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Structural behavior of inverted V-braced frames reinforced with non-welded buckling restrained braces

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Abstract. A concentric braced steel frame is a very efficient structural system because it requires relatively smaller amount of materials to resist lateral forces. However, primarily developed as a structural system to resist wind loads based on an assumption that the structure behaves elastically, a concentric braced frame possibly experiences the deterioration in energy dissipation after brace buckling and the brittle failure of braces and connections when earthquake loads cause inelastic behavior. Consequently, plastic deformation is concentrated in the floor where brace buckling occurs first, which can lead to the rupture of the structure. This study suggests reinforcing H-shaped braces with non-welded cold-formed stiffeners to restrain flexure and buckling and resist tensile force and compressive force equally. Weak-axis reinforcing members (2 pieces) developed from those suggested in previous studies (4 pieces) were used to reinforce the H-shaped braces in an inverted V-type braced frame. Monotonic loading tests, finite element analysis and cyclic loading tests were carried out to evaluate the structural performance of the reinforced braces and frames. The reinforced braces satisfied the AISC requirement. The reinforcement suggested in this study is expected to prevent the rupture of beams caused by the unbalanced resistance of the braces.

Keywords: V-type braced frames; non-welded; buckling restraining elements; cycle loading; seismic performance

1. Introduction

1.1 Background & purpose

Large-scale earthquakes causing casualties and property damages have been reported to occur increasingly often. An earthquake accompanies rapid lateral loads and sometimes paralyzes the city it strikes. Seismic design has been at the center of the discussion and effort to minimize the damages. Korean Building Codes (KBC) have been enacted and revised since the enforcement ordinances of construction law was revised in 1988 to make seismic design compulsory. Accordingly, seismic design codes have become more specific and the area of their application has been widened. Structures that were not seismically designed or do not satisfy seismic design codes

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(a) Brace buckling (b) Hysteric behavior

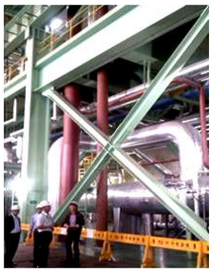
Fig. 1 Behavior of braced steel frame

are vulnerable to earthquakes. If a low and middle-rise power plant, in particular, is not earthquake-resistant, the loss will cover a wide spectrum of problems from shutdown to serious secondary damages to the industry until the facility is fully recovered. Therefore, the seismic performance of existing low and middle-rise steel structures should be reassessed to find which members need reinforcement. In low and middle-rise steel structures, braces are employed to resist most of lateral force. Braced frames and moment resisting frames are the most commonly used to resist lateral force in steel structures. Braced frames are divided into concentric braced frames (CBF) and eccentrically braced frames (EBF). Since the compressive stiffness and strength of braces resist lateral force, the smallest amount of steel is needed to provide the resistance to the horizontal loads applied to the structure and secure safety. Shear connection is used for most of the column-to-beam connections in a braced frame, so costs can be reduced. In addition, the design and structural analysis of braced frames can be easily conducted. For these reasons, braced frames are considered very efficient. However, steel braces which are subject to compressive load usually go through the deterioration in strength due to flexural buckling before yield stress as shown in Fig. 1, which interferes with energy dissipation and causes unstable behavior to the whole structure. Therefore, it is required to reinforce already-installed braces in order to ensure structural safety when seismic load is applied repeatedly. In general, welded or pin-connected steel plates are used as reinforcing members. Welding involves the risk of a fire and accompanies the inconvenience of arranging equipments. Another drawback is related to the residual stress resulting from welding thin steel plates. Therefore, this study suggests non-welded cold-formed steel elements to reinforce already-installed H-shaped slender braces in order to restrain flexural buckling and provide equal tensile strength and compressive strength. The structural performance and behavior of the compression braces reinforced as suggested were analyzed.

1.2 Structural review of already-installed braces

The structural safety of the braces in a currently used steel frame structure (OO Power plant) completed in 1990s was reviewed. Fig. 2 shows the front view and the picture of the structure. Current code requires that the K-type, V-type and inverted V-type braces in an ordinary concentric braced frame should resist both compressive force and tensile force. The braces in special concentric braced frames, in particular, should resist both compressive force and tensile force. However, the braces in the structure reviewed in this study were designed to resist tensile force only. Table 1 shows the slenderness ratio, compressive force ($P_n = F_{cr}A_s$) and yield force ($P_y = A_sF_y$) of the steel braces. The yield strength (F_y) of the steel was 210 MPa and the modulus of

(a) Braced frame elevation



(b) Inside view

Fig. 2 Mode analysis Result of OO Power Plant (In Korea)

Table 1 Axial force of members (Brace) review with OO Power plant

Member	Brace type	L (mm)	L/r_y	F_{cr} (MPa)	P_n (kN)	P_y (kN)	P_n/P_y (%)
H-300×300×15×16	K	8,239	113	120	1618	2831	57.2
H-250×250×9×14	/	7,824	124	115	1057	1936	54.6
H-200×200×8×12	X	13,816	**275**	23	149	1334	11.2
H-150×150×7×10	K	5,148	137	94	378	843	44.8

elasticity (E_s) was estimated to be 205 GPa. Thus, limit slenderness ratio ($\lambda = 4.71(E_s/F_y)^{0.5}$) was approximately 147. Ranging between 110 and 140, the slenderness ratio of K and / type braces was close to the limit. The ratio of compressive force to yield force was 40~50% in the three braces. However, the slenderness ratio of X type brace was 275 and the ratio of compressive force to yield force was 10%. The braces designed to resist tension only were evaluated as lacking the resistance to compressive force. In other words, small load can cause brace buckling and trigger unstable behavior of the whole structure. Therefore, the braces should be reinforced as required in the current code.

