Steel and Composite Structures, Vol. 8, No. 5 (2008) 389-402
DOI: http://dx.doi.org/10.12989/scs.2008.8.5.389

Full scale test and alnalytical evaluation on flexural behavior of tapered H-section beams with slender web

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(Received July 16, 2007, Accepted August 27, 2008)

Abstract. In December 2005, one(A) of the two pre-engineered warehouse buildings in the port of K City of Korea was completely destroyed and the other(B) was seriously damaged to be demolished. Over-loaded snow and unexpected blast of wind were the causes of the accident and destructive behavior was brittle fracture caused by web local buckling and lateral torsional buckling at the flange below rafter. However, the architectural design technology of today based on material non-linear method does not consider the tolerances to solve the problem of such brittle fracture. So, geometric non-linear evaluation which includes initial deformation, width-thickness ratio, web stiffener and unbraced length is required. This study evaluates the structural safety of 4 models in terms of width-thickness ratio and unbraced length using ANSYS 9.0 with parameters such as width-thickness ratio of web, existence/non-existence of stiffener and unbraced length. The purpose of this study is to analyze destructive mechanism of the above-mentioned two warehouse buildings and to provide ways to promote the safety of pre-engineered buildings.

Keywords : tapered beam; local buckling; lateral torsional buckling; peb(pre-engineered buildings) system; ductile design.

1. Introduction

The improvement of the quality of construction materials and the advance of construction technology have led to the increase in the number of long-span large steel-frame buildings. In the construction of structures with sloping roof, due to the increasing pressure of labor costs, gabled portal frame has replaced truss system. The application of constant cross-sectional area to the long-span buildings of such structure results in the waste of construction materials because the most vulnerable part under ultimate stress decides the size of cross-sectional area of all the materials used in the building. In contrast, tapered members are mainly used in PEB(Pre-Engineered Buildings) construction where material design based on design stress dictates the production of materials (Dowling *et al.* 1982, Hwang *et al.* 1991). Because stresses are appropriately loaded to materials according to the size of cross-sectional area, such

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materials contributes to the improvement of economical efficiency of long-span steel-framed buildings. PEB system was originally used for steel-framed military hangars or warehouses in the US and UK and priority was given to the reduction of the term of works and structural solidity. Today, it is widely used in private sectors such as industrial structures in advanced countries. Since it features advantages including reduction of material/ labor costs, structure weight and the term of works and convenience in construction works, the employment of the system will proliferate fast. In December 2005, over-loaded snow and unexpected blast of wind completely destroyed one(A) of the two pre-engineered warehouse buildings in the port of K City of Korea and the other(B) was seriously damaged and had to be demolished. Though the buildings collapsed due to snow-load which was three times the weight of permitted value, destructive behavior was brittle fracture resulting from local buckling at the web under compressive force and lateral torsional buckling at the flange below rafter as shown in Fig. 1. The architectural design technology of today based on material non-linear method does not consider the tolerances to solve the problem of such brittle fracture[AISC(2001)]. So, geometric non-linear evaluation which includes initial deformation, width-thickness ratio, web stiffener and unbraced length is required (Li *et al.* 2003, Chu 2005, Yau *et al.* 2006, Andrade *et al*. 2007, Larue *et al.* 2007).

Since thin steel materials are used, initial deformation caused by the influence of heat is unavoidable even though they are elaborately produced. With the tolerance provided by MBMA(2002) taken into consideration, such initial deformation does not affect structural performance. However, width-thickness ratio and unbraced length are the parameters which decide lateral torsional buckling strength (Polyzois *et al.* 1998, Lee *et al.* 1972, 1975, Prawel *et al.* 1974). The value of those parameters applied to structural design differ from designer to designer since there is no clear-cut standard. Prior to this study, a study was conducted with 4 mock-up specimens made with parameters such as web width-thickness ratio, existence/non-existence of stiffeners and unbraced length based on the rafters of warehouse building A to carry out flexure tests (Lee *et al*. 2006, Shim *et al*. 2006). This study analyzes and evaluates the structural safety of slender and tapered web members.

2. Experimental program

2.1 Plan and manufacture of specimens

The structural behavior and strength of the vast majority of laterally unrestrained beams is governed by the lateral-torsional buckling. The principal factors in flexural performance of rafter which is used in tapered H-section beams with slender web were material properties, width-to-thickness ratio and slenderness ratio and so on (Krefeld *et al*. 1959, Butler *et al*. 1963, 1966). In this study, the tests with a total of 4 specimens were conducted. Main parameters were the width-to-thickness ratio of web and the unbraced length by change of existence and nonexistence of stiffeners and lateral brace. The details of specimens are summarized in Table 1.

Dimensions and layout of full scale test specimens is shown in Fig. 2 and Fig. 3. The span of test specimens was 20 m from center of the supporting pins. The largest depth of the center was 1.6 m. Specimens were planned as the simple beam which has the supports at the zero point of moment distribution in real structure. In case of rafter, Submerged Arc Welding was used at both side of the web plate. Ten coupon tests were carried out, 2 cut from the web and flange each thickness. Actual yield stresses and tensile stresses were average 294 MPa and 430 Mpa respectively. All value exceeds the nominal values.

