sciendo

CIVIL AND ENVIRONMENTAL ENGINEERING REPORTS

E-ISSN 2450-8594

CEER 2020; 30 (3): 0068-0093
DOI: 10.2478/ceer-2020-0034
Original Research Article

APPLICATION OF BUCKLING RESTRAINED BRACES TO UPGRADE VERTICAL STIFFNESS OF EXISTING RC FRAMES

Ferdinand NIYONYUNGU[1], Ehsan NOROOZINEJAD FARSANGI[2*]
[1] School of Civil engineering, Lanzhou Jiaotong University, Lanzhou 730070, China
[2] Faculty of Civil and Surveying Engineering, Graduate University of Advanced Technology, Kerman, Iran

Abstract

In this paper, based on the RC frame structure of an industrial building, the finite element model of the structure is developed, according to the Chinese code for seismic design of buildings [9]. Considering the lack of seismic performance, the buckling restrained brace (BRB) is adopted for seismic retrofitting, and various configurations of buckling restrained support are considered for reinforcement. The elastic response spectrum analysis (RSA) and direct integration nonlinear time history analyses (NL-TH) are carried out for the frame structure before and after reinforcement using ETABS finite element software. From the joints displacement, inter-story displacement, inter-story shear force, acceleration, energy dissipation, and other aspects of the seismic response of the strengthened structure and the non-strengthened structure, the comparison has been made. The effect of buckling restrained support and common support on the existing building structure is verified through analytical modeling. After reinforcement, there is a 40%, 39.3%, 40%, 36.4%, and 38.3% reduction in the first period of vibration after the building is strengthened by inverted BRB, V BRB, two-story BRB, single BRB, and

[*] Corresponding author: Faculty of Civil and Surveying Engineering, Graduate University of Advanced Technology, noroozinejad@kgut.ac.ir

ordinary steel braces, respectively. Strengthening of the structure by buckling restrained braces and ordinary steel braces both decrease the original building displacement by more than 50% from the first to the fourth floor. Under severe earthquakes, the use of BRB reduced the column shear by 46.6%; similarly, the incorporation of ordinary steel braces reduced the column shear by 4.72%. It is concluded that using buckling restrained braces will increase the vertical stiffness of the structure to a very high extent.

Keywords: retrofitting, vertical stiffness, frame structure, buckling restrained braces, nonlinear analysis

1. INTRODUCTION

In China, due to many factors such as current earthquake resistance, urban development, strength, and durability requirements, the seismic requirements of buildings are revised regularly to adapt to the safety, durability, and practicality of the current structure[1]. Similarly, buildings designed for specific uses are sometimes used for other purposes and require re-inspection according to specifications. The revision of the design code results in the existing structure not meeting the requirements of the current code, sometimes due to differences between current methods, materials and construction techniques, and older methods. With the economic and security challenges brought by demolition and reconstruction, the reinforcement of existing structures has become increasingly important in China and the world.

In previous earthquakes that occurred in China, research has shown that many existing reinforced concrete frame structures suffered serious damage and even complete collapse [12,17,24,26]. This has called for the importance of revising the existing seismic design codes, which was done in a different subsequent period. The existing structures with insufficient seismic performance according to the current codes have potential safety hazards and need to be reinforced according to the current specifications, because demolition will be an expensive alternative [19].

There are two main classes of building strengthening methods: strengthening of individual members and strengthening of the whole structure. strengthening the individual structural members is extremely labor-intensive and it is not the preferred method over the strengthening of the whole structure.

At present, the seismic strengthening technology of reinforced concrete frame structures is still focused on improving the bearing capacity and ductility of structural members, dissipating seismic energy through the energy-dissipation capacity of structural members themselves, and ultimately achieving the goal of seismic fortification. Seismic strengthening methods of reinforced concrete frame structures mainly include the methods listed below [6].

Increasing the sections of elements: This method is the most effective and direct way to improve the bearing capacity and stability of structural components. Using the reliable connection between the new and old concrete and reinforcement[7], the structural members after reinforcement work together, the mechanical performance is good, the cost of reinforcement is low, the construction is convenient, and the quality problems such as reinforcement corrosion, protective layer shedding and local stress cracks existing in the concrete column itself can be repaired together. It has been widely used in practical engineering. However, this method is associated with many drawbacks, like moisture during maintenance, difficulty in ensuring bonds between new and old elements, additional bending, shear and torsion in structural elements, and increase in member stiffness which will affect the seismic response of the structure.

Coating with a steel sleeve method consists of pasting steel plates on the outer surface of original concrete columns or beams to make them work in coordination with the original structural members [4]. This method includes wet and dry methods. This method can improve the stiffness and bearing capacity of the members but it also has many shortcomings, and construction is difficult. If the dry method is used, the steel sleeve and the original concrete structure will not be closely combined because of the construction errors. If the wet method is used, the bonding of the cement layer with the original structure surface carbonized concrete will produce many hidden dangers. Besides, the applicable conditions for the method of wrapping steel sleeves on variable cross-section columns are also limited.

The prestressing method that had gained popularity primarily in the strengthening of existing bridge structures [20] consists of unbonded prestressing which is defined as a system in which the post-tensioning tendons or bars are located outside the concrete cross-section and the prestressing forces are transmitted to the girder through end anchorages, deviators, or saddles. The prestressing method can effectively improve the bearing capacity of structural members by applying external prestressing [2], and can be applied to the reinforcement of structural members in large-span and heavy-duty workshops [21]. It was also proven to effectively repair reinforced concrete structural members with large deformation [14], so it has been widely used in industrial buildings. However, the construction technology of this method is complex, the quality of the project is difficult to guarantee, the maintenance is difficult in the later stage, and the prestressing force will gradually decrease, which will weaken the reinforcement effect of the structure. In addition, the reinforcement method can only be carried out outside the structure members, occupying a certain space and affecting the appearance of the structure members.

Reinforcement with the CFRP method uses the tensile and shear strength characteristics of carbon fiber sheets to reinforce concrete structural members [25]. Carbon fiber sheets are bonded to the treated concrete surface through adhesives to form a tough composite layer to reinforce the tensile area of the structure. This method has been proven to increase stiffness and structural performance not only under service, but also under extreme loading conditions [1]. This method requires that carbon fiber sheets should not be exposed to sunlight, and that limited ambient temperature should be maintained during the period of use [13,23,27]. This method also requires high construction requirements. The concrete surface of CFRP needs to be smooth and clean, and it needs timely protection. It has also found that the overall strength of the beams strengthened by using this method that can be negatively affected in proper placement of stirrups is not assured [23], and the construction quality is difficult to guarantee. During later period of use, the environmental requirements are higher, and it should not be exposed to high temperature, high humidity, corrosion, and other harsh environments.

Changing the load transferring path method is mainly based on the overall structure, through the overall conformity check of the structure, adding support points to the beam members with insufficient bearing capacity, reducing the span of the beam, reducing the mid-span bending moment[22], etc.; the side is provided with a diagonal bracing to shorten the calculated height of the column, thereby changing the reverse bending point of the column and reducing the internal force of the column. The biggest disadvantage of this method is that there is a negative bending moment at the support point after changing the calculation span of the beam, and it is easy to cause negative bending moment damage at the support point; after adding the support point to the frame column, it is easy to form short columns, which is not conducive to the seismic resistance of the original structure.