2. Current codes & previous studies

2.1 Current codes

The ASCE 7-10 (Minimum Design Loads for Buildings and Other Structures) provides the limitations of the height of ordinary and special concentric braced frame structures as shown in Table 2. Standard New Zealand (SNZ) requires that concentric braced frame structures should not be higher than 8 stories. Korea Building code (KBC) 2009 provides no requirement to ensure the ductile behavior of braced frames except *for* the width-thickness ratio, slenderness ratio and minimum compressive strength of brace members. Slenderness ratio is recommended not to exceed 200 and should satisfy Eq. (1). Eqs. (2) and (3) are provided for tensile strength and compressive strength, respectively.

$$\frac{KL}{r} \leq 4\sqrt{\frac{E}{F_y}} \tag{1}$$

Table 2 Seismic design requirement (ASCE 7-10)

Case \ Type	Structural system limitation including structural height, h_n (unit: ft)				
	A or B	C	D[d]	E[d]	F[e]
S-CBF (Special Concentric steel Braced Frame)	NL	NL	160 (48.8 m)	160 (48.8 m)	100 (30.5 m)
O-CBF (Ordinalry Concentric steel Braced Frame)	NL	NL	35 (10.7 m)	35 (10.7 m)	NP

NL : Not Limited, NP : Not Permitted, h_n: Structural height

$$R_y F_y A_g \tag{2}$$

$$0.3F_{cr} A_s \quad (P_n = F_{cr} A_s) \tag{3}$$

2.2 Previous studies

Reinforcing members which have higher stiffness than core members are used as exterior stiffeners to restrain the lateral buckling of core members. The braces reinforced with exterior stiffeners are called Buckling Restrained Braces (BRB). Table 3 summarizes the equations and stiffeners suggested by the previous studies on buckling restrained braces.

Inoue *et al*. (2001) suggested equations for stiffness and strength taking into consideration the initial deformation of buckling restrained braces. According to Eqs. (4) and (5), ductile behavior cannot be displayed if the ultimate moment of a core member is greater than the moment by buckling load. Fig. 3 shows the conceptual description of a brace which is undergoing deflection because of compressive force (*Ny*).

In order for the brace to reach yield without the buckling at the core member, ultimate moment at the center of the core member needs to be smaller than yield moment ($M_c^B < M_D^B$).

$$n_E^B = N_E^B / N_y \tag{4}$$

$$m_y^B = M_y^B / N_y \mathrm{L} \tag{5}$$

Table 3 Previous research of BRB

Author (year)	Suggested formula	Exterior stiffener	Filled
Watanabe *et al*. (1998)	$P_e / P_o \geq 1.5$	Steel tube	Con'c
Tsai *et al*. (2004)	$P_{\max} = \Omega\Omega_h \beta P_y$	Steel tube	Con'c
Chen *et al*. (2001)	$P_{\max} = P_e /(1+\frac{P_e \delta_0}{M_y})$	Plate	Square steel tube
Inoue *et al*. (2001)	$n_E^B = N_E^B / N_y$, $m_y^B = M_y^B / N_y L$	Square steel tube or Con'c Panel	
Kim and Park (2008)	$P_e / P_o \geq 1.5$	Steel tube	Square steel tube

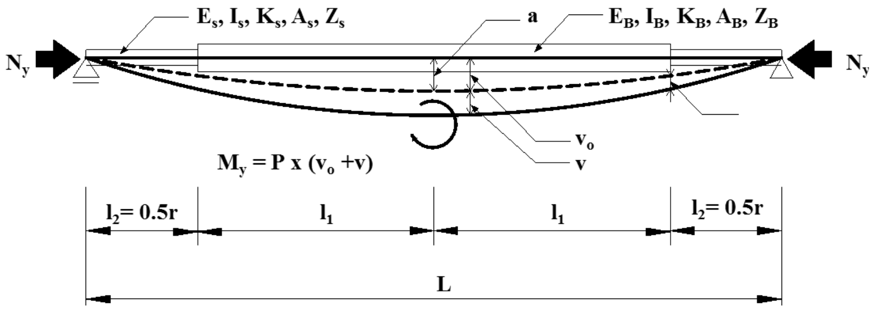
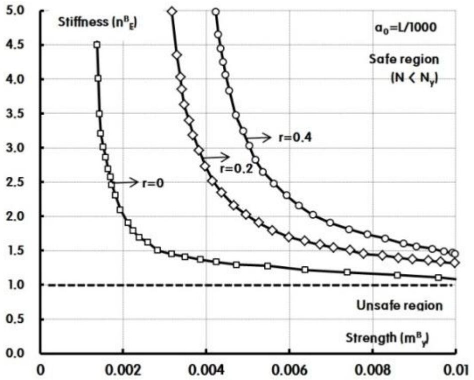
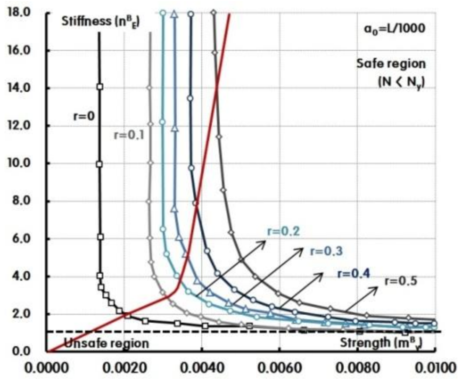


Fig. 3 Bracing Behavior Concept (Inoue *et al.* 2001)



(a) Non-reinforced



(b) Reinforcement design

Fig. 4 BRB Strength – Stiffness relationship based on reinforcement length

Fig. 4 is the graphic description of Eqs. (4) and (5) which were induced by the elastic buckling strength of the brace. X-axis and Y-axis represent stiffness and strength of the core member, respectively. Compressive force (*Ny*) greater than buckling load (*N*) means being unstable. Non-reinforced braces with higher length need the stiffeners with greater strength and stiffness. Thus, the cross-section and length of stiffeners can be estimated by the equations. In addition, the initial deformation of the brace is taken into account in deciding the cross-section of the stiffener.

