Steel and Composite Structures, Vol. 7, No. 2 (2007) 87-103
DOI: http://dx.doi.org/10.12989/scs.2007.7.2.087

Experimental investigation of force-distribution in high-strength bolts in extended end-plate connections

K. M. Abdalla†, G. A. R. Abu-Farsakh‡ and S. A. Barakat‡†

Civil Engineering Department, Jordan University of Science and Technology, P.O. Box 3030, Irbid-22110, Jordan

(Received March 24, 2006, Accepted November 20, 2006)

Abstract. This paper presents some of the results from an experimental research project on the behavior of extended end-plate connections subjected to moment conducted at the Structural Laboratory of Jordan University of Science and Technology. Since the connection behavior affects the structural frame response, it must be included in the global analysis and design. In this study, the behavior of six full-scale stiffened and unstiffened cantilever connections of HEA- and IPE-sections has been investigated. Eight high strength bolts were used to connect the extended end-plate to the column flange in each case. Strain gauges were installed inside each of the top six bolts in order to obtain experimentally the actual tension force induced within each bolt. Then the connection behavior is characterized by the tension force in the bolt, extended end-plate behavior, moment-rotation relation, and beam and column strains. Some or all of these characteristics are used by many Standards; therefore, it is essential to predict the global behavior of column-beam connections by their geometrical and mechanical properties. The experimental test results are compared with two theoretical (equal distribution and linear distribution) approaches in order to assess the capabilities and accuracy of the theoretical models. A simple model of the joint is established and the essential parameters to predict its strength and deformational behavior are determined. The equal distribution method reasonably determined the tension forces in the upper two bolts while the linear distribution method underestimated them. The deformation behavior of the tested connections was characterized by separation of the column-flange from the extended end-plate almost down to the level of the upper two bolts of the lower group and below this level the two parts remained in full contact. The neutral axis of the deformed joint is reasonably assumed to pass very close to the line joining the upper two bolts of the lower group. Smooth monotonic moment-rotation relations for the all tested frames were observed.

Keywords: steel connections; high strength bolts; moment rotation curves; experimental.

1. Introduction

Moment end-plate connections are used in industrial buildings. Their use in multi-storey, moment-resistant frame construction is becoming more common because of advancements in design methods and fabrication techniques. Current trends in the analysis and design of structures are towards considering the interaction of various components rather than the treatment of individual elements. This work has, of course, been encouraged by the increased availability of enhanced computational facilities.

†Assistant Professor, Corrresponding Author, E-mail: abdalla@just.edu.jo
‡Professor, E-mail: ghazi@just.edu.jo
‡†Associate Professor, E-mail: samer@just.edu.jo

However, most of steel frame design still firmly relies on a simplified assessment of the methods by which loads are resisted by the structure, largely due to the uncertainty regarding the exact behavior of the connections between the structural members. Thus, the two extremes of 'simple construction', in which beam-to-column connections are assumed to be incapable of transmitting significant moment, and 'rigid-frame connections', in which full rotational continuity of connection is assumed, are still used as design approaches.

It has long been recognized that most forms of steel-work joints actually function in a semi-rigid fashion, possessing some finite degree of rotational stiffness which may well be a function of the applied loading. Attempts have been made (Chen and Kishi 1989), to collect test data on the behavior of joints of various types with the aim of evaluating their restraint characteristics. Such information may then be used to study the effects of semi-rigid joint action on the performance of beams and columns as well as the overall behavior of flexibly connected frames. Several major codes permit partial restraint to be utilized, but give little guidance on how this should be done, for example AISC (2005) and BS-5950 (2001) permit partial restraint to be utilized but give little guidance on how this should be done.

Structural joints can be divided into three characteristic zones, tension, compression and shear, whose deformation contributes to the overall joint response. The joint behavioral characteristics can be represented by means of moment-rotation curves that define three main properties: resistance, stiffness and rotation capacity.

The research work reported in this paper focuses on the characterization of the properties of beam-to-column connection with extended end-plate. In this type of joint, the main source of deformability is the tension zone. In Eurocode-3 (2003) the joint is idealized by means of equivalent T-stubs (Baniotopoulos 1995, Mistakidis *et al.* 1996), which correspond to two T-shaped elements connected through their flanges by means of one or more bolt rows.

A T-stub connection may be regarded as one of the simplest types of joints apart from a simple lap joint. In the context of the design of metal and composite connections by the component method, T-stub becomes one of the principal components. A number of investigators had conducted experiments to build up the relationship between load and deflection (Baniotopoulos 1995, Mistakidis *et al.* 1996, Baniotopoulos 1996, Wald *et al.* 1997, Jaspart 1994, Ragupathy *et al.* 1995, Gebbeken *et al.* 1994, Gomes *et al.* 1995, Jolly 1995, Bursi *et al.* 1997).

