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Seismic behavior of steel beam-to-box column connections

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ABSTRACT: In order to provide the needed stiffness for a beam-to-box column joint in moment resisting frames, column diaphragm plates are installed opposite the beam flanges. As a result, beam bending moments are transferred primarily through the beam flanges into the box column. In this paper, the strength requirements for beam-column moment connections are critically reviewed. Cyclic performance of large beam-to-box column connections from recent tests are discussed. Experimental results confirmed that if the ultimate flexural capacity of the beam flanges alone is less than the strain-hardened beam moment, the conventional bolted web and welded flange beam-column connection details may not be able to provide the ductility capacity needed to survive a severe earthquake. The paper concludes with the recommendations for the seismic design of beam-to-box column connections.

INTRODUCTION

Ductile moment resisting frames (MRFs) have been widely adopted as viable structural system for buildings in high seismic risk. In particular, steel MRFs using wide flange beams connected to box columns have gained wide acceptance in the constructions of typical three dimensional MRFs and the corner columns of a framed tube system. As the panel zone in the box column joints is relatively strong, the beam-to-box column connections are required to develop the beam flexural capacity thereby dissipate seismic energy.

Past experimental research has extensively assessed the performance of beam-to-wide flange column connections (Popov et. al. 1985, Popov and Tsai 1989, Popov et. al 1989) and numerous field applications have provided a database for code provisions. Due to the cost benefits, the conventional bolted web and welded flange details commonly used for connecting the beam to wide flange column have been widely adopted in the construction of beam-to-box column connections. Nevertheless, research results on the behavior of beam-to-box column connections of realistic sizes are very limited (Linderman and Anserson 1990). Recent code provisions require that, in addition to the web bolting, supplementary welds are required to connect the beam web to the shear tab when the beam web moment capacity exceeds 30% of the entire beam flexural capacity (AISC 1990). While this is one step forward toward providing a stronger connection, the design criteria for the supplementary welds remain to be developed and validated (Popov and Tsai 1989). Further more, the relatively flexible box column plate, adjacent to the shear tab and between the two column diaphragms, may not be able to transmit the beam web moment regardless whether the beam web is fully welded to the shear tab or not.

In this paper, results of analytical and experimental investigations (Tsai and Lin 1992, Tsai and Liu 1992) on the cyclic behavior of large steel beam-to-box column connections conducted recently at the National Taiwan University are discussed. From these studies, it is confirmed that the beam web moment at the connection is primarily carried by the beam flanges due to the rigidity of the column diaphragm plates opposite the beam flanges. Based on these findings, the beam flange flexural strength criterion for the construction of seismic beam-to-box column connection using conventional bolted web and welded flange details is proposed.

STRENGTH REQUIREMENTS

As noted in many experimental tests (Linderman and Anderson 1990, Popov et. al. 1985, Popov and Tsai 1989) for the severe service intended for beam-to-column connections, traditional bolted web and welded flange beam-to-column connections show inadequate ductility and are not reliable in providing the rotational capacity needed to survive a major earthquake. While the poor ductility is generally considered being attributed to the bolt slippage (Popov et. al. 1985, Popov and Tsai 1989) it is instructive to review the strength of this particular type of connection as it can provide further insights into the failure of connections.

Considering a conventional beam-to-column moment connection where the beam flanges are welded to the column flange using full penetration weld and the web is bolted to the shear tab to resist the design shear only, then the following strength criterion should be met in order to sustain the beam bending moment as inelastic rotation develops in the beam:

$$Z_F F_u \geq \alpha Z F_y \quad \text{or} \quad \frac{Z_F}{Z} \geq \alpha \frac{F_y}{F_u} \qquad (1)$$

where Z_F and Z are plastic section moduli of the flanges and the entire beam section, respectively; F_y and F_u are the minimum yield strength and the ultimate tensile strength of the beam, respectively; α, depends on the magnitude of the beam inelastic rotation and represents the effect of strain hardening. Fig. 1 illustrates the relationships of the section moduli ratio, Z_F/Z, and the strength ratio, F_u/F_y, for various α in Eq. 1.

An extensive parametric study on beam moment-rotation relationships has been conducted for cantilever beams using elastic-plastic-strain hardened trilinear constitutive model (Tsai and Liu 1992) for beams of various grade, length and sectional moduli ratio. It is found from the study that, depends on the beam length, grade and section moduli ratio, but in general a bending moment of magnitude between *$1.1ZF_y$ and $1.2ZF_y$ will be developed at the beam-column connection* when a beam plastic rotational demand of 0.015 radian is reached.