3. Monotonic compressive loading tests

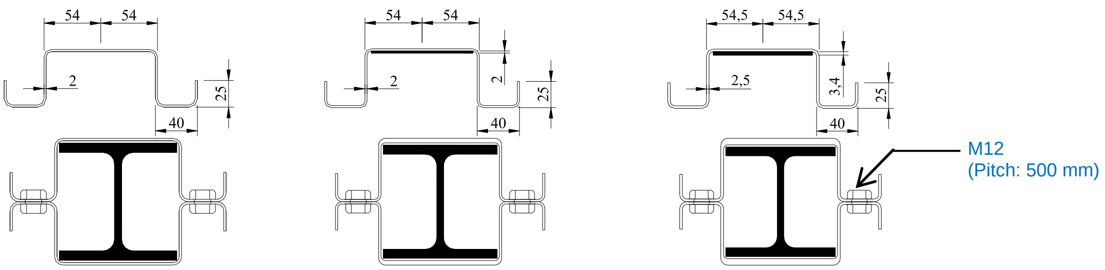
3.1 Test plan & setup

Table 4 shows the list of the brace specimens fabricated for monotonic loading tests to observe the improvement in the compressive strength of already-installed braces. The parameters in the test were reinforcement length, reinforcement area and segmentation. The cross-section and length of the H-shaped core members were H-100×100×6×8 mm and 3450 mm, respectively. In order to ensure the strength greater than the yield strength (*Py*) of the core members as well as ductile behavior, the length of the stiffeners (L_b) was set to be 75% or 90% of that of core members.

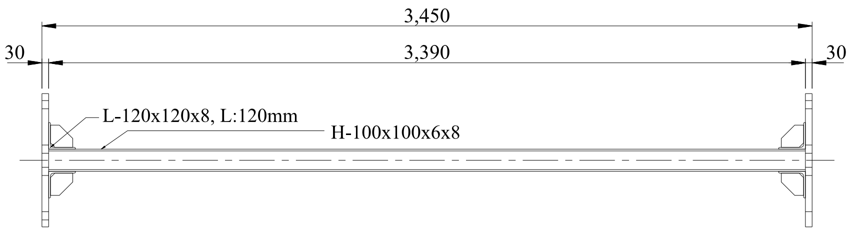
Table 4 Experimental specimen list

Specimens	I_{sy}/I_{by} (I_{sy} = 383 cm^4)	A_s/A_b (A_s = 2,164 mm^2)	L_b (mm)	Segment
M000N	-	-	-	-
M3675O	3.6	0.6	2,600	O
M3975X	3.9	0.8	2,600	X
M5075X	5.0	1.1	2,600	X
M3990X	3.9	0.8	3,100	X

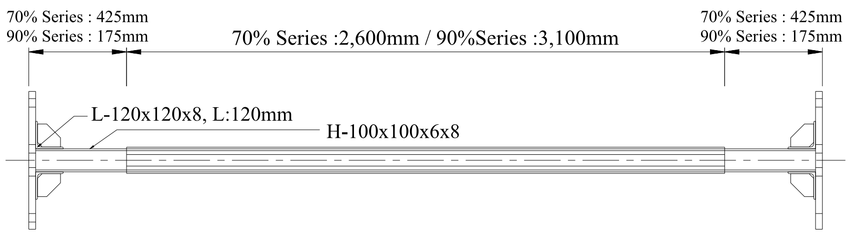
Reinforcement area (I_{sy}/I_{by}) was set to be 3.6, 3.9 or 5.0 times the area moment of inertia of core members as shown in Figs. 5(a)-(c). Segmentation was included in the parameters to consider the convenience in the process of installing the stiffeners. One of the specimens had a 3-segment stiffener. In order to prevent stress concentration in the two ends of the core members, end plates (30 mm) and angles (120×12×8 mm) were welded. Stiffeners were pin-connected (High tensile bolt, M12) and the distance between them was 200 mm. The yield strength of the steel used to make both the H-shaped core members and stiffeners was 325 MPa.



(a) Isy/Iby:3.6 (b) Isy/Iby:3.9 (c) Isy/Iby:5.0



(d) Non-reinforced specimens (M000N)



(e) Reinforced specimens

Fig. 5 Specimen detail

Table 5 Material properties

No.	Thickness (mm)	*Fy* (MPa)	Fu (MPa)	*Fy*/*Fu* (%)
1	2.0	258.1	350.6	74
2	2.5	257.3	401.8	64
3	3.4	291.8	405.7	72
4	6.0	309.9	437.3	71
5	8.0	293.3	420.7	70
Average		282.1	403.2	70

A 3,000 kN universal testing machine was used for monotonic loading at a speed of 0.01 mm per second. Loading was given up to the 80% of ultimate strength. Tensile sample test was conducted to identify the material properties of the steel plates. Table 5 shows the yield strength (F_y), tensile strength (F_u) and yield ratio observed from the test. The average yield strength and tensile strength were 282 MPa and 403 MPa, respectively.

3.2 Load-displacement relationship and failure mode

Table 6 and Fig. 6 show the load-displacement relationship of the specimens. All of the specimens exhibited higher-than-expected compressive strength. The compressive strength of non-reinforced specimen was 390 kN and that of reinforced ones ranged between 570 kN and 633 kN. Initial stiffness was similar among the specimens regardless of reinforcement, which meant that the stiffeners did not resist axial force but restrained lateral deformation. Non-reinforced specimen M0000N exhibited overall buckling and deterioration in strength from the early stage of compressive loading. Specimen M3675O with a segmented stiffener also showed rapid deterioration in strength. The others reached ultimate strength, which was followed by gradual decrease of strength.