Meng (1996) tested extended end-plate moment connections under cyclic loading. The end-plates were tested using hot-rolled beam sections ranging from 450 mm. to 900 mm. nominal depth. A design procedure for four-bolt extended moment end-plate connections, developed by Abel and Murray (1994) for monotonic loading was used to design the test specimens.

Wald and Baniotopoulos (1998) presented a two-dimensional finite element plane stress model which was constructed for the analysis of the structural behavior of a steel bolted connections. The model contains all the essential features that characterize the separation problem. Material yielding, contact interface slip and interface interaction were taken into account.

Troup *et al.* (1998) utilized the ANSYS computer program to model bolted steel connections which included extended end-plate as well as T-stub models. Special contact elements were considered for the various connections which accounted for the geometric nonlinearities resulting during the separation of contact surfaces. In their analysis, the authors considered a bi-linear stress-strain relationship for the bolts.

Coelho *et al.* (2004), carried out an experimental study of eight statically loaded end-plate moment connections. The specimens were designed to cause failure in the end-plate or bolts without development of full plastic moment capacity of the beam. The results showed that an increase in end-plate thickness resulted in an increase in the connection flexural strength and stiffness and decrease in rotation capacity. In another study, Coelho *et al.* (2004) performed thirty two tests on isolated T-stub connections composed of welded plates. Their primary intent was to provide insight into the actual behavior of this type of connection, failure modes and deformation capacity.

Concerning the evaluation of the moment rotation capacity of the end-plate connections, there were many research articles reported in the literature. Zoetemeijer (1990) proposed a criterion and simple empirical expressions for the prediction of joint deformation capacity based on a series of experiments. Later, Jaspart (1997) extended the work of Zoetemeijer which was adopted in Eurocode-3 (2003).

2. Objective and scope

The main objective of the present paper is to obtain, experimentally, the actual tension forces in the bolts connecting the extended end-plate to the column-flange. Two-types of connections are to be considered, herein; stiffened and unstiffened frame-columns. A comparison between the experimental forces in bolts and those obtained using two theoretical approaches; equal distribution (Eql) and linear distribution (Lnr), is carried out to check the validity of these approaches. In order to give more insight into the connection behavior, strains are to be measured at different locations; column-web, upper flange, web and lower flange of the various beams. The moment-rotation behavior is investigated for such type of connections.

3. Test setup

3.1 Testing frames

Six full-scale beam-to-columns with extended end-plate connections were tested. Eight high strength M20 bolts grade-8.8 (F_y = 640 N/mm^2 and F_u = 830 N/mm^2) having nominal diameter of 20 mm (21 mm hole diameter) were arranged in two vertical rows connecting the extended end plate and the column flange (see Figs. 1 and 2). For each test, the IPE-330 beam section was attached to the HEA-220 column section through an extended end-plate. Beams, columns, and end-plates were made of steel having average yield and ultimate stresses F_y = 314 N/mm^2 and F_u = 450 N/mm^2 (see Fig. 3), respectively, obtained from three different uniaxial coupons tests. Frames were categorized as: columns with stiffeners (KGS-2S, KGS-4S, and KGS-5S) and without stiffeners (KGS-1, KGS-3, and KGS-6), as indicated in Table 1. Average dimensions, geometric and material properties of columns, end-plates and beams are given in Tables 2 to 4, respectively. End-plate was connected to the beam using continuous double-sided fillet welds of size 10 mm. In addition, stiffeners were welded continuously using 10 mm size double-sided fillet welds with the column inside-flanges and web. Column top-end was braced against sidesway (no sidesway) using three link members having double-angle sections, as illustrated in Fig. 4.

3.2 Instrumentation

At beam tip, a 200 kN capacity hydraulic actuator having a compression load cell was used to measure the force induced at the load point, and linear displacement transducers (LVDT's) was used to measure the beam deflection at three locations; 140 mm, 640 mm, and 1140 mm from the face of the