In addition, there are chemical grouting methods, replacement concrete reinforcement methods, reinforcing bar planting methods, concrete reinforcement methods, and so on. These traditional reinforcement methods have made important contributions to structural reinforcement. At the same time, with the accumulation of reinforcement engineering experience, people continue to explore technology, and the quality defects of traditional reinforcement technology also exist. Gradually emerged.

Problems such as the requirement of high construction technology, difficulty guaranteeing construction quality, and poor durability after reinforcement, affect the use and application of traditional reinforcement methods, and also bring some hidden dangers to existing reinforcement projects. Therefore, exploring new reinforcement methods is an urgent problem which has important scientific value and engineering significance.

Buckling restrained braces as a new type of energy dissipation device has been widely used around the world, especially in high seismic intensity regions. This seismic device acts as a structural fuse by being the first to enter the yield state under moderate and strong earthquakes that protect other structural members. Due to the stability consideration of ordinary steel braces, steel braces will require large cross-sectional areas which in turn will increase the stiffness and seismic effect of the structure. Buckling restrained braces can avoid this kind of phenomenon and satisfy the requirement of bearing capacity and deformation of the structure without greatly increasing the stiffness of the structure.
This paper examines the applicability of BRB in modifying the vertical stiffness irregularity of reinforced concrete frame structures that suffer from a weak story mechanism.

2. CHARACTERISTICS OF BUCKLING RESTRAINED BRACES

After the failure of braced frame structures due to brace buckling, significant research has been carried out since the 1970s to address this problem, and the most significant progress of buckling restrained braces was made in the last two decades. The key characteristic of today's used type of buckling restrained brace is the axial load going through the steel core while sufficient buckling resistance is provided in another component which is a mostly concrete mortar (Fig. 1).

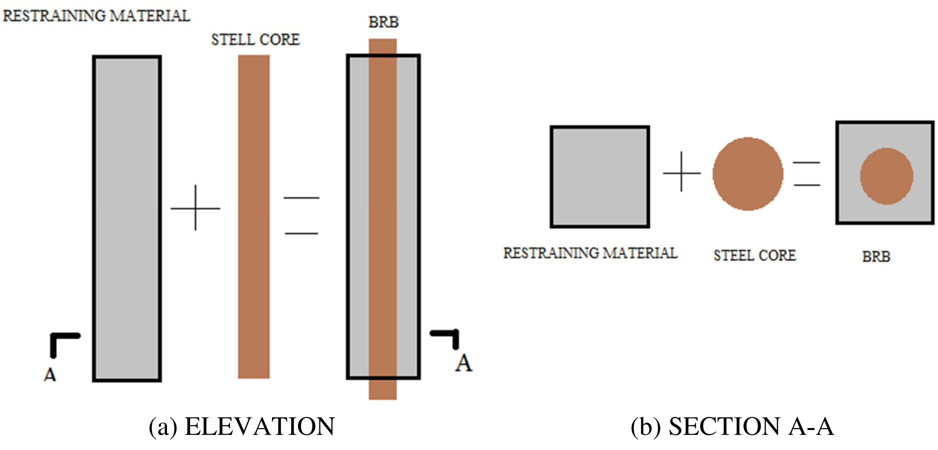
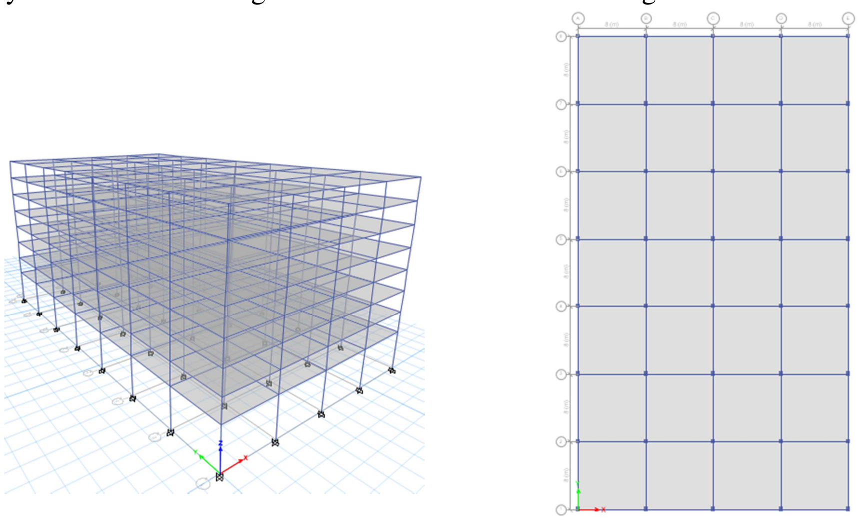


Fig. 1. A typical BRB configuration

The performance of BRB is most importantly determined by the yield strength and geometry of the yielding section. The Chinese code gives different ranges of material types used and the relationship between length and cross-sectional area of the yielding section of BRB [9]. With their full hysteresis loops in both tension and compression, BRB has good seismic energy dissipation characteristics, which are not the case for concentric braces. The compression strength of BRB is a bit larger than the tensile strength due to friction between the core section and the mortar. This friction is due to dilation of the core under compression. In contrast, concentric braces, although having good performance under tension, experiences buckling in compression which results in quick loss of compression strength upon repeated cycles, and this will impede energy dissipation compared to BRB.

3. ENGINEERING MODEL

A typical 8-story moment-resisting RC frame (Fig. 2) with a service life of 50 years was selected for this study. The frame was designed based on the seismic design requirements of GB50011-2001 and evaluated for the requirements of GB50011-2010. The building has plan dimensions of 32.0m × 56.0m with seven, 8.0m bays in both orthogonal directions, and a total height of 26.5m with the first floor 5.5m in height and 3.0m for the remaining floors. The structural system for the building consists of RC moment-resisting frames.



(a) (b)

Fig. 2. Engineering Model, (a) 3D view, (b) Plan view

The concrete grade used for structural members is assumed to be 20MPa, which corresponds to the modulus of elasticity of 2.779×10E7KN/m2 ,and the steel reinforcement grade is S225, which corresponds to the modulus of elasticity of

2×10E8KN/m2. Reinforcement details and cross-section dimensions of structural members are given in Table 1.

Table 1. Size and reinforcement details

Structural Elements	Size	Reinforcement details
Beams	250×400mm	Top Reinforcement : $4\phi18+2\phi16$ Bottom Reinforcement : $3\phi18$ Confinement: $\phi8/250$
Columns	500×500mm	Longitudinal Rebars: $8\phi20$ Confinement: $\phi8/250$
Slabs	200mm	Middle Reinforcement: $\phi8/120$ Y, $10\phi150$ X Edge Reinforcement: $8\phi200$

P-M-M hinge and one component plastic moment hinge are considered as nonlinear characteristics for columns and beams respectively in the modeling of the existing structure. The plastic hinges are defined according to FEMA356. The axial force for columns and shear force for beams is due to the combination of loads according to GB50011-2001. Seismic loads were also determined with respect to the same GB50011-2001. Connections between columns and beams are modeled as fixed joints and although there is a little influence of the floor slabs to the strength and stiffness of the beams, it is ignored in this paper. Rigid foundation with no uplift, fixed based columns, and rigid beams are also assumed in this analytical modeling.

The building is located in a region with a seismic intensity of 8.0, class III with a design base acceleration of 0.2g. The design live load is 0.5KN/m2 for roof slabs and 4KN/m2 for other floor slabs, the basic wind load is 0.65KN/m2, and the snow load is 0.3KN/m2. The characteristic ground period is 0.45 sec according to GB50011-2010.