Table 1 Details of specimen materials (unit: mm)

Specimens	Rafter 3 H_{web} : 1,600~1,200				Rafter 2 H_{web} : 1,200~900				Rafter 1 H_{web} : 900~867				parameter	
	B_{flange}	$t_{f\text{-}t}$	$t_{f\text{-}b}$	t_w	B_{flange}	$t_{f\text{-}t}$	$t_{f\text{-}b}$	t_w	B_{flange}	$t_{f\text{-}t}$	$t_{f\text{-}b}$	t_w	Stiff-ener	Flange brace
TB-200	200	7	7	6	200	10	7	6	250	12	9	8	-	-
TB-250	200	7	7	4.5	200	10	7	4.5	250	12	9	6	-	-
TB-S	200	7	7	6	200	10	7	6	250	12	9	8	10	-
TB-L/2	200	7	7	6	200	10	7	6	250	12	9	8	-	L/2

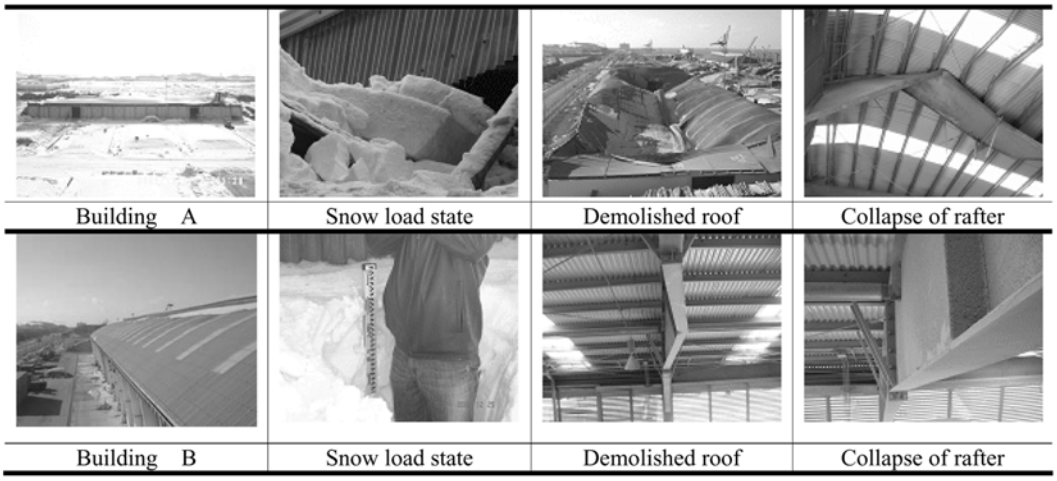


Fig. 1. Warehouse building collapse

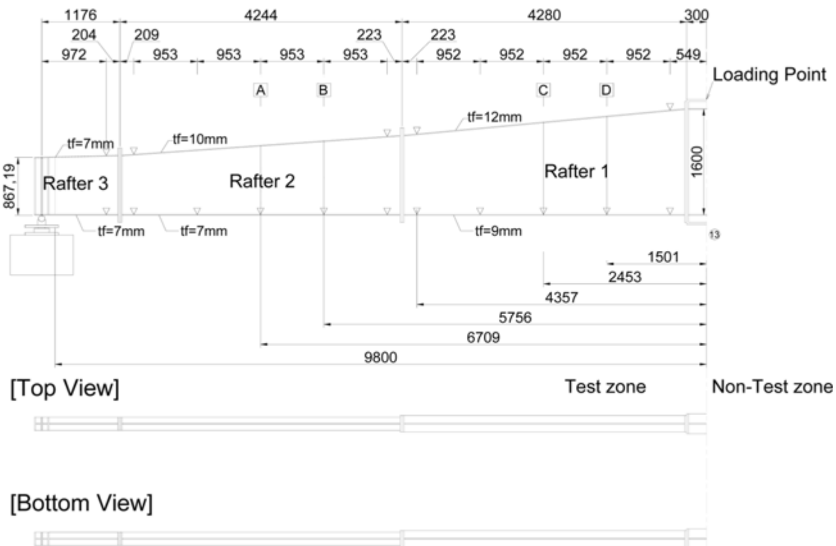


Fig. 2 Dimensions and layout of full scale test specimens

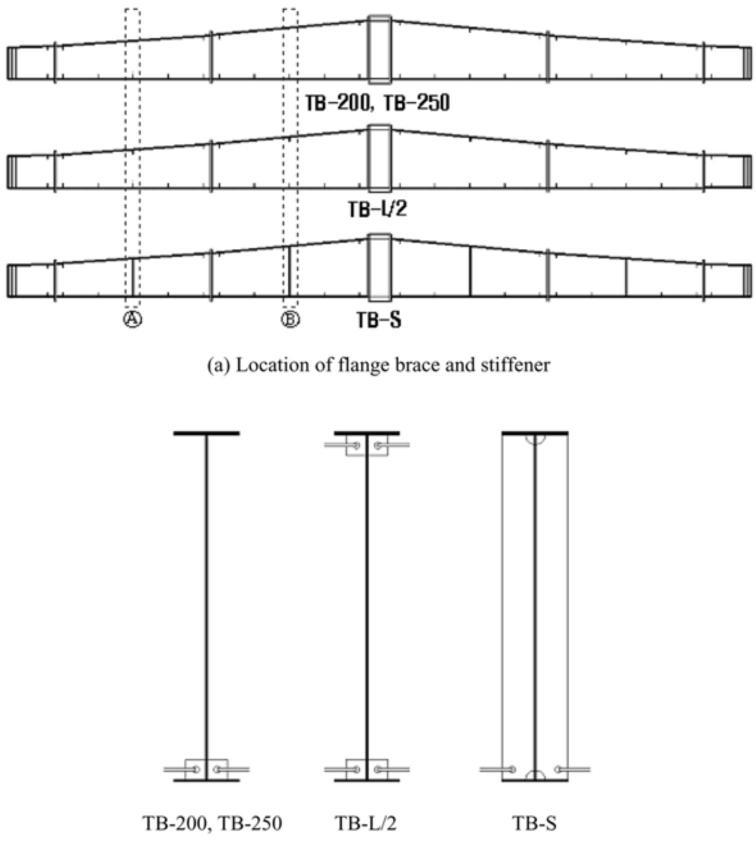


Fig. 3. Location and shape of parameters on specimens

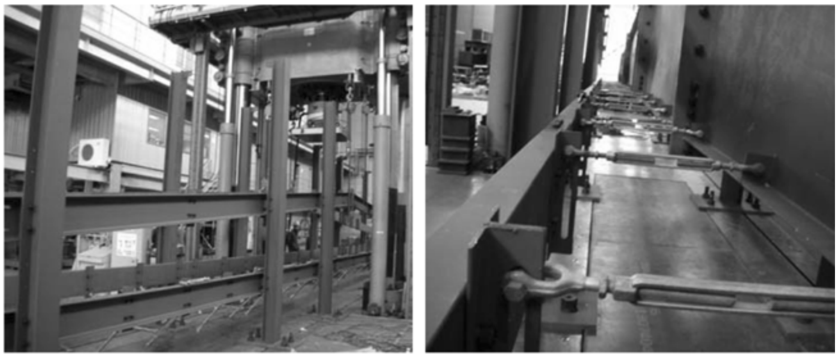
2.2 Test set-up and measurement

Full-scale testing is essential because of the uncertainty of scale effects on fabrication processes. It was desirable to test under laboratory conditions in order to minimize uncertainties of loading and