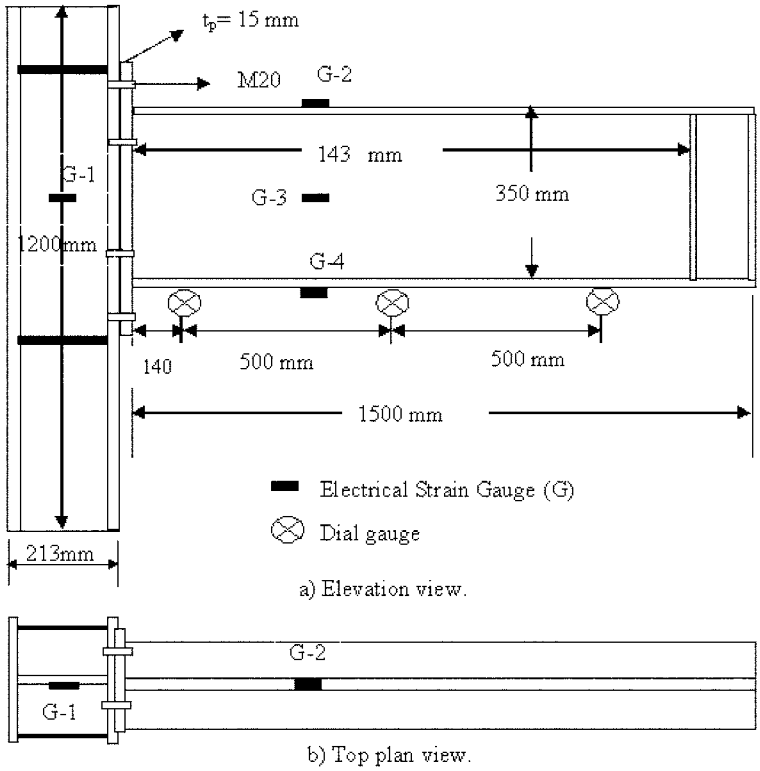


Fig. 1 Schematic representation of specimens showing dimensions, location of stiffeners and strain gauges

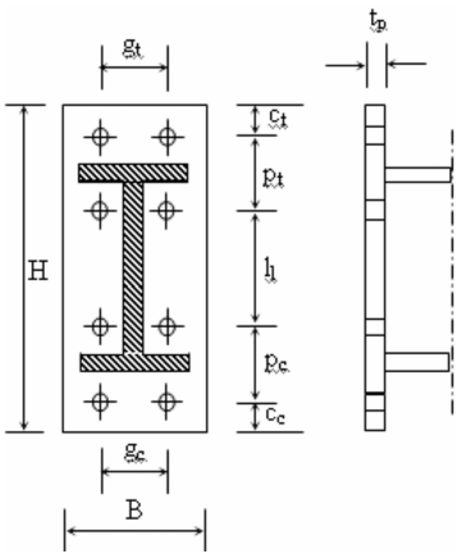


Fig. 2 Schematic representation showing end-plate geometric notations

column-flange as shown in Fig. 1. For the column, a 45-degree strain gauge rosette was placed on the column web at a central location between stiffeners for all the considered cases. For the beam three

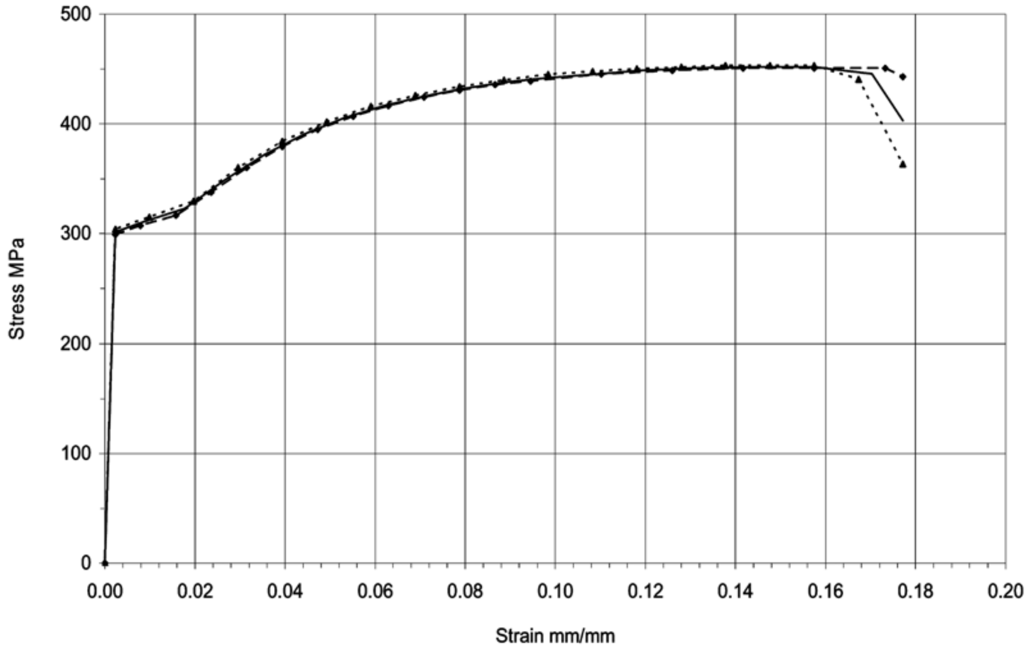


Fig. 3 Average stress-strain results obtained from column, beam and end-plate coupon tests