Since a plastic rotational capacity in the order of 0.015 radian is generally required for a typical beam-column connection in MRFs with strong beam-column panel zones (Popov et. al. 1985), an α of 1.2 in Eq.

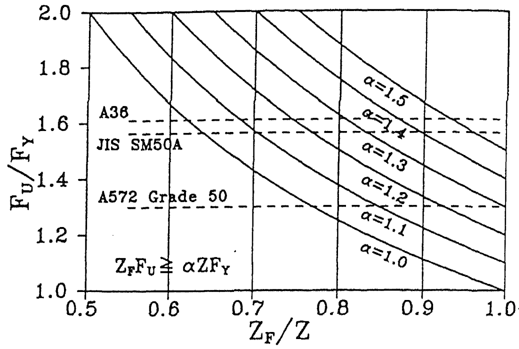

Figure 1 Sectional moduli ratio and yield ratio relationships for various strain hardening effects

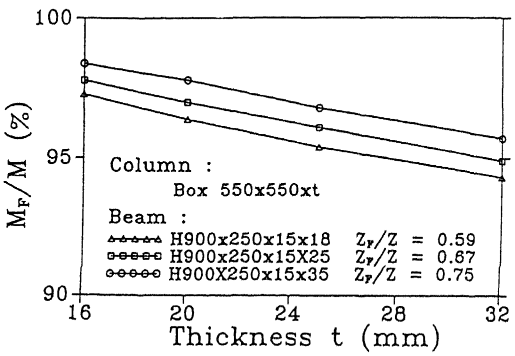

Figure 2 Beam flange moment versus column plate thickness relationships

1 is therefore appropriate for evaluating the required strength for the beam-column connection. From Eq. 1 or Fig. 1, it is clear that for certain steel grades and beam sections where web flexural capacity is significant (i.e. small Z_F/Z ratio) and yield ratio (F_y/F_u) is relatively large, beam-to-column connection using the conventional bolted web and welded flange details may not be able to sustain the beam flexural demand. In such cases, i.e. $Z_F F_u < \alpha Z F_y$ beam web should be welded to the column flange or shear tab if it is a beam-to-wide flange column connection and the design strength of the welds should at least be $\alpha Z F_y - Z_F F_u$ from the flexural strength point of view. Additional bolting and web welds should be provided to develop the beam shear force.

However, for a typical beam-to-box column connection where column interior diaphragm plates are provided opposite the beam flanges, beam flanges carry most of the beam bending moment regardless whether the beam web is fully welded to the column or not.

