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New Steel Divergent Braced Frame Systems for Strengthening of Reinforced Concrete Frames

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ABSTRACT

The seismic strengthening methods are very important in earthquake-prone countries. Steel divergent bracing with replaceable link beam tied in steel frame and embedded in a concrete frame is a new method for a concrete frame strengthening. That is low cost and easy repairable after an earthquake. In this article six concrete frame strengthening methods have been investigated, including X-bracing, reverse chevron bracing, divergent bracing with concrete link beam, divergent bracing with steel link beam connected to steel columns in the steel frame, divergent bracing with steel link beam connected to the steel frame and with steel columns between those two, divergent bracing with steel link beam connected to the steel frame. All strengthening models are attached to concrete frames by a steel frame surrounding them . These models are investigated by ETABS and PERFORM-3D softwares. In concrete frame strengthed by steel divergent bracing with steel link beam, the base shear is decreased about 20%, steel consumption decreased to 40% in 6-story, and 15% in 14- and 20-story compared to X-bracing, and the existing to allowable stress ratio decreased to 50% in 6-story, to 40% in14-story and 35% in 20-story. As the structure's height is increased, the interaction between the frame and the brace, and the lateral force in the frames increased. Nonlinear static and dynamic analysis have shown more elastic hardness, ductility, behavior coefficient, and base shear in strengthed concrete frame with divergent bracing with steel link beam connected to the steel frame model than others.

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1. INTRODUCTION

Some existing buildings need to be strengthed because of extra story construction, poor performance, design deficiencies, usage and regulation changes [1-3]. In comparison of strengthening methods by the shear wall and steel braces, steel bracing method was more preferred for its easy implementation, low weight, stiffness increase and lateral displacement decrease [4]. Bracing systems include convergent, divergent and unbuckling bracings, and different bracing dampers. Proper hysteretic plasticity will be achieved by design to prevent early buckling of braces [5]. Reinforced concrete buildings needed appropriate strength, stiffness, and ductility to resist properly high intensity earthquakes. The ductility of reinforced concrete structures depended on the details of its components and the location of the plastic hinges. Coaxial bracing was a very custom lateral resistant system, but its hysteresis behavior showed rapid collapse under cyclic loads due to local buckling. Buckling-resistant braces show appropriate hysteresis energy dissipation even under large axial deformation [6]. Studies indicated that conventional seismic design codes were more conservative than Code No. 360. In other words, design of steel moment frames with conventional design codes leads to stronger beams and columns. This phenomenon could be due to controlling lateral interstory drift and weak beam-strong column criterion in moment frames that control final design results of these structures [7]. The effect of three indirect strengthening models of concrete frames by steel braces, was tested cyclically (Figure 1). The experimental results

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revealed the buildings strength improvement compared to numerical values [8]. The reliability of reinforced concrete frames with steel braces was investigated by Liu et al. [9]. The modeling and the steel brace failure mode are shown in Figure 2. Experimental results showed the effect of strengthening by steel bracing models on the resistance, ductility, and energy dissipation capacity of the strengthened concrete structures [9]. In addition, investigations showed that the braced structures had more stable hysteresis behavior, more energy dissipation capacity, and significantly, lesser damage than the case strengthened with shear panels [10].

The seismic performance of the unbuckling braced frame with a direct connection to the reinforced concrete frame showed that it increased stiffness and ductility to the level of proper seismic performance. The energy dissipation was about 5 times, and the lateral load capacity was about 4 times larger than the concrete frame [11]. Performance evaluation of the existent reinforced concrete frame and strengthened reinforced concrete frame showed that the strengthened system worked suitably [12]. Based on seismic damage analysis, the concrete frames strengthed by steel x-bracing reduced the possibility of damage more than chevron braces [13]. Investigation on concrete frames structural performance with 16 different bracing models showed better performance, and the different bracing showed a significant seismic performance effect on the buildings [14]. Past studies showed strengthened concrete buildings by external braces had more ductility than the strengthened concrete buildings with internal bracings [15]. The results of the seismic performance analysis of steel building revealed that seismic design codes generally had more limited criteria than the improvement codes [16]. Investigation and comparison of seismic behavior of divergent braced frames with vertical and knee joint beams in strengthened concrete frame, showed that knee bracing systems were more effective than vertical bracing, in increasing stiffness, reducing lateral displacement and stress ratio in concrete frame members; but, drastically reduced ductility [17]. Also, the short link beams in experiments showed more resistance than the code specific design. Seismic damage assessment of reinforced concrete frames conducted by convergent steel brace, it showed strengthened concrete frames with steel braces could reduce the possibility of damage [18]. Nateghi and Vatandoost [19] experimentally investigated steel braces for strengthening concrete frames with three indirect bracing models and observed that the the structural resistance was increased in comparison to the results obtained by numerical analysis. In this paper, a new method of braced steel frame with divergent bracing is introduced to strengthen the existing reinforced concrete frame, with the aim of rapid return of the building to service with minimal changes in the existing concrete frame. This method of seismic strengthening, with the ductile behavior of the beam as a shear fuse, will reduce or not damage other members of the structure at different levels of risk. Limiting damage to replaceable shear fuses reduces the time and cost of repairing the structure and returning the building to service quickly. Due to the extensive studies of recent years on different methods of concrete frame strengthening with convergent and divergent braces, in this article it is necessary to evaluate and compare the performance of these methods and the new method of concrete frame strengthening with steel braced frame with divergent bracing.

