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

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1. INTRODUCTION

Some existing buildings need to be strengthened 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...
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 strengthened 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.

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/m², 1000 N/m², and 2000 N/m², 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].

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 bracing. 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.
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 basesshear 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

| 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|

| 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|

| 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
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 strengthened models showed more lateral resistance capacity than unstrengthened 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 concrete frame. Also in 14-story, results revealed that 80% of the lateral load is supported by the braces and on average 15% of the lateral load is supported by the concrete frame. 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 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 strengthened. 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 displacement 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 unstrengthened concrete frame stiffness almost 2 times and in the 20-story models all strengthened models increased the stiffness of the unstrengthened 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
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        |

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 stories, 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.
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:

\[ R = R_b R_p \Omega_0 \]  

where \( R_b \) 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 (\( R_p \)). 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:

\[ \Omega_0 = \frac{V_y}{V_s} \]  

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
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 strengthened by a steel braced frame was more than the concrete frame strengthened 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 and 1 times of design base shear. So in high-rise structures the proposed models nonlinear behavior were not appropriate.
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چکیده
طراحی لرزه‌ای سازه‌ها و مقاوم سازی ساختمان‌های موجود از جمله مسائل مهم در کشورهای لرزه خیز است. مقاوم
سازی قاب خمشی بتن‌آزمایی با مهاربند واگرا و کلاف شده در قاب فولادی و تعمیق آن در مقاوم‌سازی قاب با
مهندسی همگرا است. 

یک کلاس سازه به شکل سیری در سطح خطر متوسط در این روش، به عنوان محور شدن نسبی ها به تیر پیوند
سریع و کم هزینه است. در این مقاله 8 روش مقاوم‌سازی قاب بتنی شامل: مهاربند ضربدری، مهاربند و شکل 8,
مهاربند واگرا با تیر پیوند، مهاربند واگرا با تیر پیوند فولادی، مصالح به دو ستون فولادی در کلاف، مهاربند,
اکرما و تیر پیوند فولادی متصل به کلاف، با دو ستون فولادی بین کلاف و تیر فولادی، مهاربند واگرا و تیر
پیوند فولادی متصل به کلاف در نوسازه‌های 6 طبقه، 14 طبقه و 20 طبقه مورد مطالعه قرار گرفت است. همچنین
دارای کلاف فولادی محاط کننده 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 می‌تواند در نوسازه‌های
14 طبقه و 20 طبقه رشد 15 درصد و در سازه‌های 6 طبقه و 14 طبقه و 20 طبقه رشد 40 درصد را نشان داده است. در همه
مدل‌های مقاوم سازی در این مقاله، کاهش قابل ملاحظه‌ای در تغییرات کاهش نسبی و تنش در اعضای قاب و نشان
نتیجه‌گیری کاهش نسبی و تنش در اعضای قاب و نشان داده است. 

بررسی رفتار غیرخطی استاتیکی و افزایش ضریب تنش در نوسازه‌های 6, 14 و 20 طبقه با سختی استاتیکی,
شکل ضربدری، ضریب تیر پیوند و برخورد با استاتیکی تایید شد. در این مقاله، مدل قاب بتنی با تیر فولادی
مقاوم سازی شده با مهاربند واگرا و اتصال تیر فولادی به ستون می‌تواند در نوسازه‌های 6, 14 و 20 طبقه
داده است.