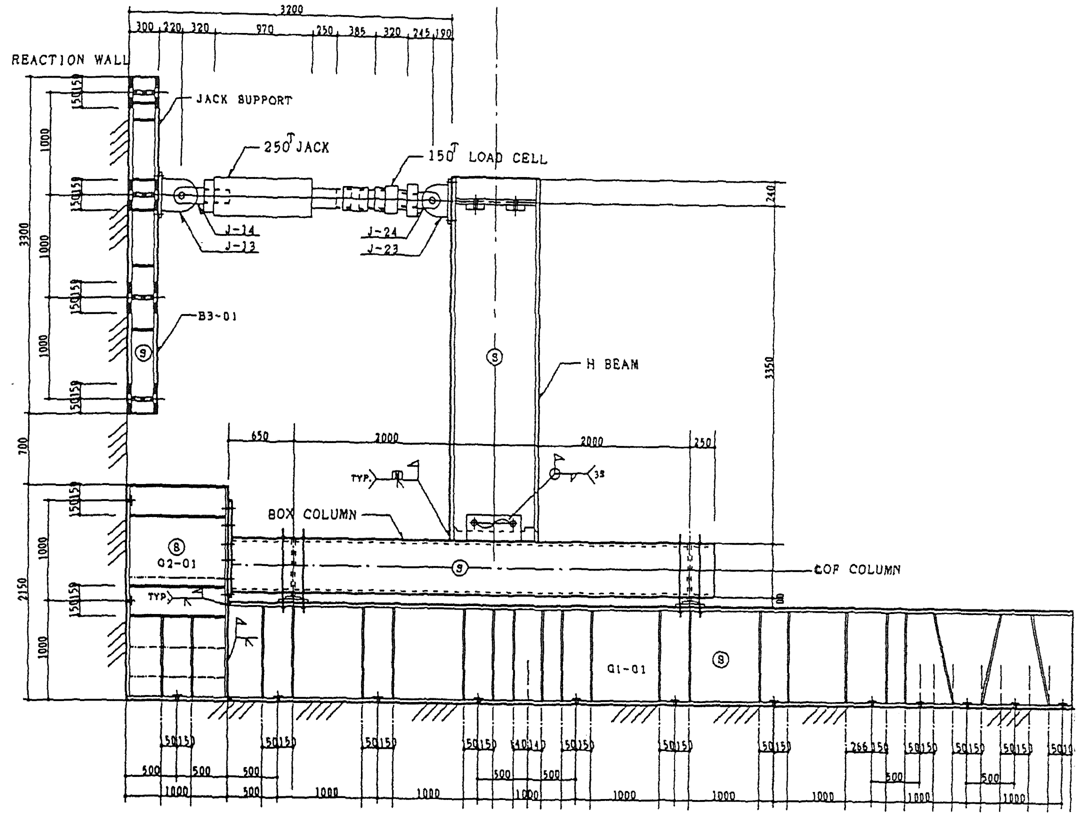

Figure 3 Experimental Set-up

This is illustrated in Fig. 2 for three different cantilever beams connected to box columns using finite element analyses (Tsai and Liu 1992). From the ratios of the bending moment carried by beam flanges and the entire beam moment, M_F/M, shown in the figure, it can be found that beam moments are transferred primarily through the beam flanges into the column. Since the stiffness at the beam web-to-column region is not pronounced as demonstrated in these finite element analyses, it appears that even for box columns with relatively thick plates, the beam flange reinforcing plates may be required for the beam-to-box column connection in MRFs when the connection flexural strength requirement is violated. The stiffening requirement for the beam flange should be based on Eq. 1 in which Z_F includes the effects of the beam flange stiffeners. The beam web should be adequately bolted or welded in order to develop the beam ultimate shear.

Using the strength criterion Eq. 1 and the corresponding α of 1.2 for the design of test specimens, ductile behavior in recent tests of several beam-to-box column subassemblages discussed later in this paper have been achieved. The strength requirements for both the beam-to-wide flange column and beam-to-box column connections described in this section, seem not adequately addressed in the current model steel seismic building code (AISC 1990), have been incorporated into the draft of the building code of Republic of China (CSSC 1991).

EXPERIMENTAL PROGRAM

All specimens were fabricated as cantilevers attached to column stubs. As shown in the experimental set-up in Fig. 3, the column is place horizontally and the cantilever beam loads are applied laterally through a actuator mounted between the reaction wall and the beam end. The beam and column sizes used are given in Table 1, where the ratios of the plastic beam flange moduli Z_F to the respective plastic beam moduli Z are given.

All columns are built-up box sections. Columns

Table 1 Schedule of specimens

Specimen (1)	Column	Beam			Beam Strength (t/cm^2)			
	Size (mm) (2)	Size (mm) (3)	Length L_b (4) (mm)	Z_F/Z [1] (5)	F_{YF} (6)	F_{UF} (7)	F_{YW} (8)	F_{UW} (9)
TB1	□ 550 × 550 × 24 × 24	H690 × 240 × 14 × 24	2305	0.73	2.67	4.55	2.88	4.59
TB2	□ 550 × 550 × 24 × 24	H772 × 250 × 16 × 27	2305	0.71	2.76	4.89	2.93	4.80
TB3	Same as TB1	H690 × 320 × 14 × 24	2305	0.78	2.64	4.52	2.86	4.55
TB4	Same as TB2	H772 × 380 × 16 × 27	2305	0.79	2.85	4.85	2.93	4.80
TB5	□ 900 × 900 × 32 × 32	H700 × 350 × 15 × 38	3000	0.86	3.03	4.88	3.10	4.76
TB6	Same as TB5	H700 × 350 × 15 × 38	3000	0.86	3.03	4.91	3.10	4.76
SB2	□ 550 × 550 × 16 × 16	H900 × 250 × 15 × 18	3175	0.59 (0.85)[2]	3.93	5.30	4.64	5.73
SB3	Same as SB2	H900 × 250 × 15 × 18	3175	0.59 (0.82)[3]	3.90	5.32	4.72	5.78
SB4	□ 550 × 550 × 25 × 25	H900 × 250 × 15 × 25	3175	0.67 (0.95)[2]	4.22	5.68	4.37	5.71
SB5	Same as SB4	H900 × 250 × 15 × 25	3175	0.67 (0.94)[3]	4.07	5.69	4.53	5.70

