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PERFORMANCE OF DUCTILE STEEL BEAM-COLUMN MOMENT CONNECTIONS

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ABSTRACT

The performance of twelve seismic beam-to-column moment connection specimens using modified bolted web-welded flange (BWWF) details is critically evaluated. The concept of strength demand and supply for the seismic-resistant design of BWWF beam-column connections is reviewed first. Twelve modified steel BWWF beam-column moment connection specimens are fabricated using the A36 W21×62 beam section and considering the strength unbalance between the possible demand and supply. The modifications to the conventional BWWF connection details include two schemes: (1) increasing the connection strength supply by stiffening the beam flanges at the beam-to-column juncture or fully welding the beam web to the thickened shear tab, and (2) decreasing the connection strength demand by reducing the beam flange cross-sectional area in the proximity of the beam-column connection. Test results indicate that both two schemes of modification reduce the stresses in the beam-to-column flange welds effectively. Therefore, the rotational capacity of the beam-column connections are significantly enhanced. The paper concludes with the recommendations for the seismic-resistant design of steel beam-column moment connections.

KEYWORDS

Steel beam-column connection, Steel moment resisting frame, Moment connection, Welded connection, Ductility, Rotational capacity, Cover plate, Stiffener.

INTRODUCTION

Steel moment resisting frame (MRF) has been widely accepted as a viable system for earthquake-resistant structures. However, widely-spread fractures of beam-to-column moment connections have been discovered in more than 100 steel MRFs following the 1994 Northridge Earthquake. Many of the affected buildings, ranging from one to 24 stories, are designed and constructed since about 1980. The most common fabrication details found in these damaged connections are shown in Fig. 1. Many of them adopted the most recent industry standard practice and were in conformance with the modern seismic building standards (Uniform 1994). Although no casualties or collapses occurred as a result of the moment connection failure, however, it has drawn great concerns not only on how to repair the damaged connections but also on how to construct new steel MRFs. Consequently, a well coordinated research program has been launched in the US in order to examine the full range of issues pertaining to the seismic steel MRFs (SAC 1994). In order to collect additional data, a combined analytical and experimental research program assessing possible causes and solutions for these damages was conducted recently in the National Taiwan University (Tsai and Chen 1995). In this paper, the cyclic performance of twelve seismic beam-to-column moment connection specimens using various modified bolted web-welded flange (BWWF) details is critically evaluated. These twelve modified steel BWWF beam-column moment connection specimens were fabricated using A36 W21×62 beams and considering the strength unbalance between the possible demand ($\alpha Z F_Y$) and supply ($Z_f F_u$). Applying a large strain hardening factor, α, it is illustrated that both the supply-enhancement and the demand-reduction schemes are effective means in enhancing connection ductility.

POSSIBLE DEMAND VERSUS LIMITED CAPACITY

Steel MRFs designed according to the current seismic building practices are expected to deform well into the inelastic range thereby dissipating seismic energy. Test results have indicated that a bending moment demand substantially greater than the beam plastic moment capacity, ZF_y, can be developed in a seismic beam-column moment connection. This is primarily because of the beam material strain hardening, and the larger the beam plastic rotational demand the larger the amplified bending moment, αZF_y, will be developed (Tsai and Popov 1993). However, experimental results (Tsai et al. 1995) have confirmed that the ultimate flexural capacity of a well constructed BWWF connection is rather limited, and can be satisfactorily predicted by the ultimate beam flange flexural strength $Z_f F_u$. Therefore, the strength criterion:

$$Z_f F_u \geq \alpha Z F_y \quad or \quad \frac{Z_f}{Z} \geq \alpha \frac{F_y}{F_u} \tag{1,2}$$

has been proposed for the general design of seismic steel beam-column moment connections using BWWF details (Tsai et al. 1995). It is illustrated that the smaller the cantilever beam-to-depth ratio, the larger the flexural demand may develop at the connection. In addition, the larger the variations of F_u or F_y, the larger the load factor α has to be adopted in order to achieve the required safety margin and a specified beam rotation (Tsai and Popov 1993).