instrumentation. Frames were tested in a 9,806 kN test rig designed and fabricated in the structural engineering laboratory at the RIST(Research Institute of Industrial Science and Technology).

The design of overall lateral restraint system was carefully considered in relation to the test rig. As shown in Fig. 4(a), the lateral bracing columns and channels were planed additionally in order to embody the role of the purlin and flange brace[stay] which is identical with reality. It was used turn-buckles instead of purlin and flange brace, it gave shape to lateral restraint(see Fig. 4(b)). All tapered beams were simply supported with hinge point where moment was zero and load was applied at one point as shown in Fig. 5. Simple supported specimens were subjected to equal concentrated vertical loads(P) applied to the tapered beam with slender web. Vertical loads were applied by 0.05 mm/sec displacement controlled UTM.

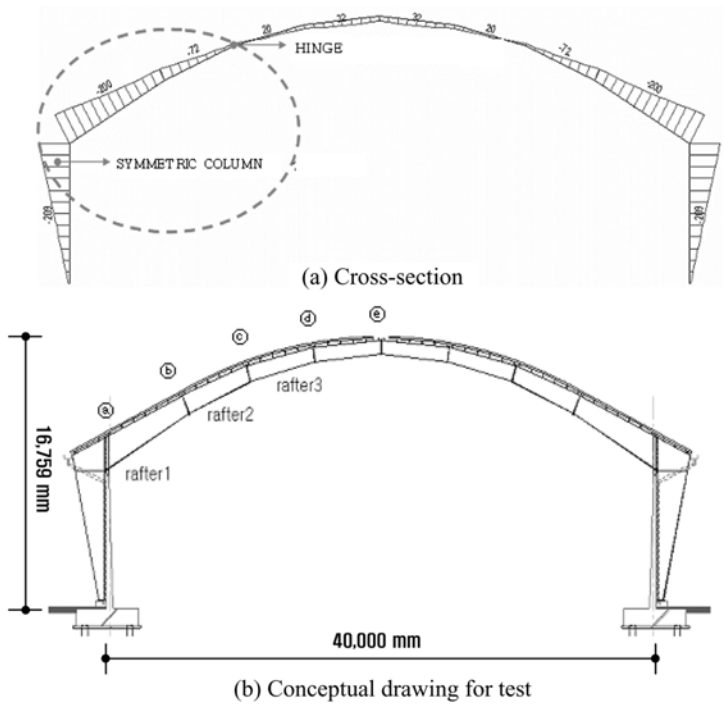
2.3 Experimental results

In Fig. 6 Load-Displacement relation, the maximum load capacity of the specimens is approximately