The building was first designed as a 40.0m×32.0m, 6-floor building and after the construction, 16.0m×32.0m of area was added and the building height was increased by adding two floors. This called for the diagnosis of the new building and reinforcement where needed.

To check the reinforcement requirements of the original structure, the modal decomposition response spectrum and modal analysis of the structure were performed by using ETABS software [6]. This paper uses the complete quadratic combination (CQC) mode combination method and the square root of the sum of the squares (SRSS) direction combination method, so as to conform to the corresponding provisions of Chinese code. The damping ratio of 5% is used in this paper.

According to the site conditions of the actual project and referring to the Chinese code for seismic design of structures, it can be seen that the influence coefficient of the frequent earthquakes in the 8-degree area is 0.16.

According to the code, the seismic influence coefficient is calculated by the following formulas:

$$\left.\begin{aligned} &\alpha=\left(\frac{T_g}{T}\right)^{\gamma}\eta_2\alpha_{\max}, T_g \prec T \prec 5T_g \\ &\alpha=0.2\gamma\eta_2-\eta_2-\eta_1 T-5T_g\alpha_{\max}, 5T_g \prec T \prec 6.0 \\ &\eta_2=1+\frac{0.05-\zeta}{0.05+1.6\zeta} \\ &\gamma=0.9+\frac{0.05-\zeta}{0.3+6\zeta} \end{aligned}\right\} \tag{3.1}$$

According to the code, the seismic effect is calculated by CQC method as follows:

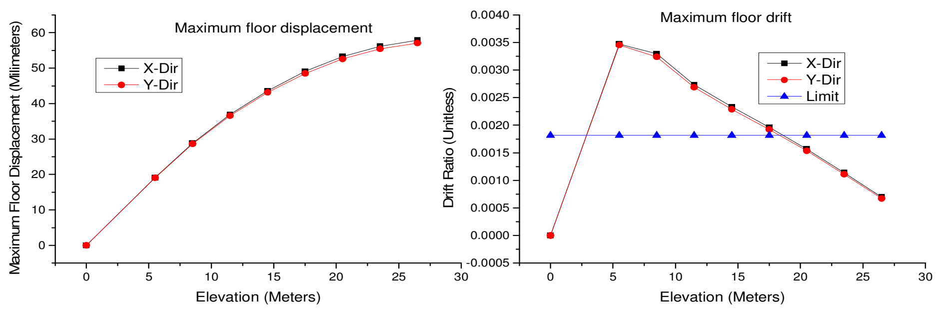
$$\left.\begin{aligned} &S_{EK}=\sqrt{\sum_{j=1}^{m}\sum_{k=1}^{m}\rho_{jk}S_jS_k} \\ &\rho_{jk}=\frac{8\zeta_j\zeta_k\left(1+\lambda_T\right)\lambda_T^{1.5}}{\left(1-\lambda_T^2\right)^2+4\zeta_j\zeta_k\left(1+\lambda_T\right)^2\lambda_T} \end{aligned}\right\} \tag{3.2}$$

Where, S_{EK} is the standard value of seismic action effect considering the torsion effect. S_j, S_k are standard effects for j and k modes. ρ_{jk} refers to the coupling coefficient of J mode and K mode. λ_T Refers to the ratio of the natural period of K mode and j mode. ζ_j, ζ_k refers to the damping ratio of the structure under J and K modes.According to the results from modal analysis under frequent earthquakes shown in table 2, the period ratio, which is the ratio between the torsional period and the translation period in the Y direction, was equal to 0.913, and was greater than 0.9, which is the maximum allowed by the code, therefore, the building will experience torsional effect under earthquake loads.

Table 2. Modal periods of the original building

Modal	Original Building	
	Period(sec.)	direction
1	2.043	Y Translation
2	2.022	X Translation
3	1.866	Torsion
4	0.614	Y Translation
5	0.608	X Translation
6	0.562	Torsion

The maximum story displacement (Figure 3-a) and drifts (Figure 3-b) from response spectrum analysis show that the stories from one to five need to be strengthened since their drift ratio is greater than the maximum allowed by the code. Under medium and strong earthquakes, the frame structure will fail before entering the plastic stage due to excessive displacement.



(a) (b)

Fig. 3. Displacement response characteristics. a. floor displacement, b. Floor drifts

According to the Chinese code, the lateral stiffness of the story must be equal or greater than the maximum of 70% of the lateral stiffness of the adjacent upper story and 80% of the average lateral stiffness of the lateral stiffness of the adjacent three stories. From response spectrum analysis results (Fig. 4), the building has an irregularity due to the lateral rigidity of the first story. Under medium and strong earthquakes, the frame structure will fail due to the weak first floor.

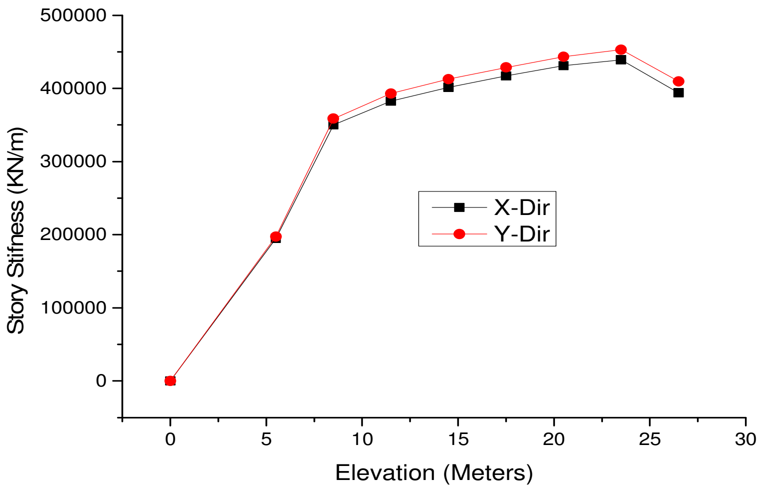


Fig. 4. lateral stiffness of the original structure

The results from the response spectrum of the original structure show that the strengthening objective is the reinforcement of the horizontal (torsional) irregularity and vertical irregularity of lateral rigidity. For the first floor, the required braces are designed by considering the maximum difference between stiffness and drift requirements, and for the second to the fifth floor, the required braces are designed according to drift requirements in buckling restrained braces parameters.

4. BUCKLING RESTRAINED BRACES PARAMETERS

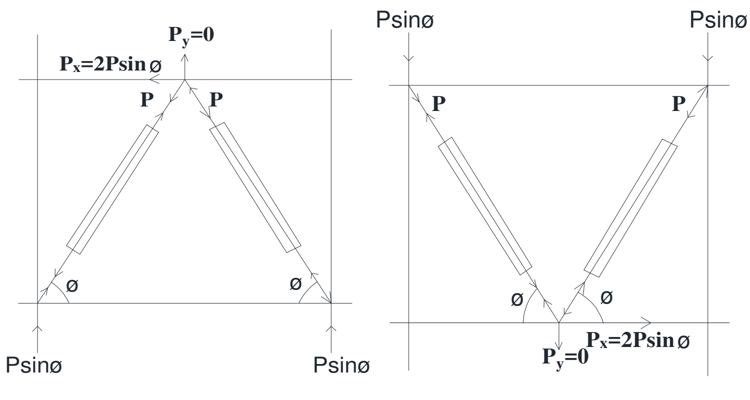
In this paper, the seismic effect of different configurations of buckling restrained braces on reinforced concrete frame structure was analyzed. There are two main types of buckling restrained braces: energy dissipation restrained braces and load-bearing restrained braces. Energy-dissipating buckling restrained braces can reduce the seismic effect of other structural members by their yield energy dissipation mechanism; load-bearing restrained braces can provide larger lateral stiffness for structures with smaller sections, but cannot yield energy dissipation under moderate earthquakes. The reason why the energy-dissipating buckling restraint braces are mostly preferred is that the energy-dissipating buckling restraint braces can meet the requirements of bearing capacity and deformation under earthquake action, and their yield energy dissipation under medium and large earthquakes can ensure that other structural components will not be seriously damaged[15,16].