Figure 1. Strengthening of a concrete frame by indirect bracing [8]

Figure 2. Concrete frame strengthed by steel brace and its fracture mode [9]

2. MODELING AND STUDY METHODOLOGY

In this article, six models of strengthened reinforced concrete frames with four bays in X-direction, five bays in Y-direction, span length of 5 meters and story height of 3.5 meters in 6-story , 14-story and 20-story were considered as a short, middle, and high-rise structures, respectively. Figure 3 presents the 3-D view of reinforced concrete frames strengthened by steel braces. The cross-sections of beams and columns were considered by Iran profiles sections (IPE). The number and location of bracing in the structures showed that the structures did not need local strengthening, and the performance of frames was similar to each other. In all strengthened models, the steel frame was made by studs that were attached to the existing concrete frame. The studs dimensions for connecting the bracing system to the existent concrete frame were considered based on braces cross-section. In this study, seven models including reinforced concrete frame (M0), concrete frame with X-convergent bracing (M1), concrete frame with reverse chevron bracing (M2), concrete frame with divergent bracing with concrete link beam (M3), concrete frame with divergent bracing with steel link beam connected to steel columns in the steel frame (M4), concrete frame with divergent bracing with steel link beam connected to the steel frame and with steel columns between those two (M5) and concrete frame with divergent bracing with steel link beam connected to the steel frame (M6) were considered. All strengthening models were attached to concrete frames by a steel frame surrounding them. The strengthened models are shown in Figure 4. The dead load of the floor, the partition load and the live load of floors were considered to be 4000 N/m^2, 1000 N/m^2, and 2000 N/m^2, respectively [20]. Also, the compressive strength of concrete and yield strength of steel were considered as 24 MPa and 400 MPa, respectively. Linear dynamic analysis and design of models were performed by ETABS software. The Iranian seismic code(4th edition) was applied for seismic loading [21]. In this article, it was assumed that unstrengthened concrete frames could sustain only 40% of seismic load. The ACI 318-14 and AISC 360-10 have been used for the reinforced concrete and steel members design, respectively [22, 23]. The nonlinear static and dynamic analysis were done by PERFORM-3D software. The link beam length and geometric properties were considered to shear behaviour of link beam that increase the energy dissipation and ductility. All braces and columns of steel frames were designed basis on their capacity.

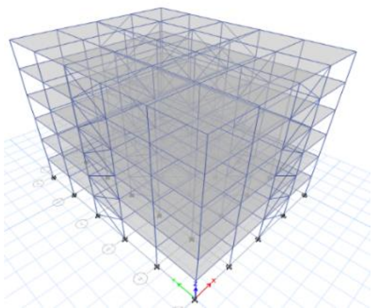

Figure 3. 3D-view of models

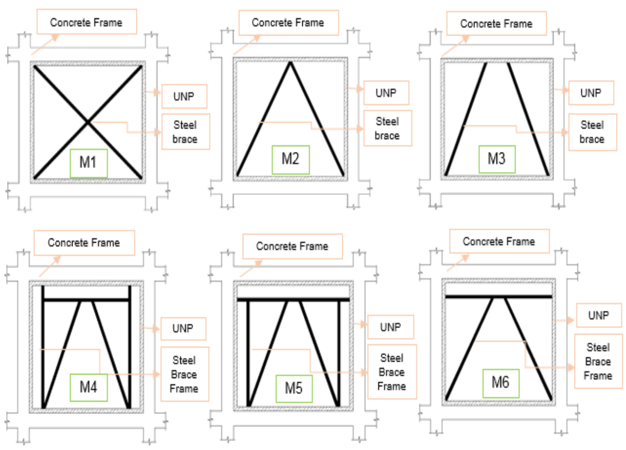

Figure 4. Strengthed concrete frame models

3. VALIDATION OF STRUCTURAL MODELS

As a steel bracing is commonly used to enhance the seismic shear strength of existent reinforced concrete frames, numerous experimental studies have been conducted on strengthened concrete frame systems with steel braces. In order to control the accuracy of the results and make sure the modeling and analysis process of frames; in this study, an experimental strengthened model of a single-span and single-story concrete frame of 1:3 in scale was used to validate the structural models. Then the lateral load capacity curves of the models were investigated. In experimental setup, a link beam in a divergent bracing system was attached directly to the concrete frame and were subjected to a lateral reciprocal resistance capacity. Figure 5 presented the details of the reinforced concrete frame and its experimental setup. The length of the link beam was assumed to be 50mm in the reinforced concrete frame system, and the steel bracing system was considered by normal strength steel. Reinforced concrete frame with compressive strength of 28 MPa of class C20/25 was considered in the experimental model. In this experimental specimen, the link beam and steel sections were considered to be IPE100 and M16 bolts and ST37 steel connectors. Experimental models of the strengthened concrete frame were modeled in PERFORM-3D software. The FEMA element was used for modeling of the concrete beams and columns [24]. Nonlinear static analysis was done on analytical model. In the nonlinear static analysis method, the lateral load was increased, such that the displacement at a given point exceeds from the code specified limit. Thus, the deformation and forces were constantly monitored by load increment. The control point in analytical model was considered to be the mass center of the beam. Also based on FEMA356, lateral load distribution at height was considered based on the static linear method.

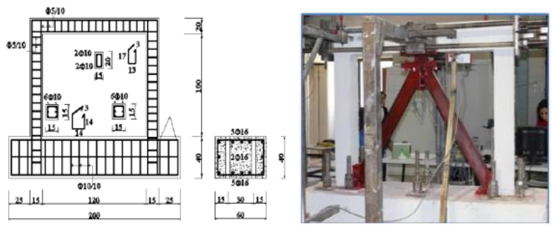

Figure 5. Details of reinforced concrete frame and experimental setup [24]

In Figure 6 the base shear-displacement curves of experimental and analytical models by the PERFORM-3D software was presented. The curves of this figure were shown insignificant difference between the analytical model results and experimental model results. So the PERFORM-3D software could be used for investigating the behavior of models.