(a) installation of lateral brace (b) connection of turn-buckle

Fig. 4 Test rig



(a) Cross-section

(b) Conceptual drawing for test

Fig. 5 Summary of test

350 kN and their self-load is about 34 kN. Consequently, the self-load of 34kN exerts an influence on the initial droop in the specimens. That's why the self-load is set as the starting point in the load-displacement graphs in Fig. 6. Load-mid span deflection relationships for tapered beam with slender web tests are presented in Fig. 6 and final failure mode and buckling shape are presented in Fig. 7 and Fig. 8. Table 2 shows the results of the test. In taking a side view of strength and stiffness, the specimens, TB-200, TB-S and TB-L/2, have same tendency in elastic region, whereas the specimen, TB-250 having different width-thickness ratio, shows a evident strength and initial stiffness decreasing rather than other

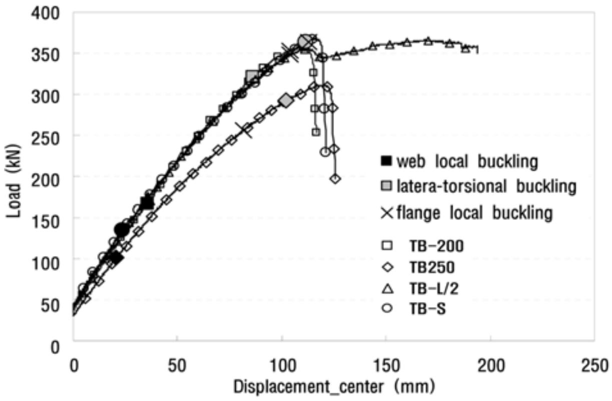
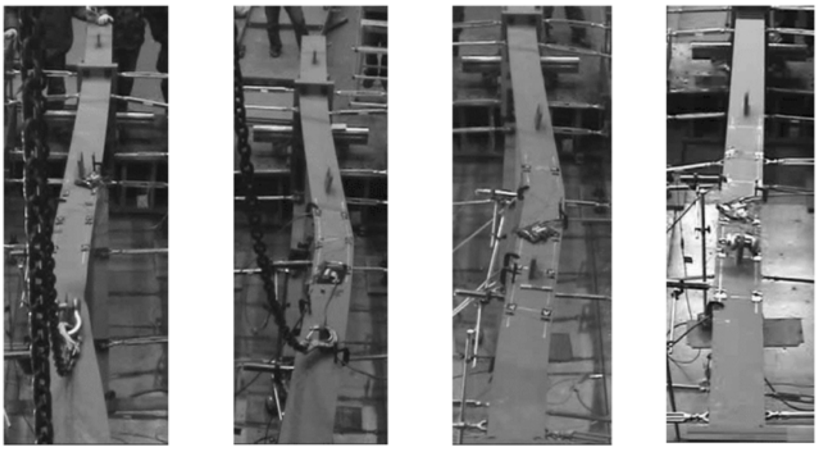


Fig. 6 Load-displacement relationship



(a) Lateral torsional buckling(TB-200) (b) Lateral torsional buckling (TB-250) (c) Lateral torsional buckling(TB-S) (d) Flange local buckling (TB-L/2)

Fig. 7 Final failure mode

specimens. After reaching maximum strength, the specimens, TB-200, TB-S and TB-250, showed brittle phenomenon which made the strength rapidly decreased, whereas the specimen, TB-L/2, show ductility conduct. In TB-250 specimen, local web buckling appeared about 46% less than the maximum load and stiffness deterioration was also observed. The specimen TB-S, which has the stiffener at the center(Fig. 3(a)) of rafter where flange braces are set in TB-200, had early local web buckling. Specimen TB-L/2, which has the flange brace installed the center of rafter and has the half of the unbraced length, had the buckling strength 15% greater than TB-200.

3. Buckling analysis

3.1 Analysis sumary

As shown in Fig. 5(a), the arch-shaped pre-engineered warehouse buildings in K City of Korea are

Fig. 8 Buckling shape

Table 2 The results of the test

	Web local buckling (kN)	Flange local buckling (kN)	Lateral Torsional buckling (kN)	Collapse mechanism	Ultimate strength (kN)
TB-200	169	352	320	A-C-B	356
TB-250	105	256	293	A-B-C	311
TB-S	137	362	361	A-C-B	366
TB-L/2	174	347	-	A-C-B	364

A : Web local buckling *B* : Flange local buckling *C* : lateral torsional buckling

made of tapered members. Table 1 shows that web height and flange thickness increase as they approach the column. In this study, flexure-moment of the warehouse building A of 40 m-span (Fig. 5(b)) was drawn and the part featuring large flexure-moment was simplified to be beam-shaped (Fig. 9). The objects of analysis were 4 specimens with parameters such as web width-thickness ratio (200-250), existence/non-existence of stiffeners and unbraced length.

