The Structural Behavior of Connections of Cold-formed Steel Portal Frames

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ABSTRACT

A series of connection tests of portal frames which were composed of cold-formed steel columns and rafters, were carried out to study the moment-rotation relation, the rotational rigidity, and the yield and ultimate moment capacity of the connections. The connection test specimens were composed of column-base, column-rafter and rafter –rafter connections, and the closed cold-formed sections were used for the column and rafter members. The main factors of the test were the thickness and the shape of mild steel connecting members. The connection test results were com pared with those obtained by using advanced analysis procedures. The se cant stiffness of the connections which was estimated from the moment- rotation curves was proposed as the rotational rigidity of semi-rigid connections considered in the frame analysis. Simple formulas for the ultimate shear strength of the screw fastener connections based on the test results and AISI specifications were also proposed.

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Introduction

The structural behavior of connections should be investigated with the development of the new cold-formed steel sections in order to be used effectively in the construction of steel structures. The structural behavior of connections composed of cold-formed steel sections includes local buckling, bearing failure and shear strength of screws and bolts. The flexural-torsional buckling problem should certainly be considered in case the one symmetric section like C-section is used as a flexural member (Hancock, 1998; Yu, 1991). The joints of chord and web members of a truss jointed are connected directly with a gusset plate and are assumed as a hinged connection (AISI Specifications, 1996). However, the connections which composed of the thicker plate than structural members could be divided into the rigid or semi-rigid connections according to the flexural strength. In order that the flexural rigidity could be considered in the analysis and design of frames, the nonlinear moment versus rotation relation of connections is studied.

In this paper, the structural behavior of the connections of the portal frame composed of closed cold-formed steel sections which have been recently developed (Kwon, 2002), were studied experimentally and theoretically. A series of connection tests were carried out to investigate the moment-rotation relation, flexural rigidity, yield and ultimate moment of the connections. The main factors of the test are connection shape, thickness of connecting members and the restraint against torsion of the rafter who se end is loaded with special grip. The numerical analysis were executed using THIN-WALL and LUSAS. The numerical results were compared with the test results. The nonlinear analysis of the portal frame was carried out considering various conditions of the connections where the secant stiffness of the connections which was estimated from the moment-rot action curves was proposed as the rotational rigidity of semi-rigid connections. Simple formulas for the ultimate shear strength of the screw fasten er connections based on the test results were proposed and were compared with AISI specifications(1996).

Section Properties

The section used for column and rafter members is closed cold-formed steel sections which was made by cold-rolling with clinching techniques. The nominal yield strength of the cold-formed steel section is 570MPa. Structural steel grade of connection material was SM400 with nominal yield strength of 240MPa. The diameter of screw fasteners was 4.8mm and nominal shear strength of a screw fastener used was 1700MPa. The section geometry is shown in Fig. 1 and the section properties are given in Table 1.

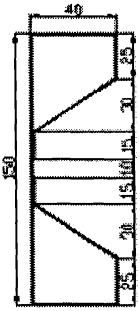


Fig. 1 150PRY08 section geometry

Table 1. 150PRY08 section properties

Specimen	Depth (mm)	Thickness (mm)	Area (cm^2)	Section Modulus (cm^3)	Inertia Moment	
					Ix (cm^4)	Iy (cm^4)
150PRY08	150	0.8	3.28	9.39	83.24	7.71

Buckling Strength of Section

The elastic buckling analysis of the section under pure bending was carried out using THIN-WALL and the numerical results are shown in Fig. 2. The local buckling stress of the section was 320MPa at half- wavelength 40mm.

The distortional buckling is practically not liable to occur at intermediate half-wavelength since the distortional buckling strength of the closed section is so high. According to the increase of half- wavelength, the section showed global flexural buckling mode with negligible amount of torsional deformation.