Table 1 Full scale test details

Test Ref.	Beam Size	Column Size	Column Stiffeners	End Plate Size (mm)
KGS-1	IPE-330	HEA-220	No	528×218×15
KGS-2s	IPE-330	HEA-220	Yes	528×218×15
KGS-3	IPE-330	HEA-220	No	528×218×15
KGS-4s	IPE-330	HEA-220	Yes	528×218×15
KGS-5s	IPE-330	HEA-220	Yes	528×218×15
KGS-6	IPE-330	HEA-220	No	528×218×15

Table 2 Column geometric dimensions and its yield and ultimate stresses

Depth d (mm)	b_f mm	t_w mm	t_f mm	A mm^2	I_x mm^4	I_y mm^4	Wt kg/m	L mm	F_y N/mm^2	F_u N/mm^2
213	222	7	10	5791	4.9973×10^7	1.8241×10^7	45.5173	1900	314	452

Table 3 End-plate geometric dimensions and its yield and ultimate stresses

Thickness t_p (mm)	B (mm)	H (mm)	c_t (mm)	p_t (mm)	l_l (mm)	p_c (mm)	c_c (mm)	g_t (mm)	g_c (mm)	F_y (N/mm^2)	F_u (N/mm^2)
15	218	528	43	90	262	90	43	140	140	314	452

single-grid strain gauges were fixed to the top of flange, mid-web and lower flange to measure axial strains at these points, see Fig. 5. A strain gauge was fitted inside the shank of the considered bolt (numbered 1 to 6) to obtain the axial force induced in each bolt by measuring the bolt-strain during the test. The pre-tension force in the bolt was achieved using a manual wrench by rotating the bolt-

Table 4 Beam geometric dimensions and its yield and ultimate stresses

Depth d (mm)	b_f mm	t_w mm	t_f mm	A mm^2	I_x mm^4	I_y mm^4	Wt kg/m	L mm	F_y N/mm^2	F_u N/mm^2
360	170	8	12	6768	1.4886×10^8	9.8403×10^6	53.1965	1500	314	452

a) Column top-end of Frame 6 braced against sidesway. b) Column top-end of Frame 5 braced against sidesway.

Fig. 4 Typical frame showing column bracing system and end-plate connection before testing

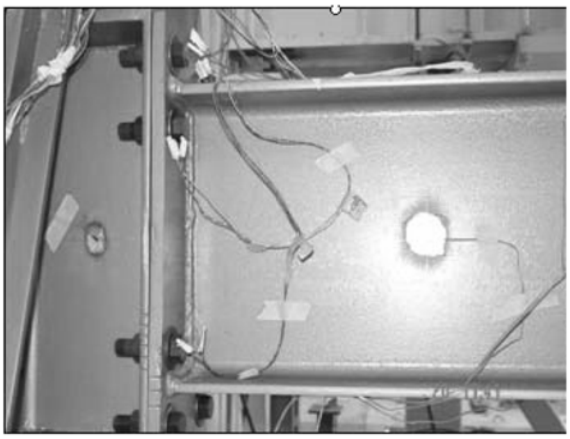


Fig. 5 Typical beam-to-column connection showing locations of bolts and strian gauges

nut one-third turn. This force is needed to bring the end plate and the column flange into full contact before testing.

The strain data readings for each of the instruments, mentioned above, were taken at several loading increments. The data points were recorded using a PC- based data-acquisition system at the Structural Laboratory of the Civil Engineering Department at Jordan University of Science and Technology (JUST). The data were transferred via disk media using commercial software for further analysis.

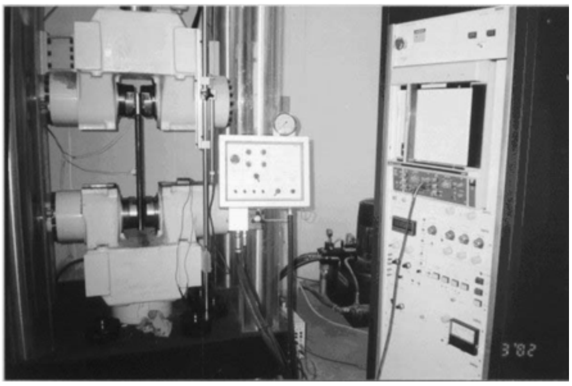


Fig. 6 Tension test setup

Fig. 7 Bolts tested to failure

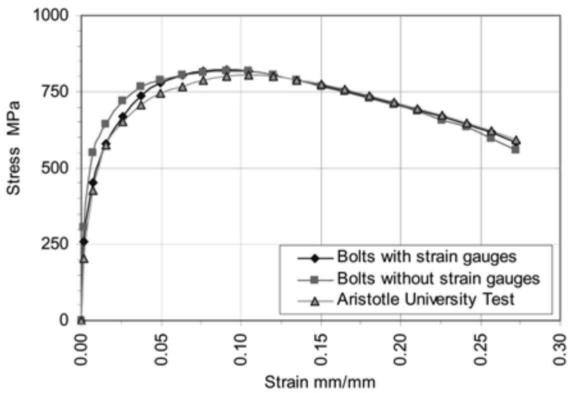


Fig. 8 Stress–strain curves for three bolts, tested with strain gauges, without strain gauges, and Aristotle University results