3.2 Modeling method for specimens

ANSYS 9.0, the finite element analysis program with proven reliability in terms of material and geometric non-linear analysis was used and Shell 181 Geometry consisting of 4 nodes and 6 degrees of freedom (displacement and rotation of X, Y, Z axes) was used. Material properties were bi-linear. 210 GPa was used

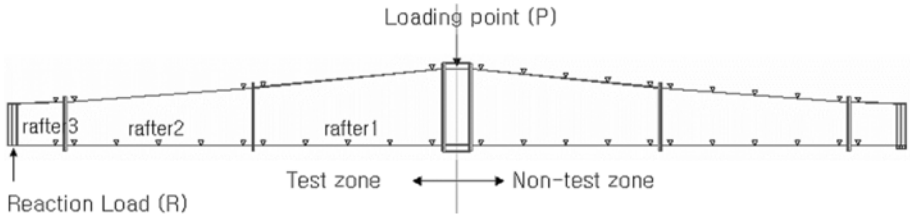


Fig. 9 Summary of analysis model

as the modulus of elasticity of materials. And, 2.1 GPa (1/100 of modulus of elasticity) and 349.7 MPa which was obtained from previously conducted structure test were employed as modulus of elasticity after yield and yield strength of material, respectively. And, mass density 7.85 g/cm^2 and acceleration of gravity 9,800 mm/sec^2 were applied to calculate self-weight. Regarding boundary condition of the analysis model, lateral degree of freedom at the node where flange and web met was fixed in order for specimens' purlin and flange stay to control flange. As regards left and right supporting points of the simplified model, roller condition and hinge condition were provided for non-test zone and test zone, respectively. Load was applied to the center of the specimens. Bolt connection was used in the specimens, but analysis model was made on the assumption that welding connection was employed for simplicity's sake. Final analysis model are presented in Fig. 10.

Material non-linear analysis uses Newton-Raphson Method on the assumption that the cross-sectional areas of materials always satisfy material properties such as modulus of elasticity, yield strength, Poisson's ratio, the 2nd modulus of elasticity and stress-deformation relation. However, in geometric non-linear analysis, geometric deformation resulting from residual deformation and errors in production is taken into consideration and Arch-length Method based on Lanzos' Eigen value analysis is used.

3.3 Evaluate method of buckling strength

Comparison of stress caused by external load with material strength using non-linear analysis program is the common way to examine the safety of rolled steel structures which are free of buckling thank to

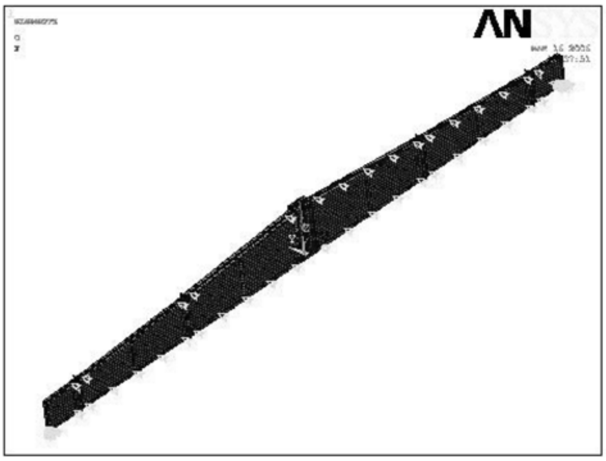


Fig. 10 Analysis model

low slenderness ratio. However, structures made of slender materials such as plate girder featuring large width-thickness ratio exhibit various collapse mechanism due to the influence of initial deformation, width-thickness ratio and unbraced length. Accordingly, for the comparison with the previous test results, strengths for 3 types of buckling (web local buckling, flange local buckling and lateral torsional buckling) were driven as follows. The 1st eigenvalue obtained from Lanzos' Eigen Value Analysis Method was used as web local buckling strength. The 1st eigenvalue obtained from Lanzos' Eigen Value Analysis after elasticity analysis was conducted and lateral displacement at the web part displaying tensile stress was controlled was used as flange local buckling strength. Ultimate load where stiffness became negative based on Arch-length Method was used as lateral torsional buckling strength. Analysis was ended when the droop at the center increased rapidly since we assumed that it would mean failure.

3.4 Analysis results

Fig. 11 shows load-displacement relation comparing the result of geometric non-linear analysis with that of material non-linear analysis. The self-load is set as the starting point at the load-displacement relation in Fig. 10 to take the influence of self-load into consideration in the relation. Web local buckling strengths of 197 KN at TB-200, TB-S and TB-L/2 and 109 kN at TB-250 were obtained. Flange local buckling strength was 321 kN at TB-200, TB-S and TB-L/2 and 177 kN at TB-250. Lateral torsional buckling strength was 280 kN at TB-200, TB-S and TB-L/2 and 254 kN at TB-250. As shown in Table 3, collapse mechanism at TB-200, TB-S and TB-L/2 was in order of web local elastic buckling, lateral torsional buckling and flange local buckling and the largest value of ultimate strength was 321 kN