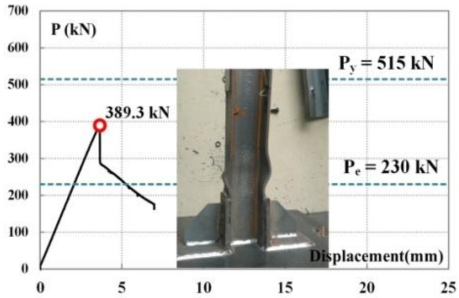
Failure modes were different among the specimens depending on segmentation and reinforcement length. Non-reinforced specimen M0000N and segmented specimen M3675O experienced overall buckling in the direction of weak axis, which was followed by failure. Specimens M3975X and M5075X having reinforcement length ratio of 75% exhibited local buckling at the edges of the core members and failed. Specimen M3990X underwent overall buckling in the direction of strong axis, which was followed by failure.

3.3 Influence of reinforcement ratios on strength and displacement

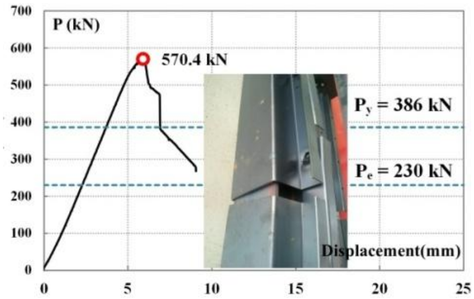
Fig. 7 shows the non-dimensional comparison of ultimate strength and displacement among the

Table 6 Result of experiment

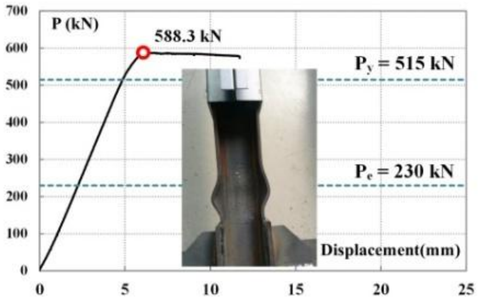
Specimens	*Py* (kN)	*Pu* (kN)	*Ki* (kN/mm)
M000N	379	390	110.4
M3675O	562	570	103.2
M3975X	575	588	103.7
M5075X	574	584	109.0
M3990X	615	633	108.6



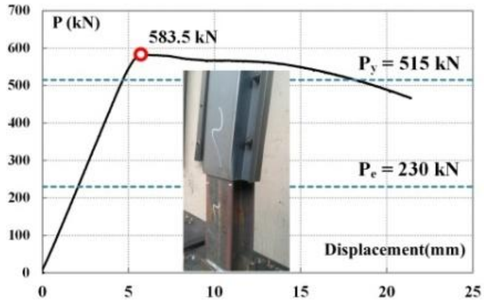
(a) M000N



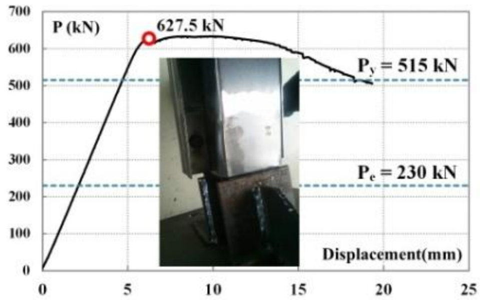
(b) M3675O



(c) M3975X



(d) M5075X



(e) M3990X

Fig. 6 Load-Displacement curve

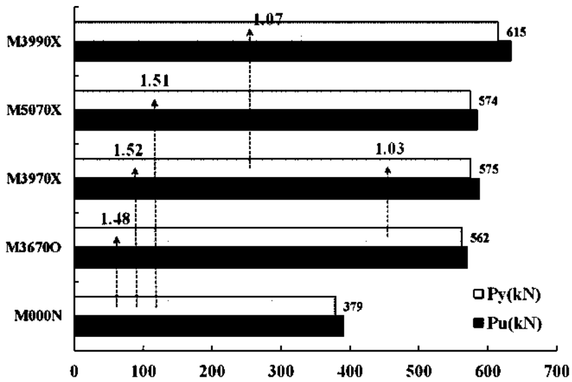


Fig. 7 Effect on reinforced ratio

specimens. When compared with non-reinforced specimen M0000N, specimens reinforced with non-welded stiffeners exhibited the increase in strength to approximately 1.5 times. The increase in reinforcement length ratio from 75% to 90% resulted in additional increase of about 10%. In short, the strength of the specimen with relatively small reinforcement length and area ratios increased to 1.5 times when compared with non-reinforced specimen. It was also observed that the increase of reinforcement area exerted greater influence on the improvement of ductility than the increase of reinforcement length.

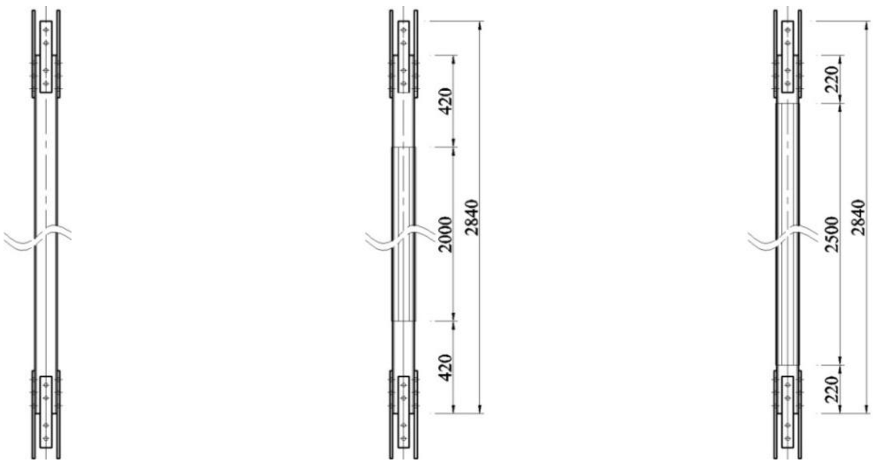
4. Inverted V-type braced frame test

4.1 Test plan

The specimens of the test were fabricated by duplicating the frame of a currently used structure in terms of brace shape, span and steel type. Since the 1-axis compression test stated above observed the behavior of a single brace, structural test was carried out to analyze the overall behavior of the frame for field application. Parameters in the test were reinforcement length, reinforcement area and segmentation. Four inverted V-type braced frame specimens listed in table 7 were fabricated with ASTM A36 Grade steel. The slenderness ratio (140) and the cross-section (H-100×100×6×8) of braces as well as the cross-section of columns and beams (H-400×400×13×

