

Behavior of Steel Beam-Column Subassemblages with Semi-Rigid Connections

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Abstract

In this study, fifteen full size semi-rigid connection specimens and three full size specimens of restrained beam-column subassemblages having semi-rigid connections were conducted. The connection details were flange and web cleat type, flange cleat type and double web cleat type and they were tested by applying equal concentrated loads to both ends of steel beam cantilevers connected to a column in symmetrical manners. In the paper, the moment rotation curves of semi-rigid and simple connections were presented and were compared with three-parameter power function and with the results of finite element analysis. For beam-column subassemblage tests, the column was semi rigidly connected to the restraining beams and rigidly connected to the loading beams at both ends. The column was loaded to a constant axial load and equal joint moment was increased up to and beyond the instability limit. The results of the subassemblage tests were reported and were verified with the results of the inelastic frame analysis program. Analytical study on portal frames with semi-rigid connections was conducted. It was found that the strength of semi-rigid frames decreases with the decrease in connection rigidity and with the increase in the column slenderness ratio.

1. Introduction

Steel rigid frames of high rise buildings subjected to high axial load generally fail due to in plane instability of the beam-columns. In such the frames, the beams provide full degree of restraint to the columns. However, for low rise frames where full demand of beam-to-column connection fixity can be relaxed, semi-rigid connection construction is allowed in AISC specification [1], for ease in fabrication

practice. There have been experimental and analytical studies for the strength and stiffness of steel frames with rigid connections [2]. However, limited studies were conducted on the behavior of semi-rigid connections [3], [4], [5], and of steel frames with semi-rigid connections [6]. According to the connection classification system [4], the limits of strength and stiffnesses of semi-rigid connections are given as follow:

$$\text{In term of strength: } 0.7 > \bar{m} > 0.2 \quad (1)$$

$$\text{In term of stiffness: } 2.5 \bar{\theta} > \bar{m} > 0.5 \bar{\theta} \quad (2)$$

and for

$$\text{Rotation capacity: } \bar{m} = (5.4 - 2\bar{\theta})/3 \quad (3)$$

where:

$$\bar{m} = M / M_p, \quad \bar{\theta} = \theta_r / (5M_p d / EI_b)$$

M = moment at connection

M_p = plastic moment of beam

d = the depth of beam

θ_r = rotation at connection

E = the modulus of elasticity of beam

I_b = the moment of inertia

Empirical formulae for moment rotation characteristic of flange and web cleat angle connection and web cleat angle connection was suggested in term of three-parameter power model as follow:

$$M = R_{ki} \theta_r / [1 + (\theta_r / \theta_0)^n]^{1/n} \quad (4)$$

where:

M_u = ultimate moment capacity

R_{ki} = initial connection stiffness

n = shape parameter

θ_0 = reference plastic rotation ($\theta_0 = M_u / R_{ki}$)

The objective of this study is to report on fifteen tests of full size connections and on three tests of subassemblages with semi-rigid connections. Connection test results were

compared with three-parameter power function. In addition, analytical verification using finite element program ABAQUS was conducted and compared with the connection and the subassembly test results. The parametric study of portal frames with semi-rigid connections was also investigated.

2. Experimental Studies

2.1 Connection Test

Details of semi-rigid connection tested are shown in Fig. 1. Connections in group 1 and 2 were of flange cleat and double web cleat angle type. For connections in group 3, only flange cleat angles were used and for connections in group 4, only double web cleat angles were fabricated. The size of flange cleat angles is L130x130x9 mm and the size of web cleat angles is L75x75x6 mm, respectively. Full size self contained test arrangement was used and the test plane was horizontal as shown in Fig. 2 and the characteristic of 15 semi-rigid connections tested was summarized in Table 1.

