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Strengthening of concrete structures with buckling braces and buckling restrained braces

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Abstract. The purpose of this article is to strengthen concrete structures using buckling and non-buckling braces. Connection plates are modeled in three shapes including the effect of 1.5t hinge zone length, 2t one and without the zone (1.5t-CP, 2t-CP and WCP). According to the verification performed with ABAQUS software, the connection plates which are superior in ductility and strengthening are found. The results show adding steel braces in concrete moment frames increase the strength and stiffness of the structures up to about 12 and 3 times, respectively. The frame strength increased about 21 and 25 percent with considering the effect of 2t hinge length in connection plates compared to 1.5t-CPs and WCPs. Also the ductility of retrofitted frames with 2t-CP improved 2.06 times more than WCP ones. Thus, 2t-CP sample is the best choice for connecting steel braces to concrete moment frames for retrofitting them. Afterwards, optimum conditions for elemental coating in braces with no buckling are assessed. The length of concrete coatings could be reduced about 30 percent, and buckling did not occur. Therefore, the weight of restraining coating decreased, and its performance improved. It is worth noting that BRBs could be constructed with only steel materials, which have outer steel tubes too. In fact, only the square cross sections of the tube profiles are appropriate for removing the filler concrete, and the rectangular ones are prone to buckle around their weak axis.

Keywords: gusset plate connections; 2t hinge zone length; concrete frame; buckling restrained frame; steel tube

1. Introduction

Braces are used as resistant elements against earthquakes in various types of buildings. Using bracing systems in steel structures has been an economic and effective method for resisting them in lateral loads (Gholipour and Mazloom 2018). Convergent steel braces are one of the earthquake resistant systems that are installed in different shapes in structures (Mazloom and Salehi 2017). In concrete structures, lateral stiffness is usually provided with flexural frames individually or the combination of them with concrete shear walls (Mazloom 2010). Using flexural moment frames

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combined with steel braced systems can be used too (Badoux and Jirsa 1990). The reason for it is the operation of this kind of system in a lower cost, as well as the ability to repair and replace the system after earthquakes. Also it is possible to use this type of system for concrete structures that are weak against lateral loads (Bush *et al.* 1991).

Elfath and Ghobarah (2000) and Elfath and Ghobarah (2001) examined the concentric and eccentric braces of concrete frames under various earthquakes by conducting spectral analyses. Maheri and Memarzadeh (2001) did an empirical research on the use of flexible knee braces and compared them with steel braces in concrete moment frames. This study showed that reinforced concrete frames equipped with knee braces had superior ductility. They used ANSYS software to evaluate the nonlinear behavior of concrete frames with convex braces based on their laboratory work. They studied the behavior of frames and braces based on weak beams and braces philosophies. It was seen that the strengthening of frames with braces was effective. Shin et al. (2016) examined the dynamical performance of reinforced concrete frames with FRP (fiber reinforcement polymer) coating columns. They retrofitted the poorly armed double-deck frames, which their destruction mode was a brittle rupture of columns in the first level, by using FRP coating here.

He *et al.* (2017) examined using steel-jackets as a way to retrofit reinforced concrete columns with constructing 16 samples. Their results indicated that the use of steel restrainer tubes had significant effects on strength, stiffness and ductility of concrete columns. Truong *et al.* (2017) also tested various strategies for retrofitting concrete columns such as steel jackets, CFRP (carbon fiber reinforcement polymer) coatings and new concrete coatings. The laboratory samples were investigated under cyclic loadings considering the initial axial force of columns. The results indicated that the projected retrofit methods could partially improve the seismic capacity of the beam–column joints; steel jackets increased the ductility and load-carrying capacities of the structures. CFRP wrapping methods did not significantly affect the seismic capacity of the beam–column joints. Rahai and Lashgari (2006) studied concentric and buckling restrained braces in reinforced concrete structures. These systems were examined in a nine story reinforced concrete structure. The results showed that both systems could improve the strength and stiffness of the structure but BRBs had better performance than the other especially in nonlinear phase and under compressive forces.

Youssef *et al.* (2007) and Ghaffarzadeh and Maheri (2006) tested various concentric bracing systems in a laboratory. According to their studies, using bracing systems for both new buildings and the retrofitted ones were efficient enough and led to reasonable results. Subsequently, Maheri and Ghaffarzadeh (2008) investigated the interaction between concrete frames and steel braces and calculated the excessive strength caused by this interaction. Viswanath *et al.* (2010) examined the effect concentric steel braces in the behavior of 4, 8, 12, and 16 story concrete frames. According to their results, the X type of steel bracing systems significantly contributed to the structural stiffness and reduced the maximum interstory drift of the frames. Ozel and Guneyisi (2011) examined the behavior of concrete frames with eccentric braces using seismic fragility curves. They studied the original structure and the retrofitted one with K, and V type eccentric bracing systems. Each of these braces were applied with four different distribution in the concrete frame height, and they reported improvements of these frames by comparing the average values of the fragility curves of the present building before and after retrofits.

Maheri *et al.* (2003) analyzed reinforced concrete frames with steel X and knee braces with pushover method. According to their results, the strength and ductility of the systems increased, and its total displacements decreased up to the preferred levels. In addition, Maheri and Akbari

(2003) calculated the behavior of these dual systems in numerical studies. How to attach steel braces to concrete frames was the subject of a research by Maheri and Sahebi (1997).

Goel and Lee (1992) investigated the seismic resistance of reinforced concrete structures with steel braces; the results of them showed that the system had appropriate ductility. Also, some retrofitted structures with this method are reported by Sekiguchi (1988), Del Val Calderon et al. (1988), Badoux and Jirsa (1990). Tagawa *et al.* (1992) tested concrete frames with K-shaped braces. The results indicated that the strength of the retrofitted frames were equal to the total concrete frame strength plus steel brace strength. In this method, the connection of braces to concrete frames played an important role in seismic performance of the system. Aydin *et al*. (2015) researched the optimal placement of elastic steel diagonal braces with bee colony algorithm. They presented a new algorithm to catch the optimal distribution of steel diagonal braces.

The buckling restrained bracing system (BRB) is one of the newest systems due to restraining of braces that cannot buckle. They have the potential to absorb much more energy than conventional bracing systems (Zhang *et al.* 2018). Structures including BRBs are among the few earthquake resistant systems that have extreme stiffness property and great ability to deplete earthquake energy (Guerrero *et al.* 2018). In addition, this system not only is used in new structures but also is utilized to improve the existing steel and concrete structures in abundance. This type of bracing system, in addition to the significant improvement in seismic performance, reduces the dimensions of members in structures and also reduces the cost of repairing of the damaged structures in earthquakes (Yazdi *et al*. 2018). Meanwhile, these reasons make BRBs applicable in tall buildings (Mohammadi *et al*. 2018).

Uang *et al.* (2004) and Xie (2005) studied different types of BRBs in the world and showed their results in abstract. In this field, Watanabe et al. (1988) estimated optimized amount of P_e/P_y in BRBs. Chou and Chen (2010) suggested central plates that were between two amplifier elements with some screws in BRBs. Each of restrained elements in prefabricated concrete panels including steel coating were constructed to resolved problems caused by non-adherent material and concrete placement. Hoveidae and Rafezy (2012) examined the overall buckling behavior of steel BRBs. One of the basic requirements for the proper mechanical behavior of BRBs under severe earthquakes was to prevent general buckling until it experienced plastic deformation and great ductility. They studied BRBs with different values of distances between restrained and central elements, and the initial discontinuities to know the overall buckling behavior of such braces. They concluded that the flexural stiffness of the restrained elements could affect the overall buckling behavior of the braces. Therefore, a minimum ratio of Euler buckling load from the restrained element to the core was proposed for design purposes.