4. LINEAR DYNAMIC ANALYSIS RESULTS

The most important reason of structural weakness was gravity and lateral load, or lateral displacement. So by determining the structural weakness reason, a suitable strengthening system could be suggested. If the structural weakness was due to lateral load and displacement, it could be strengthened by different bracing systems. But if it was due to the gravity loads, the bracing systems could not simply eliminate the structural weakness and unsuitable members should be retrofitted. Figure 7 showed the baseshear comparison of M1-M6 models in 6-, 14- and 20-story. The base shear coefficient was the sum of the lateral seismic force divided by the weight of the structure.

As shown in Figure 7, M1 model showed the maximum base shear coefficient compared to other models, Investigation of the new proposed models including M4, M5 and M6 revealed the reduction of the base shear coefficient in this models compared to the M1 model. The base shear of the M4, M5 and M6 models were reduced to 18, 15 and 10%, respectively in 6-story; and to 15, 16 and 17%, respectively in 14-story and finally in the 20-story the base shear of M4, M5 and M6 were reduced to 17%. Thus, by reducing the base shear coefficient in the this models, the lateral force on the structure in the floors was also reduced and the stress of the structural members were reduced. Tables 1-3 summarized the average of the computational stress to allowable stress ratio of the concrete columns in 6-,14- and 20-story models compared with M0 model. In this tables, a negative sign indicate a decrement and a positive sign indicate an increment of computational stress to allowable stress ratio compared with M0 model.

Generally, except in the columns of the braced spans, the computational stress-to-allowable stress ratio of the concrete frame members in strengthened models members was reduced about to 50%. The maximum computational-to-allowable stress ratio, in the M1 to M6 models, were observed on the upper floors. That was found to decrease to 50, 40 and 30% in the 6-, 14- and 20-story, respectively. It could be observed by results that the computational stress to allowable stress ratio in concrete columns in 6-story has decreased to 30% in

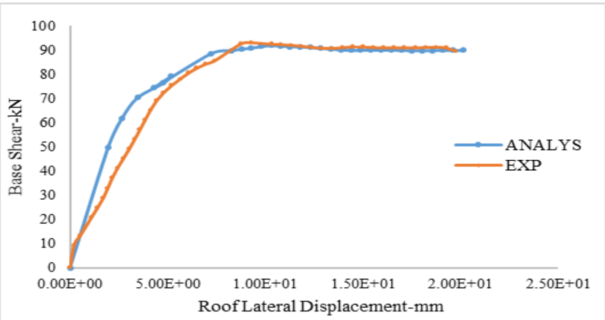

Figure 6. Base shear-displacement curves of experimental and analytical models

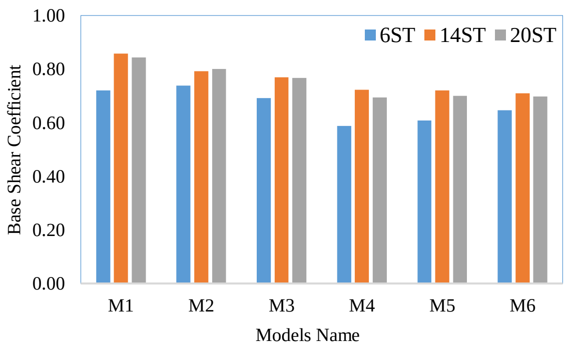

Figure 7. Base shear coefficient. 6-, 14- and 20-st.

TABLE 1. computational-to-allow stress ratios of concrete columns in 6-story models compared to M0 model

Story	M1	M2	M3	M4	M5	M6
1-2	-28	-26	-26	-26	-17	-26
3-4	-3	19	-42	-43	-37	-38
5-6	-50	-3	-51	-45	-47	-46

TABLE 2. computational-to-allow stress ratios of concrete columns in 14-story models compared to M0 model

Story	M1	M2	M3	M4	M5	M6
1-4	-8	-9	-9	-13	-7	-10
5-8	-25	-25	-26	-29	-24	-25
9-11	-33	-33	-34	-35	-31	-27
12-14	-38	-39	-39	-42	-41	-40

TABLE 3. computational-to-allow stress ratios of concrete columns in 20-story models compared to M0 model

Story	M1	M2	M3	M4	M5	M6
1-4	8	7	7	2	6	6
5-8	4	-1	2	-3	1	2
9-12	-2	-2	-3	-7	-3	-3
13-16	-26	-25	-27	-30	-26	-26
17-20	-32	-33	-34	-36	-34	-34

lower floors and to 50% in the upper floors, and in 14-story, it decreased to 10% in lower floors, 30% in the middle floors and 40% in the upper floors. Similarly, in 20-story models, this stress ratio has decreased to 20% in the lower floors and up to 30% in the middle and upper floors. Generally, braced systems for reducing the stress ratio is more effective in 6-story. By increasing the height of the structure, it will be less effective in the stress ratio reduction. Figures 8-10 show the models' relative lateral displacement-relative height by linear dynamic analysis.