The test set-up of semi-rigid connection specimens consists of two pairs of cantilever beams connected by cleat angles to the ends of a column. The beam and the column sections are W200x150x30.6 kg/m and W175x175x40.2 kg/m, respectively. The members were bent about the strong axis and were supported by angle bents which were attached to concrete stubs and resting on the laboratory floor. The beam ends were loaded by applying the tension force to 4- ϕ 28 mm steel rods connected by 600x550x25 mm bearing plates which were clamped to a pair of reaction beams. A 100-ton compression jack was put between the reaction beams and in line of the column and two 5-ton proving rings were used to measure the loads at the beam cantilever ends. Instrumentation devices consist of deflection gages to measure the cantilever beam end deflections and rotation of the connection. The strain gages were attached at several lines to the connection angles and the strain gage wires were connected to the PDA-320A data logger to record the strain during the test. Figure 3 shows the moment-rotation characteristic of the connection tests. In each group of connections,

the increase of stiffness and strength is observed with the degree of connection fastening enhancement details from standard angle bolting to welding of angles to the beam. It was obvious that rotation of connections of groups 1 and 2 resulted mainly from the deformation of the leg of flange angles on the tension side.

2.2 Subassembly Test

Three beam-column subassemblies with semi-rigid connections had been tested to observe the behavior and the strength of the frames. For the three specimens, the column and restraining beam sections were W175x175x40.2 kg/m and W200x150x30.6 kg/m, respectively. The slenderness ratio of the column (L_c/r_x) is equal to 69. Three semi-rigid connections T1A, T1E and T2C were selected to be incorporated in the frame specimens as described in Table 2.

The test set-up of the subassembly tests was modified from that of the connection tests (see Fig. 2). A semi-rigid frame subassembly specimen consists of a pair of restraining beams, a column, and the loading beams. All members were bent about the strong axis and the test plane was horizontal. The column end lateral supports were used to prevent overall lateral movement but allowed axial movement of the ends of the column. The loading beams were fabricated by welding two W200x150x30.6 kg/m sections in order to be elastic over the entire range of loading during the test. The extended end-plate rigid connections were used to join the loading beams to the column. A ϕ 50.8 mm reaction rod was attached to the restraining beam ends and was connected in series to a tension load cell to monitor the reaction force during the test.

The loading beams and the column were loaded by 100-ton jacks inserting between the bearing plates at the beam and column ends. The loads were transferred to the members by means of series of tie bars and the beam and column loads were measured by a 25-ton proving ring and by a 100-ton load cell, respectively. The strain data were recorded by strain gages having the gage factor of 2.08. The

PDA-320A data logger was used to record the strain readings. Dial gages were used to determine the midspan column deflection and the movement of rotation arms. During the test, the column was loaded to a predetermined level and was kept constant and the load at the end of loading beam was monotonically increased until the maximum strength of the frame was reached. Beyond the instability limit, the test was monitored by the controlled deflection technique.

It was observed that substantial yielding occurred along the column and in the connections at the maximum column strength level. At each stage of loading, the column displacement, the connection rotation and strain data were recorded. The relationship between horizontal displacement at the middle of column and the applied load at the end of the loading beams is shown in Fig. 4(a) and the applied moment and strain data at the flanges of the column section for specimen FT1A are shown in Fig. 4(b).

The variation of bending moments at column and beam end sections with the column midspan deflection is plotted in Figs. 4(c), 4(d) and (4e). The joint moment and restraining beam moments are measured at the column flanges and column end moment is at the center of the web plate. The ratios of column end moment to the joint moment are 0.59, 0.54 and 0.44 for specimens FT1A, FT1E and FT2C, respectively. Due to P-delta effect, the ratios of midspan moment to the joint moment are 1.09, 0.92 and 0.84, respectively. These ratios are observed at the instability limit and the restraining beams remain elastic.