In this paper, energy dissipation buckling restrained braces were used. The material for the core used was steel of the Q235 type with a modulus of elasticity (E) of 206000MPa and a yield stress (f_y) of 235E3KPa. Because the connection between braces and columns was assumed to be fixed, the effect of braces to the columns was neglected and the braces were assumed to resist 80% of the total lateral shear forces.

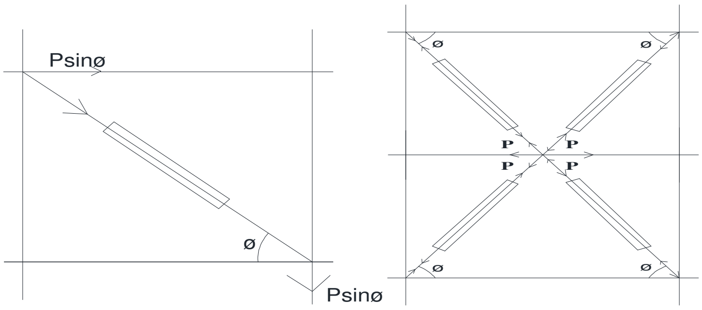
Normally, for the design of new buildings, the effect of braces on the vertical load-bearing of frame structures is considered in the design of the whole building and other structural elements specifically. The compression yield strength of the brace is 10% higher than the tensile strength which will cause a remaining imbalance of upward force even after all braces have yielded, causing additional bending moments in beams for V type braces, but in the current research, which is focused on strengthening of existing structures, this effect was avoided by using equal tension and compression strengths of BRB components, which will make the sum of their vertical elements cancel each other out.

Two-story X-BRB(TSXB) configurations have an advantage in that with the same section used for braces in alternating stories, both horizontal and vertical component of brace forces will be zero at the brace-beam joint, which will solve the problem of additional load in the middle of the beam and moment at the brace-column joint. For strengthening of existing structures, zero unbalanced force that braces exert to the beam will make the strengthening of the beam unnecessary.

The length of the core brace is taken as 70% of the total length of the brace and the angle of inclination is defined by the geometry of the frame bay (Fig. 5)



(a) Inverted V (b) V



(C) Single (d) two-story X

Fig. 5. Brace configurations and free body diagrams

The core section is assumed to bear all the total axial force exerted to the buckling restrained brace and it is assumed that there is no effect of axial forces on the encasing part of the buckling restrained brace. The area of the core section is obtained from the maximum of the results between the results obtained from the displacement method (equation 4.1) and that from equating the axial force in the brace with its maximum elastic bearing capacity by equation 4.1.

$$N_b = \phi A_1 f_y \tag{4.1}$$

ϕ is the strength reduction factor and 0.9 is used in this paper [7,12]

Where A_1 is the cross-section area of the yielding core, and f_y is the yield strength value of core element material.

From the relationship between brace displacement and story displacement (Fig. 6), the displacement of the brace is computed by taking the maximum target displacement of the story as the maximum between 2% of the maximum story drift and twice the design drift, which is 1/550 according to the code.

The stiffness of the floor can be computed as the sum of stiffnesses of all vertical load resistance members located on the same floor, the contribution of the floor to the stiffness is neglected and only columns are considered to provide the horizontal stiffness. The stiffness is given by equation 4.2 given as follows:

$$d = \frac{12(\mathrm{EI})_c}{h^3} \tag{4.2}$$

Where (EI)C is the elastic modulus of the column and ‘h’ is the column height.

The stiffness of the brace is geometrically determined by equation 4.3.

$$k_{BRACE} = \frac{12(\mathrm{EI})_b \sin\theta(\cos\theta)^2}{h} \tag{4.3}$$

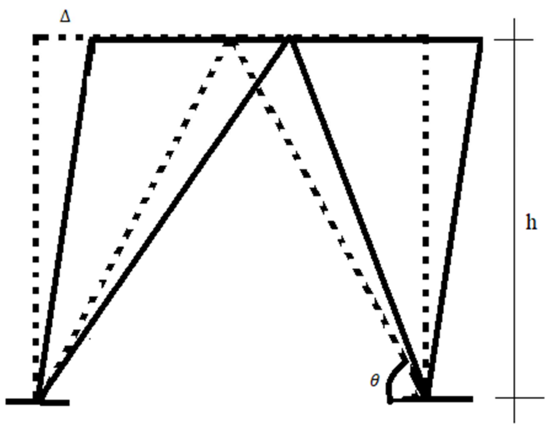


Fig. 6. Displacement relationship between buckling restrained brace and frame

The required dimensions of buckling restrained braces used for V and inverted V configurations are presented in table 3, and the maximum required area of the yielding core was obtained from equating the axial force in the brace with the linear bearing capacity. For a single brace, twice the area is used and the inclination is defined by the geometry of the bay.

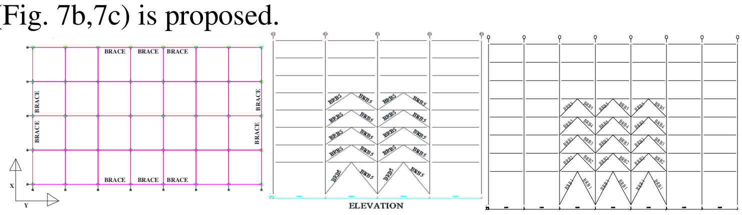
Table 3. BRB dimensions for V and inverted V braces

Floor	Braces	Total length(m)	Inclination(ø)	Yield section area (mm2)
1	BRB1	6.8	54	1849
2	BRB2	5	37	2304
3	BRB3	5	37	2116
4	BRB4	5	37	1936
5	BRB5	5	37	1600

According to Chinese technical regulations and seismic reductions of buildings (JGJ297 – 2013), buckling restrained braces should be arranged where energy dissipation can be maximized without affecting the function of the building and sets of windows, doors, and other openings, to meet the requirements of overall loading of the structure.

Installation of the braces in the structure will add loads to the columns that were not initially considered during design. For inverted V and single brace types, interior columns will carry the vertical components of the brace's forces from the above stories (Fig. 5a), for V types, these brace forces will be first carried by the beam which in turn transfers them to the column of the story below through pinned connection (Fig. 5b). In this paper, braces were installed by also taking into account the vertical load-bearing capacity of the building. It is assumed that the frame structure and individual structural members can bear the additional load from braces.

From the response spectrum results, the displacement is almost homogeneous in the joints within the same floors. By conforming to the principles of placing buckling restrained braces in frame structures [20], the first reinforcement scheme (Fig. 7b,7c) is proposed.