Considering a strength unbalance between the demand $1.2ZF_y$ and the supply $Z_f F_u$ for sizing the supplemental web welds, it has been demonstrated that code prescribed strength for supplemental web welds, 20% of beam web plastic moment capacity, is not adequate (Tsai et al. 1995). It is more serious for high strength steel where F_y/F_u ratio is significantly larger than that for A36 steel. In addition, test results have also confirmed that a lightly strengthened BWWF connection, employing the proposed strength criterion for the supplemental web welds, may be acceptable only for steel MRFs having moderate inelastic demand. For moment connections likely to experience large inelastic deformations, it has been recommended that a large α factor and other modified connection details be considered (SAC 1994). Noted that most of the standard AISC section may not be able to sustain a large α factor in Eq. 2 especially for high strength steel where F_y/F_u ratio is relatively large.

EXPERIMENTAL PROGRAM

Following the Northridge earthquake, experimental tests conducted at the University of Texas (SAC 1994) have confirmed that well detailed and constructed flange cover plates, significantly increase the flexural strength at the beam-column juncture, can effectively enhance the rotational capacity of large beam-column connections. Experimental studies conducted prior to the Northridge earthquake have indicated that the rotational capacity of steel moment connection can be effectively enhanced by reducing the beam flange cross-sectional area in the proximity of the connection (Plumier 1990, Chen and Yeh 1994). In order to collect additional data, twelve modified BWWF moment connection specimens were tested at the National Taiwan University (Tsai and Chen 1995).

All beam-column connection tests were conducted on specimens fabricated as cantilevers attached to column stubs. As shown in the experimental set-up in Fig. 2, the column was placed horizontally,

Table 1 Schedule of specimens

Specimen (1)	Beam (2)	Column (3)	Connection detail (4)	Shear tab thickness(mm) (5)
SB1	W21x62	W14x176	T&B 2x5-36ϕ flange holes	15
SB2	W21x62	W14x176	T&B 4x9-18ϕ flange holes	15
SB3	W21x62	W14x176	T&B 4x6-22ϕ flange holes	15
WB1	W21x62	□ 450x450x32x32	T&B wing plates	15
WB2	W21x62	□ 450x450x32x32	T&B wing plates	15
CB1	W21x62	□ 450x450x32x32	T&B triangular cover plates	15
CB2	W21x62	□ 450x450x32x32	T&B triangular cover plates	15
CB3	W21x62	□ 450x450x32x32	Top triang. /BOT rect. cover plates	15
CB4	W21x62	□ 450x450x32x32	Top triang. /BOT rect. cover plates	15
NB1	W21x62	□ 450x450x32x32	bolted web	15
NB2	W21x62	□ 450x450x32x32	welded web	25
NB3	W21x62	W14x176	welded web	25

All beams are A36 and all columns are A572 Grade 50
Z_F/Z of W21x62 is 0.73
All web bolting consists of 5-1"ϕ A325X bolts

Table 2 Summary of test results for all specimens

Specimen (1)	θ_p (%rad)		M_U/M_P		Flange failure mode (6)
	+ (2)	- (3)	+ (4)	- (5)	
SB1	2.89	2.50	1.19	1.09	buckled & TF cracked
SB2	1.94	2.01	1.19	1.12	BF fractured
SB3	2.68	2.17	1.18	1.16	TF fractured
WB1	3.10	2.85	1.52	1.44	buckled & BF cracked
WB2	2.81	2.85	1.66	1.51	buckled & TF cracked
CB1	3.32	3.44	1.65	1.45	buckled
CB2	4.10	3.82	1.59	1.48	buckled
CB3	2.99	2.93	1.34	1.28	buckled & BF cracked
CB4	2.88	2.49	1.76	1.60	BF fractured
NB1	1.68	1.35	1.33	1.21	TF fractured
NB2	2.80	2.86	1.52	1.34	buckled & BF fractured
NB3	2.11	1.95	1.43	1.36	BF fractured