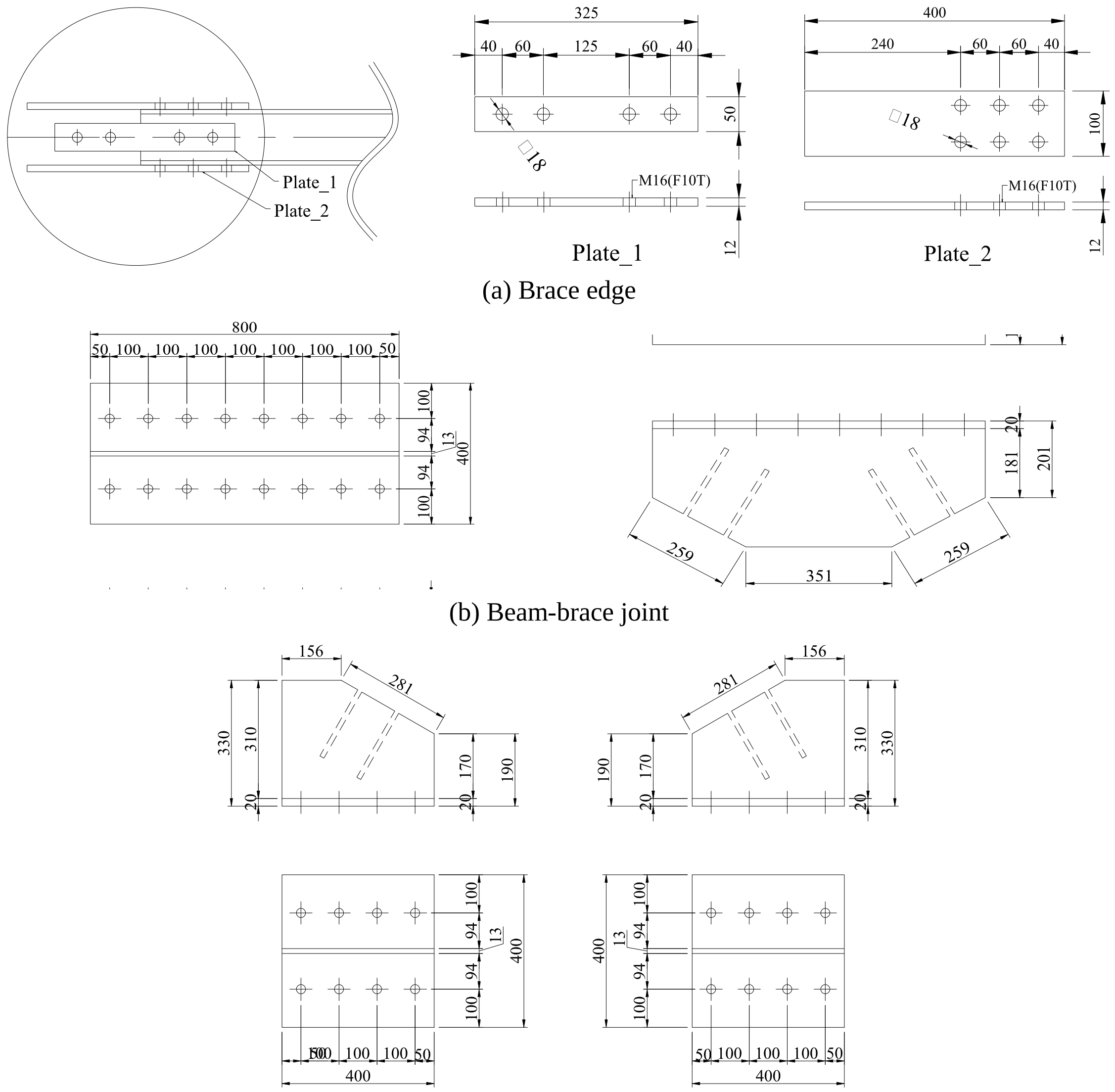
Table 7 Frame Specimen list

Specimens	*Isy*/*Iby*	*As*/*Ab*	*Lb* (mm)
Y0000N	-	-	-
Y3670X	3.6	0.6	2,000
Y5090X	5.0	1.1	2,500
Y5090O	5.0	1.1	2,500



(a) Y0000N (b) Y3670O (c) Y5090X, Y5090O

Fig. 8 Specimen (brace) detail



(a) Brace edge

(b) Beam-brace joint

(c) Base-plate joint

Fig. 9 Detail of connecting element

21) were identical among the specimens. Because the purpose of the test was to observe the behavior of the braces, base plates of the specimens were pin-connected to prevent stress transfer to columns and beams. Fig. 8 shows the details of the specimens. The braces were 2,840 mm in length. So, the length of the stiffener was 2,000 mm in Y3670X and 2,500 mm in Y5090X and Y5090O. All of the connecting elements were pin-connected. Figure 9 shows the details of base plate and gusset plate.

As shown in Fig. 10, a 1,000 kN actuator was used in the test. Since the specimens were made of slender elements, 4 lateral supports were placed to prevent lateral buckling. Teflon plates were placed to minimize the friction between the lateral supports and columns.