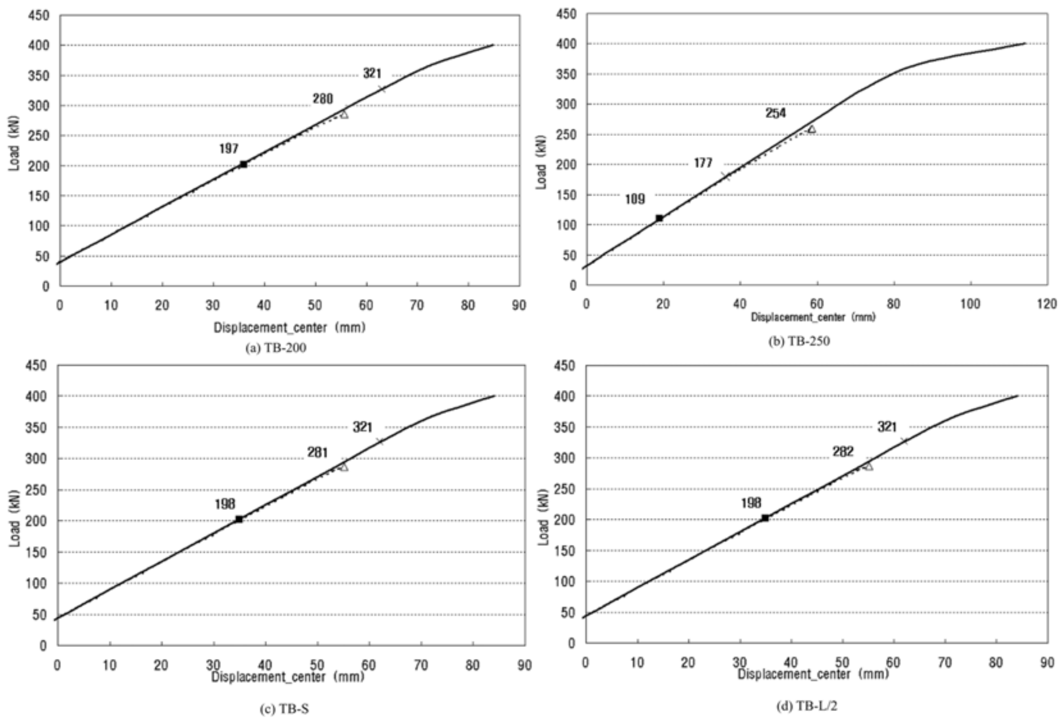


Fig. 11 Load-displacement relation and collapse mechanism of non-linear analysis

Table 3 Buckling strength of specimens

	Web local buckling (kN)	Flange local buckling (kN)	Lateral Torsional buckling (kN)	Collapse mechanism	Ultimate strength (kN)
TB-200	197	321	280	A-C-B	-
TB-250	109	177	254	A-B-C	-
TB-S	198	321	281	A-C-B	-
TB-L/2	198	321	282	A-C-B	-

observed at flange local buckling. However, collapse mechanism at TB-250 was in order of web local buckling, flange local buckling and lateral torsional buckling and ultimate strength exhibited the largest value of 254 kN at lateral torsional buckling. In all specimens, buckling was observed at rafter 2 as shown in Fig. 12. It is because web width-thickness ratio is the largest at rafter 2 as shown in Fig. 13. And then, in Fig. 13, web width-thickness ratio is similar at rafter 1 and rafter 2. But, the length of rafter 2 over limit width-thickness ratio is longer thank rafter 1. Also slender element of rafter 1 is restrained by the stiffener of loading point, Accordingly, buckling occurred at rafter 2. Consequently, it appears that the use of web with large width-thickness ratio exerts influence on collapse mechanism and buckling strength and strength drops dramatically after ultimate strength.

3.5 Verification of analysis models

Table 4 and Fig. 14 show buckling strengths and load-displacement relations of test and analysis to verify that the analysis models are reliable. Initial stiffness values are similar throughout all specimens, and the difference in stiffness between test results and analysis results seems to result from the assumption that the rafter connection in the analysis models did not involve high-tension bolts but was thoroughly welded while the rafter connection in the specimens was made by F10T-M26 high-tension bolts. Collapse mechanism is in order of web local buckling, lateral torsional buckling and flange local buckling at TB-200, TB-S and TB-L/2 and web local buckling, flange local buckling and lateral torsional buckling at TB-250. In comparison between TB-200 and TB-250 both of whose parameter is width-thickness ratio, the latter exhibits lower strength than the former and web local buckling, in particular, is approximately 40% lower. It seems that low value of initial web buckling load causes low flange buckling strength due to web deformation. TB-S which includes stiffeners exhibits buckling strength similar to that of TB-200 and these two specimens show similar collapse mechanism. It seems that the existence of stiffeners does not exert influence on buckling strength. Analysis result shows that TB-L/2 which has an additional brace and 1/2 as long unbraced length as TB-200 exhibits similar buckling strength to that of TB-200. However, test results indicates that strength drops rapidly upon flange local buckling or lateral torsional local buckling at TB-200, TB-250 and TB-S while load remains constant and ductility is high at TB-L/2. So, it seems that shorter unbraced length leads to improved ductility. Consequently, to control lateral buckling, it is required that web width-thickness ratio be low if buckling seems possible at pre-engineered building's rafter with tapered web, flange brace be elaborately designed according to installation angle based on lateral force and the size of cross-sectional area and unbraced length of flange brace be short. Analysis results obtained from the analysis method mentioned in Section 2 was almost similar to test results in spite of the difference by simplify of model. It seems that the analysis of steel-framed structures using the same analysis method can be used for the analysis of collapse mechanism of steel-framed structures.