M_P (744 kN-m) is based on coupon tensile strength
$Z_F F_U$ = 818 kN-m
+ : indicates tension in beam bottom flange

and the cyclic loads were applied laterally through an actuator mounted between the reaction wall and the beam end. As summarized in Table 1, a total of twelve specimens consisting of different modifications to the same beam section were tested. Specimen NB1 (Fig. 1) represents the standard BWWF connection. The modifications to the conventional BWWF connection details include two main categories: (1) increasing the connection strength supply by stiffening the beam flanges at the beam-to-column juncture or by fully-welding the beam web to the thickened shear tab, (2) decreasing the strength demand by reducing the beam flange cross-sectional areas in the proximity of the beam-column connection. The stiffening schemes for the connection include fully welding the beam web to the shear tab (NB2 and NB3), applying flange cover plates (Specimens CB1 through CB4), flange wing plates (WB2) and widened beam flanges (WB1). The strength demand-reduction scheme employs drilled holes in the top and bottom flanges (SB1, SB2 and SB3). All beam sections are W21×62 built up from A36 plates with Z_f/Z ratio of 0.73. Tensile coupon test results indicate that the yield strengths F_y are 317 and 306 Mpa, tensile strengths F_u are 487 and 469 Mpa, for beam flange and web, respectively. All beams were about 2.1-m-long, measured from the point of applied load to the top column flange.

A total of three column stubs was used for the cantilever beams. As illustrated on the column in Fig. 2, beams in Specimens SB1, SB2, SB3 and NB3 were connected to a W14×176 (HC1) column. Specimens CB1 through CB4 were made using a 450×450×32 box column (BC1), while Specimens WB1, WB2, NB1 and NB2 were fabricated using another box column (BC2) of the same size as Column BC1. All column stubs were built-up sections, including W14×176 ($H380\times395\times19\times30mm$), of A572 Grade 50. These sections were chosen to provide relatively strong panel zones so that, except for Specimen NB3, limited inelastic deformations could occur in the panel zones before significant flexural yielding developed in the beam sections. No doubler plate was welded to the column panel zone for Specimen NB3.

Connection details for all specimens are schematically illustrated in Fig. 3. All specimens were provided with continuity plates (Column HC1) or diaphragm plates (BC1 and BC2) of a thickness equal to the beam flange thickness. The continuity plates were welded to columns using partial penetration double-bevel groove welds while diaphragm plates were attached to the column applying electroslag welds. Complete penetration single-bevel groove welds were used to connect the beam flanges to the column flange in all specimens. Except for Specimens NB2 and NB3, where two A325-X 1-in-diameter erection bolts and 15-mm size three-side fillet welds were used attaching the beam web to the 25-mm thick shear tab, all other beam webs were connected to the 15-mm thick shear tabs using five A325-X 1-in-diameter bolts. All web bolts for shear were designed for bearing using the LRFD specifications (1993). They all satisfied the requirements to resist a beam shear corresponding to the ultimate beam bending moment developed at the connection.

The expected ultimate bending moments were $Z_f{}^* F_u$ for Specimens CB1 through CB4, WB1 and WB2; $Z_f F_u + Z_w F_y$ for Specimens NB2 and NB3; and $Z_f F_u$ for Specimens SB1, SB2, SB3 and NB1. Noted that $Z_f{}^*$ represents the plastic sectional modulus considering the contributions of the cover plates, the wing plates or the widened part of beam flanges. The shear tabs for Specimens NB2 and NB3 were designed to carry the design shear and the beam web plastic moment capacity $Z_w F_y$. Noted that all bolt spacings were 75-mm, and all bolt edge distances were 60-mm. Although the bolts were designed for bearing, all bolts were carefully tightened before making flange welds. All bolts were pretensioned, as that required for the slip critical connection (*Load* 1993), by using a calibrated torque wrench. A 9 mm × 25 mm cross-section backup bar was provided for each flange groove weld. The backup bars were 100-mm wider than the beam flange width and remained in place after the welds were completed. All flange welds were made by using Self-shielded Flux Cored Arc Welding procedures. The weld electrodes were E70T-7, 2.4-mm-diameter of NR-311 grade, made by Lincoln Electric Company. All flange groove welds were made in the Structural Testing Laboratory of the University. They were all made by a certified welder with the specimen in an upright position to simulate field conditions. Ultrasonic tests (UT) were conducted for all flange groove welds, and they