Quan *et al*. (2014) examined the effect of BRB model parameters on structural seismic response. In this study, the susceptibility of BRB models was considered to provide a tool for evaluating the effect of fixed parameters of this type of braces on structural behaviors. The effect of general and local buckling on fixed parameters of this type of braces was determined with modeling of one of the previous experiments (Qin *et al.* 2016). Bin and Yang (2015) investigated buckling mechanism of steel cores in BRBs. Ziqin *et al*. (2015) developed a finite element model to evaluate the contact force between the core and external restrained members to investigate the BRB performance. The influences of strength and stiffness of external restrained members, core length, and other geometric parameters on the BRB performance were examined too. They proposed recommended values of core width-to-thickness ratio, core thickness and gap based on numerical results.

Piedraftia *et al.* (2015) tested a new perforated steel core in BRBs. They presented two models with three different loading protocols. Some of the specimens showed stable responses, but the others suffered the loss of compression capacities caused by local bucklings. Mirtaheri *et al.* (2018) studied how to improve the behavior of buckling restrained braces. One of the disadvantages of this system was the possibility of buckling in restrained braces under compressive loads. The length of BRB steel cores could have significant effects on their general behaviors since they influenced the energy dissipation capability of the members directly.

The motivation for this research is the promotion of weak concrete structures with using steel braces. They improve the structural performance effectively with less involvement of other members. Using buckling restrained braces has increased in recent years. Thus both two types of braces, which are common and restrained, were designed to evaluate how much improvement occur in the concrete structural performance. The first one was steel buckling braces that were used in concrete moment frames. The lateral strength of the structures were investigated by the effect of considering 2t hinge zone length at the junction of braces to frames (Astaneh *et al.* 2006) as well as changing the thicknesses of gusset plate connections. In the second part, buckling restrained braces (BRBs) were modeled. In this step, at first, an assessment was made to find the optimum conditions for coating BRB elements. Eventually, all steel BRB models were studied. One of the goals of this work was to obtain braces that had both sufficient resistance to lateral forces and economically feasible for construction projects.

2. Buckling braced systems

Steel buckling braces are used in this part. Initially, a flexural reinforced concrete frame with a steel bracing system is examined and verified. Then, gusset plate connections with different thicknesses are used in this frame, and then the effect of 2t hinge zone length, 1.5t hinge zone length and without considering hinge zone length (Astaneh *et al.* 2006) are studied (t is the thickness of gusset plate). Finally, the connection samples that had superior efficiencies in ductility and strength are specified.

2.1 Verification of the analytical model

Laboratory samples of Massumin and Absalan (2013) was used for numerical verification of the models. The numerical results of ABAQUS software were compared with the experimental results, and the modelling accuracy was evaluated. The dimensions and geometry of numerical samples were modelled exactly in accordance with laboratory samples. In the software, C3D8R elements which represented an eight-node hexagon component was used for concrete. A two-node truss element called T3D2 was used to simulate the reinforcement. This element showed a two-node linear element in three dimensional space. A shell element called S4R was used to simulate the gusset plate connections.

Compressive and tensile stress-strain curves are shown in Figs. 1 and 2. They are used for concrete considered with concrete plastic damage (CPD) modelling of materials. The utilized isotropic bilinear stress-strain curve of the steel in the model is shown in Fig. 3.

The embedded region option is used to model the interaction between reinforcement and concrete. The steel brace jointed to the connection plates and the connection plates jointed to concrete frames with a Tie type connection. The specifications of the steel used in this research is shown in Table 1.

The compressive strength of the concrete used in this model was 25 MPa. The modulus of elasticity and Poisson's ratio of the steel bars were 200 GPa and 0.3, respectively. The values of yield and ultimate strengths of the bars were 300 MPa and 520 MPa, respectively. The value of ultimate strain of steel bars was 0.09. Concrete coating was considered 20 mm for beams and columns. The mechanical properties of concrete is shown in Table 2.

Pushover analysis is utilized to consider the nonlinear behavior of the frame. The boundary conditions in numerical modeling were used to prevent the movement of the frame outside its plane (Fig. 4).

The mesh size was 25 mm. In fact, the accuracy of calculations increased a little with making smaller the size of elements, but the volume of calculations increased dramatically.

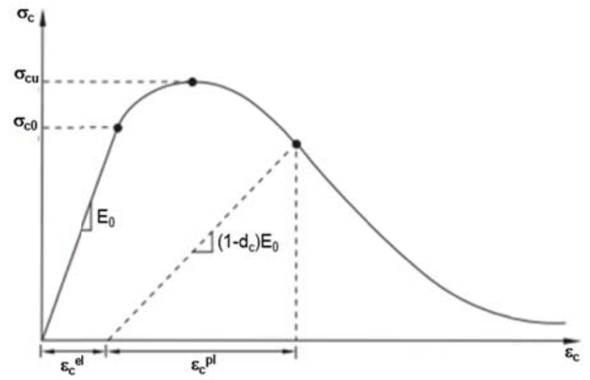


Fig. 1 Compressive Stress-strain curve of concrete with CPD model

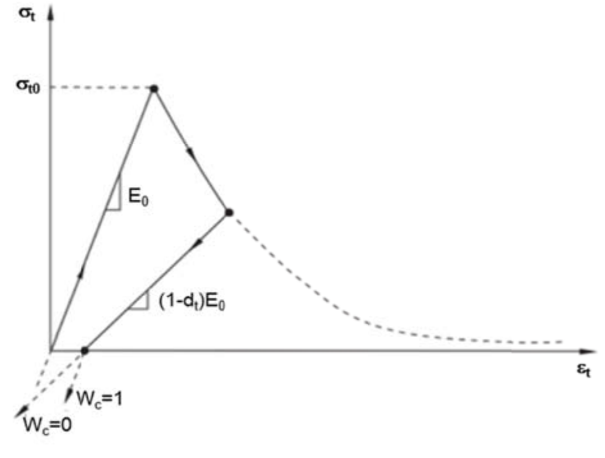


Fig. 2 Tensile stress-strain curve concrete with CPD model

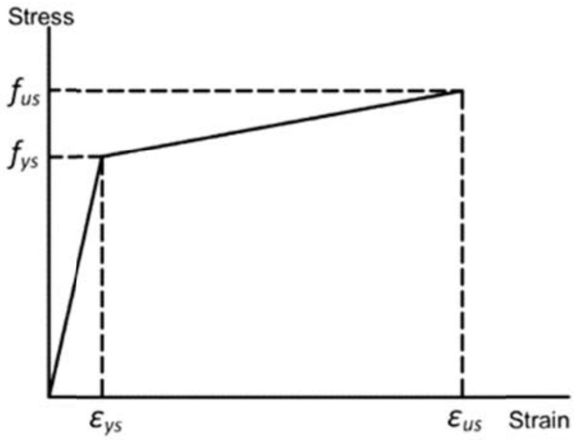


Fig. 3 Stress-strain curve of steel

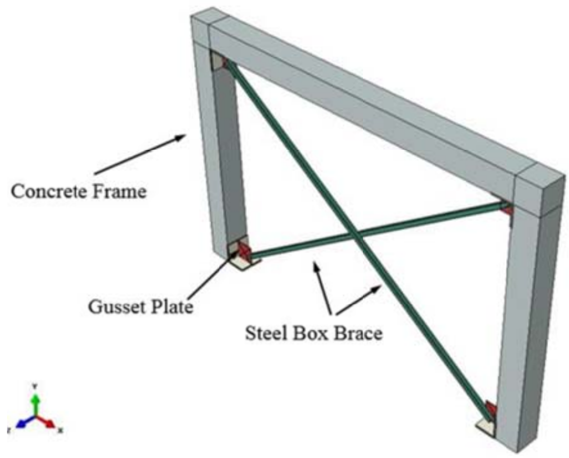


Fig. 4 Numerical model of the reformed frame

Table 1 Properties of steel materials

Elastic modulus	Poisson’s ratio	Yielding tension	Ultimate stress	Ultimate strain
200000 MPa	0.3	300 MPa	520 MPa	0.09

Table 2 Properties of concrete materials

Elastic modulus	Poisson’s ratio	Viscosity
25744.1 MPa	0.2	0.01

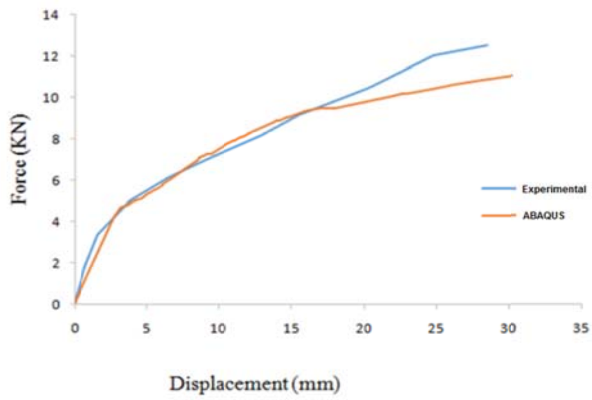


Fig. 5 Comparing base shear-roof displacement diagrams of experimental and numerical investigations

Experimental and numerical base shear – roof displacement diagrams of the frame are compared in Fig. 5. It is clear that the model was exact enough for predicting the behavior of the structure in different conditions.