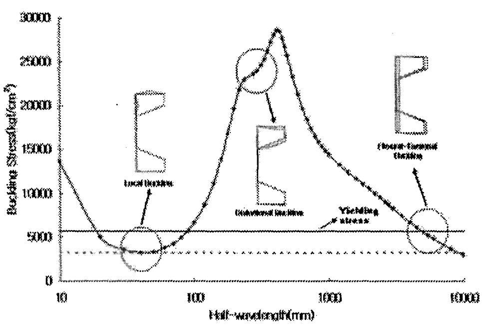


Fig. 2 Buckling stress of PRY08 section

The expected buckling mode of long flexural members is a mixed mode of local and flexural buckling. Local buckling, yield and plastic moment of the section are summarized in Table 2. The local buckling moment is the bending moment which causes the local buckling to occur. The yield moment is assumed as a bending moment when the maximum stress of the extreme fiber of the section reaches the yield stress. The effective width according to AISI specifications was used for the section modulus to calculate the yield moment.

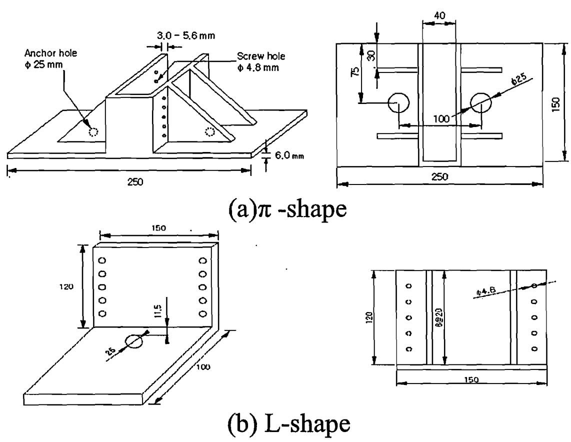
Table 2. 150PRY08 section strength

Section	M_{cr} (kNm)	M_y (kNm)	M_p (kNm)
150PRY08	3.6	5.4	8.2

Shape of Connection Members

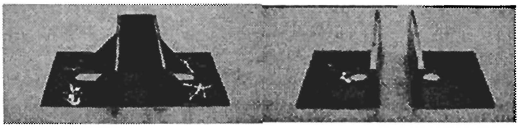
The test connection specimens of the portal frame were made for column base, column-to-rafter at corner and rafter-to-rafter connections at the ridge.

Two types of column base connection plate are shown in Fig. 3. First connection shape was π-shape and second one was double angles, where the column member was inserted between. The column was anchored into the massive concrete block using anchor bolts which had enough strength against shear and pull-out failure.



(a)π -shape

(b) L-shape



(c) π-shape (d) Double L-shape

Fig. 3 Column base connection members

The shape of basic rafter-to-column connection is shown in Fig. 4. The four connection types of thin-gauge cold-formed steel sections are based upon the specific connection member plates, screwed on the back of PRY08 sections of rafter and column end parts.

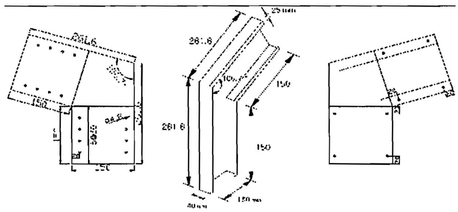
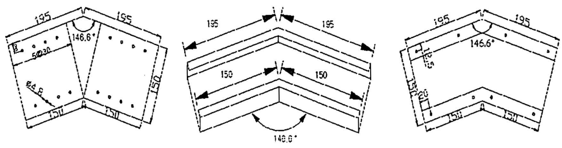


Fig. 4 Rafter-to-Column connection member

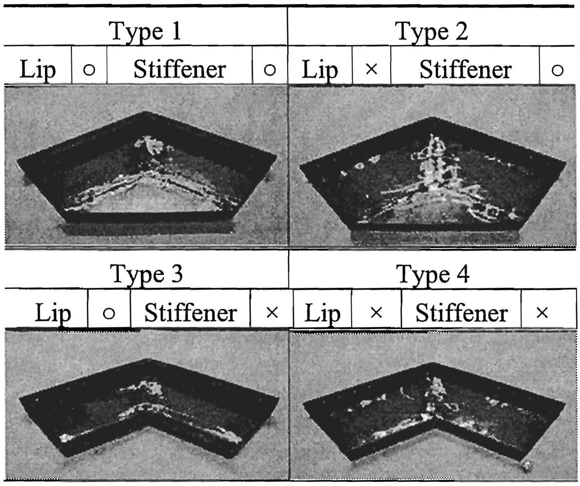
The types of connection member studied are divided into four categories according to the presence of additional lip and stiffener as shown in Fig 5(b). The lip was attached outwardly at the edge of the connection plate and the stiffener was added by welding the Δ-shape plate beneath the L-shaped connection plates. The angle between column and rafter was 106.7 degrees.