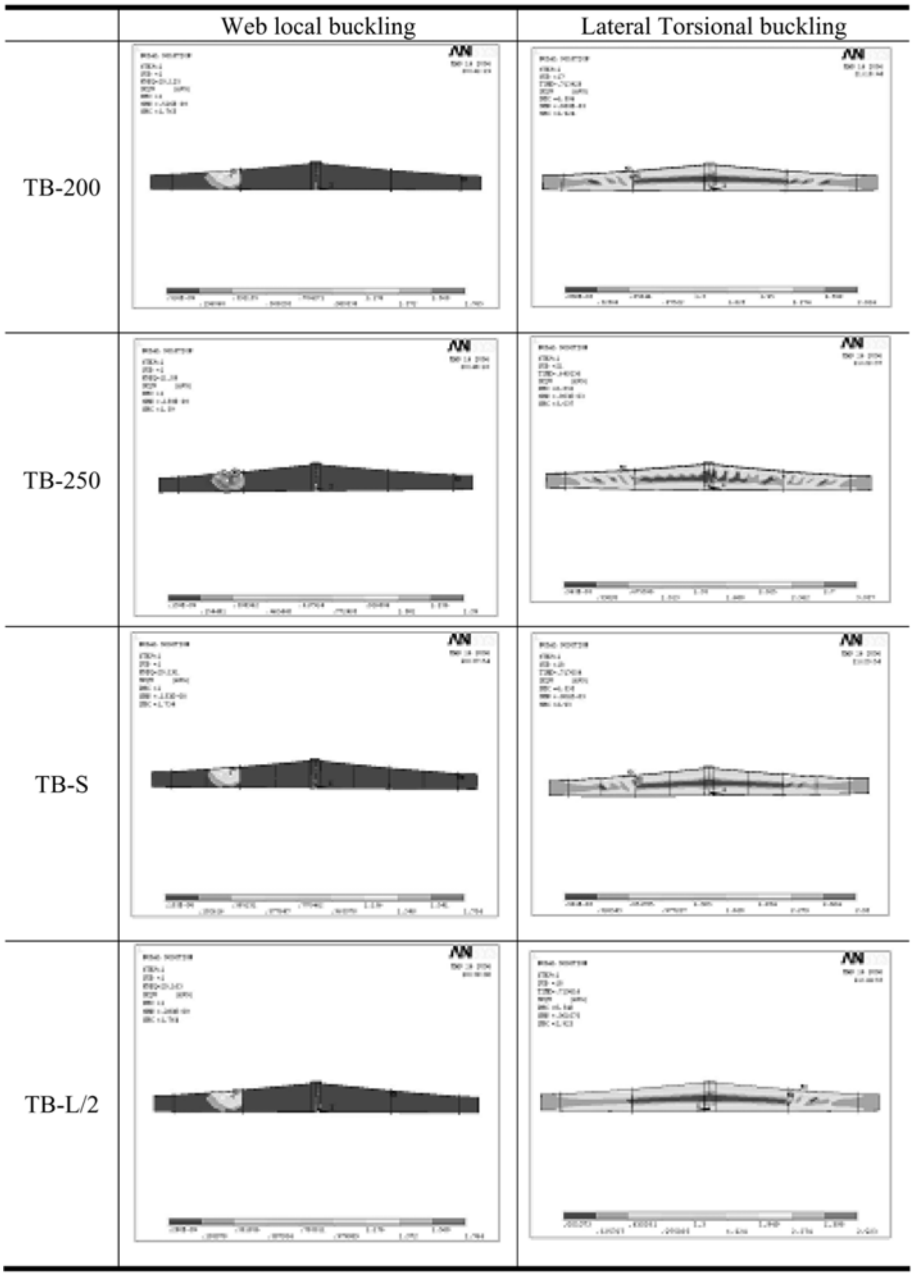
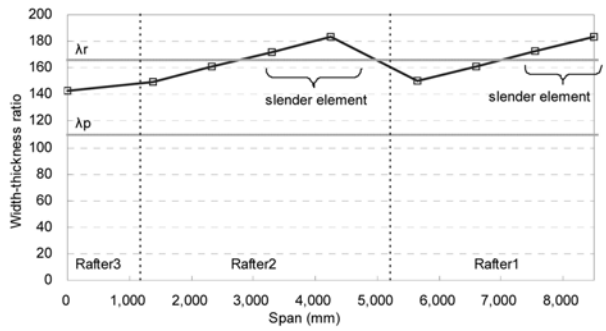