2.2 Introducing numerical models

Three gusset plate connections were modeled considering the effects of 1.5t (Fig. 6) and 2t hinge zone length (Fig. 7) and do not having the hinge zone length (Fig. 8) with different thicknesses including 4, 8 and 12 mm. The analysis of numerical samples was done in two separated parts. The impact of connection plate thicknesses and the effect of 2t hinge zone length were considered in the performance of retrofitted concrete frames, respectively. There were three different sample groups used in this part. The first one reflected to the effect of 2t for gusset plate connection (2t-CP). The second one related to the effect of 1.5t, and the last one was stimulated without having hinge zone length (WCP).

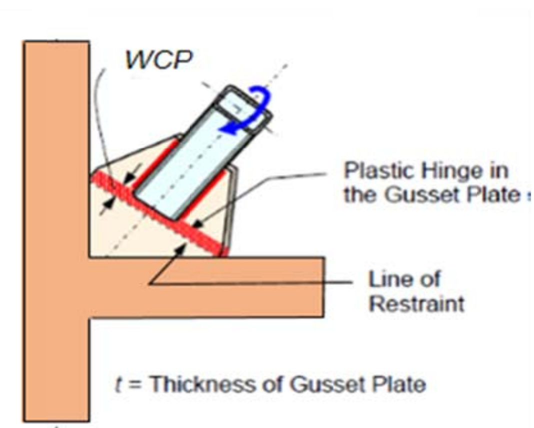


Fig. 6 Properties of WCP gusset plate connection

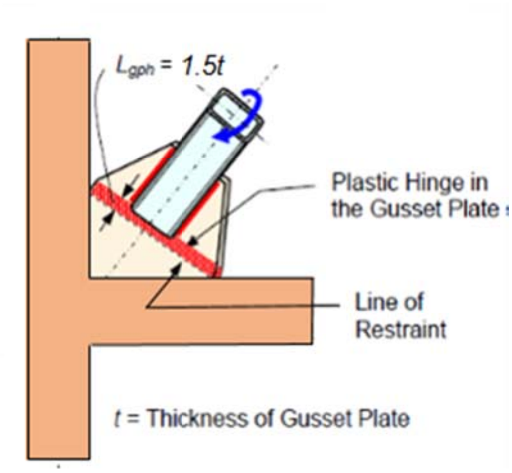


Fig. 7 Properties of 1.5t-CP gusset plate connection

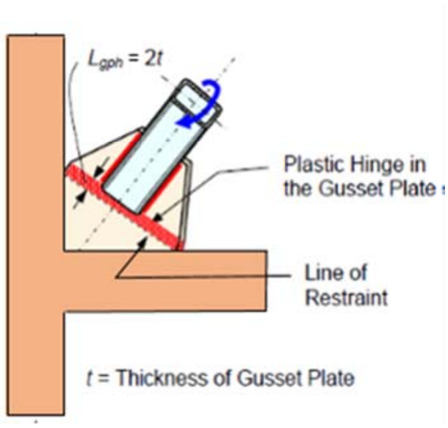


Fig. 8 Properties of 2t-CP gusset plate connection

2.3 Effect of different thicknesses on gusset plate connections

The distribution of internal stresses and buckling of braces are shown in Fig. 9. The brace buckled under compressive axial load. Then it yielded in tensile stresses.

The distribution of cracks in the concrete frame is presented in Fig. 10. The maximum amount of plastic hinges were in the braces and their connections. This result was consistent with Maheri and Akbari (2003).

As shown in Fig. 10, in a position where braces were in tension, the inside of the frame had reached to tensile cracks, and in a position where braces were in compression, the outer part of the frame cracked. Also compressive damages occurred at the beam-column connections.

Base shear - roof displacement diagram shows the amount of stiffness and strength of the frame with increasing the thickness of gusset plates (Fig. 11). Considering the effect of 2t, this change was noticeable, and the ductility of the plates increased too. If the plate thickness increased, the amount of buckling capacity raised and its strength was greater than before, so the braces buckled in higher loadings.

As shown in Fig. 12, yielding strengthen of gusset plate connections were represented due to their ifferent thicknesses in the braced frame system and unbraced one. The braced frame had more strength than the unbraced one about 12 times. These results are comparable to the findings of Goel and Lee (1992). In fact, the results show that the strength and ductility of reinforced concrete structures with steel braces increase considerably.

The stiffness of connection plates with different thicknesses are shown in Fig. 13. The stiffness of braced concrete frame systems were improved up to 3 times approximately by increasing the thicknesses of gusset plate connections compared to unbraced ones. It is worth noting that the stiffness of retrofitted concrete frames with steel convergent braces gained considerable improvement according to Viswanath and Prakash (2010).

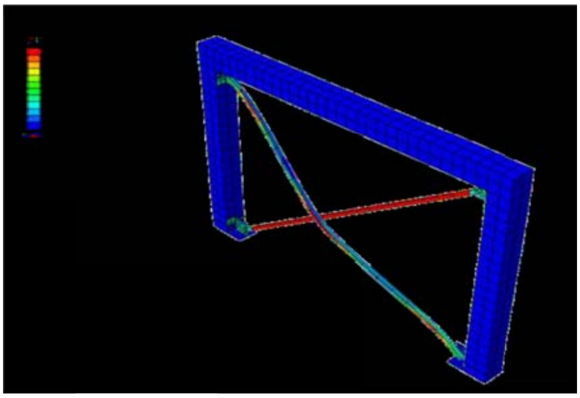
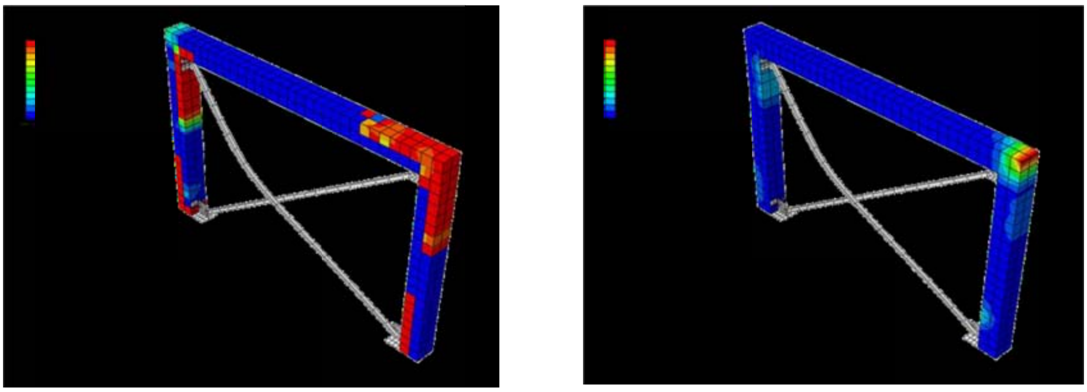


Fig. 9 Distribution of internal stresses and buckling of braces in numerical model