Notes: 1. Values in parentheses give ratio of Z_F/Z including flange stiffeners.
2. With cover plates.
3. With wing plates.

in Specimens TB1 through TB6 were made from ASTM A572 Grade 50 steel while columns in Sepcimens SB2 through SB5 were made from JIS SM50A grade. All beams are built-up wide flange sections. Beams in Specimens TB1 through TB6 were made from ASTM A36 material while beams in Specimens SB2 through SB5 were made from JIS SM50A grade. In Table 1, values of F_{YF} and F_{UF} indicate the actual tensile yield strength and ultimate strength of beam flanges, respectively, while values of F_{YW} and F_{UW} give actual yield strength and ultimate strength of beam webs. Since the Z_F/Z ratios of the beam section for Specimens SB2 through SB5 are smaller than that required from Eq. 1 for a corresponding α of 1.2 (see Fig. 1), Specimens SB2 and SB4 are reinforced with cover plates at each exterior side of the beam flanges, while in Specimens SB3 and SB5, triangular wing plates were added to the side edges of the beam flanges. Ratios of Z_F/Z considering the effect of flange reinforcements are also given in parentheses in Table 1.

All the beam-to-column connections were made in the structural testing laboratory of the Department of Civil Engineering of National Taiwan University by certified welders using bolted web and full penetration welded flange details. All web copes for the flange welds were ground smooth before welding. The beam flange welding for Specimens TB1 through TB4 were fabricated using flux-cored arc welding while the remainders were made using shielded metal arc welding. All full penetration flange welds were ultrasonically tested. All beam flange welds satisfy the specified requirements except the one in the beam bottom flange of Specimen SB2. The defect was removed and rewelded twice before passing the requirements. During the test of each specimen, increasing cyclic displacements were applied at the cantilever beam end.

EXPERIMENTAL RESULTS

The resulting cantilever beam load versus beam plastic rotation hysteretic diagrams for the specimens are illustrated in Fig. 4, in which a circle in the diagram indicates a fracture at or near the flange weld occurred whereas a square indicates a fracture near the diaphragm electro-slag weld occurred. A summary of the test results for the specimens is given in Table 2. It can be seen from Fig. 4 and Table 2 that except in the tests of Specimens SB3 and SB5 in which premature fractures of diaphragm weld occurred, most of the remaining beam-to-box column connections (except in Specimen TB4) sustained significant cyclic plastic deformations before fracture occurred in the beam flange. In Specimens SB3 and SB5 where fractures of diaphragm weld occurred, lacks of fusion of the asssociated electro-slag weldings were found after disassembling the beam-column joints.