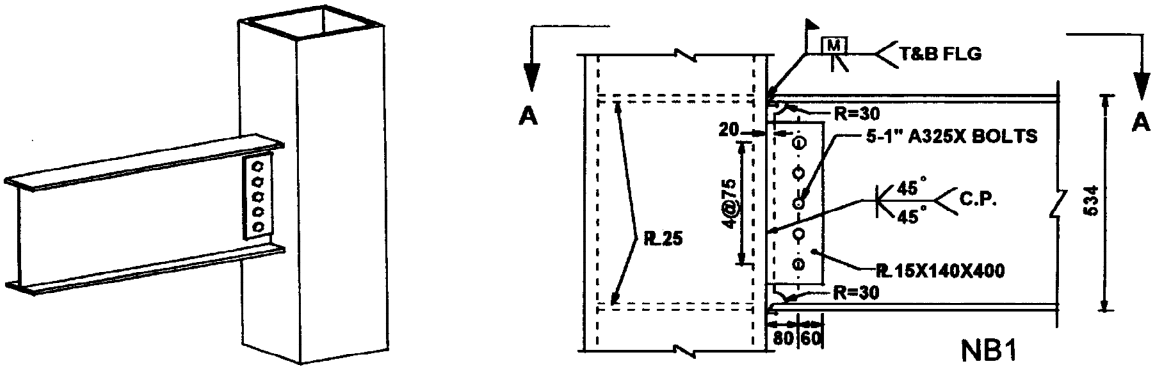
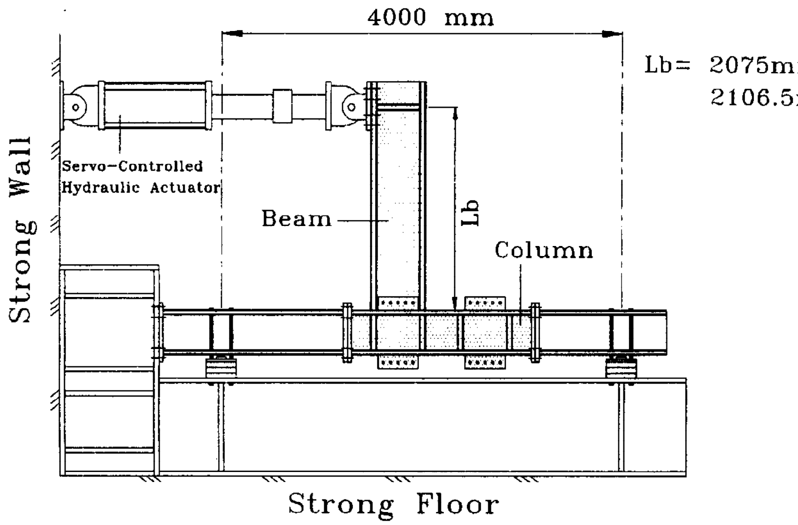