(a) Distribution of tensile cracks (b) Distribution of compressive demolitions

Fig. 10 Distribution of tensile and compressive cracks in concrete frame

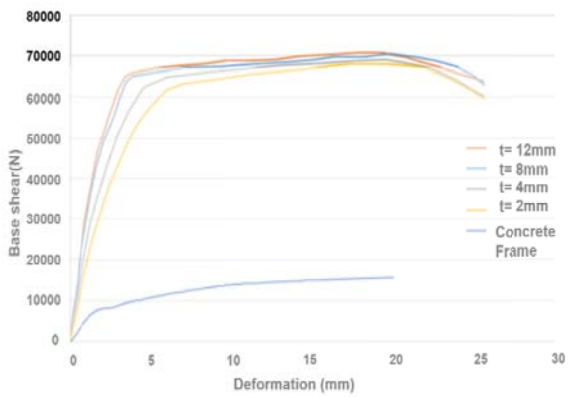


Fig. 11 Base shear-roof displacement diagrams of concrete samples with and without braces

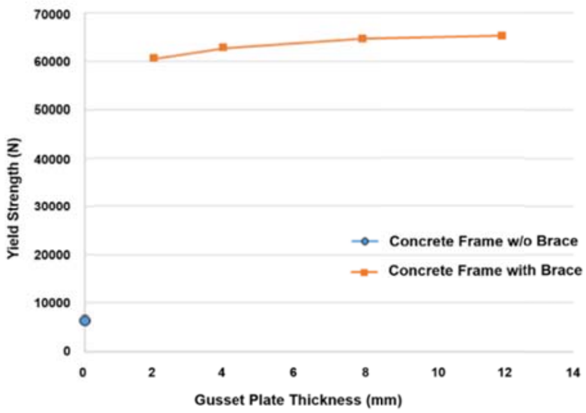


Fig. 12 Comparing yield strength of braced and unbraced frame systems including connection plates with different thicknesses

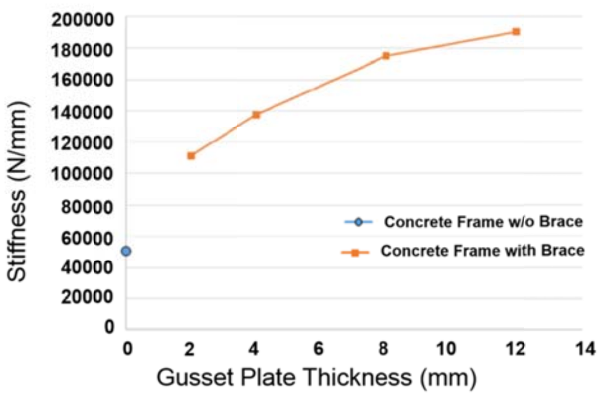


Fig. 13 Comparing stiffness of braced and unbraced frame systems including connection plates with different thicknesses

2.4 effect of 2t hinge zone length in gusset plate connections

Base shear – roof displacement diagrams for the retrofitted frame considering the effect of 2t hinge zone length (2t-CP), 1.5t hinge zone length (1.5t-CP) and without it (WCP) including different thicknesses (4 mm, 8 mm and 12 mm) are shown in Figs. 14-16. The operation of retrofitted concrete braced frames including 2t-CP samples was better than WCP and 1.5t-CP. In other words, the loading capacity of braced frames including 2t-CPs was much more than the other samples.

In calculation of ductility, final displacement in 80 percent of lateral resistance was divided to the end of linear behavior displacement. The ductility of concrete frame is presented in Table 3 having 1.5t-CP, 2t-CP and WCP in different thicknesses. The ductility of 2t-CPs in each thickness was more than the others. As shown in this table, the best choice is 2t-CP with 8mm thickness. It should be noted that choosing this connection plate optimized between different thicknesses because the results approximately were close in 8 and 12 mm thicknesses. It means, improving the thickness of gusset plate improves the ductility of the system up to a point. Afterwards, improving the thickness of the plate do not change the ductility results considerably.

Table 4 shows the lateral strength of concrete frame with different types of connection plates. 2t-CP samples had the maximum resistance compared to others in each thickness. Also it had the same results in 8 mm and 12 mm thickness; therefore, the optimized choice could be 2t-CP with 8 mm thickness. The ductility was calculated with dividing the displacement in 80 percent of lateral resistance into the displacement of the end of linear behavior of the model.

Table 3 Frame ductility with different types of connection plates

Connection plate	t=4 mm	t= 8 mm	t=12 mm
WCP	5.12	5.96	6.25
1.5t-CP	5.62	7.26	7.53
2t-CP	11	12.31	13.33

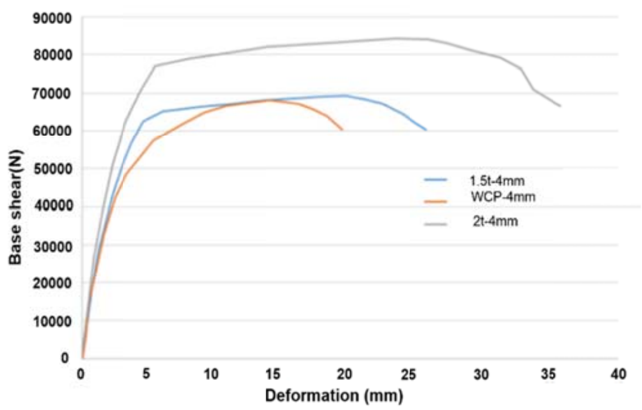


Fig. 14 Base shear- roof displacement diagram of the frame with 1.5t-CP, 2t-CP and WCP connection plates with 4 mm thickness

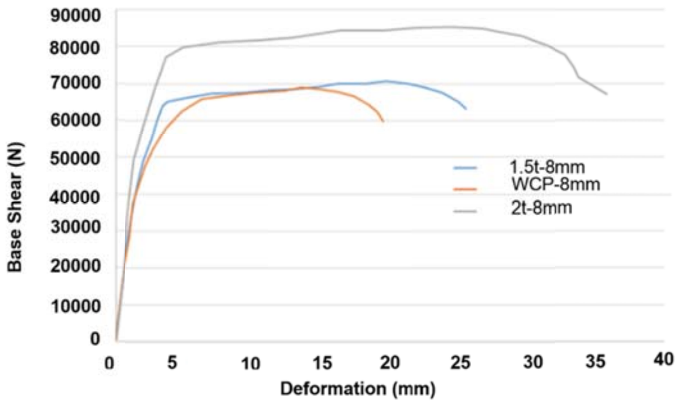


Fig. 15 Base shear- roof displacement diagram of the frame with 1.5t-CP, 2t-CP and WCP connection plates with 8 mm thickness

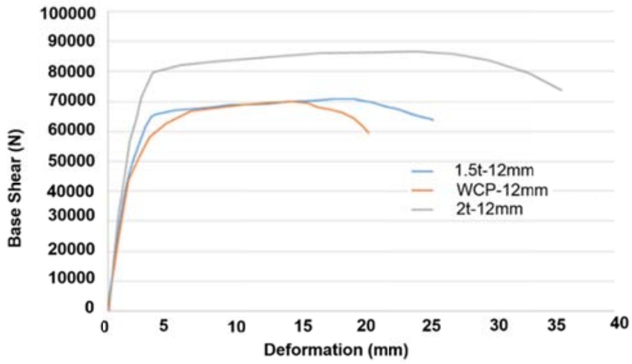


Fig. 16 Base shear- roof displacement diagram of the frame with 1.5t-CP, 2t-CP and WCP connection plates with 12 mm thickness

Table 4 lateral resistance of the frames with different types of connection plates (Newton)

Connection plate	t=4 mm	t= 8 mm	t=12 mm
WCP	68055	69167	70064
1.5t-CP	69234	70736	70947
2t-CP	84317	85333	86603

2.5 Examining the behaviour of concrete retrofitted frames with steel braces under cyclic loads

It was observed that gusset plates with 2t hinge zone length were very ductile, and their cyclic behaviour were desirable (Astaneh *et al*. 2006). This type of gusset plate easily qualified to be a part of braced frames. Rigid-body buckling of a brace occurred when two hinges formed in the gusset plates, due to the free length of the gusset plates between restrained points. The formation of three hinges resulted in the buckling mechanism. The 2t hinge zone length of the installed brace-gusset plate connection was observed. The gusset plates were stable while providing adequate cyclic rotational ductility to accommodate the brace cyclic buckling expected in special concentrically braced frames. It is worth noting that 2t was required in the gusset plate to provide space for the formation of the plastic hinge in the gusset (Astaneh *et al.* 2006).