The connections in the ridge joint with using a specific connection shape on the back of PRY08 sections of rafter ends was made in a similar way as the rafter-to-column connections as shown in Fig.5. The angle between rafter ends was 146.6 degrees.

The specific connection member plates were made of mild steel (nominal yield strength F_y=240 MPa). The structural section was inserted into the connection plate and fixed using self-drilling screw fasteners. The holes of connection plate were pre-punched at the exact position to be screwed before fabrication.



(a) Rafter-to-Rafter connection member



(b) Types of column/rafter connections

Fig. 5 Rafter-to-rafter connection members

The connection specimens are listed in Table 3. The thickness of connection plate of column-to-rafter and rafter-to-rafter were 1.6mm, 2.3mm and 3.2mm, but mainly 2.3mm. The specific connection plates and shape s fabricated by fillet welds. The number of screw fastener was decided using shear strength of screw connections of AISI specifications (1996) so that the bearing and tilting failure of the structural sections could not occur before the local buckling of sections or deformation of the connection plate.

Table 3. Test Specimens

Connection Category	Types	Thickness(mm)	Specimen
Column base	Double Angle	3.0	CB-L-3.0
		4.3	CB-L-4.3
		5.6	CB-L-5.6
	π -shape	3.0	CB- π -3.0
		4.3	CB- π -4.3
		5.6	CB- π -5.6
Rafter-Column	Type-1	2.3	RC-1-2.3
	Type-2		RC-2-2.3
	Type-3		RC-3-2.3
	Type-4		RC-4-2.3
Rafter-Rafter	Type-1	2.3	RR-1-2.3
	Type-2	2.3	RR-2-2.3
	Type-3	1.6	RR-3-1.6
		2.3	RR-3-2.3
		3.2	RR-3-3.2
	Type-4	1.6	RR-4-1.6
		2.3	RR-4-2.3
		3.2	RR-4-3.2

Connection Tests

The connection tests were carried out by the displacement control method of 1 mm/min using MTS Dynamic Actuator (Capacity 250kN). Out side of the shape and thickness, the restraint against the torsion of section was considered as a factor in the connection tests. The connection specimen was loaded in the direction of upward or downward to find out the effects of torsion of sections. The specially designed grip attached at the loaded end of the section prohibited the torsion of section at the time of upward loading. However, the torsion of loaded end of the section was not prevented at the downward loading. The lateral displacements were measured with using displacement transducers (LVDT) located at column and rafter in the joining region of rafter and column ends. Strain gauges were attached in the connection plate to measure the stress level of connection plate. For moment-rotation relationship of the connection, the rotations are calculated from the measured displacements. The test arrangements are shown in Fig. 6.

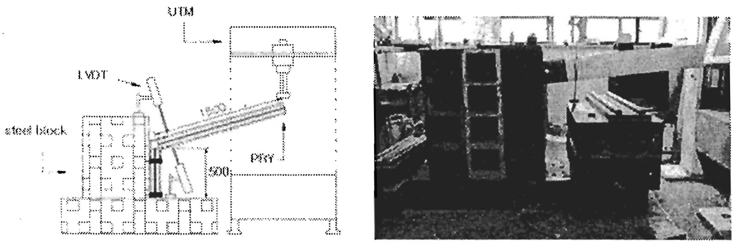
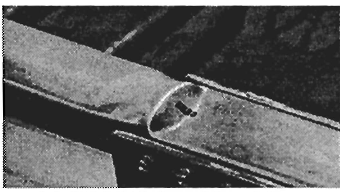


Fig. 6 Test arrangements

Test Results

The final failure mode of the connections tested were the local buckling of the column or rafter section near connection member and then fracture of clinching as shown in Fig. 7 and 8.