Figure 1 Typical BWWF details for beam-column connection



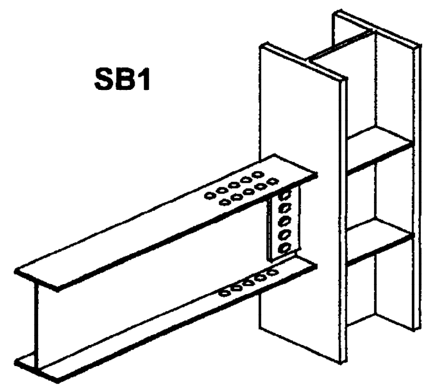


Figure 2 Experimental set-up for steel moment connection tests

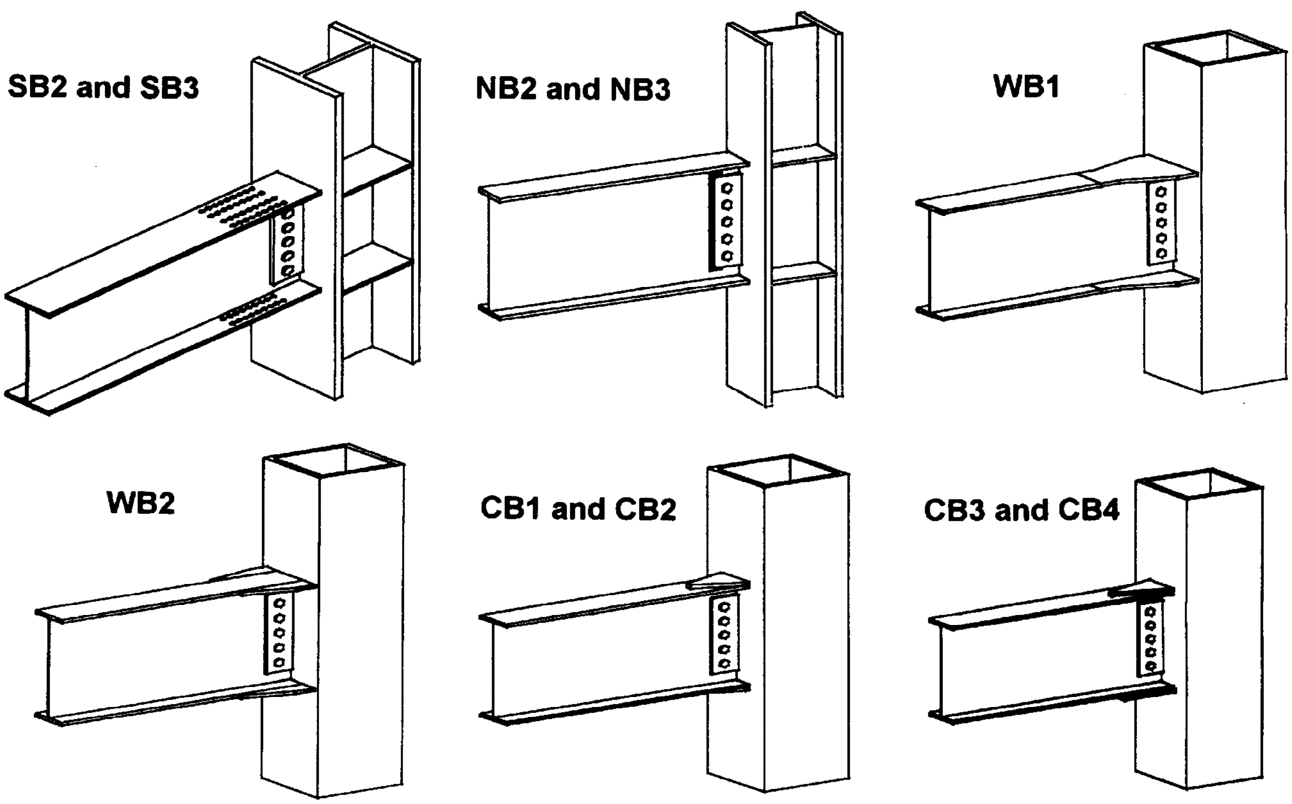


Figure 3 Schematic details for specimens

all satisfied the code prescribed acceptance criteria for statically loaded structures (*Structural* 1992). The top beam flange of each specimen was connected to the hydraulic actuator.