In this research, retrofitted concrete frames with steel braces including different types of connection plates and the frame without retrofitting were tested under the cyclic load presented in Fig. 17.

The hysteresis diagrams of the samples with different types of connection plates are shown in Fig. 18. Unbraced concrete frames had the smallest energy absorption and lateral strength compared to the braced concrete frames. Using 2t-CPs as gusset plate connections in retrofitted concrete frames with steel braces had bigger lateral resistances than the others. Also their ductility was better than the other ones because of the more area below the graph compared to the other models.

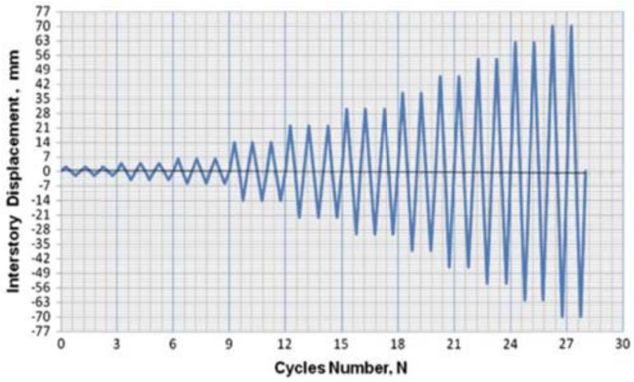


Fig. 17 Protocol of cyclic loading

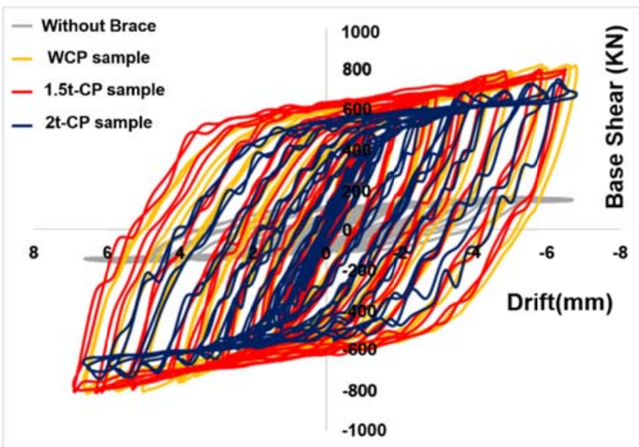


Fig. 18 Comparing hysteresis diagrams of the frame with 1.5t-CP, 2t-CP and WCP samples and the one without bracing

2.6 Examining failure modes of concrete retrofitted frames with steel braces

Retrofitted concrete frames with steel braces including different types of connection plates were analysed under increasing static loads and were loaded until the structural collapse. In Figs. 19-22 the distribution of internal stresses are shown in concrete frames before and after retrofitting with steel braces.

Based on Fig. 19, the concrete frame before retrofitting reached to rupture in 27 cm displacement on the top of the frame. This rupture was related to the tensile stresses in the bottom of the columns. Considering Figs. 20-22, it was noticeable that retrofitted concrete frames with steel braces including WCPs, 1.5t-CPs and 2t-CPs carry 39 m, 42 cm and 45 cm lateral displacement, respectively until they collapsed. This tensile rupture occurred in the bottom of the columns and gusset plates of the braces. Therefore, the load bearing capacity and ductility of the frame increased considerably with adding steel braces in the concrete frames.

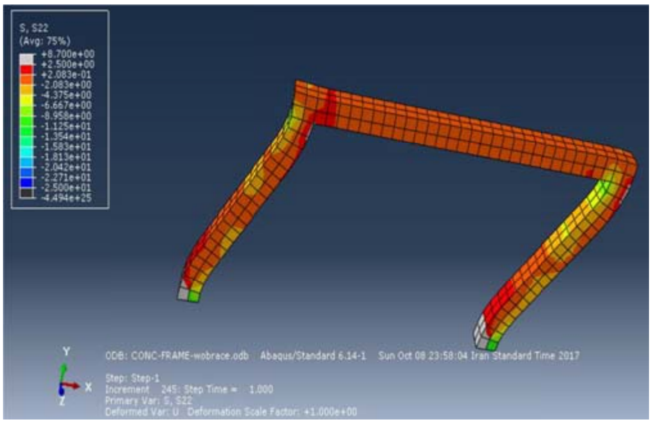


Fig. 19 Distribution of internal stresses in concrete frame without retrofitting

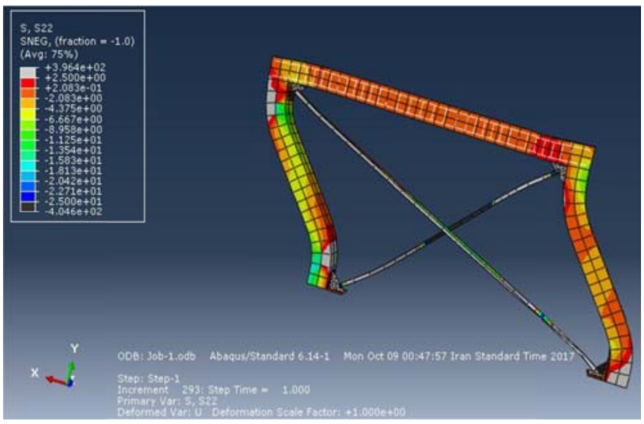


Fig. 20 Distribution of internal stresses in retrofitted concrete frames with steel braces including WCPs

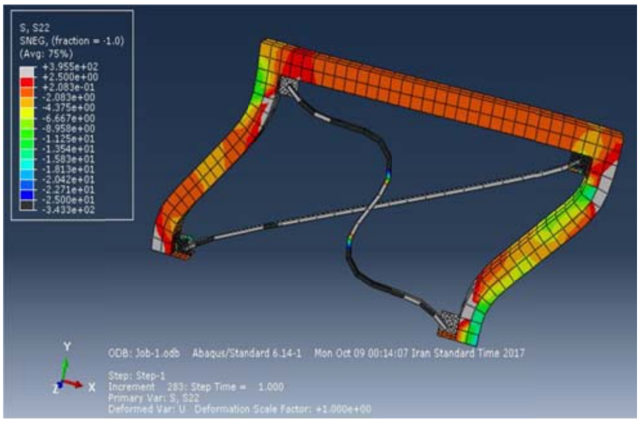


Fig. 21 Distribution of internal stresses in retrofitted concrete frames with steel braces including 1.5t-CPs

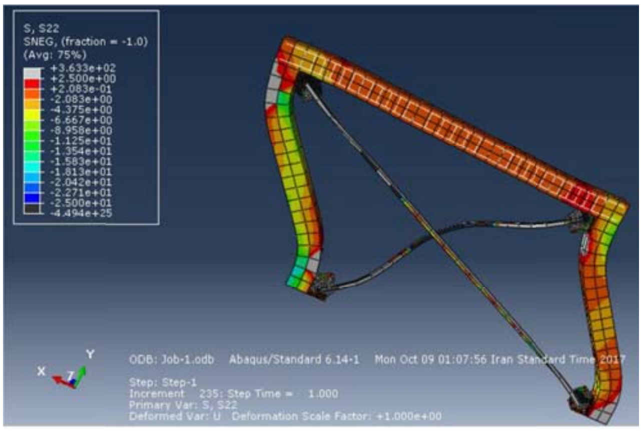
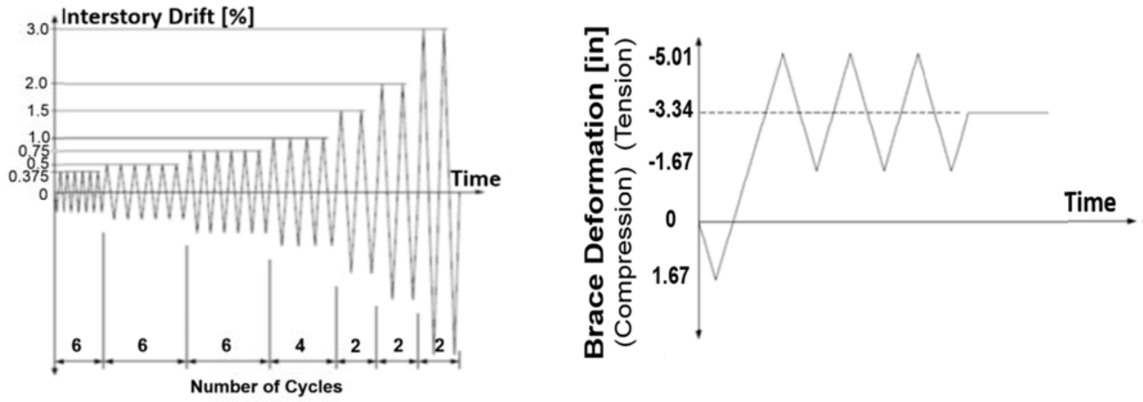