(a) (b) (c)

Fig. 7. Reinforcement scheme, (a) Plan view, (b) X-Dir elevation view, (c) Y-Dir elevation view

5. ANALYSIS RESULTS FOR STRENGTHENED BUILDING

5.1. Seismic responses under small earthquakes by the response spectrum method

The dynamic equation for a moment frame without braces is government by equation 5.1 as follows:

$$[M]\{\ddot{x}(t)\}+[C]\{\dot{x}(t)\}+[K]\{x(t)\}=-[M]\{\ddot{x}_g\} \tag{5.1}$$

Where, $\ddot{x}_g$ is the three-dimensional vector of earthquake acceleration.

When buckling restrained braces are incorporated in frame structures, their mass contribution in the mass matrix is very small and can be ignored, the dynamic equation can be expressed as follows,

$$[M]\{\ddot{x}(t)\}+\{[C]+[C_{BRB}]\}\{\dot{x}(t)\}+\{[K]+[K_{BRB}]\}\{x(t)\}=-[M]\{\ddot{x}_g\} \tag{5.2}$$

Where, C_{BRB} and K_{BRB} are damping matrix and stiffness matrix of Buckling restrained braces (BRB), respectively.

According to the Chinese code for seismic analysis of building structures, the period ratio, which is the ratio between the third mode to the first mode period of vibration for class A high rise buildings, must be less than 0.9 [5] to avoid torsional irregularity. Based on elastic response spectrum results, both buckling restrained braces and ordinary steel braces can solve the torsion irregularity of the building (table 4). This means that including braces in horizontal irregular RC building structure can substantially improve its performance by bringing both centers of stiffness and that of gravity as close as possible. The period ratio for the strengthened building conforms to the Chinese code for seismic design of structures and this is for both BRB and ordinary steel braces.

The original building is prone to sudden failure of individual structural elements or even complete collapse due to excessive distortion during an earthquake. Adding braces will provide considerable ductility that will help by reducing the period of vibration. For the first mode of vibration of the first reinforcement scheme, there is a 40%, 39.3%, 40%, 36.4%, and 38.3% reduction in the period of vibration after the building is strengthened by inverted BRB, V BRB, two-story BRB, single BRB, and ordinary steel brace, respectively.

In the strengthening of existing structures, much attention must be paid to the vertical lateral resisting system because there is an additional load that is added to the structure by this process. Therefore, in addition to the seismic stability of the strengthened structure, the whole gravity load stability must be ensured. For this paper, it has also found that there is a huge reduction in the horizontal load exerted on the column. This is beneficial in achieving the string column mechanism.

Table 4. Modal periods of original and strengthened frame models

Period ratio	3	2	1		Modal
0.9133627	1.866	2.022	2.043	Period/seconds	Original building
0.6971429	0.854	1.135	1.225	Period	Inv.VBRB
0.708871	0.879	43101	45292	Period	VBRB
0.706798	0.863	1.163	1.221	Period	2 Floor BRB
0.721324	0.937	1.238	1.299	Period	singe BRB
0.72279587	0.91	1.199	1.259	Period	Ordinary brace
	Torsion	X Translation	Y Translation		Direction
LESS 0.9					Requirement

The results from response spectrum analysis show that during the elastic state, braces will reduce more than 60% of the horizontal force exerted on columns, the selected first-floor column forces (Tables 5 and 6) shows that single BRB and ordinary steel braces are the best in accommodating horizontal forces on the column with reduction of more than 70% of the forces compared to the original building. This is explained by the size and geometrical configuration of the braces, since there is a demand for a small inclination to the horizontal axis and a bigger cross-section area for single and ordinary braces, and this will increase their load-bearing capacity.

Table 5. Reduction in first floor's column force, Vx of the strengthened building (in % of the original building)

Column	Single BRB	VBRB	inverted V BRB	Two-story BRB	Ordinary BRACE
C4	69.23	64.99	66.81	66.64	78.79
C5	68.57	64.99	66.81	66.64	78.79
C9	70.21	65.49	66.90	67.26	79.81
C12	68.40	64.78	66.37	66.21	77.77
C17	69.00	66.69	65.81	66.69	80.71
C25	70.26	65.49	66.90	67.26	79.81
C28	68.39	64.78	66.37	66.21	77.77
C35	69.23	65.19	66.74	66.77	79.11
C36	68.58	64.99	66.74	66.64	78.79

By comparing the maximum deflections of the original and strengthened building under small earthquakes, it was found that the seismic performance of the strengthened building is much better. The poor performance is explained by the existence of the first soft-story, which will cause concentrated heavy damage on the first story. The strengthened building demonstrated very good performance with a maximum story drift of less than 1/550, which is the maximum allowed by the code under small and moderate earthquakes (Fig. 7). In the X-direction, the maximum drift for RC frame strengthened by ordinary steel braces was greater than the maximum allowed by the code (Fig. 9).

It was shown that strengthening of the structure by buckling restrained braces and ordinary steel braces both decrease the original building displacement by more than 50% from the first to the fourth floor and 35% for other upper floors (Fig. 8).

Table 6. Reduction in first floor's column force Vy of the strengthened building (in % of the original building)

Column	Single BRB	VBRB	inverted V BRB	Two-story BRB	Ordinary BRACE
C4	69.44	66.46	66.02	66.22	80.94
C5	72.45	66.46	66.02	66.22	80.94
C9	70.79	66.00	66.97	66.94	80.83
C12	70.45	65.87	66.39	66.68	80.30
C17	71.092	65.65	66.40	66.71	80.73
C25	70.45	66.00	66.06	66.94	80.83
C28	70.26	65.87	65.57	66.68	80.30
C35	69.43	65.69	65.31	66.78	80.29
C36	72.41	66.46	64.27	66.22	80.94

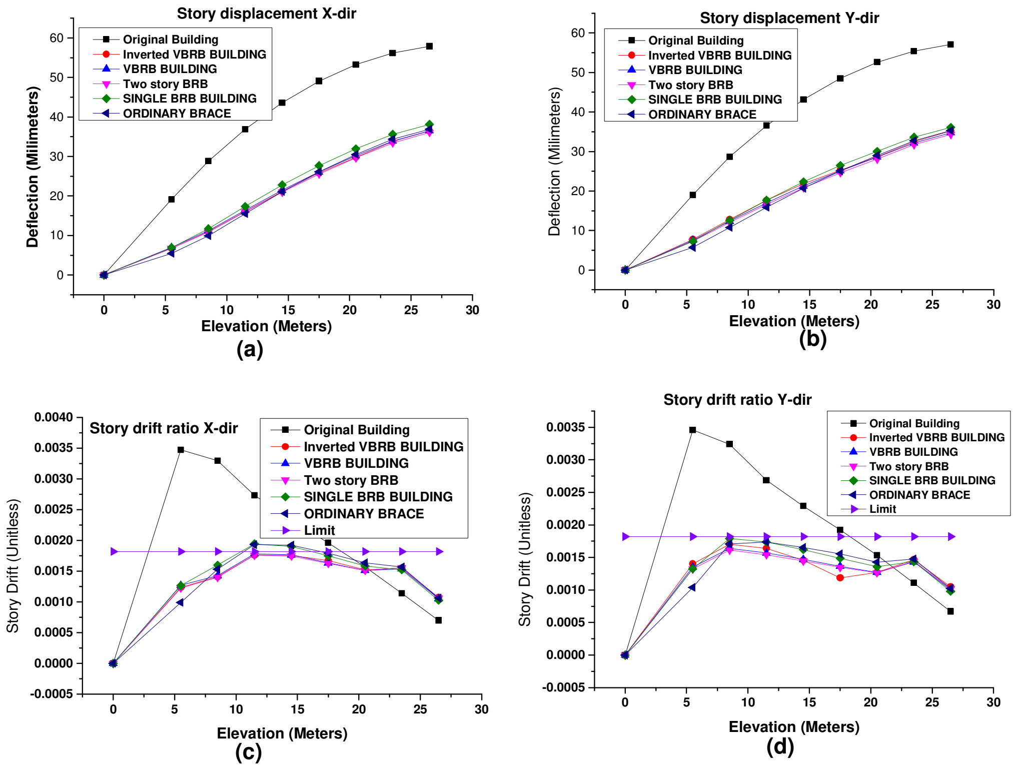


Fig. 8. Maximum story displacement

The two stories' X buckling restrained braces demonstrated better performance than other types of braces in terms of story deflections and maximum story drifts.