(a) PRY08 local buckling

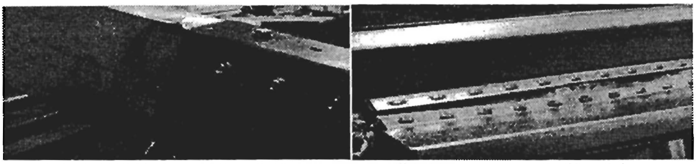
(b) Fracture of clinching

(c) Deformation of connection member

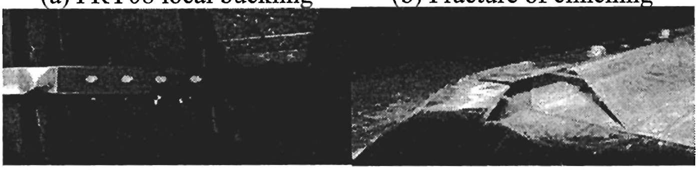
(d) Local failure

Fig. 7 Failure modes of column base connections

The connections of thin connection plate had shown large deformation of connection member plate before local buckling of the column and rafter sections and consequently the welds were fractured as shown in Fig. 8(d).



(a) PRY08 local buckling (b) Fracture of clinching



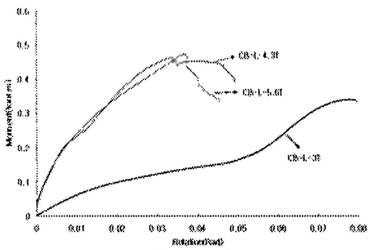
(c) Deformation of connection (d) Fracture of welds

Fig. 8 Failure modes of column-rafter/rafter-rafter connections

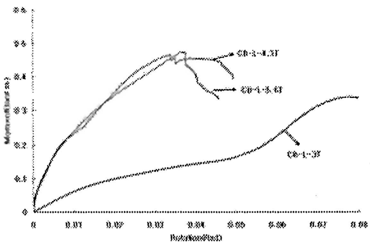
The yield and ultimate moment were significantly different in relation with the type and the thickness of the connection member plate. It was found that the torsion of structural section had significant effects on the flexural strength of the connections.

Column Base Connections

Moment-rotation relations of column base connections are given in Fig. 9.



(a)π –shape



(b) L-shape

Fig. 9 Moment-rotation curves of column base connections

The connection composed of 3mm thickness connection plate had shown large deformation of connection plate before local buckling of the column and lower flexural strength and ultimate moment compared with the connections of thick plates (thickness of 4.3mm and 5.6mm). The CB-L-3T curve is different from other test results, since the deformation of column was negligible and the failure was due to the deformation of connection member plate. The ultimate moments of π-type and Double L-type are similar but the flexural stiffness and the yield moment are significantly different as shown in Fig. 10.

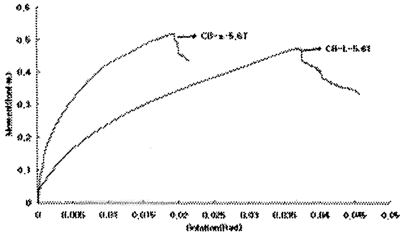


Fig. 10 Comparison of π and L-type connections

The yield moment was assumed as the lower value between the yield moment of the connection plate and the local buckling of column section. The torsion had bad effects on the flexural stiffness and the ultimate moment caused by the fracture of clinching. In Table 4, the initial flexur al stiffness is listed as well as the yield and ultimate moments. The initial flexural stiffness k_0 on the moment-rotation curve is the secant stiffness at the rotation of 0.005 radians. It is half the slope of 0.001 radian of a simple beam corresponding to the central deflection of L/300 which is allowable deflection in Korean Building Specifications. The ratios of ultimate moment to yield moment are ranged approximately from 2.0 for Double L-type to 3.0 for π-type connections.