The results of this tables showed a significant decrease in the relative lateral displacement of the M1 to M6 models in compared with the M0 model. The relative lateral displacement of the structures was reduced to the regulation limits. The interaction of the existent concrete frame and the bracing systems of the M1-M6 models were investigated. Figures 11-13 showed the interconnection curves of the existent concrete frame and the strengthening systems added to the concrete frame in the 6-,14-, and 20-story. In these Figures, the letter F

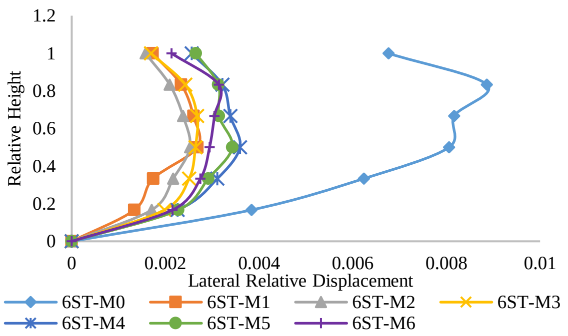

Figure 8. Lateral relative displacement, 6-st models

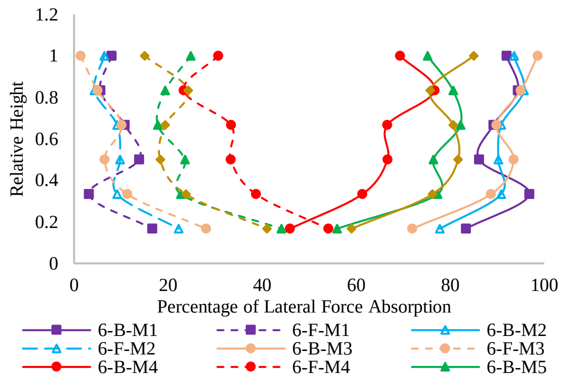

Figure 11. Concrete frame and brace Interaction,6-st.

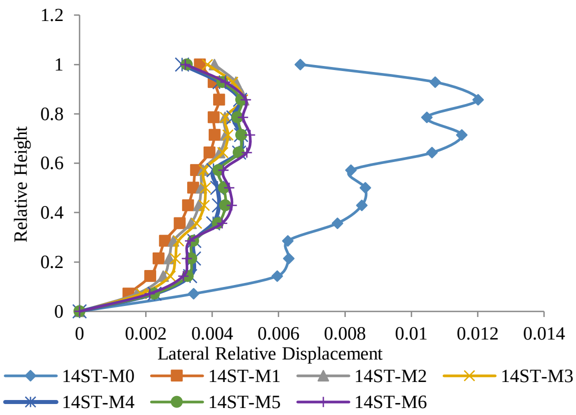

Figure 9. Lateral relative displacement, 14-st.

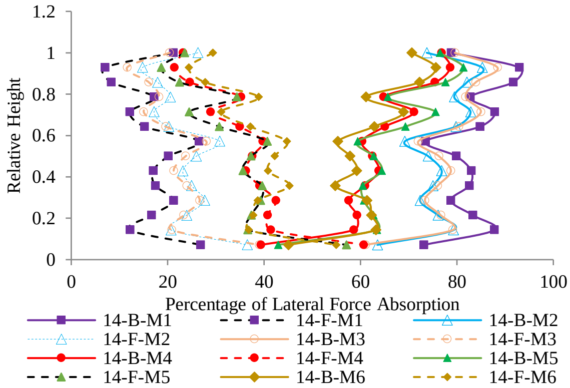

Figure 12. Concrete frame and brace Interaction14-st.

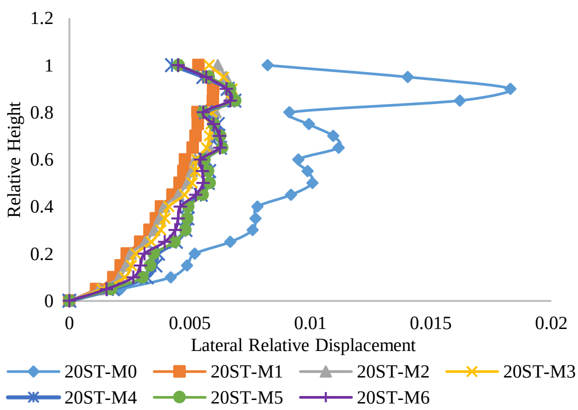

Figure 10. Lateral relative displacement, 20-st.

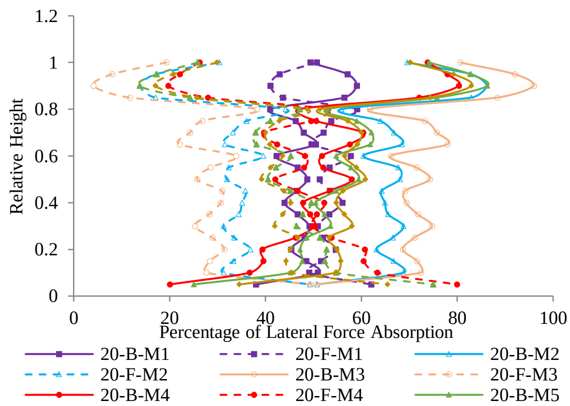

Figure 13. Concrete frame and brace Interaction20-st.

means the absorption of the lateral load by the concrete frame and letter B means the absorption of the lateral load by the braces. The concrete frame in strengthed models, showed more lateral resistance capacity than unstrengthed models. The concrete frame participation in the lateral resistance system in M1, M2 and M3 models was less than M4, M5 and M6 models.

Therefore, the concrete frame of M4, M5 and M6 models showed better ductility and performance than other models. The results in M1 to M3 models of 6-story models showed more than 85% of the lateral load is supported by the braces and on average 15% of the lateral load is supported by the concrete frame. Also in 14-story, results revealed that 80% of the lateral load is supported by the braces and on average 20% of the lateral load is supported by the concrete frame, while in M6 model of 6- and 14-story these percentages were obtained 60% and 40%, respectively. In 20-story 65% of the lateral load is supported by the braces and on average 35% of the lateral load is supported by the concrete frame while in M6 model of 6- and 14-story were obtained 50% and 50%, respectively. Undoubtedly, the columns and foundations of the braced spans need to be strengthed. However, the 85% participation of the braces in the lateral load has made their strengthening costly, This is despite the fact that the capacity of other members of the concrete frame and columns has not been well used.