5.2. Seismic response under severe earthquakes by nonlinear time history analysis

When performing elastic response spectrum analysis, the structure response is assumed to remain in a solely elastic manner, but because of the material non-linearity of some structural members, especially buckling restrained braces, geometrical nonlinearity of the structure, and possible seismic non-linearity behaviors of some structural members, it is advantageous to also consider nonlinear analysis.

In this paper, nonlinear time history analysis of the strengthened building was conducted under strong earthquakes. Time history analysis is a step by step analysis of the dynamic response of the structure subjected to time function load. In time history analysis, earthquake loads are regarded as input, and the structure is used as a vibration system [8]. The principle of time history analysis is to divide the earthquake action time into a large number of very short time periods, and assume that the damping and stiffness of the structure are constant values in each small time period, the acceleration of ground motion and the acceleration of particle change linearly with time, and then input the selected earthquake motion into the structure, directly integrating the dynamic equation of the building structure, so as to calculate the structure in the earthquake. Every instantaneous response in the process of action is used to obtain the dynamic response of the structure with the change of time in the earthquake [10].

For single support excitation, the same earthquake motion excites all the masses and the equation of multi-degree of freedom system is expressed as follows [3]:

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=[M][\mathrm{I}]\ddot{x}_g(t) \tag{5.3}$$

Where,[M],[C], and [K] are the mass matrix, stiffness, and damping matrix of the system, respectively.

$\ddot{x}$, $\dot{x}$, x are absolute acceleration, relative velocity, and relative displacement vector of the system, respectively.

$\ddot{x}_g$ is the ground acceleration vector.

I is the influence coefficient matrix

By dividing the duration of seismic wave t into a large number of small-time periods $0 \le \tau \le \Delta t$, at the time i, the parameters of the equation 5.4 will become $\ddot{x}(t)$, $\dot{x}(t)$, $x(t)$, $\ddot{x}_g(t)$, which will change at every θ of time.

The increase in short time Δt changes equation 3.1 by the following:

$$[M]\{\Delta\ddot{x}\}+[C]\{\Delta\dot{x}\}+[K]\{\Delta x\}=[M][\mathrm{I}]\Delta\ddot{x}_g(t) \tag{5.4}$$

By using the linear acceleration method, Wilson-θ method, Newmark, or other numerical integration methods, equation 5.5 can be solved to obtain the response increment of the structure under earthquake action. To obtain the response of the structure for the whole process of the earthquake, step by step integration is performed by considering the end state of $t+\Delta t$, taken as the start of the next step, and so on. A widely used method is Wilson-θ method, which assumes that the variation in acceleration between two-time intervals is linear.

When the time interval is very small, the system can be assumed to be linear in time Δt and the damping and stiffness of the system remain constant with time. With an extended period of time,

$$\tau=\theta\Delta t,(\theta \succ 1.37) \tag{5.5}$$

Assuming linear acceleration,

$$\ddot{x}_{i+\tau}=\ddot{x}_i+\frac{\tau}{\theta\Delta t}\left(\ddot{x}_{i+\theta}-\ddot{x}_i\right) \tag{5.6}$$

Where, $0\le\tau\le\Delta t$

By integrating equation 5.6 within the interval $\tau=\theta\Delta t$, corresponding velocity and displacement can be obtained by:

$$x_{i+\tau}=x_i+\ddot{x}_i\tau+\frac{\tau^2}{2\theta\Delta t}\left(\ddot{x}_{i+\theta}-\ddot{x}_i\right) \tag{5.7}$$

$$x_{i+\tau}=x_i+\dot{x}_i\tau+\ddot{x}_i\frac{\tau^2}{2}+\frac{\tau^3}{6\theta\Delta t}\left(\ddot{x}_{i+\theta}-\ddot{x}_i\right) \tag{5.8}$$

For $\tau=\theta\Delta t$, the equations 5.7 and 5.8 can be deduced to:

$$x_{i+\theta}=x_i+\frac{\theta\Delta t}{2}\left(\ddot{x}_{i+\theta}+\ddot{x}_i\right) \tag{5.9}$$

$$x_{i+\theta} = x_i + \dot{x}_i\theta\Delta t + \ddot{x}_i\frac{(\theta\Delta t)^2}{6}(\ddot{x}_{i+\theta} + 2\ddot{x}_i) \tag{5.10}$$

From equations (5.7) and (5.8), velocity and accelerations for the interval $\tau + \theta\Delta t$ are obtained:

$$\dot{x}_{i+\theta} = \frac{6}{(\theta\Delta t)^2}(\ddot{x}_{i+\theta} - x_i) - \frac{6}{\theta\Delta t}(\dot{x}_i - 2\ddot{x}_i) \tag{5.11}$$

$$\ddot{x}_{i+\theta} = \frac{3}{\theta\Delta t}(x_{i+\theta} - x_i) - 2x_i - \frac{\theta\Delta t}{2}x_i \tag{5.12}$$

By substituting equation 5.12 into equation 5.3, we get:

$$F^* x_{i+\theta} = F^*_{i+\theta} \tag{5.13}$$

Where:

$$K^* = [K] + \frac{6}{(\theta\Delta t)^2}[M] + \frac{3}{\theta\Delta t}[C] \tag{5.14}$$

$$F^*_{i+\theta} = F_i + \theta(F_{i+1} - F_i) + [M]\left(\frac{6}{(\theta\Delta t)^2}x_i + \frac{6}{\theta\Delta t}\dot{x}_i + 2\ddot{x}_i\right) + [C]\left(\frac{3}{\theta\Delta t}x_i + 2\dot{x}_i + \frac{\theta\Delta t}{2}\ddot{x}_i\right) \tag{5.15}$$

By equation 5.14, $x_{i+\theta}$ can be obtained, and be used to obtain $\ddot{x}_{i+\theta}$ and $\dot{x}_{i+\theta}$ from equations 5.11 and 5.12, respectively. And this is repeated at every interval for the whole process.

The following steps flow in performing a dynamic analysis of the structure, first seismic waves are selected according to the location of the structure, the numerical model of the structure is established, the force of inertia established, and the system is solved using appropriate numerical solution method.

The following formula (equation 5.15) shows that the mass of the structure is a very important factor in seismic response analysis and additional gravity load on the structure will not only disturb the gravity load-bearing capacity, but will also increase the seismic effect on the structure.