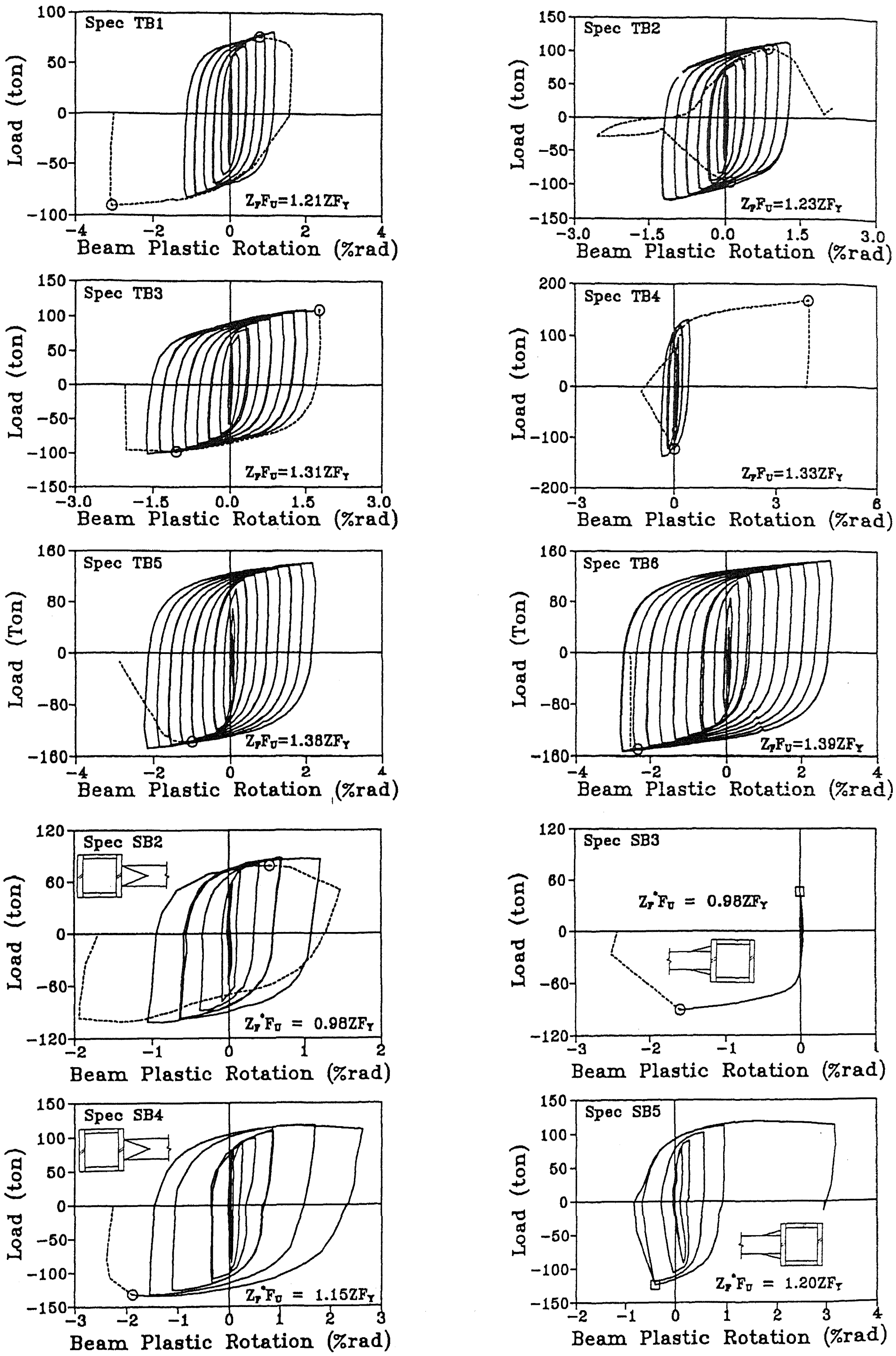

Figure 4 Beam plastic rotation versus cantilever load relationships

Table 2 Summary of experimental results

Specimen (1)	θ_{bp} (% rad)		P_u (ton)		P_u / P_p		P_u / P_F		α (10)
	+ (2)	− (3)	+ (4)	− (5)	+ (6)	− (7)	+ (8)	− (9)	
TB1	1.16	3.07	81.4	91.1	1.30	1.46	1.08	1.20	1.21
TB2	1.29	1.20	114.3	123.0	1.32	1.42	1.07	1.15	1.23
TB3	1.76	1.60	109.6	101.1	1.43	1.32	1.09	1.01	1.31
TB4	3.97	0.37	168.1	136.9	1.39	1.13	1.05	0.85	1.33
TB5	2.17	2.14	141.3	147.8	1.36	1.42	0.99	1.03	1.38
TB6	2.80	2.75	145.2	153.4	1.40	1.48	1.01	1.07	1.39
SB2	1.20	1.90	88.3	97.8	0.90	0.99	0.91	1.01	0.98
SB3	---	1.60	51.0	91.0	0.54	0.96	0.55	0.98	0.98
SB4	2.60	1.90	116.7	134.0	0.97	1.11	0.84	0.97	1.15
SB5	3.20	0.40	118.8	125.2	1.04	1.10	0.87	0.91	1.20

θ_{bp} : max. bam plastic rotation attained before failure
P_u : max. cantilever load attained before failure
"+" indicates tension in top flange

$P_p = (Z_F F_{YF} + Z_W F_{YW}) / L_b$
$P_F = (Z_F F_{UF}) / L_b$
$\alpha = (Z_F F_{UF}) / (Z_F F_{YF} + Z_W F_{YW})$

CONCLUSIONS

Using cyclic applied beam loads, test results of ten full scale beam-to-box column subassemblies using conventional and the stiffened beam-to-column connection details are introduced. It is illustrated that if the ultimate flexural capacity of the beam flanges alone is greater than strain-hardened beam moment, the conventional beam-to-box column connection details are very likely to provide the beam rotational capacity needed to survive a severe earthquake. For beam sections violate the proposed strength criterion, it is demonstrated that the strength and ductility of the beam-to-box column connections can be enhanced by properly detailed cover plates or stiffeners at the beam flanges. Based on these limited tests, it appears that the proposed beam flange strength criterion with a corresponding strain-hardening factor α of 1.2 is appropriate for the design of beam-to-box column connections intended for severe seismic service.

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