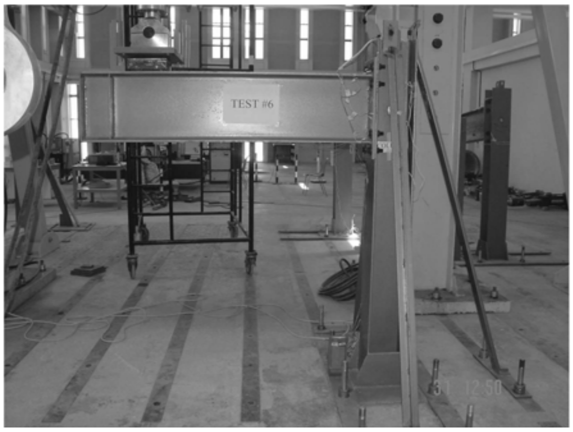


Fig. 9 Typical frame setup showing column-strong floor connection before testing

3.3 Bolt tensile testing

Bolts used were designated according to various specifications: SBE-8.8, M20×1.5×70, UNI- 5738, DIN-960 and ISO 8765. Three specimens were tested with internal strain gauges (bolt-gauge BTM-6C) and another three bolts were tested without internal strain-gauges to obtain the tensile stress-strain diagrams of the bolts. Figs. 6 and 7 show, respectively, the tension test setup and the two tested groups of bolts mentioned above. As a double check of the test results, a third sample of these bolts was sent to be tested at the Structural Laboratory of The Aristotle University of Thessaloniki-Greece. The test results of the above three groups of bolts are plotted in Fig. 8 and it is clear that they are all in good agreement. Also, it should be noted that measuring the uniaxial strain in bolts by introducing gauge requires; drilling a 2 mm diameter hole through the head and extending at least 13 mm down the shank, filling the hole with a low viscosity adhesive, then inserting the gauge and letting the adhesive to cure before testing. This technique is much simpler and more accurate than installing foil gauges on the surface of small diameter bolts. In the conventional strain gauging technique there is always a problem on how to keep the gauges from damage when the bolts are put into service and also the surface mounted gauges are, especially, susceptible to extraneous localized bending strains. Therefore, and to avoid the above mentioned problems, the bolt gauge technique is used herein and the results in Fig. 8 show that drilling out the core was almost insignificant to the bolt behavior.

3.4 Test arrangement and instrumentation

The main features of the test apparatus are illustrated in Fig. 9. The column was bolted to the steel strong floor of the Structural Laboratory at JUST. The length of the beam was chosen to ensure a realistic stress pattern developed in the connection. The primary requirements of the instrumentation were the measurement of applied load, vertical displacement of the beam at three different locations, strains in beam and column, deflected shape of the end-plate and strains in bolts (see also Figs. 1, 2 and 5).

4. Experimental results

The results presented in the following section are related to the data acquired during each test. The measured data are referred to the applied load, displacement and direct strain readings.

4.1 Behavior of high strength bolts

Figs. 10 and 11 plot the applied moments versus the bolts strains for the frames KGS-3 and KGS-4S, respectively. The experimental results indicate that the bolts in each row of the tension-zone have unequal strains, with the upper tension bolts (B1 and B4) experiencing larger strains than bolts (B2 and B5), while strains in bolts (B3 and B6) are almost equal to zero which indicates that the neutral axis is very close to the line connecting these two bolts. The measured experimental strains have been converted into bolt forces using a standard procedure and compared to theoretical values of the bolt forces computed using two methods; equal distribution (Eql) on the upper-flange bolts (B1, B2, B4 and B5) and linear distribution (Lnr) on all bolts along each row (bolts B1 to B3 and B4 to B6). The two methods are:

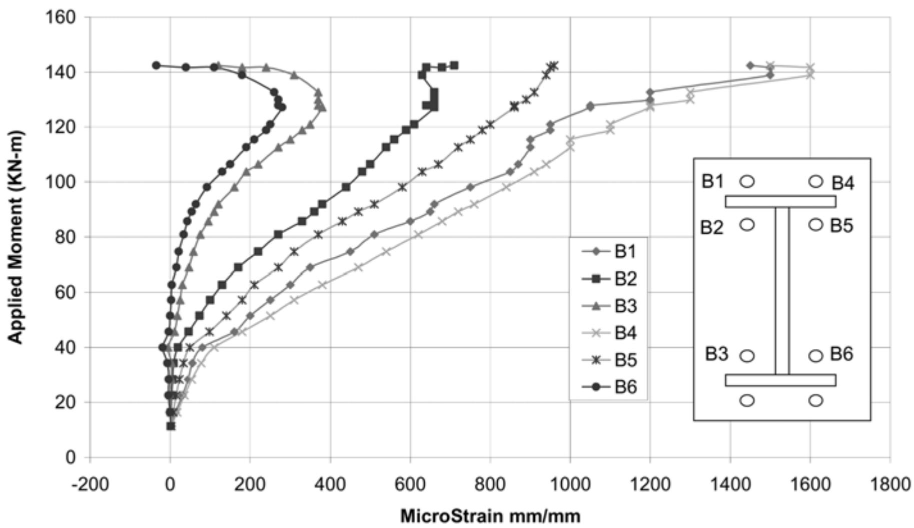


Fig. 10 Applied moment versus bolt strain for connection KGS-3

4.1.1 Equal distribution method

In this method the connection moment is resisted by equal couples (see Fig. 12) in which:

$$T = C \quad \text{and} \quad M = T.d \quad \text{then}$$

$$T = \frac{M}{d}$$

This tension force T is assumed to be equaly distributed to the bolts in tension such that,

$$F_{bolt} = \frac{T}{4} = \frac{M}{4d}$$

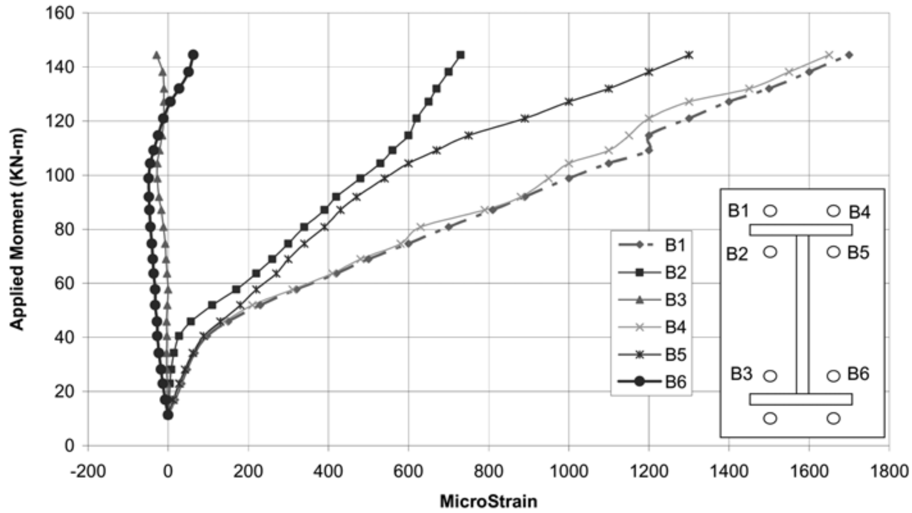


Fig. 11 Applied moment versus bolt strain for connection KGS-4S

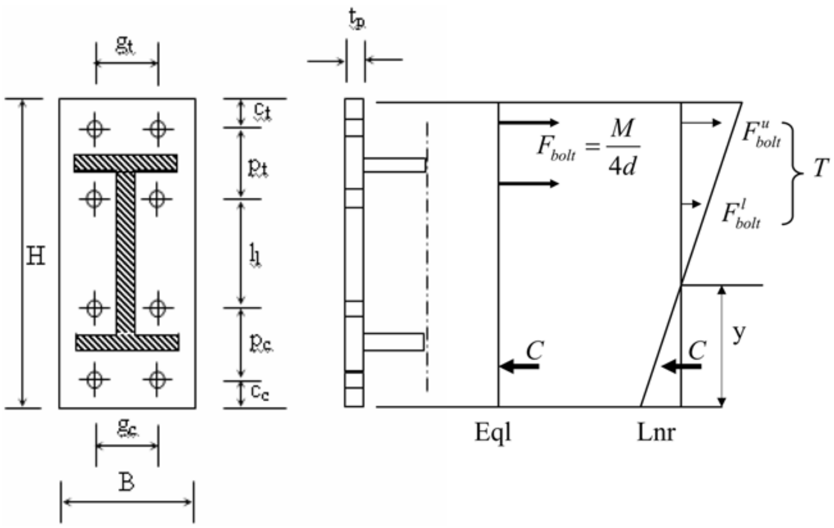


Fig. 12 Assumed connection force distribution, Equal and Linear distribution methods

4.1.2 Linear distribution method

In this method it is assumed that the center of gravity of the compression and tension areas is located at a distance y from the bottom of the connection area as shown in Fig. 12. Then,