5.2.1. Selection of seismic loads

According to china's code for seismic design of building GB50010-2010, the selection of seismic waves should take into account the amplitude, frequency, and duration of the wave. Amplitude represents the amount of energy released from the ground during earthquakes, frequency represents the seismic characteristics at the specific site, and the duration of earthquake controls the duration of earthquake energy input. Three seismic waves should be selected and shall be selected according to the site classification and the earthquake design severity. In the three selected waves, two natural waves and one artificial wave should be selected, and the three selected seismic waves and the seismic influence coefficient curve selected by mode decomposition response spectrum method shall be consistent. The selected wave duration should be at least 5 to 10 times the basic period of the structure. Two actual strong earthquake records and one synthetic earthquake are selected according to the types of building sites and design earthquakes grouping. The selected waves are shown in the figure below.
The selected earthquakes are all of the severe earthquake records with a 2% probability of exceedance. The Nakanoshima Nagaoka earthquake of 2007 (figure 1.a and b), the Darfield New Zealand earthquake of 2010 (figure 1. c and d), and ELC (figure 1.d and e) were used for this time history analysis and their acceleration time histories were derived from ground motion records.
The spectral characteristics of the selected seismic waves were as close as possible to the characteristic period of the building site, and the duration of the seismic waves selected conformed to the code.
Graphical representation of the seismic waves used are presented in the Fig. 10.

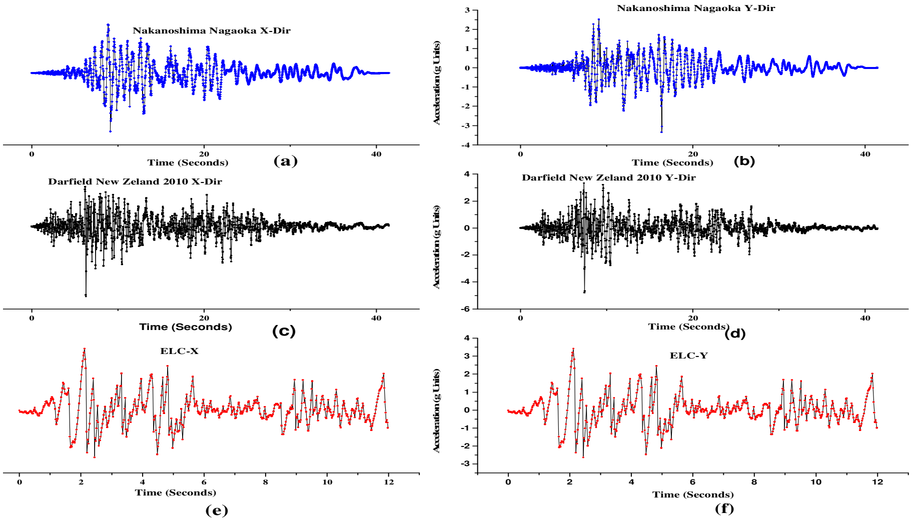


Fig. 10. Seismic waves for time history analysis

The selection of seismic waves was based on the requirements for the seismic design of the building is shown in table 6 below.

Table 6. The maximum time for seismic time history analysis (in cm/sec2)

Earthquake level	6degree	7 degree	7.5 degree	8 degree	8.5 degree	9degree
Small	18	35	55	70	110	140
Medium	51	98	147	196	294	392
Severe	125	220	310	400	510	620

5.2.2. NLTH results

Seismic performance indices, which are the base shear time history curve, the peak acceleration time history curve, and the peak displacement time history curve of the elastic-plastic time history analysis of the structure with ordinary steel braces and BRB, were compared.
The shear time history curves of two typical columns in the direction of seismic waves were extracted to observe whether the pure frame structure will increase the shear force of the columns due to the introduction of buckling restraint braces.

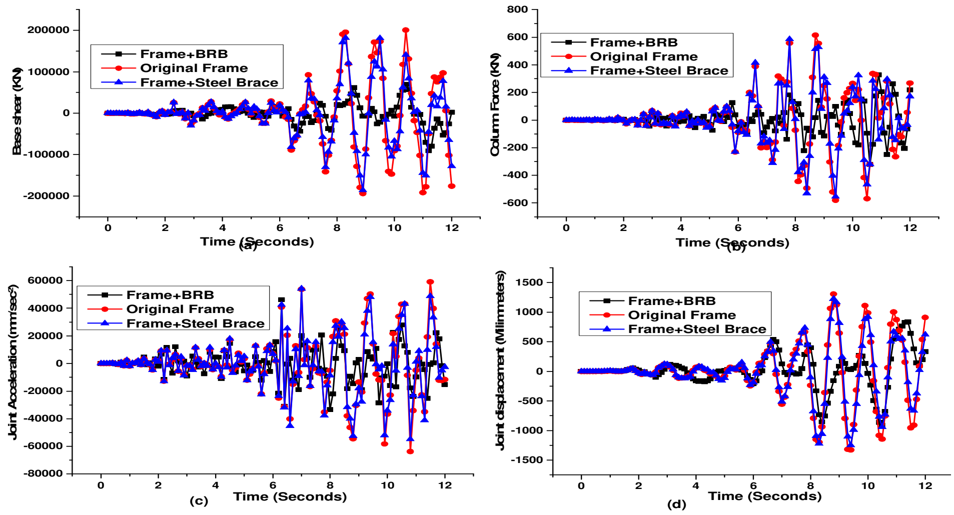


Fig. 11. Time history curves under TH048TG040_DARFIELD NEW ZEALAND 9-3-2010 DFHS Earthquake

To sum up, the results of (1) (2) (3), because the base shear force, peak acceleration, and peak displacement time history of the BRB structure were less

than that of the original frame and steel brace structure under rare earthquakes, the buckling restrained brace provided more additional damping ratio for the structure, reduced the displacement response of the structure under earthquake, and reduced the damage of the main structure caused by the earthquake.
It can be seen from the above analysis results that for the vertical members of the mainframe structure, although the additional buckling restrained support causes the period to be smaller and the structure to be rigid, under rare earthquake action, the buckling restrained support not only bears part of the horizontal earthquake action but also reduces the input energy of the earthquake action due to its yield energy consumption. Generally speaking, ds shown in Figure 5.14, the shear forces of corner columns in the frame structure with buckling restrained braces are smaller than that of ordinary steel braces and original frame structure. According to statistics, for the original frame structure in 5.14, the maximum shear force is 614.4713kN, while for the BRB structure, the maximum shear force is 328.507kN, and for ordinary steel brace the maximum shear force is 585.4309. This means that the use of BRB reduced the column shear by 46.6%; similarly, incorporation of ordinary steel braces reduced the column shear force by 4.72%. This is because, under severe earthquakes, ordinary steel braces will lose their load-bearing capacity due to excessive buckling.

6. CONCLUSION

In this paper, the use of buckling restrained braces in strengthening the existing reinforced concrete frame structures through correcting the vertical stiffness irregularity was studied. Through response spectrum analysis of a typical 3D model with vertical stiffness irregularity, reinforcement schemes have been determined. It has been determined that the inclusion of BRB in frame structures will reduce the horizontal internal forces of the structure acting in the columns up to 70% and above depending on the location and number of braces. Two-story X BRB has been found to have better performance compared to other brace configurations. Ordinary steel braces also can be used to strengthen reinforced concrete frame structures against vertical stiffness, but a bigger cross-section area is needed compared to Buckling restrained braces. The Response spectrum analysis should be used together with time history analysis in order to capture the time-dependent on load variation and structure and material nonlinearity.