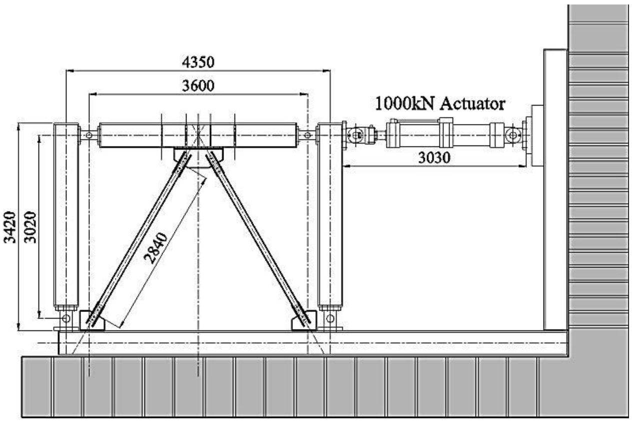


Fig. 10 Test setting

4.2 Loading & evaluation method

The loading program for buckling restrained braced frame test provided by the AISC Seismic Provision (2010b) was used. The provision recommends that loading be based on the yield displacement (D_{by}) of core members in Step 1 and target interstory drift (D_{bm}) increasing by 0.5 from Step 2.

- Yield displacement (D_{by}) of core member

- Core member (H-100×100×6×8) → (A_s = 2,190 mm^2 / Total length: 3,450 mm)
- $D_{by} = \frac{PL}{AE} = \frac{F_y A_g L}{A_g E} = \frac{325MPa \times 3450 \text{ mm}}{205000 \text{ MPa}} = 3.96 \text{ mm}$

- Target interstory drift (D_{bm})

According to the provision, interstory drift (D_{bm}) should not be smaller than 1% of floor height and should be smaller than 5D_{by}. 1% of floor height was 34.2 mm and 5D_{by} was 19.80 mm. Therefore, target interstory drift was decided to be 19.80 mm.

- Interstory drift (D_s) of frame

The horizontal displacements of frames similar to the axial deformations of braces need to be estimated. Table 8 shows the horizontal displacements.

Table 8 Displacement according to the story drift

Loading No.	1	2	3	4	5	6
	1.0D_{by}	0.5D_{bm}	1.0D_{bm}	1.5D_{bm}	2.0D_{bm}	2.5D_{bm}
Axial deformation	3.9 mm	9.8 mm	19.5 mm	29.3 mm	39.0 mm	48.8 mm
Story drift	3.4 mm	8.4 mm	16.9 mm	25.3 mm	33.8 mm	42.2 mm
Story drift ratio	0.1%	0.25%	0.49%	0.73%	0.98%	1.22%

4.3 Hysteretic behavior of inverted V-type braces

All of the specimens showed slip of pinned connections in early stage of loading. As frictional force changed to bolt bearing force, the behavior of the specimens deteriorated rapidly after ultimate strength. Fig. 11 shows load-displacement relationship of the specimens under cyclic loading. Table 9 summarizes compression and tension at each interstory drift.

Although estimated buckling load (230 kN) was reached in non-reinforced specimen Y0000N, both right and left braces buckled between step 4 and step 5. Since the specimen was made of slender elements, overall buckling after elastic behavior caused rapid deterioration in strength.

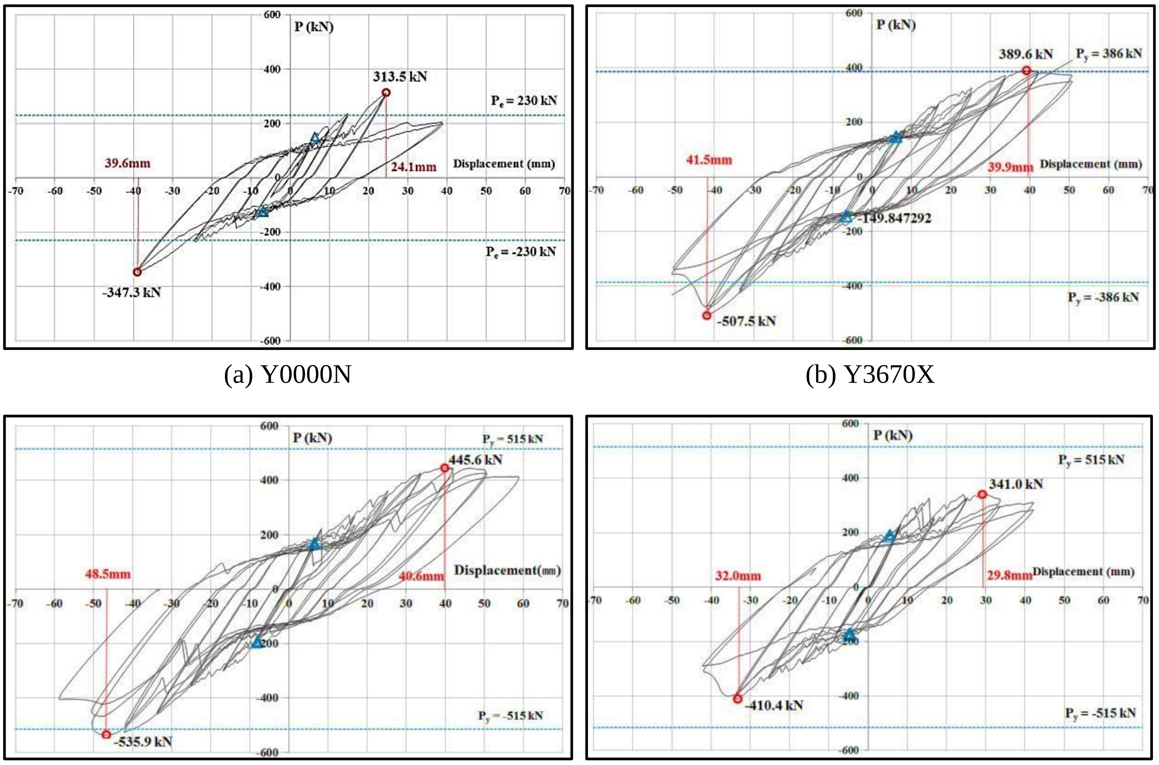
Specimen Y3670X displayed strength greater than expected (386 kN) but the compressive strength was 1.3 times as great as the tensile strength. In spite of the supports to prevent lateral buckling, eccentricity occurred at ultimate strength as deformation increased. Both braces showed lateral local buckling and the overall frame slanted laterally after ultimate strength. But, it was more stable than non-reinforced specimen Y0000N in terms of strength and post-buckling behavior. The reinforced specimen failed due to the local buckling at the edges of its core member. Its behavior was relatively stable even after buckling.

