ای راجع به سیستم‌های سازه‌ای لرزه‌ای، از جمله میراگرها، انجام داده اند، ولی توجه کمتری به بحث ضریب رفتار قاب مجهر به میراگر شده است. در این تحقیق تأثیر میراگر اصطکاکی پال، به عنوان یک ابزار جاذب انرژی، بر روی پارامترهای رفتار لرزه‌ای قاب فولادی مهاربندی ارزیابی شده است. به منظور تعیین مولفه‌های ضریب رفتار، علاوه بر تحلیل استاتیکی غیرخطی تحلیل دینامیکی غیرخطی نیز انجام شده است. بدین منظور سه قاب ۵، ۸ و ۱۰ طبقه بدون میراگر بررسی شدند، سپس به هر قاب میراگر با بارهای لغزش مقاومت (۳، ۲۰، ۱۵، ۸) درصد وزن سازه که به طور یکسان بین میراگر تقسیم شده است) اضافه شده و مورد مطالعه قرار گرفتند. نتایج نشان می‌دهند که ضریب رفتار قاب شورون مجهر به میراگر اصطکاکی به بار لغزش آن بستگی داشته و با تغییر ضریب رفتار می‌توان یک روش مناسب برای تعیین بار لغزش بهینه میراگر بیان نمود.

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