EXPERIMENTAL RESULTS

All specimens were tested by imposing cyclically increasing displacements at the cantilever end. The cantilever displacement increments between each cycle were 8 mm. All tests were conducted using a servo-controlled hydraulic actuator, with a pre-programmed displacement history, until failure occurred. Then, the cantilever end displacements were switched to manual control. Finally, a monotonic half-cycle reversed load terminated each test before either a failure occurred again or a large beam end displacement was achieved. The recorded beam end displacements of each specimen can be interpreted as the total deformations consisting of column rotations, deformations of the column panel zone and the beam segment. The beam plastic rotation is obtained by subtracting the elastic beam responses and all other deformation components. The beam plastic rotation versus cantilever load relationships for all specimens are shown in Fig. 4. Positive force indicates the beam bottom flange under tension. All fractures occurred either in the flange groove welds or in the adjacent beam or column flange heat affected zones. Table 2 summarizes the maximum beam plastic rotations, θ_p, and maximum bending moment, M_u, attained before failures occurred in all specimens. Specimen CB2 possessed the best rotational capacity (0.038 radian) and Specimen NB1 had the worst (0.014 radian). In terms of strength, all specimens developed the beam plastic moment capacity computed from the measured yield strength (Table 2).

Specimens' Performance

Quality of Beam Flange Groove Welds

Difficulties in assuring good quality groove welds for beam flange welds often arise when the welding procedure specifications or the welding electrode are not properly conformed (SAC 1994). The backup bars project 50-mm beyond both sides of the flange. Using end tabs for the twelve specimens, each pass of flange groove welds was initiated, and terminated when possible, at a point outside the flange edge. This was done to prevent poor quality welds, normally occurring at the initiation of the weld, from entering the beam flange-to-column juncture. Of the twelve specimens tested, six specimens experienced fractures in the beam flange (Table 2). In addition, the quality of the groove welds for all the beam top flanges appears to be about the same as that for the bottom flanges in this series of tests.

Strength of Specimens

As described above, all connections developed the beam plastic moment capacity. In addition, it is shown in Table 2 that, except for Specimens SB1, SB2 and SB3, where drilled flange holes were provided, the effects of cover plates and wing plates on the connection's ultimate moment capacity is very significant. Nevertheless, the effects of fully welding the beam web to the thickened shear tab on connection strength are evident when comparing either Specimens NB2 or NB3 with NB1.

Beam Plastic Rotational Capacity

From Table 2 and Fig. 4, it can be seen that the cyclic beam plastic rotational capacity, θ_p, of all specimens ranges between about 0.014 and 0.038 radian. These two values are based on the plastic deformations attained before fracture occurred for all specimens, since the specimen was no longer capable of resisting cyclic loads after the fracture. This range of plastic rotational capacities is considered to be satisfactory, compared with other test results (Engelhardt and Husain 1993, SAC 1994, Tsai et al. 1995). In this series of tests conducted on modified BWWF connection tests, it is evident that both the strength supply-enhancement scheme (Specimens CBs, WBs, NB2 and NB3) and the strength demand-reduction scheme (Specimens SBs) have effectively enhanced the beam plastic rotational capacity.