Fig. 22 Distribution of internal stresses in retrofitted concrete frames with steel braces including 2t-CPs

Generally, load bearing capacity and stiffness of the frame increased about 12 and 3 times, respectively if steel braces were added to flexural concrete frames. The strength and ductility of these frames with three types of connection plates, and their thicknesses were studied. 2t-CP samples increased the strength of the frame about 21 and 24 percent compared to 1.5t-CPs and WCPs, respectively. Also 2t-CPs increased the ductility of the frame about 2.06 and 1.65 times compared to WCPs and 1.5t-CPs, respectively, too. It should be noted that the frame capacity against earthquake improved with increasing its ductility and strength. In fact, using 2t-CP was the best choice in comparison with the others for retrofitting flexural concrete frames and the optimum thickness in this regard was 8 mm. The results showed that in higher thicknesses these values were similar. The results showed that 1.5t-CP samples were almost the same as WCPs. Therefore, using gusset plate connections with considering less than 2t hinge zone length were not effective because of the ductility and strength of the system.



(a) SAC basic loading history (b) SAC near field loading protocol

Fig. 23 SAC loading for verification

3. Buckling restrained brace (BRB)

Brace systems with no buckling are developed recently. The performance of these types of braces against earthquakes are better than the other braces due to their high lateral stiffness and ability to absorb much more earthquake energies. Recently, they are used in retrofitted concrete and steel structures Therefore, in the following some of these types of BRBs are studied. The models were in four shapes, and they were compared with the braces manufactured in previous section.

In this section, the second goal was to assess the optimum conditions for braces with no buckling in the concrete frames. Concrete used for coating the restrainer may cause some problems in performance due to its heavy weight. In order to overcome this problem, it is tried to investigate the effect of friction coefficient between the core and filler concrete. Then coating length was reduced without changing in core surface area. The results showed that the length of the concrete coating could be reduced up to 30 percent without the risk of buckling. The models were simulated in ABAQUS software, and they were analysed with static nonlinear analysis. The models were designed to determine the effect of non-adherent friction coefficient, the length of concrete coating and its removal, the shape of core profile and optimal dimensions of restrainer when the concrete coating was removed. Takeuchi *et al.* (2010) and Takeuchi *et al.* (2012) have some investigations in this field too.

3.1 Verification with ABAQUS software

Some tests were done with different axial loads on BRBs including different specifications by Blake et al. (2004). One of these specimens called Specimen 99-1 (Fig. 24) was used to verify the true modelling of the actual sample in this investigation. Time history analysis was applied based on the SAC basic loading history and the SAC near-field loading protocol of Black *et al.* (2004) that are shown in Fig. 23.

Simulation of elements were done in the part module of ABAQUS software. In this software, three types of elements were used to model three types of materials utilized in buckling restrained braces. For steel part, the two dimensional SHELL element was used. This element had the ability to model large nonlinear strains, large periods, nonlinear properties of materials and creep. Solid modelling was used to model concrete. This element had the properties to model cracking and crushing of concrete, as well as placing steel in concrete, but in this sample non-reinforced concrete was used.

The steel and concrete specifications were used in accordance with the research of Black *et al.* (2004). It should be noted that in the modelling of the concrete, only the value of the elastic modulus was introduced, and the nonlinear states of concrete was neglected. In other words, concrete is modelled in the elastic state. This assumption was valid before concrete cracking. Therefore, after analysing the values of tensions in concrete, the accuracy of the aforementioned assumption was investigated. The steel behaviour was considered as isotropic in the elastic and plastic forms.

In all elements considered in the major mesh, the standard library and linear geometric degree was used. Four-node S4R network was used in this modelling for the shell elements, and 8-node C3D8 network was considered for the solid ones (Table 5).

The inner surface of steel restrainer tube with concrete was in contact with the Tie fastener. Communication through this constraint resulted in the closure of all the degrees of freedom between surfaces, and no relative movement occurred between them. The tie constraint allowed linking the connections with different grids. There were also two rigid plates at the beginning and the end of the braces attached to the steel cores. Also, to simulate the contact between the steel and concrete, the interaction with contact behaviour of friction type was used with the friction coefficient of 0.1.

Table 5 Type and number of elements and meshes

Modeling part	Type of element	Number of elements	Mesh dimension (mm)
Concrete part	C3D8	1548	80*80
Inner core	S4R	308	80*80
Outer tube	S4R	860	80*80
Rigid plate	R3D4	32	60*60

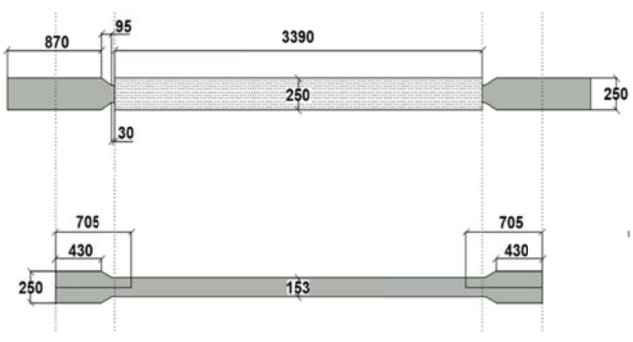


Fig. 24 The geometry of BRB- sample 99-1

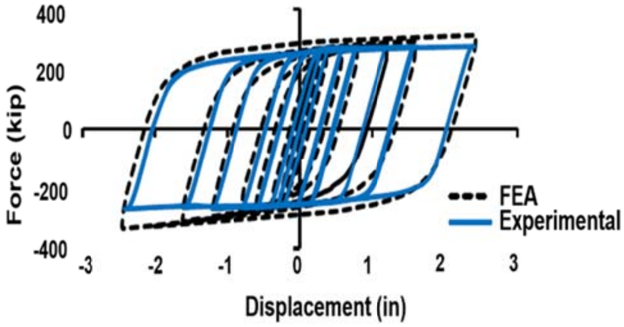


Fig. 25 Comparison of displacement- force curves of ABAQUS models and laboratory sample 99-1 for SAC basic loading history

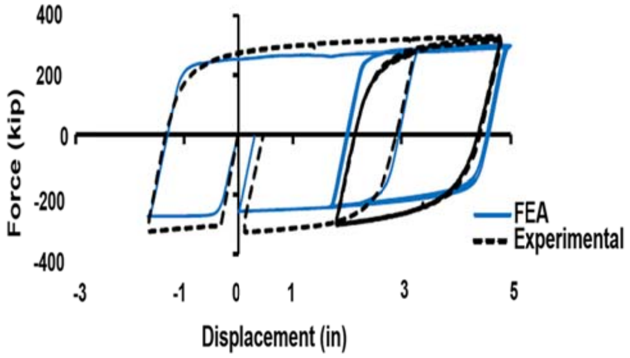


Fig. 26 Comparison of displacement- force curves of ABAQUS models and laboratory sample 99-1 for SAC near-field loading protocol

Also, the upper surface of the braces (loading location) were tightened in all directions except the loading direction.