REFERENCES

1. Achudhan, Deepavarsa, Vandhana, Shalini; Strengthening and Retrofitting of RC Beams Using Fiber Reinforced Polymers;' *Materials Today: Proceedings,* **16**, 361-366'. https://doi.org/10.1016/j.matpr.2019.05.102.
2. Allawi, AAM 2017. Behavior of Strengthened Composite Prestressed Concrete Girders under Static and Repeated Loading. *Advances in Civil Engineering*, **2017**, 1-13. https://doi.org/10.1155/2017/3619545.
3. Bai, Y Xu ZD 2019. Multi-Degree of Freedom System. In: Bai, Y Xu ZD *Structural Dynamics*. Wiley. 127–205. http://doi.org/10.1002/9781119605775.ch4.
4. Belal, MF, Mohamed, HM and Morad, SA 2015. Behavior of reinforced concrete columns strengthened by steel jacket. HBRC Journal, **11(2)**, 201-212. https://doi.org/10.1016/j.hbrcj.2014.05.002.
5. CMC. 2010b. Technical Specification for Concrete Structures of Tall Building (JGJ3-2010). China Ministry of Construction, China Architecture and Building Press: Beijing, China. (in Chinese).
6. Computers and Structures Inc. (CSI), Structural and Earthquake Engineering Software, ETABS, Extended Three-Dimensional Analysis of Building Systems Nonlinear Version 15.2.2, 2015 (Berkeley, CA, USA).
7. Darain, K et al. 2016. Strengthening of RC Beams Using Externally Bonded Reinforcement Combined with Near-Surface Mounted Technique. *Polymers*, **8(7)**, 1-23. doi:10.3390/polym8070261.
8. Dowrick, D 2009. Design and Detailing of New Structures for Earthquake Ground Shaking. (2009). In: Dowrick D. (ed.) *Earthquake Resistant Design and Risk Reduction*, Wiley, 337–449. doi:10.1002/9780470747018.ch10.
9. GB 50011-2010, Code for Seismic Design of Buildings, China Building Industry Press, Beijing, China, 2010.
10. Kaliszky, S, Vásárhelyi, A and Lógó, J 1991. The Time History Analysis of Viscoelastic Structures by Mathematical Programming. In: Brüller O.S., Mannl V., Najar J. (eds) *Advances in Continuum Mechanics*. Berlin, Heidelberg, Springer, 488–499. doi:10.1007/978-3-642-48890-0_39.
11. Kaplan, H and Ylmaz, S 2012. Seismic Strengthening of Reinforced Concrete Buildings. Earthquake-Resistant Structures - Design, Assessment and Rehabilitation. In. Moustafa A. (ed) Earthquake-Resistant Structures, IntechOpen, 407-428. doi:10.5772/28854.
12. Li, HN, Xiao, SY and Huo, LS 2010. Lessons Learnt from Building Damages in the Wenchuan Earthquake. *Earth and Space 2010: Engineering, Science, Construction, and Operations in Challenging Environments,* 12th Biennial International Conference on Engineering, Construction, and Operations in Challenging Environments; and Fourth

NASA/ARO/ASCE Workshop on Granular Materials in Lunar and Martian Exploration 3253-3261 doi:10.1061/41096(366)310.
13. Li, S, Zhu, T, Lu, Y and Li, X 2016. Effect of Temperature Variation on Bond Characteristics between CFRP and Steel Plate. *International Journal of Polymer Science*, **2016**, 1-8. https://doi.org/10.1155/2016/5674572.
14. Manisekar, R 2018. Effect of External Post-tensioning in Retrofitting of RC Beams. *Journal of The Institution of Engineers (India): Series A*, **99(3)**, 495–501. doi:10.1007/s40030-018-0312-9.
15. Mohsenian, V, Gharaei-Moghaddam, N and Hajirasouliha, I 2020. Reliability analysis and multi-level response modification factors for buckling restrained braced frames. *Journal of Constructional Steel Research*, **171**, 106-137. doi:10.1016/j.jcsr.2020.106137.
16. Momenzadeh, S, Seker, O, Faytarouni, M and Shen, J 2017. Seismic performance of all-steel buckling-controlled braces with various cross-sections. *Journal of Constructional Steel Research*, **139**, 44–61. doi:10.1016/j.jcsr.2017.09.003.
17. Naser, AF and Wang, Z 2011. Damage Investigation, Strengthening, and Repair of Jilin Highway Double-Curved Arch Concrete Bridge in China. *Procedia Engineering* **14**, 2294–2300. doi: 10.1016/j.proeng.2011.07.289.
18. Nguyen, PL, Vu, XH and Ferrier, E 2019. Thermo-mechanical performance of Carbon Fiber Reinforced Polymer (CFRP), with and without fire protection material, under combined elevated temperature and mechanical loading conditions. *Composites Part B:Engineering*, **169**, 164-173. https://doi.org/10.1016/j.compositesb.2019.03.075.
19. Pun, SK, Liu, C and Langston, C 2006. Case Study of Demolition Costs of Residential Buildings. *Construction Management and Economics,* **24(9)**, 967–976. doi:10.1080/01446190500512024.
20. Recupero, A, Spinella, N, Colajanni, P and Scilipoti, CD 2014. Increasing the Capacity of Existing Bridges by Using Unbonded Prestressing Technology: A Case Study. *Advances in Civil Engineering*, **2014**, 1-10. https://doi.org/10.1155/2014/840902.
21. Romanichen, RM and Souza, RA 2019. Reinforced concrete corbels strengthened with external prestressing. *Revista Ibracon de Estruturas e Materiais*, **12(4)**, 812-831. https://dx.doi.org/10.1590/s1983-41952019000400006.
22. Scheerer, S, Zobel, R, Müller, E, Senckpiel-Peters, T, Schmidt, A and Curbach, M 2019. Flexural Strengthening of RC Structures with TRC—Experimental Observations, Design Approach and Application. Applied Sciences **9**, 13-22. https://doi.org/10.3390/app9071322.
23. Siddika, A, Saha, K, Mahmud, MS, Roy, SC, Mamun, MAA and Alyousef, R 2019. Performance and failure analysis of carbon fiber-reinforced

polymer (CFRP) strengthened reinforced concrete (RC) beams. *Applied Sciences*, **1**:1617. doi:10.1007/s42452-019-1675-x.

24. Wang, Q, Li, J, Liao, W, Zhang, L and Qin, X 2011. Building Damages in Deyang City by the 2008 Wenchuan Earthquake. *Geodesy and Geodynamics* **2(4)**, 59–63. doi:10.3724/sp.j.1246.2011.00007.2.
25. Wang, W and Guo, L 2006. Experimental study and analysis of RC beams strengthened with CFRP laminates under sustaining load. *International Journal of Solids and Structures*, **43(6)**, 1372-1387. https://doi.org/10.1016/j.ijsolstr.2005.03.076.
26. Wang, Y 2010. Revision of Seismic Design Codes Corresponding to Building Damages in the '5.12' Wenchuan Earthquake. *Earthquake Engineering and Engineering Vibration* **9(2),** 147–155. doi:10.1007/s11803-010-0001-9.
27. Yuan, X, Zhu, C, Hu, J and Zhang, Y 2019. Crack and mechanical behavior of CFRP plate-reinforced bridge roofs under high temperature with different anchoring measures. *Latin American Journal of Solids and Structures*, **16(6)**, 1-20. 2019.https://dx.doi.org/10.1590/1679-78255575.

Editor received the manuscript: 21.08.2020