Y5090X presented the best result among the specimens. Thanks to the increase of reinforcement length and reinforcement area, strength and the behavior after ultimate strength were quite stable. However, estimated tensile strength (515 kN) was not reached in left brace due to the deformation of base plate. Right brace (compression brace) resisted load up to 535.9 kN, which was greater than expected. Its behavior after buckling was also stable.

The only difference between Y5090O and Y5090X is that the stiffener of the former was composed of 3 segments for the sake of convenience in fabrication. Estimated load (515 kN) was

Table 9 Compressive and Tension each story drift

No.	AISC	Story drift (%)	No. of cycle	Y0000N		Y3670X		Y5090X		Y5090O	
				C_{max} (kN)	T_{max} (kN)	C_{max} (kN)	T_{max} (kN)	C_{max} (kN)	T_{max} (kN)	C_{max} (kN)	T_{max} (kN)
1	D_{by}	0.1	1	72	107	69	76	67	82	64	86
			2	71	109	70	77	67	82	63	88
2	$0.5D_{bm}$	0.25	1	156	188	169	169	186	223	180	229
			2	153	173	169	175	187	219	179	235
3	$1.0D_{bm}$	0.49	1	185	236	245	268	300	235	281	325
			2	165	227	245	267	297	271	281	292
4	$1.5D_{bm}$	0.73	1	234	313	313	325	360	360	336	339
			2	230	304	308	303	352	343	326	325
5	$2.0D_{bm}$	0.98	1	347	205	424	370	457	425	410	340
			2	338	197	412	360	454	421	393	303
6	$2.5D_{bm}$	1.22	1	-	-	507	389	525	445	399	309
			2	-	-	485	381	502	441	289	283
7	$3.0D_{bm}$	1.47	1	-	-	473	380	535	443	-	-
			2	-	-	339	349	468	424	-	-



(a) Y0000N (b) Y3670X

(c) Y5090X (d) Y5090O

Fig. 11 Load-displacement relationship

reached in none of the braces in Y5090O. Since the segmentation of the stiffener interrupted continuous resistance to lateral force, cyclic loading caused buckling at the discontinued parts, which resulted in the deterioration in confinement effect of the stiffener. The specimen also experienced the deformation of the base plate. In designing reinforced braces, base plates should be revised appropriately.

5. Analysis of frame test result

5.1 Deformation capacity

The AISC requires that the displacement of braces after buckling (δ_{max}) be 10~20 times yield displacement (δy) in order for them to be employed in seismic design. FEMA-356 suggests ductility 5 for LS (Life Safety) standard and ductility 7 for CP (Collapse Prevention) standard. Table 10 shows the deformation capacity of the specimens in terms of yield displacement and post-buckling displacement obtained from the test.

As shown in the table, none of the specimens satisfied the AISC's ductility requirement (10~20). It was expected that the improvement of boundary condition or the increase of reinforcement area would enhance deformation capacity. Presenting ductility above 5, mostly

Table 10 Deformation Capacity each specimen

Specimens	*Ki* (kN/mm)	P_{max} (kN)	δy (mm)	δ_{max} (mm)	$\delta_{max}/\delta y$
Y0000N	21.3	313.5	8.01	24.1	3.02
	21.5	347.3	8.20	39.6	4.83
Y3670X	21.4	389.6	7.50	39.9	5.32
	21.6	507.5	6.20	41.5	6.69
Y5090X	25.5	445.6	7.50	40.6	5.41
	21.2	535.9	9.50	48.5	5.11
Y5090O	27.0	341.0	6.98	29.8	4.27
	22.0	410.4	6.50	32.0	4.92

reinforced specimens satisfied LS (Life Safety) standard. Specimen Y3670X showed quite satisfactory deformation capacity considering relatively small reinforcement area and reinforcement length ratios.

5.2 Compressive strength adjustment factor (β)

Compressive strength adjustment factor (β) is the ratio of maximum compressive force (C_{max}) to maximum tensile force (Tmax). The AISC recommends β not be greater than 1.3 because big difference between tensile strength and compressive strength in an inverted V-type braced frame increases the possibility of beam failure due to horizontally-applied unbalanced force after the yield of tension brace. Fig. 12 shows the adjustment factors obtained from Table 12. In all of the specimens, compressive force was greater than tensile force. The stiffeners provided lateral confinement to prevent buckling of the core members when compressive force was applied. The adjustment factors in non-reinforced specimen ranged between 0.65 and 1.72, which did not satisfy the AISC recommendation. Ranging between 0.72 and 1.3, the adjustment factors in reinforced specimens met the recommendation. It was deduced that weak-axis reinforcing stiffeners employed in braces would prevent beam failure caused by the unbalanced forces applied to the braces.