5. NONLINEAR STATIC ANALYSIS RESULTS (PUSHOVER)

The static nonlinear behavior was evaluated based on FEMA356 by PERFORM-3D software. The structural members were defined by its geometrical, material, and plastic hinge properties and were modeled by FEMA356 elements in PERFORM-3D software [25-27]. Figures 14-16 illustrated the base shear-roof lateral displacemen results concluded by nonlinear static analysis of the M0-M6 models in the 6-, 14-, and 20-story, respectively. The M1, M2 and M6 models in the 6-story and 14-story increased the unstrengthed concrete frame stiffness almost 2 times and in the 20-story models all strengthened models increased the stiffness of the unstrengthed concrete frame by about 1.5 times. Based on the pushover curves results, all the models improved the nonlinear performance and ductility of the existent concrete frame. Comparison of the pushover curves obtained by nonlinear static analysis of the studied models showed the M6 model concluded more ductility than other models.

6. NONLINEAR DYNAMIC ANALYSIS RESULTS

The models' seismic performance was evaluated by nonlinear dynamic analysis with a strong far-fault record set. These records were obtained from FEMA P695 and listed in Table 4. PERFORM-3D software was used for performing the nonlinear dynamic analysis of the models [28].

Relative lateral displacement results were concluded by nonlinear dynamic analysis in relative height of the 6-, 14- and 20-story models, are shown in Figures 17-19.

It was revealed by Figure 17 that the maximum relative lateral displacement was equal to 0.035 and in the M0 model in the 6-story, and the minimum relative

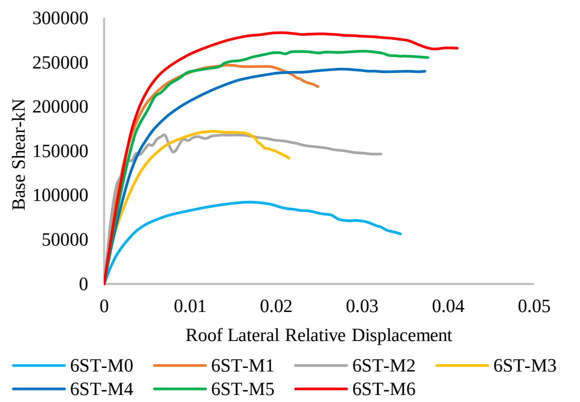

Figure 14. Pushover curves of 6-story models

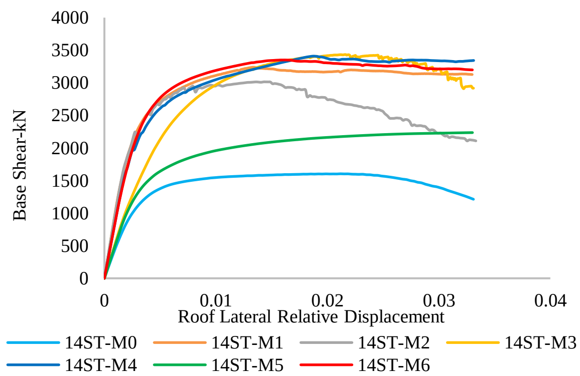

Figure 15. Pushover curves of 14-story models

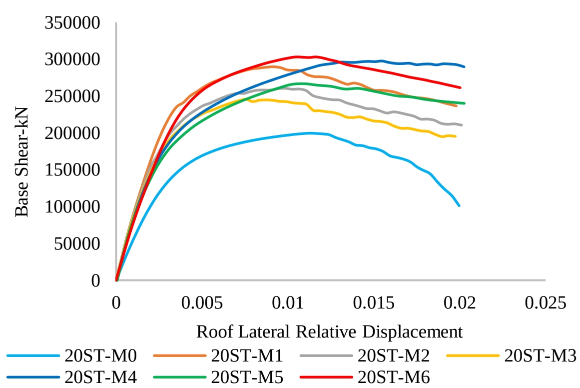

Figure 16. Pushover curves of 20-story models

TABLE 4. Records for nonlinear dynamic analysis

Record number	Record Name	Record year
1	CHI-CHI, Taiwan	1999
2	Imperial Valley	1979
3	Manjil, IRAN	1990
4	Tabas, IRAN	1978
5	Kobe, JAPAN	1995

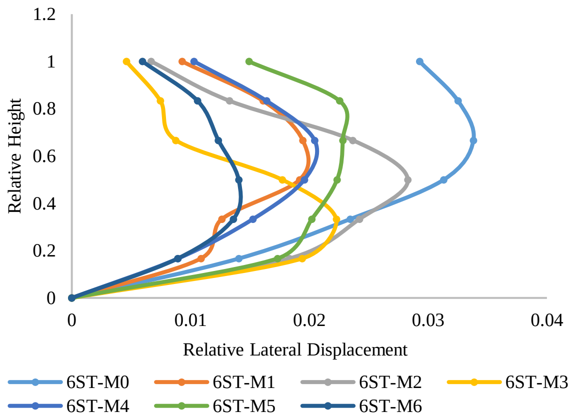

Figure 17. Relative lateral displacement by nonlinear dynamic analysis in 6-story

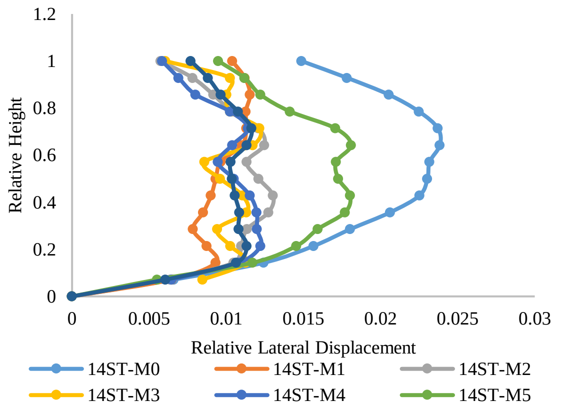

Figure 18. Relative lateral displacement by nonlinear dynamic analysis in 14-story