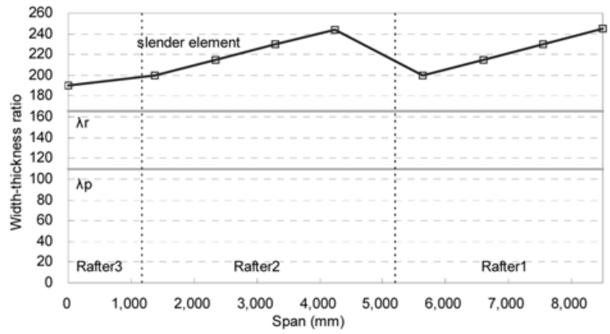
Fig. 12 Location of buckling

4. Conclusions

In this study, to evaluate of structural performance for web tapered H-shaped beam with large width-thickness ratio mainly using in PEB(Pre-engineered Building), we conducted the test and analysis for four specimens with parameters of width-thickness ratio, flange brace with or not, stiffener with or not



(a) TB-200



(b) TB-250

Fig. 13 Condition of width to thickness ratio of web along the span

Table 4 Buckling strength comparison (kN)

		Web local Buckling Load		Lateral Tortional Buckling Load		Flange Local Buckling Load		Ultimate Load	
		Load	%	Load	%	Load	%	Load	%
TB-200	Test	169	1.00	320	1.00	352	1.00	356	1.00
	Analysis	197	1.17	280	0.88	321	0.91	-	-
TB-250	Test	105	0.62	293	0.92	256	0.73	311	0.87
	Analysis	109	0.64	294	0.79	177	0.50	-	-
TB-S	Test	137	0.81	361	1.13	362	1.03	366	1.03
	Analysis	198	1.17	281	0.88	321	0.91	-	-
TB-L/2	Test	174	1.03	-	-	347	0.99	364	1.02
	Analysis	198	1.17	282	0.88	321	0.91	-	-

and analyzed the results.The evaluation of structural safety of specimens led to the following conclusion. Large width-thickness ratio of web causes sudden brittle fracture and decreases buckling strength. Short unbraced length does not exert influence on collapse mechanism and ultimate strength, but improves ductility more than a little. The result of geometric non-linear analysis shows that collapse mechanism at specimens was web local buckling→flange local buckling→lateral torsional and in both specimen, flange local buckling strength and lateral torsional buckling strength were nearly reached after web local buckling. Consequently, to control lateral buckling, it is required that web width-thickness ratio be

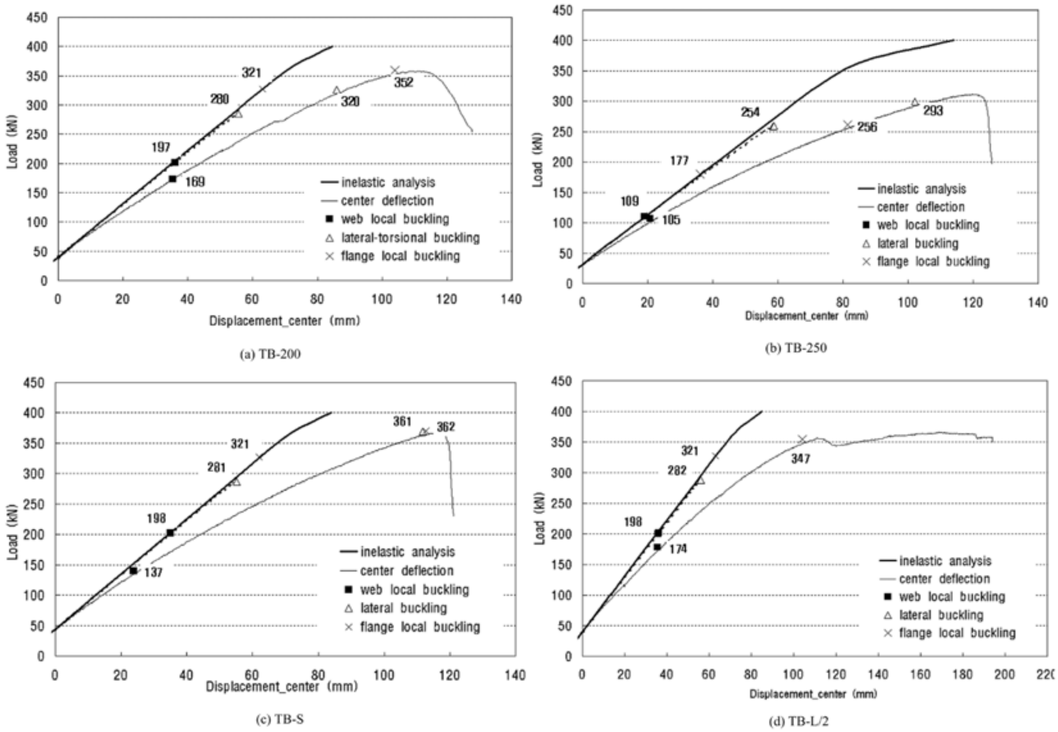


Fig. 14 Load-Displacement comparison between test and analysis

low if buckling seems feasible at pre-engineered building's rafter with tapered web, flange brace be elaborately designed according to installation angle based on lateral force and the size of cross-sectional area and unbraced length of flange brace be short. Collapse mechanism of specimens mentioned in this study was derived through simplification using finite element analysis, so the collapse mechanism of test and analysis result may not be identical to each other.

Acknowledgements

This work was supported by the Korea Science and Engineering Foundation(KOSEF) grant funded by the Korea government(MOST) (No. R0A-2007-000-10047-0)

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