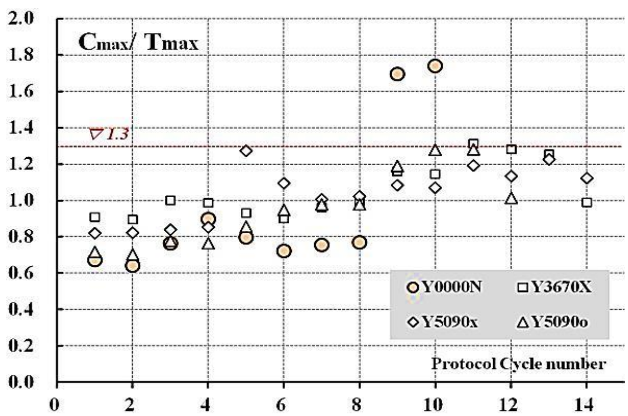


Fig. 12 Compressive strength of adjustment factor (β)

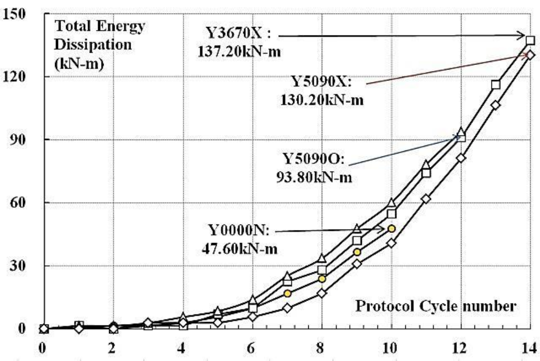


Fig. 13 Energy dissipation

5.3 Energy dissipation

Energy dissipation is one of the major structural factors to resist earthquakes and estimated as the sum of the areas obtained from load-displacement relationship graph. Fig. 13 shows energy dissipation of the specimens estimated by adding the areas together. The pattern of energy dissipation was similar among the specimens until cycle 5 and began being divergent from cycle 6.

When compared with non-reinforced specimen Y0000N, the toughness of reinforced specimens Y3670X and Y5090X increased to 2.7 times and that of Y5090O having a segmented stiffener to 1.9 times. Thus, it was deduced that the reinforcement of braces would double the deformation capacity and improve seismic performance.

6. Field application

The built-up non-welded stiffeners suggested in this study were applied to the inverted V-type braces on the 1st floor of a 154 kV transformer in ❍❍ Substation. Fig. 14 shows the Y5090X stiffeners applied to the transformer. The purpose of this study is to reinforce braces to resist compressive force and tensile force equally and prevent beam failure caused by unbalanced forces. The reinforcement can be made by dry construction method and does not require long term of works. A follow-up research is planned to be made to find the spots requiring reinforcement and the amount of reinforcement and draw the optimum seismic design.

(a) Before

(b) After

Fig. 14 Field application

7. Conclusions

This study suggests how to reinforce the braces in a frame under compressive force to solve the problem of unstable behavior caused by buckling before yield stress. Structural tests and finite element analysis were carried out with variables such as reinforcement area and reinforcement length to evaluate the compressive behavior of reinforced braces. In addition, cyclic loading tests were made with full-scale frame specimens to evaluate the seismic performance and behavior of inverted V-type braced frames. The reinforcement method suggested in this study was applied to a transformer currently in use.

- In the monotonic loading tests conducted with H-shaped braces reinforced with weak axis stiffeners, the reinforced braces showed strength greater than the yield strength of shear plane and displayed ductile behavior after ultimate strength. It was deduced that the suggested non-welded stiffeners would prevent the buckling of H-shaped core members.
- Reinforced braces showed similar stiffness and strength regardless of the variables. The similarity of stiffness among the braces implied that only the H-shaped core members resisted the compressive force in the elastic zone. That the stiffeners resisted buckling stress perpendicular to the axis after yield implied that all of the reinforced braces provided strength greater than yield strength.
- Displacement ductility increased by 60~80% depending on reinforcement area and reinforcement length. The increase of the second area moment of the stiffeners was more effective in improving deformation capacity than the increase of reinforcement length.
- When the reinforcement length ratio of the analysis models was 70%, the ductility of the braces increased as much as 40% or 80% depending on reinforcement area ratio. It was found that the braces with reinforcement length ratio 70% would need reinforcement area ratio 5 to satisfy LS (Life Safety) standard provided by FEMA.
- When the reinforcement length ratio of the analysis models was 90%, the ductility of the braces increased as much as 24% or 37% depending on reinforcement area ratio. It was found that the braces with reinforcement length ratio 70% would need reinforcement area ratio 3.9 to satisfy LS (Life Safety) standard.
- The AISC recommends that the ratio of maximum compressive force to maximum tensile force not exceed 1.3 to prevent beam failure caused by the unbalance force applied to the braces in a frame. The ratio ranged between 0.72 and 1.3 in the reinforced specimens. Reinforced specimens showed the increase of accumulated energy dissipation, one of the major factors in seismic performance, to 1.9~2.7 times when compared with non-reinforced one.

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Symbols

A_b	(mm^2)	Area of stiffeners
A_s	(mm^2)	Area of core member.
A_a	(mm^2)	Area of gross section ($A_b + A_s$)
D_{bm}	(mm)	Target interstory drift
D_{by}	(mm)	Yield displacement
F_y	(MPa)	Yield strength of Material
F_u	(MPa)	Tensile strength of Material
I_{sy}	(mm^4)	Moment of inertia of core member
I_{by}	(mm^4)	Moment of inertia of stiffeners
I_{sy}/I_{by}	Non	Reinforcement ratio
KL	mm	Effective Length
K_i	kN/mm	Initial stiffness
L_b	Mm	Length of the stiffeners
M_C^B	kN-m	Maximum Moment
M_y^B	kN-m	Yield Moment
N_y	kN	Applied axial load ($A_S \times F_y$)
P_e	kN	Euler's buckling load (= $N^B{}_E$)
P_o	kN	Compressive strength for gross section
$P_{\max}$	kN	Maximum Compressive load of specimen
P_r	kN	Normal compressive strength
P_y	kN	Yield strength of specimen
δ_o δ_o	mm	Initial deflection
$\delta_{\max}$	mm	Maximum displacement
λ	Non	Slenderness ratio
μ	Non	Displacement ductility ratio
r	mm	Radius of rotation