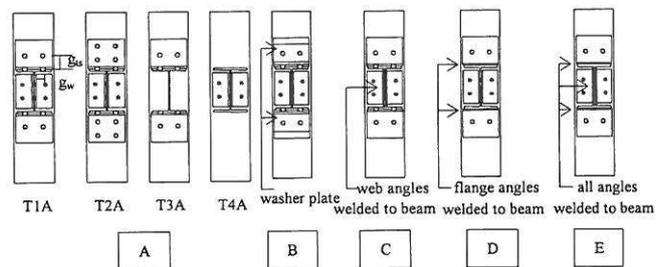


Fig. 1: Details of connections of groups 1 to 4

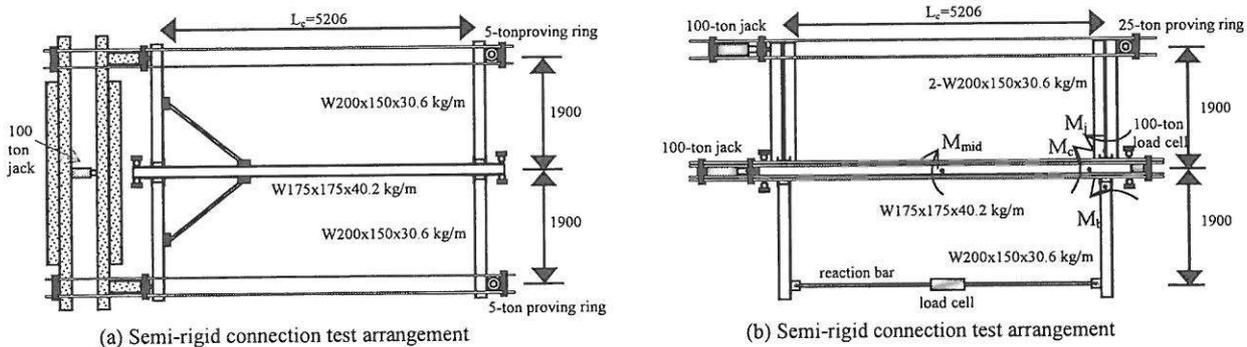


Fig. 2: Test set-up arrangement

Table 1: Test of semi-rigid connection

Name	Connection Type	Connection Detail	g_s (mm)	g_w (mm)
T1A	FWC - Type T1	standard bolting	60	40
T1B	FWC - Type T1	washer plates added to flange angle	60	40
T1C	FWC - Type T1	web angles welded to beam	60	40
T1D	FWC - Type T1	flange angles welded to beam	60	40
T1E	FWC - Type T1	all angles welded to beam	60	40
T2A	FWC - Type T2	standard bolting	40	40
T2B	FWC - Type T2	washer plates added to flange angle	40	40
T2C	FWC - Type T2	web angles welded to beam	40	40
T2D	FWC - Type T2	flange angles welded to beam	40	40
T2E	FWC - Type T2	all angles welded to beam	40	40
T3A	FC - Type T3	standard bolting	60	-
T3B	FC - Type T3	washer plates added to flange angle	60	-
T3D	FC - Type T3	flange angles welded to beam	60	-
T4A	WC - Type T4	standard bolting	-	40
T4C	WC - Type T4	web angles welded to beam	-	40

FWC=Flange and Web Cleats; FC=Flange Cleats; WC=Web Cleats; g_s =gage length of top and seat angles; g_w =gage length of web angle

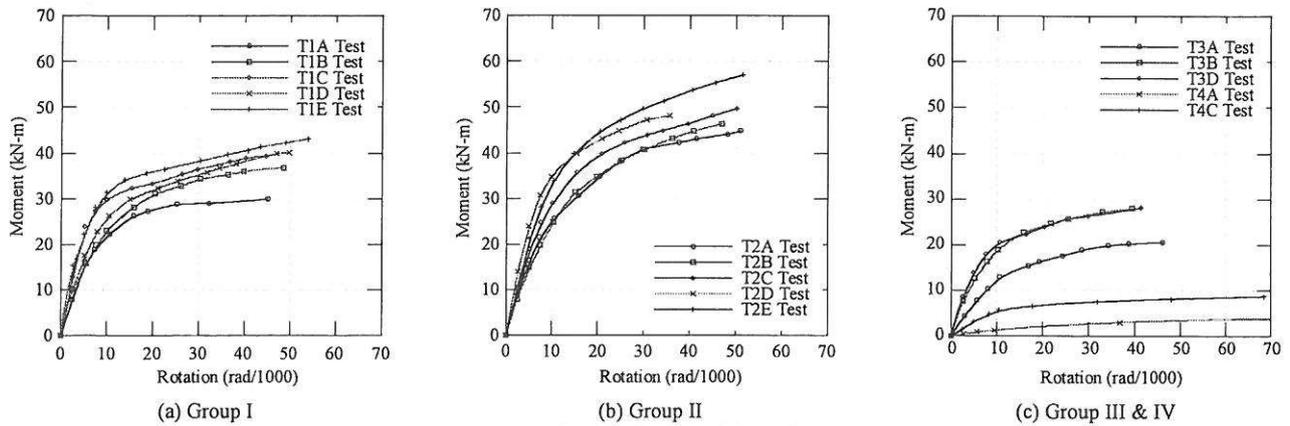
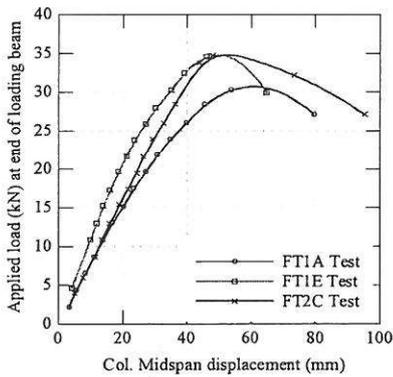