In order to ensure the accuracy of modelling in ABAQUS software, the hysteresis loops were compared in both software and laboratory conditions. As shown in Figs. 25 and 26, the experimental and numerical results were quite close to each other.

3.2 Analysis and design of the first model

This part involves four types of buckling restrained brace models for strengthening concrete frames; each of them consist of a core profile, concrete coating and a steel restrainer tube. There was a gap (S) to deform the internal core profiles. The 0.5S step was also created for all the specimens. The difference between four models was only in friction coefficient of non-sticky materials. Concrete was used to as a core filler and non-sticky material between the core plate and filler concrete. This difference in coefficient for models A1 to A4 were 0, 0.1, 0.2 and 0.3, respectively. Fig. 27 shows the core and pillars ingredients of the steel and concrete restrainers as filler materials and non-adhesive materials between the central sheet and filler concrete of group A.

The hysteresis curves derived from the models in the group A are shown in Fig. 28. It could be observed that increasing the friction coefficient up to 0.3 did not affect the whole hysteresis loops, but increased the maximum compressive load in some cycles. In order to control this increasing, the β parameter was determined from Eq. (1). For all models in Group A, the values of β parameter are given in Table 6. According to AISC-341 (2005), the maximum allowed value of this parameter is 1.3. Therefore, all models A1 to A4 are within acceptable limits. It should be noted that the maximum load bearing capacity in each cycle indicates the ability of braces to withstand axial loads.

$$\beta = P_{compressive}/P_{tensile} \tag{1}$$

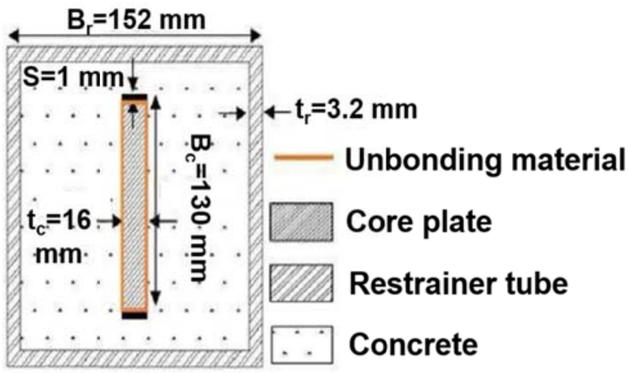


Fig. 27 Cross section of group A

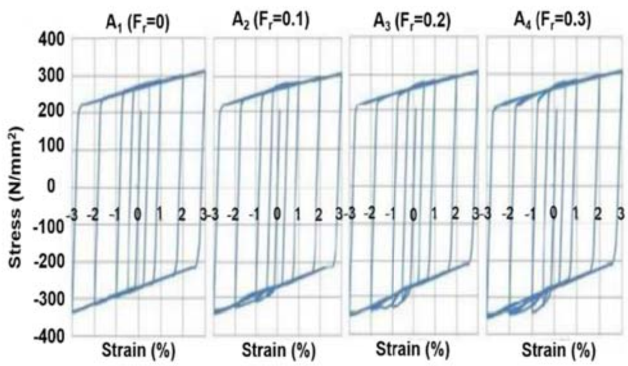


Fig. 28 Hysteresis behaviors of the models in group A

Table 6 The β parameter for group A models

Model	A1	A2	A3	A4
	1.071	1.107	1.150	1.194

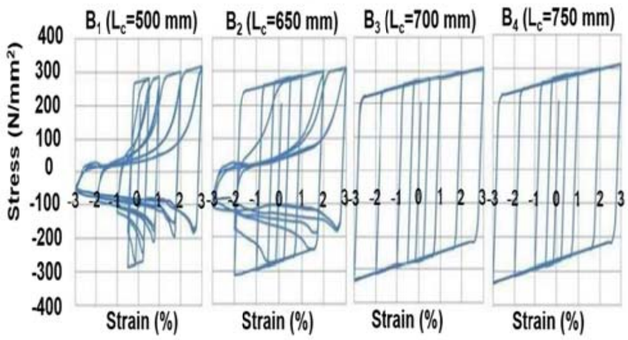


Fig. 29 Hysteresis behaviors of the models in group B

3.3 Analysis and design of the second model

The main components of the models of this group that were used in the concrete frame with BRB system were the same as the ones in group A. The difference between these two models was the length of restrained elements. Their lengths were changed in the models, but the length of plastic core plates remained constant. The friction coefficients among models were assumed to be zero. The lengths of concrete coatings in models B1 to B4 were 50%, 65%, 70% and 75% of the total length of the core plates, respectively. In this group, the optimal coating length was created without any changes in the core. This length was the shortest one in which no buckling occurred in the uncoated parts. Static nonlinear analyses showed that no buckling happened in braces B3 and B4 with the concrete coating lengths of 70% and 75% of the core plates of the braces. Buckling occurred along uncoated sections of B1 and B2 braces. Fig. 29 showed how the hysteresis behaviour of braces were affected by the lengths of the coatings. The curves of Fig. 29 show that decreasing the lengths of coatings up to 30% of the lengths of the core plates did not significantly alter in the hysteresis loops of the braces. Shrinking the coating up to half of its initial length (B1) caused buckling in the fifth cycle, which indicated a significant reduction in energy bearing capacity of the system. Also shrinking the coating up to 35% of its core plate length (B2) caused buckling in the 10th cycle.

3.4 Analysis and design of the third model

There were four models in this part; three models (C1 to C3) contained concrete filler, and C4 only controlled with steel coating in the structure of BRBs. The core profile in this group of braces were I-shaped ones. The general shape of the braces in this group are shown in Fig. 30. Models C1 to C3 were studied to see the effects of gap between core and filler concrete. Model C4 with a core profile like the other models in this group, showed bracing behaviour without any filler material as the restrainer. The boundary conditions were similar to the other groups, and the details are given in Table 7.

The obtained hysteresis loops are shown in Fig. 31. All the models with filler concrete (C1 to C3) exhibited sustained hysteresis behaviour. It is worth noting that only the I-shaped core of model C4 buckled and did not have acceptable behavior.

Table 7 Details of group C models

	I shaped core dimensions (mm)				Gap size (mm)	
Specimen	B_w	B_f	t_w	t_f	S1	S2
C1	50	84	11	7.5	0.5	0.5
C2					1	0
C3					0.5	0
C4					1	1