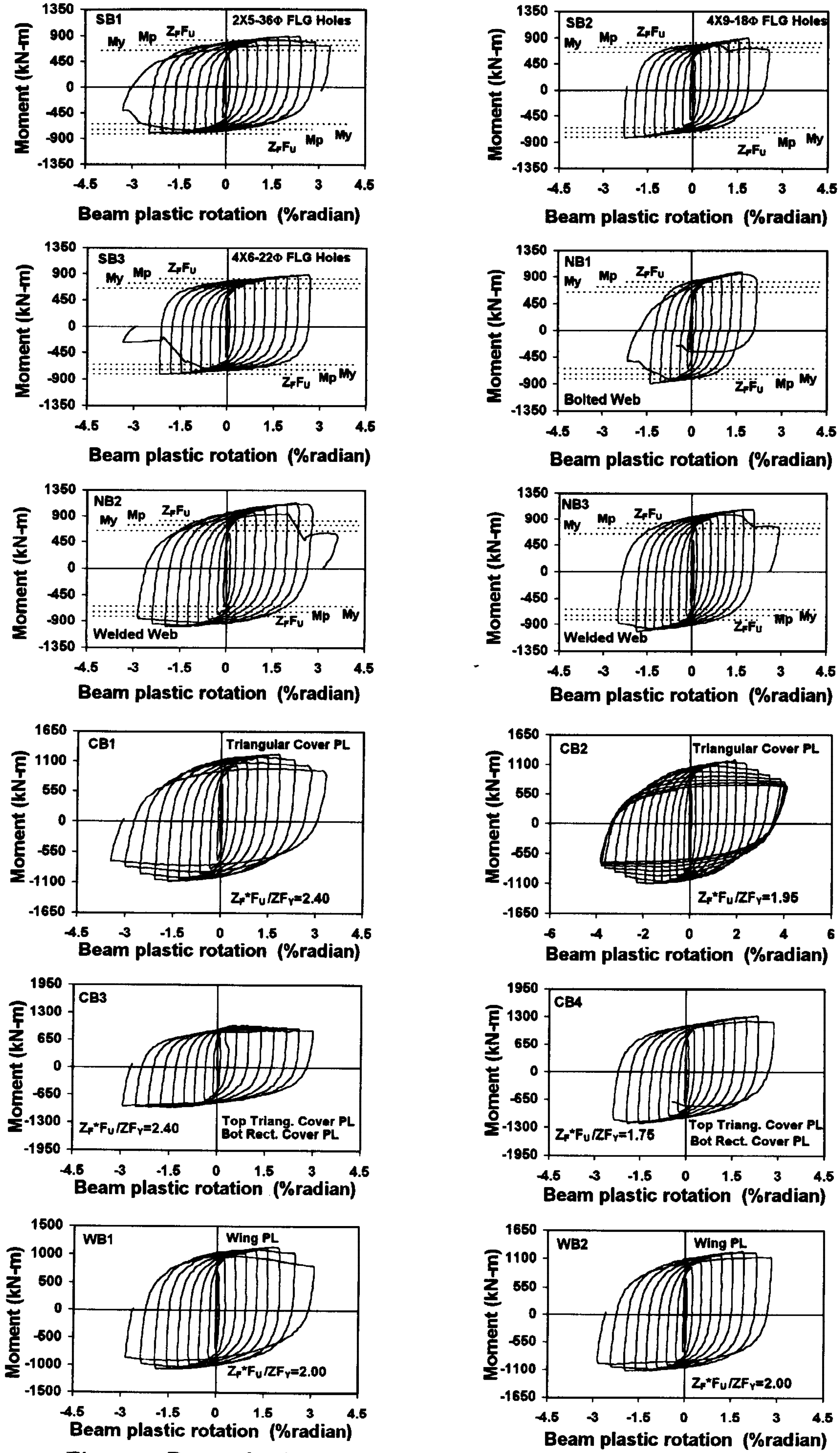


Figure 4 Beam plastic rotation versus cantilever moment relationships

CONCLUSIONS

Since the disclosures of the steel moment connection failures following the Northridge Earthquake, it has been a general consensus that the historical practices and designs are no longer appropriate for the design and construction of steel MRFs likely to experience large inelastic demand from earthquake (SAC 1995). It is found from the nonlinear finite element analyses and the tests that both two schemes noted above effectively reduce the flexural stress in the beam-to-column flange welds and the heat affected zones before a large beam plastic rotation is developed. In this series of tests, the averaged cyclic rotational capacity of the modified beam-column connections is significantly enhanced (greater than 0.025 rad). In addition, if a thickened shear tab is properly designed and constructed, it has shown that moment connections using the fully welded beam flange and web details can achieve a cyclic plastic rotational capacity significantly greater than that found in the conventional BWWF steel moment connections. From this series of test, it is concluded that the rotational capacity of the steel beam-column moment connections can be greatly enhanced by using the proposed modification schemes. It is confirmed (not shown in this paper) that if a strain hardening factor α of 1.35 is applied in computing the strength requirements for the beam flange stiffeners ductile response of steel beam-column moment connections can be achieved.

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