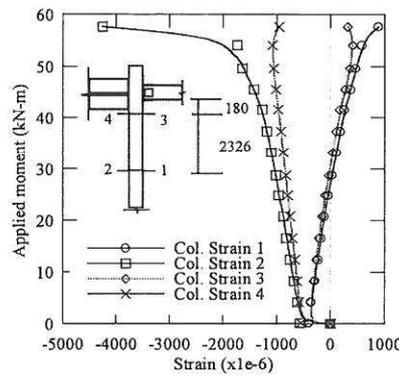
Fig. 3: Moment rotation relationship of connection tests

Table 2: Experimental program for semi-rigid frame test

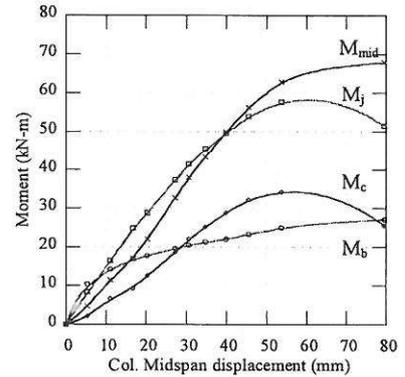
Test No.	Name	Incorporated Connection	Connection Detail	g_{is} (mm)	g_w (mm)	L_c/r_x
1	FT1A	T1A	standard bolting - Type T1	60	40	69
2	FT1E	T1E	all angles welded to beam - Type T1	60	40	69
3	FT2C	T2C	web angles welded to beam - Type T2	40	40	69



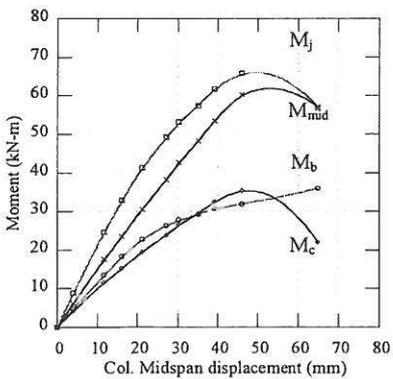
(a) Load displacement relationship for FT1A, FT1E and FT2C



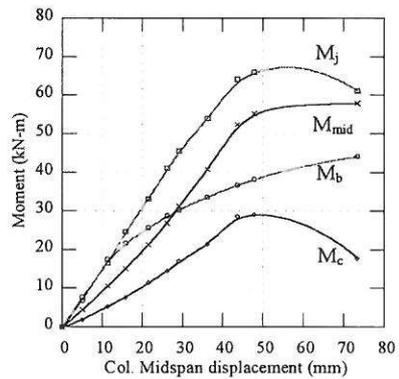
(b) Strain results at column sections of FT1A



(c) Moments at various sections of FT1A



(d) Moments at various sections of FT1E



(e) Moments at various sections of FT2C

Fig. 4: Test results for semi-rigid frame subassemblages

3. Verifications and Finite Element Model

3.1 Connection

Figure 5(a) summarizes the results of all the range of connection tests. In this figure, the moment rotation curves of the tested connections were plotted and compared with the semi-rigid connection classification system [4]. It can be seen that the test results of all the range of flange and web cleat angle connections lie within semi-rigid connection classification (see equation (1), (2)). The moment rotation curves of standard bolting flange cleat connection (type T3A), and web cleat connections fall in the simple connection classification.