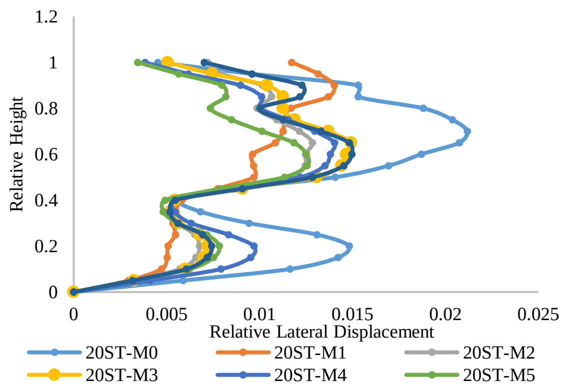

Figure 19. Relative lateral displacement by nonlinear dynamic analysis in 20-story

lateral displacement have seen in M6 model in the 6-story and was equal to 0.015. The relative lateral displacement in the M6 model was decreased 25% compared to M1 model and was decreased 60% compared to M0 model. In 14-story models, the results of the relative lateral displacement in the M1 to M4 and M6 models were close to each other. The relative lateral displacement in the M6 model was reduced about 60% compared to the M0 model. Also, in 20-story models, lateral displacement reduction was significant in all M1 to M6 models compared to M0 model. However, in the M1-M6 models, the lateral displacement reduction in the middle storys , to 40% compared to the M0 model, was significant.

In Figures 20-22 was presented the maximum lateral force to weight ratio of the 6-, 14 and 20-story models by nonlinear dynamic analysis. In 6-story models, the M0 and M5 models showed the maximum lateral force to weight ratio on all floors.

However, the lateral force to weight ratio of the M6 model was about 50% less than other models. Also, in the 14-story models, M3 and M6 models showed the maximum and minimum lateral force to weight ratio respectively, and their difference was about 40% on all floors. As shown in Figure 22, all the 20-story models revealed close results together, but in the M6 model, that is almost 15% less than other models.

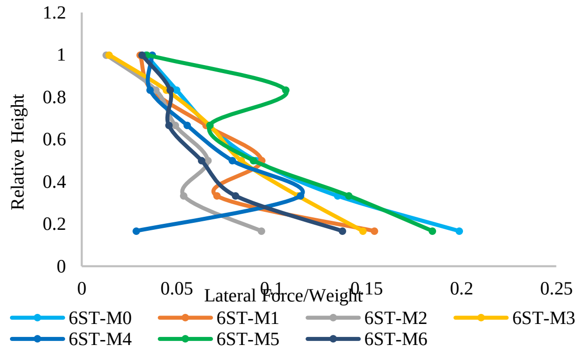

Figure 20. Lateral force/ weight by nonlinear dynamic analysis in 6-story

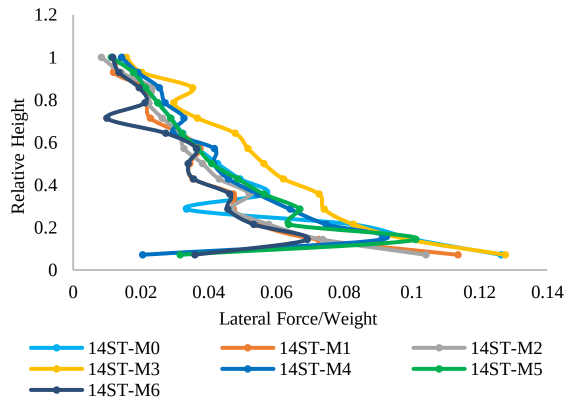

Figure 21. Lateral force/ weight by nonlinear dynamic analysis in 14-story

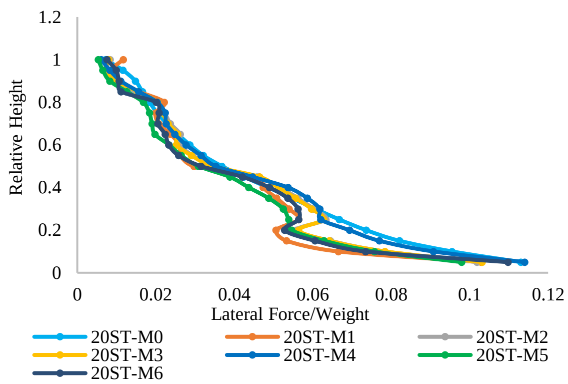

Figure 22. Lateral force/ weight by nonlinear dynamic analysis in 20-story

7. INVESTIGATIONS OF BEHAVIOR COEFFICIENT

New proposed strengthening models were not specifically mentioned in the design codes. Therefore, determining the structural design parameters was very important. The most important method for calculating the behavior coefficient was ductility theory method, energy method, capacity spectrum method, and Uang ductility method. In this article, the Uang plasticity method was used to determine the multiplication of behavior. In 1994, a new formulation was presented for the behavior coefficient known as the Uang method [29,30]. The behavior coefficient was defined as:

$$R=R_R.R_\mu.\Omega_0 \tag{1}$$

where R_R was the indefinite coefficient. In the structural models were presented for strengthening due to the high indefinite degrees, this value was considered to be maximum, i.e. 1. Due to the structural ductility, a significant amount of earthquake energy was depreciated by hysteresis behavior, which depends on the total ductility value of the structure. The energy dissipation capacity could be reduced by the elastic design force to the yield resistance (R_μ). When the plastic hinge was formed in one of the structural members from a design standpoint, the operation of the structural resistance was finished; but this phenomenon was not the end of the ultimate structural resistance, Because the deformed plastic member could still absorb the input energy to the point of destruction. The process of forming plastic hinges continued with increasing external force, and more hinges were created in the structure. Therefore, the resistance of that structure after the first plastic hinge deformation (V_s) to the mechanism step (V_y) was called as incremental resistance and could be expressed as:

$$\Omega_0 = V_y/V_s \tag{2}$$

It has been observed that as the structural height was increased, the behavioral coefficients (R) and elastic stiffness (K_y) of the models were decrease. The elastic stiffness of 6-, 14- and 20-story models are shown in Figure 23. In 6-story models, dual resistant concrete frame systems in M1, M5 and M6 models revealed the maximum elastic stiffness. In the 14-story the M3, M4 and M6 models and in 20-story the M1, M2, M3, and M6 models showed the maximum elastic stiffness. Figure 24 showed the behavior coefficient of 6-, 14- and 20-story models. The M2, M3 and M6 models in 6-story models, the M1, M2 and M6 models in 14-story and the M2 and M6 models in 20-story revealed the maximum behavior coefficient values. Figures 25-27 concluded base design shear (V_{design}) and structural yield shear (V_y) in 6-, 14- and 20-story models. In dual structural strengthened concrete frame models, the results showed that in the M6 model yielding base shear (V_y) was more about 1.25

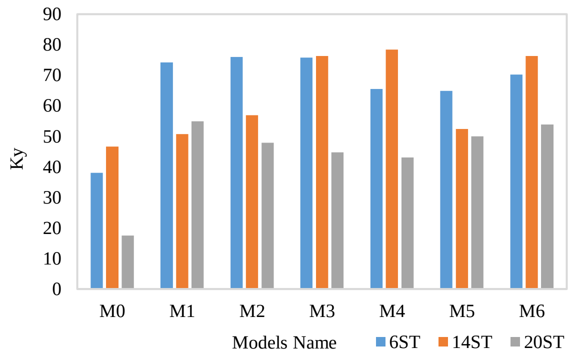

Figure 23. Elastic stiffness 6-,14-, 20-st.

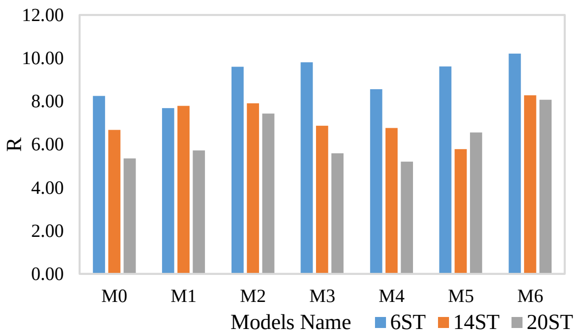

Figure 24. Behavior coefficient.6-,14-,20-st.

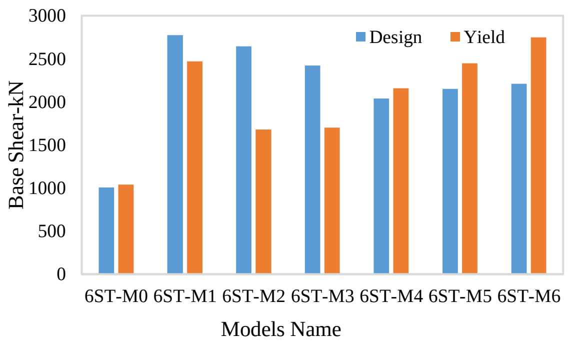

Figure 25. Design baseshear and yield base shear in 6st.

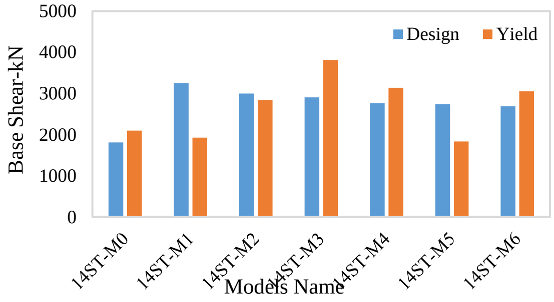

Figure 26. Design baseshear and yield base shear in 14st.

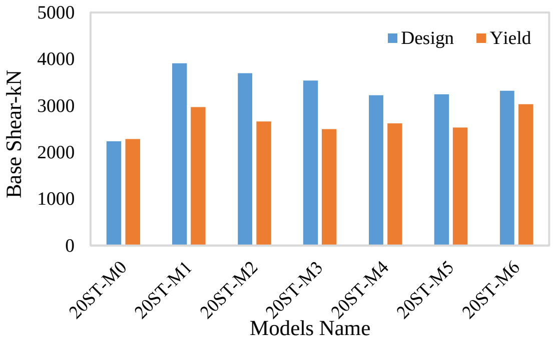

Figure 27. Design baseshear and yield base shear in 20st.

times (Figure 25), 1.10 times (Figure 26), and 1 time (Figure 27) than design base shear (V_{Design}) respectively in 6-story, 14-story and 20-story.

8. CONCLUSION

The steel braced frame systems as a new strengthening system of the concrete frame were introduced and compared with other strengthening systems. These systems were remarkable for their structural fuse, economical cost, and good structural behavior. The performance of concrete frame with six indirect strengthed models were investigated, including steel X-reverse chevron and eccentric braced frame (EBF) with cocrete link beam, and steel EBF braced frame by steel column connection to concrete beam, steel EBF braced frame by steel beam connection to the concrete column and concrete frame with EBF bracing by steel beam connection to a concrete column in 6-, 14-, and 20-story. At first, a linear dynamic analysis was conducted on the structures. Based on linear dynamic analysis results, the determination of the type of structural weakness to provide a suitable strengthening solution was necessary. If the structural weakness was due to lateral force and displacement, it could be strengthened by using different bracing models. Then, using the nonlinear static and dynamic analysis, the seismic performance of the structures were investigated. According to these analysis, the conclusions could be summarized as follows:

In strengthened concrete frame models in 6-, 14- and 20-story there was significant increase in the stiffness and base shear of models compared to that of the M0 concrete frame. But the results of strengthened concrete frame models with steel braced frame in 6-, 14- and 20-story structures showed about a 20% reduction in base shear compared to the strengthened concrete frame with X-bracing by linear dynamic analysis.