Moment of compression area (full contact):

$$\frac{B \times y^2}{2} \tag{1}$$

Moment of tension area (bolts only):

$$2A_b((H-c_t-y)+(H-c_t-p_t-y)+(H-c_t-p_t-h-y)) \sum M_{area} = 0 \tag{2}$$

$$\frac{B \times y^2}{2} = 2A_b((H-c_t-y)+(H-c_t-p_t-y)+(H-c_t-p_t-h-y)) \tag{3}$$

$$By^2 + 6A_b y - 2A_b(3H-3c_t-2p_t-h) = 0 \tag{4}$$

Therefore

$$y = \frac{-6A_b \pm \sqrt{(6A_b)^2 + 8BA_b(3H-3c_t-2p_t-h)}}{2B} \tag{5}$$

Moment of inertia about C.G.:

$$I = \frac{1}{3}By^3 + 2A_b((H-c_t-y)^2+(H-c_t-p_t-y)^2+(H-c_t-p_t-h-y)^2) \tag{6}$$

Tension stress on bolts:

$$f_t = \frac{Mc}{I} \tag{7}$$

$$f_t^u = \frac{M(H-c_t-y)}{I}, \quad f_t^l = \frac{M(H-c_t-p_t-y)}{I} \tag{8}$$

$$F_{bolt}^u = f_t^u A_b, \quad F_{bolt}^l = f_t^l A_b \tag{9}$$

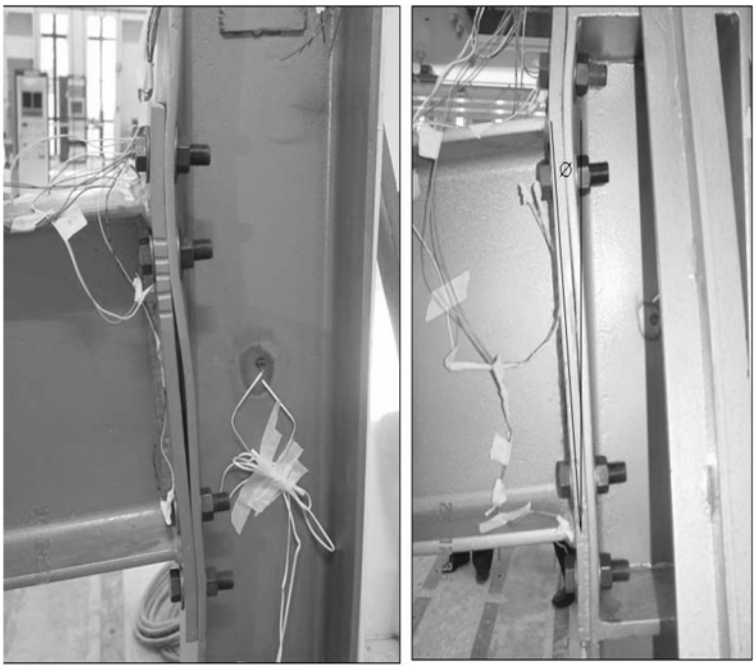
The experimental and the theoretical bolt forces are presented in Table 5.

4.2 End-plate deformation

The most significant characteristic describing the overall end-plate deformation is the separation between the extended end-plate and the column-flange in the upper part of the tension-zone. The deformed configuration of the column-flange connected to the extended end-plate is presented at final loading stage in Fig. 13 for the tests KGS-1 and KGS-2S. The deformations of the two connections show separation of the column-flange from the extended end-plate almost up to the point of lower bolts (B3 and B6) below which the two parts remain in full contact. This supports again the above conclusion that the neutral axis is very close to the line joining bolts (B3 and B6). Furthermore, the connections

Table 5 Distribution of forces in the bolts

Test Reference		KGS-1	KGS-3	KGS-6	KGS-2S	KGS-4S	KGS-5S
Load-P (kN)		113.1	126.0	120.4	106.3	127.8	128.0
R1, R4	Exp	93.3 (82.5%)	105.4 (83.7%)	103.2 (85.7%)	95.4 (89.8%)	108.8 (85.1%)	110.0 (85.9%)
	Eql	91.8	102.3	97.7	86.3	103.7	103.9
	Lnr	73.3	81.7	78.1	68.9	93.6	93.8
	$(Exp/P)_{ave}$		(84%)			(86.9%)	
R2, R5	Exp	72.7 (64.3%)	62 (49.2%)	71.2 (59.1%)	39.3 (37%)	51.8 (40.5%)	52.4 (40.9%)
	Eql	91.8	102.3	97.7	86.3	103.7	103.9
	Lnr	57.5	64.0	61.2	54.0	73.4	73.5
	$(Exp/P)_{ave}$		(57.5%)			(39.5%)	
R3, R6	Exp	3.868	22.80	5.0	1.2	1.2	2.2
	Eql	------	-------	--------	-------	--------	-------
	Lnr	11.39	12.69	12.13	10.71	14.55	14.55



a) Without stiffeners Test KGS-1 b) With stiffeners Test KGS-2S

Fig. 13 Deformed shape of a beam-to-column connection at final loading stage

without stiffeners showed more separation than those with stiffeners. This indicates again the effect of stiffeners on distributing the force among the rows of bolts. Also, local buckling of the column-flange was observed in all tested frames within the gap region, along a line connecting bolts B2 to B3 and the line connecting bolts B5 and B6.