The three-parameter power model has been suggested to describe the estimated moment rotation relationship of cleat angle connections [5], as given in equation (4). For connections of standard bolt fastening (type A connections), the M_u and R_{ki} using the geometry of the connections are incorporated in equation (4). However, for connection T2A with two rows of bolts in the cleat angle, the gage distance, $g_{ts} + 10$ mm is used in calculation of M_u and R_{ki} . This is to account for the offset of yielding zone from the center of bolt center line. The estimated moment rotation curves are shown in Fig. 5(b).

In determining the estimated moment rotation curves for type E connections, the calculated moment capacity and initial stiffness of type A connections are modified by multiplication with the ratios of the tested values $M_{u,E} / M_{u,A}$ and $R_{ki,E} / R_{ki,A}$, respectively before determination of the estimated moment and rotation. The estimated curves are shown in Fig. 5(c).

To determine the connection behavior by the inelastic analysis, finite element model was developed. The finite element model is symmetric about the center of the column web; therefore only one side of the plane of symmetric shown in the dashed rectangle in Fig. 6(a), is input in ABAQUS 3D finite element analysis program [7]. The 8-node brick elements shown in Fig. 6(b) were used to model the geometry of all the specimen parts. The high strength hexagon bolts and nuts were

modeled but the washers were omitted. In this study, the "CONTACT PAIR, SMALL" and "SURFACE BEHAVIOR, PRESSURE-OVERCLOSURE = HARD" options were used (see Fig. 6(c)), to specify the interactions between the angles and the beams or column and between the bolt shanks and the bolt holes. The pre-stress tightening forces were applied to the bolts in order to simulate the minimum bolt tension. The pre-stress force of 104 and 161 kN were applied to $\phi 16$ mm and $\phi 19$ mm bolts, respectively. The displacements were prescribed at the end of the loading beams. The software package PATRAN2003 is used as pre-processing and post-processing units. The total number of elements and nodes is approximately 12500 and 21000, respectively. The moment-rotation relationship of the test results is verified with the inelastic analysis using the finite element program as shown in Figs. 7(a) and 7(b).

3.2 Subassemblage Frame

The 8-node brick elements are used to model for all the subassemblage members. The interaction model between two surfaces of connection elements previously discussed is incorporated in the frames. The geometric finite element model is input as shown in Fig. 2(b). The idealized pinned supports were used at both ends of column to prevent the end lateral movement. At the end of restraining beams, the idealized hinged supports were utilized. The 50-TON column loads were constantly applied to the column ends. At the end of loading beams, the displacements were monotonically applied at the end of the loading beams until beyond the instability limit. The total number of elements and nodes is approximately 40000 and 61000, respectively. The results of applied cantilever end load and column midlength relationship from the tests and finite element analysis are shown in Fig. 8(a), 8(b) and 8(c). It can be seen that the calculated curves tend to be stiffer than the tested curves and the maximum beam end loads are in good agreement.