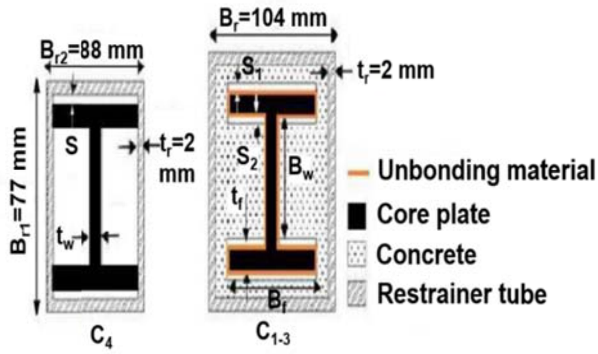


Fig. 30 Cross section of group C

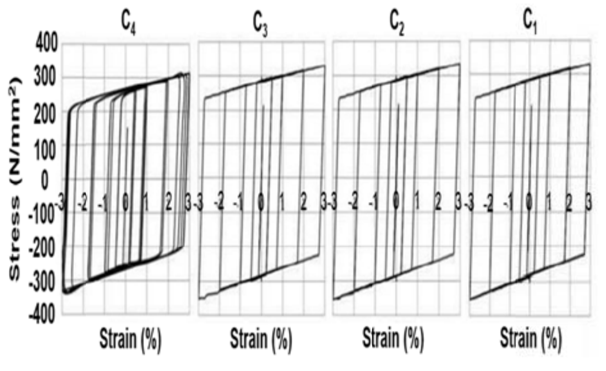


Fig. 31 Hysteresis behaviors of the models in group C

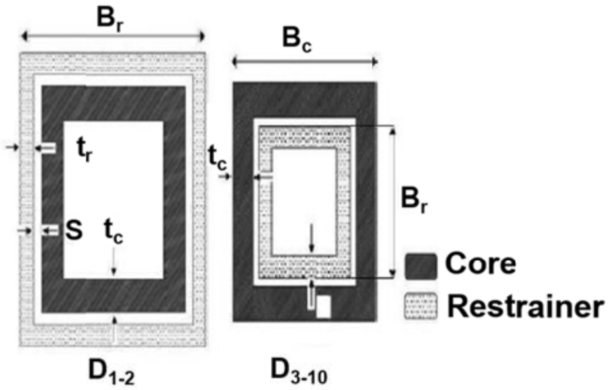


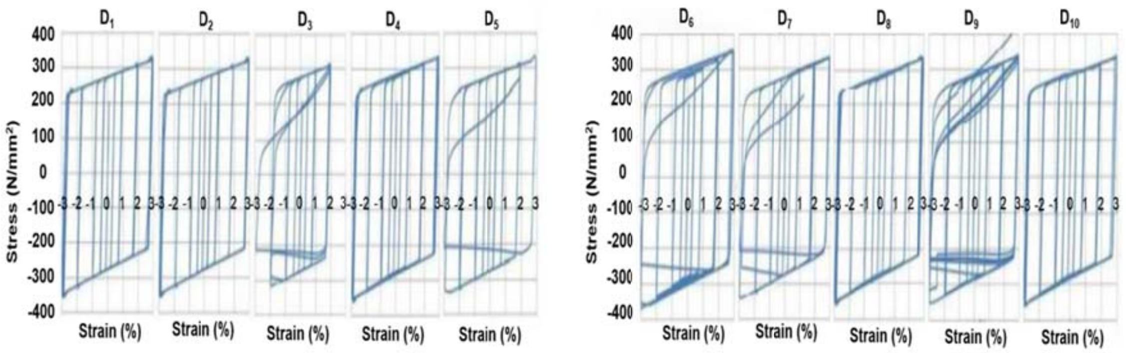
Fig. 32 Cross section of group D

3.5 Analysis and design of the fourth model

In the last group of the buckling restrained braces in the concrete frames, the concrete coating parts were removed. Each sample contained a steel core coated with a steel restrainer tube (Fig. 32). The steel tubes could restrain the buckling of the braces. In fact, the core profile changed to a box element with the same cross-sectional area. This group included ten models with the same core dimensions. The differences between the models were in the thicknesses and dimensions of the restrainer tubes. This group of models was designed to evaluate the possibility of removing filler coatings in BRBs. The thicknesses of the restrainers varied only between D1 and D2, and in the bracing behaviours with just one exterior restrainer tube, the only variable parameter was the restrainer tube thicknesses of the restrainers. In models D5 to D7, the effects of the gap between nucleus and the restrainer tube were studied on the cyclic behaviours of the bracings. Models D8 to D10 were designed for this purpose but with higher thicknesses. The boundary conditions were similar to the other groups. As shown in Table 8, the P_e / P_y ratio in the most models were less than 1.5. It should be noted that Pe and Py were the buckling load of restraining members and the yielding load of the core plates, respectively (Chou and Chen 2010).

The models in this group were studied to find the possibility of reducing the filler concrete without increasing the cross-section of restrainer tubes and cores. Consequently, eliminating the filler concrete from BRBs with rectangular outer steel tubes could lead to the buckling of the braces into the inside of the empty spaces. In fact, only the square cross sections of tube core profiles were appropriate for removing the filler concrete.

In general, increasing the P_e/P_y ratio led to growing the maximum load bearing capacity of the braces. However, Samples D4 and D10 showed different results due to the effect of parameter S. Regarding the models with internal restrainers, the cyclic behaviours improved with decreasing the gap S. This was due to the squeezing of cores with box profiles and the expansion of interiors. In this case, lower gap intervals had better behaviours. Comparing models D3 and D5 to D7 showed that reducing S size from 1 mm to zero did not help at all. However, according to D9 and D10 models, the effect of gap reduction was visible with just 0.1 mm increasing in restrainer thickness. Both hysteresis and compression behaviours were improved by reducing the value of S from 1 to



(a) (b)

Fig. 33 Hysteresis behaviors of the models in group D

0.5 mm without any significant change in the P_e / P_y ratio. Finally, in D-series models subjected to cyclic loadings, hysteresis loops in some cases became unstable for more than 2 percent core strains (Fig. 33). Moreover, the curves for models D3 and D5 to D7 showed buckling in strain values of less than 3 percent. The models D1 and D2 that had external restrainer tube had desired behaviours.

Among models with internal restrainers D4, D8 and D10 did not have buckling, and they had consistent hysteresis behaviours. Except for the gap between nucleus and restrainer, all other dimensions remained constant among D3 and models D5 to D7. The thicknesses of restrainers in models D3 and D1 were kept constant to examine the effect of core and restraining positions in the outer or inner restrainer tubes. Fig. 30 shows that model D3 with the size gap of 1 mm and the thickness of restrainer tube of 1.3 mm had an unstable behaviour. As noted, the bracing behaviour was expected to improve by reducing the size of the gap. In this case, models D5 to D7 were tested with the same D3 conditions but different S values. But again, its behaviour was not similar to a stable curve. Considering the proportional behaviour of model D4, it was observed that the thicknesses of 1.3 to 2 mm and the gap sizes less than 1 mm had acceptable behaviour. Accordingly, models D9 and D10 were designed with 1.4 mm thickness of restrainers. Model D9 including 1 mm gap had an unstable behaviour (Fig. 33). However, with decreasing the gap equal to 0.5 mm in model D10, the behaviour of this model improved and gained ideal hysteresis loop. Between the models that had interior restrainer tubes, the ones having the P_e/P_y ratio of less than 0.53 buckled in the strain range of less than 3 percent. Among the models with 1.3 mm thickness of restrainer, model D3 had the most inappropriate behaviour, and in the fourteenth cycle, there was a very high decreasing in strength at the strain of 3 percent. Decreasing the gap up to 0.7 mm in model D5 and 0.5 mm in model D6 could improve the behaviour in fifteenth and seventeenth cycles, respectively, but it could not prevent the bucking in these models. With increasing 0.1 mm into the thickness of the restrainer and having 1mm gap in model D9, the brace could endure the whole of the loadings without destruction. Model D10 had acceptable behaviour without any decreasing in strength and buckling up to the eighteenth cycle.

Table 8 Details of group D models

Model	B_r(mm)	t_r(mm)	B_c(mm)	t_c(mm)	S(mm)	P_e/P_y
1	77.6	1.3	73	8	1	1.337
2	79	2	73	8	1	2.11
3	55	1.3	73	8	1	0.466
4	55	2	73	8	1	0.689
5	55.6	1.3	73	8	0.7	0.481
6	56	1.3	73	8	0.5	0.492
7	57	1.3	73	8	0	0.520
8	56	2	73	8	0.7	0.824
9	55	1.4	73	8	1	0.499
10	56	1.4	73	8	0.5	0.527

4. Conclusions

Based on the analysis of the results, the following conclusions can be presented.

1. Increasing the gusset plate thicknesses developed the structural strength up to specific amounts. In fact, using 2t-CP samples with 8 mm thickness was the best choice between different types of samples studied here to provide room for the formation of plastic hinges in the gusset plates. They could improve the ductility up to 2.06 and 1.65 times compared to the ones without having hinge zone length (WCP), and the samples considering the effect of 1.5t hinge zone length (1.5t-CP). Also 2t-CP samples could improve the strength of the retrofitted concrete frames up to about 25 and 21 percent compared to WCPs and 1.5t-CPs, respectively.
2. Buckling restrained braces (BRBs) containing concrete coating had acceptable behaviors and stable hysteresis loops. The optimum coated length for BRBs was 70 percent of the length of plastic core transformation. It means, 15 percent of the coated length could be reduced from the two ends of the BRBs. This reduction decreased the weight of the BRBs considerably.
3. Only BRBs with the square cross sections of tube core profiles were appropriate for removing the filler concrete. Removing it from BRBs having different cross sections could lead to the buckling of the braces into the inside of the empty spaces.

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