The new strengthening method of a concrete frame with a steel braced frame reduced steel consumption compared to reinforced concrete frame strengthened with X-bracing. The reduction in steel consumption was 30, 10, and 15%, in 6-, 14- and 20-story models, respectively. Also, these models reduced significantly the computational stress-to-allowable stress ratio of concrete frame members. As such the maximum stress ratio reduction was observed in the upper stories of models which is 50, 40, and 35% in 6-, 14- and 20-story models, respectively and this value is more than the concrete frame with X-bracing.

Linear dynamic analysis results showed that the relative lateral displacement of the existent concrete frame decreased by using strengthening models. The relative lateral displacement of the concrete frame strengthed by a steel braced frame was more than the concrete frame strengthed by X-bracing. Generally, by increasing the models height, relative lateral displacement results of strengthened models are approximately similar .

Interaction of concrete frame and strengthening system shows that the lateral load absorbed by the strengthening system and the existent concrete frame was on average 80 and 20%, respectively in 6-story models; 40 and 60%, respectively in 14-story models; and 50 and 50%, respectively in 20-story.

Nonlinear static analysis curves showed that all strengthened models had better performance and energy absorption than the concrete frame. However concrete frame strengthened with the steel EBF braced frame model had more energy absorption and hardness than the other models which was about 3 and 2 times of concrete frame, respectively.

By increasing the height of models, the behavior coefficient of a dual concrete frame with bracing strengthening systems decreased. Concrete frame with EBF bracing and with steel beam connection to the concrete column model showed more behavioral coefficient than other models. This model increases the ductility of the concrete frame by about 2 times and elastic hardness of concrete frame by 3 times in 6-story models and by 2 times in 14-story and 20-story models. Also, this model showed that the yielding base shear was 1.25, 1.10 and1 times of design base shear. So in high-rise structures the proposed models nonlinear behavior were not appropiate.

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Persian Abstract

چکیده

طراحی لرزه ای سازه ها و مقاوم سازی ساختمان های موجود از جمله مسائل مهم در کشورهای لرزه خیز است. مقاوم سازی قاب خمشی بتن‌آرمه با مهاربند واگرا کلاف شده در قاب فولادی و تعبیه آن در قاب بتنی با امکان تعویض پذیری تیر پیوند، روش جدید مورد مطالعه در این مقاله است. مطالعات انجام شده در این مقاله نشان داده است که عملکرد اندرکنشی و رفتار لرزه ای قاب بتنی با مهاربند واگرای الحاقی منجر به رفتار لرزه‌ای بهتر قاب بتنی در مقایسه با مقاوم‌سازی قاب بتنی با مهاربند همگرا است. بازگشت سازه به خدمت رسانی در سطح خطر متوسط در این روش، به علت منحصر شدن آسیب ها به تیر پیوند، سریع و کم هزینه است. . در این مقاله ٦ روش مقاوم‌سازی قاب بتنی، شامل مهاربند ضربدری، مهاربند به شکل٨، مهاربند واگرا با تیر پیوند بتنی، مهاربند واگرا با تیر پیوند فولادی، متصل به دو ستون فولادی در کلاف، مهاربند واگرا و تیر پیوند فولادی متصل به کلاف، با دو ستون فولادی بین کلاف و تیر فولادی، و مهاربند واگرا و تیر پیوند فولادی متصل به کلاف در سازه های ٦ طبقه، ١٤طبقه و ٢٠طبقه مورد مطالعه قرار گرفته است. همه مدل‌ها دارای کلاف فولادی محاط کننده‌ی مهاربند و متصل به قاب بتنی است. نتایج تحلیل دینامیکی خطی، در مدل قاب بتنی مقاوم سازی شده با تیر فولادی با مهاربند واگرا و اتصال تیر فولادی به ستون بتنی نسبت به مدل قاب بتنی مقاوم سازی شده با مهاربند ضربدری، کاهش برش پایه تا ٢٠ درصد و کاهش مصرف فولاد در سازه ٦ طبقه تا ٤٠ درصد و در سازه ١٤ و ٢٠ طبقه تا ١٥درصد را نشان داده است. در همه مدل های مقاوم سازی در این مقاله، کاهش قابل ملاحظه در تغییرمکان جانبی نسبی و تنش در اعضای قاب بتنی مشاهده شده است، بطوریکه کاهش تنش در مدل های مقاوم سازی شده با تیر فولادی و مهاربند واگرا در مدل‌های ٦ طبقه تا ٥٠ درصد، ١٤ طبقه ٤٠درصد و ٢٠ طبقه تا ٣٥درصد است. همچنین با افزایش ارتفاع سازه، اندرکنش قاب و مهاربند افزایش یافته، جذب نیروی جانبی در قاب افزایش و در مهاربند کاهش یافته است. بررسی رفتار غیرخطی استاتیکی و دینامیکی مدل ها با نرم افزار پرفرم، مدل قاب بتنی با تیر فولادی مقاوم سازی شده با مهاربند واگرا و اتصال تیر فولادی به ستون بتنی را در سازه‌های ٦ ، ١٤ و ٢٠ طبقه با سختی الاستیک، شکل پذیری، ضریب رفتار و برش پایه تسلیم بالاتری نسبت به بقیه مدل‌ها نشان داده است.