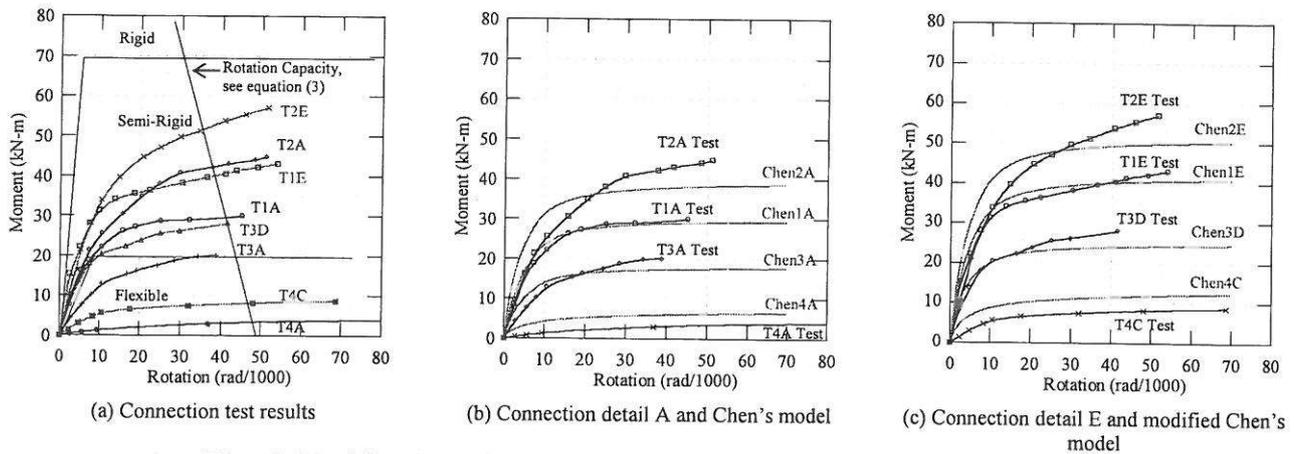


Fig. 5: Verification of connections and three-parameter power model

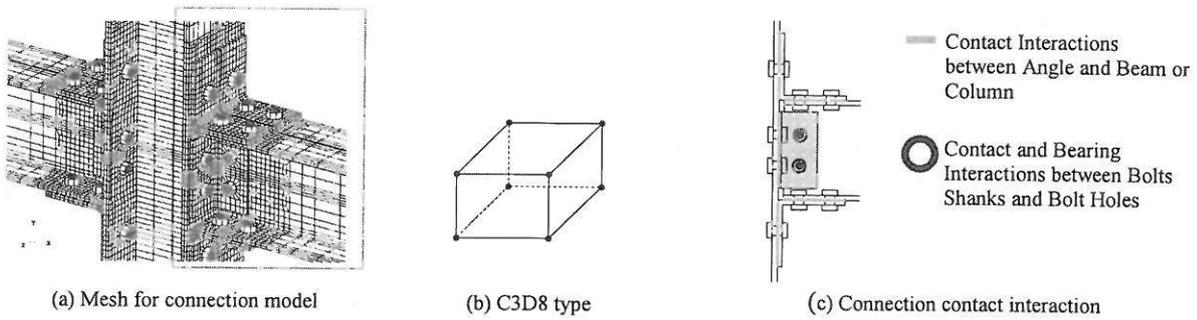


Fig 6: Finite element model of semi-rigid connection

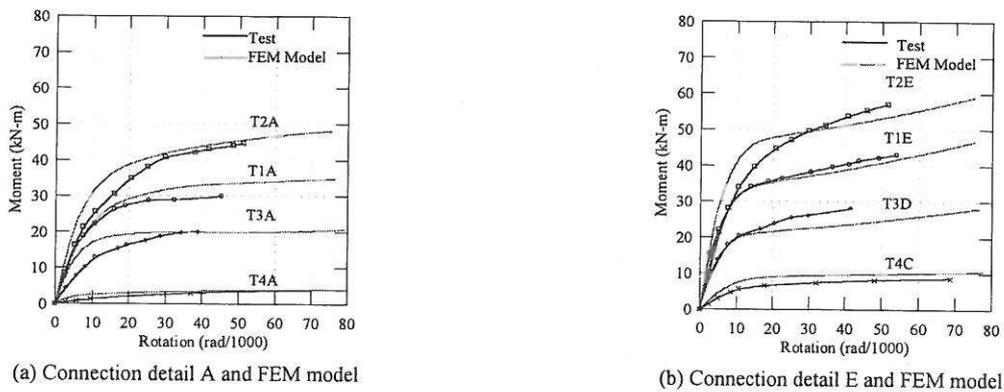


Fig. 7: Verification of connections and FEM model

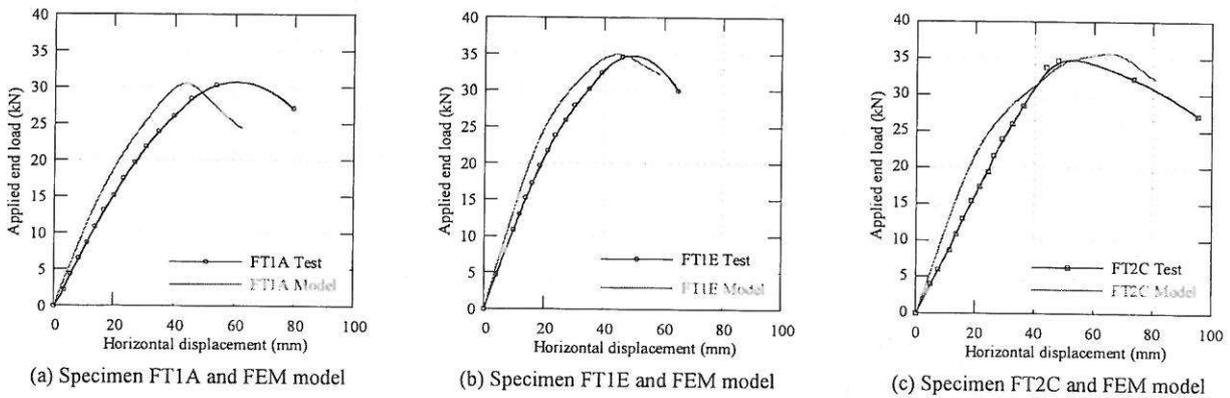


Fig.8: Verifications of frames and FEM model

4. Parametric Study of Unbraced Frames

Figure 9 shows the configuration of the unbraced portal frames studied. The beam and column sections are W250x146x31.2 kg/m (W10x21) and W203x203x46.1 kg/m (W8x31), respectively. The column slenderness ratios (L_c/r_x) are 40 and 80 and all members are bent about the strong axis. The ratio of the column stiffness to beam stiffness ($G=(E_c I_c/L_c)/(E_b I_b/L_b)$) is equal to 1.0 for all the frames. The study on the portal frames with rigid connection was previously reported [8].

In this study, the effect of the rigidity of the beam to column connections and the effect of the slenderness ratio of the columns on the frame strength are investigated. The semi-rigid connection types T1A, T1E and T2E are selected to be incorporated in the frames. For these connections, the ratios of the maximum moment capacity of the connections to the plastic moment capacity of the beam are 0.49, 0.58 and 0.73 respectively.