Table 4 Test results of column base connections

Specimens	My (kNm)	Mu (kNm)	Mu/My	K0 (kNm/rad)
CB-L-3.0T	Failure of connection plates			-
CB-L-4.3T	2.4	4.6	1.92	32.8
CB-L-5.6T	2.1	4.7	2.24	32.3
CB-π-3.0T	1.3	4.4	3.38	36.2
CB-π-4.3T	1.6	5.1	3.19	50.9
CB-π-5.6T	1.4	4.2	3.00	41.5
CB-π-5.6T(TR)	3.1	5.2	1.68	63.4

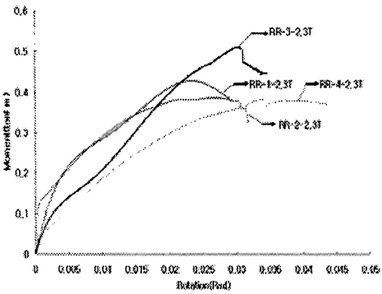
Rafter-to-Rafter Connections

Rafter-to-rafter connection test series were composed of four types for the connection plate thickness of 2.3mm and type4 and type5 for the thickness of 1.6mm and 3.0mm. The test results are shown in Figs. 11(a)-(c) and Table 5.

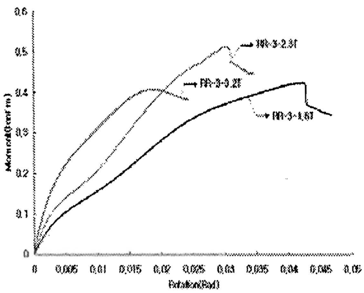
Table 5 Test results of rafter-to-rafter connections

Specimens	My (kNm)	Mu (kNm)	Mu/My	K0 (kNm/rad)
RR-1-2.3T	1.7	4.3	2.53	59.2
RR-2-2.3T	1.2	3.8	3.17	83.9
RR-3-1.6T	2.0	4.2	2.10	23.3
RR-3-2.3T	1.8	5.1	2.83	39.0
RR-3-3.2T	1.5	4.1	2.73	71.9
RR-4-1.6T	1.4	3.4	2.43	25.4
RR-4-2.3T	1.4	3.8	2.72	23.2
RR-4-3.2T	1.5	3.9	2.9	60.3

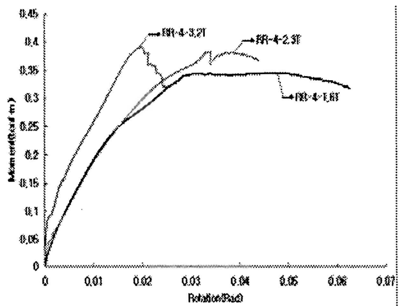
The type-3 with a lip and stiffener had shown large yield and ultimate moment and basic type-1 connection had shown lower flexural stiffness and strength. The difference in ultimate moment between thickness of 1.6mm, 2.3mm and 3.2mm is negligible but the flexural stiffness and yield moment are rather significant as shown in Figs. 11(b) and (c). The reason is that the final failure was due to the fracture of clinching of cold-formed steel sections. Therefore, the thickness of connection member plate could be decided to be used. The ratios of yield moment to ultimate moment are ranged from 2.1 to 3.17 and average value is 2.7. The ultimate moment is clearly picked up from the moment-rotation curves but the yield moment could not be easily obtained from the moment-rotation curve. Therefore the basic design strength would rather be ultimate moment than the yield moment. Since the buckling moment of the cold-formed section (given in Table 1) lies between the yield moment and the ultimate moment, the nonlinear moment-rotation relationship should be considered in the structural analysis and design of the portal frame to expect accurate structural behavior of portal frames.



(a) Comparison of types (2.3T)



(b) Type3 connections



(c) Type4 connections

Fig. 11 Moment-rotation curves of rafter-to-rafter connections

Rafter-to-Column Connections

Rafter-to-column connection tests were focused on the types with a constant thickness of 2.3mm. Moment-rotation curves are shown in Fig. 12 and 13. The Test results are summarized in Table 6. The initial flexural stiffness had no significant difference among connection types but the ultimate moment varied with large amount. The fact is due to that the restraining capacity against the torsion of the rafter section was different from connection types.