The unbraced portal frames were analyzed by ABAQUS program considering the effect of geometrical and material nonlinearity. The 3D-C3D8 solid elements are used to model the semi-rigid connections and the beam and the columns are modeled by using 1D-beam elements. The nodes between 3D-C3D8 solid brick elements and 1D-beam elements are linked using the MPC, BEAM option [7]. The total number of elements and nodes is approximately 9500 and 16000, respectively.

The details of the geometry of the parametric frames are shown in Table 3. In the analysis, the columns are subjected to constant axial load (P_2) and the horizontal load (H) is monotonically increased by specifying the predetermined values of sway displacement (Δ) up to and beyond the instability limit.

The results of the parametric analysis are summarized in Table 4 and the load-displacement relationship of the portal frames is shown in Fig. 10. All the frames except frame RF1 fail by reaching the instability limit. For frame RF1, the plastic hinge develops at the leeward end of the beam. The strength of the frames with semi-rigid connection

decreases significantly compared with those of the rigid frames. For Group I frames, the ratios of horizontal load capacity of frames SRF11 to RF1, SRF12 to RF1 and SRF13 to RF1 are 0.46, 0.48 and 0.56, respectively. The strength of frames decreases with the increase in the slenderness ratio of the columns. By comparing the results of Group I and Group III frames, the ratios of the strength of frames RF3 to RF1, SRF31 to SRF11, SRF32 to SRF12 and SRF33 to SRF13 are 0.3, 0.21, 0.22 and 0.24, respectively.

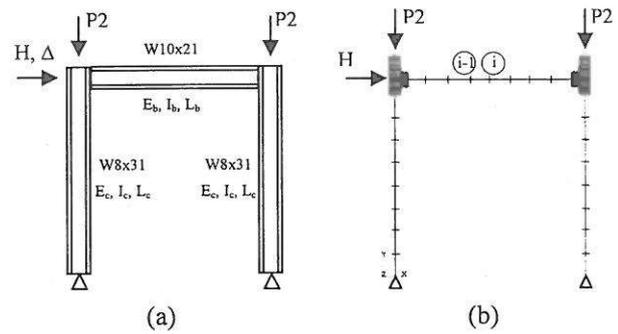


Fig. 9: Portal frame configuration

Table 3: Details of portal frames

Group	Name	Connection Type	L_c/r_x	L_c (mm)	L_b (mm)	Column Load (P_2)
I	RF1	Rigid	40	3556	3429	$0.1P_{y,Col}$
	SRF11	T1A	40	3556	3429	$0.1P_{y,Col}$
	SRF12	T1E	40	3556	3429	$0.1P_{y,Col}$
	SRF13	T2E	40	3556	3429	$0.1P_{y,Col}$
II	RF2	Rigid	40	3556	3429	$0.3P_{y,Col}$
	SRF21	T1A	40	3556	3429	$0.3P_{y,Col}$
	SRF22	T1E	40	3556	3429	$0.3P_{y,Col}$
	SRF23	T2E	40	3556	3429	$0.3P_{y,Col}$
III	RF3	Rigid	80	7112	6858	$0.1P_{y,Col}$
	SRF31	T1A	80	7112	6858	$0.1P_{y,Col}$
	SRF32	T1E	80	7112	6858	$0.1P_{y,Col}$
	SRF33	T2E	80	7112	6858	$0.1P_{y,Col}$

Table 4: Summary of frame analytical results

Group	Name	Max. Δ (mm)	Max. H (kN)	Type of failure
I	RF1	88.9	48.0	Beam plasticity
	SRF11	74.7	21.9	Instability
	SRF12	80.6	23.3	Instability
	SRF13	85.7	27.0	Instability
II	RF2	76.2	31.8	Instability
	SRF21	55.1	10.1	Instability
	SRF22	55.0	11.6	Instability
	SRF23	60.6	14.5	Instability
III	RF3	266.7	14.4	Instability
	SRF31	185.4	4.6	Instability
	SRF32	178.0	5.2	Instability
	SRF33	191.9	6.4	Instability

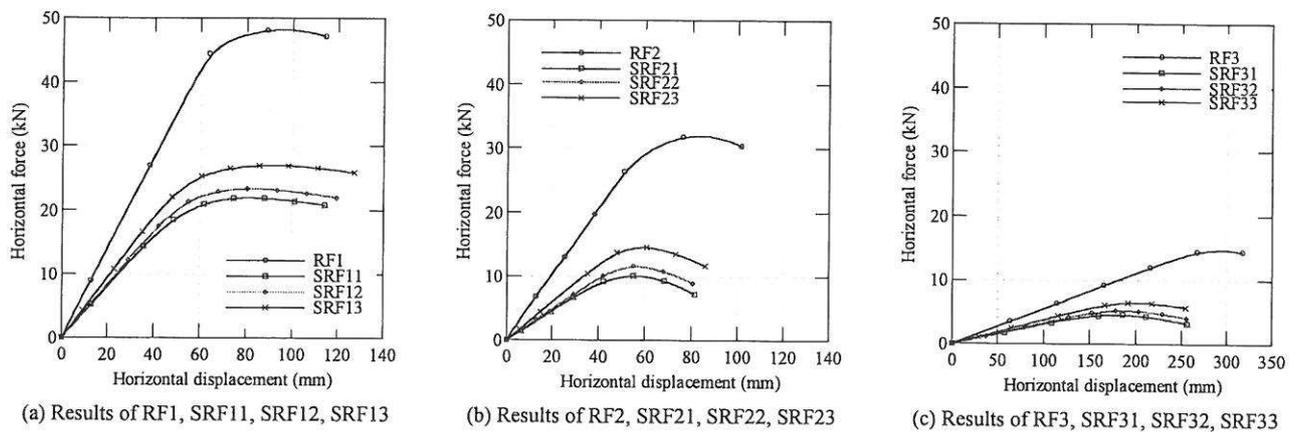


Fig. 10: Load-displacement relationship of portal frames

5. Conclusions

The experimental behavior of fifteen full size semi-rigid connections and three subassemblages with semi-rigid connections was conducted. It was observed that the stiffness and strength of the connections increases over the standard bolting type, depending on the details of enhancement. For all the three subassemblage specimens tested, the restraining beams remained elastic. The P-delta effect and significant degree of yielding causes instability failures of the columns. The test results of the connections and the subassemblages agree with the verification by the finite element inelastic analyses program.

The parametric analysis of portal frames with varying degree of connection rigidity was conducted. It was found that the horizontal load capacity of the frames decreases with the decrease in connection rigidity and with the increase in the column slenderness ratio.

6. Acknowledgement

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