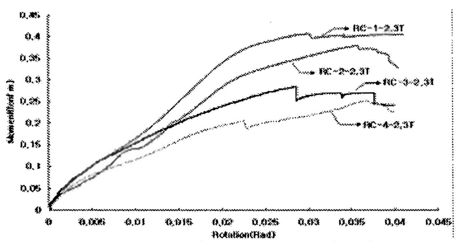


Fig. 12 Moment-rotation curves of rafter-to-column
(torsion unrestrained)

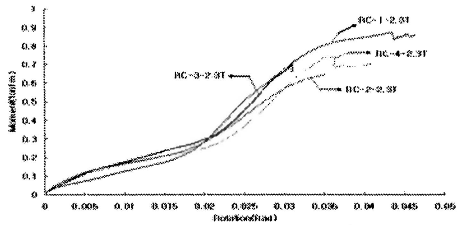


Fig. 13 Moment-rotation curves of rafter-to-column (torsion restrained)

Table 6 Test results of rafter-to-column connections

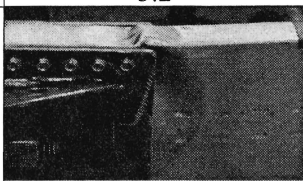
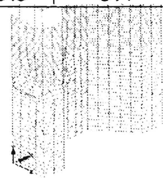
Specimens	My(kNm)	Mu(kNm)	Mu/My	K0 (kNm/rad)
RC-1-2.3T	2.4	4.0	1.67	19.4
RC-1-2.3T(TR)	0.5	8.7	17.40	14.8
RC-2-2.3T	2.1	3.8	1.81	14.0
RC-2-2.3T(TR)	0.9	6.5	7.22	23.7
RC-3-2.3T	1.5	2.8	1.87	20.0
RC-3-2.3T(TR)	0.7	7.0	10.00	22.7
RC-4-2.3T	1.0	2.5	2.50	15.2
RC-4-2.3T(TR)	0.8	7.4	9.25	21.3

The ultimate moment of connection type-1 with lip and stiffener was 4.0kNm, and that of type-4 was 2.5kNm. Type-2 and type-3 had shown the values of 3.8 and 2.8 kNm respectively. It was concluded that the lip and stiffener could increase the flexural strength significantly. The upward loading test with specially designed grip which prevent section from torsion had similar results to downward loading without grip. In comparison of upward and downward loading test results, both the flexural strength and the flexural stiffness had shown serious difference between test methods owing to the torsion of cross sections. The ultimate moment obtained with torsion of section being restrained was much larger than that affected by the sectional torsion. The results about flexural stiffness were similar to the ultimate moment. However, the results of the yield moment obtained were not clear since the rotational angle could not be obtained accurately.

Numerical Analysis

Buckling and yield moment of column base connections were compared between tests and numerical analysis results in Table 7. The yield moment of test and numerical analysis are well agreed. However, buckling moment is not agreeable since the local buckling of section occurred early at low load stage and determination of buckling stress on the moment versus rotation curves was difficult. The failure mode of the section is quite similar.

Table 7. Buckling and ultimate moments (kNm)

Moment	Test results	Numerical analysis	
		Lusas	Thin-Wall
M_{cr}	3.1	3.7	3.6(3.1*)
M_u	5.2	5.5	5.4**
Failure mode			
Final failure	Fracture of clinching	Member yielding	

*$M_{cr} = f_{cr}$ (local buckling stress × $S_{f(}$ (section modulus), values in ( ) effective section modulus

** $M_u = f_y$ (yield stress) × S_e (effective section modulus)

Conclusions

The experimental and numerical study into the structural behavior of the connection of cold-formed steel portal frames was executed. The connections tested had shown nonlinear moment-rotation relationship and could be divided into the semi-rigid connections. The numerical results showed that the semi-rigid connection could be applied to the portal frame and cause moment distribution effectively from the rigid connection part. Consequently, the specific connection members developed could be used for the portal frame with cold-formed PRY sections. The restraint of torsion of cross section should be properly provided by attached panel, girt or purlin sections for the portal frames in order to delay the flexural-torsional buckling at a lower loading stage. The final failure was caused by the rupture of the clinching part following local failure of the cross section. Therefore, so as to enhance the ultimate moment strength, the clinching part should be certainly tightened.

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