4.3 Moment-rotation curves

The moment-rotation curves (M-F) were obtained for the stiffened frame KGS-5S and unstiffened frame KGS-6 using the vertical-displacement and applied-load readings at two different locations, and shown in Figs. 14 and 15, respectively. The bending moment (M) acting on the connection was obtained as the applied load (P) times the distance (L) between the point of load application and the face of the end plate, i.e.,

$$M = P * L \tag{10}$$

The rotational-deformation of the joint (Φ) is given as the sum of shear deformation of the column-web in the panel zone (γ) and the beam-rotation (β), as illustrated in Fig. 16, i.e.,

$$\Phi = \gamma + \beta \tag{11}$$

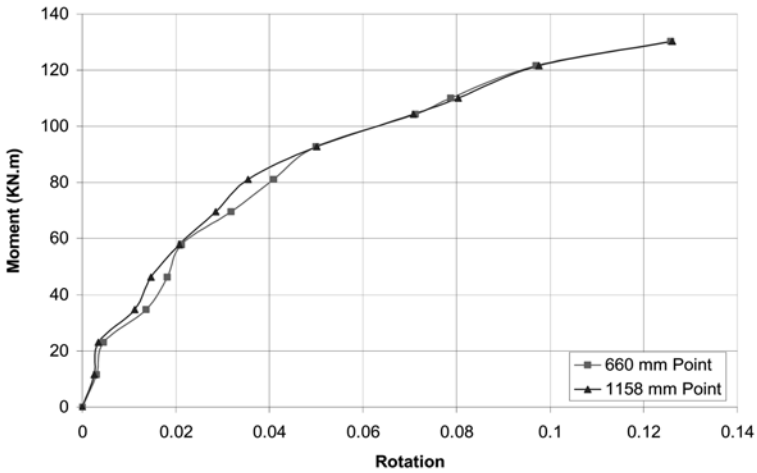


Fig. 14 Moment rotation curve for connection KGS-5S

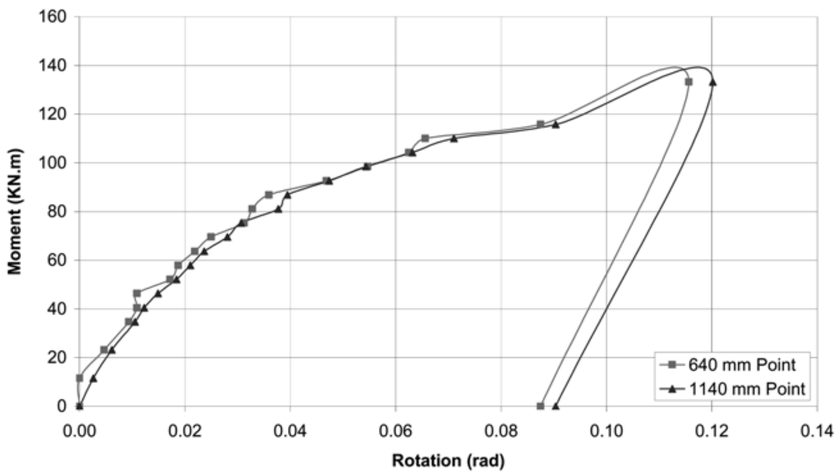


Fig. 15 Moment rotation curve for connection KGS-6

Fig. 16 Deformed shape of beam-to-column connection with stiffeners at final loading stage for test KGS-2S

Fig. 17 Final rigid beam deformed shape of Frame KGS-2S after testing

The beam-rotation (β) is obtained approximately as: the measured vertical displacement divided by the corresponding length. This assumption is reasonable since the depth of the beam (360 mm) is more larger than the depth of the column (213 mm) and therefore, the beam can be considered so rigid that it will not deform but will rotate, as can be seen in Fig. 17.

5. Discussion of test results

The measured tension force in each bolt was obtained experimentally, and compared with the corresponding theoretical values derived from equal forcedistribution (Eql) and linear force-distribution (Lnr)

Table 6 Measured loads and strains

Test Reference	Load (kN)	G-2 (μ mm/mm)	G-3 (μ mm/mm)	G-4 (μ mm/mm)
KGS-1	113.1	+ 482	+ 3.75	-550
KGS-3	126	+ 480	+ 71.57	-500
KGS-6	120.4	+ 510	+ 41.82	-510
KGS-2S	106.3	+400	-58.4	-460
KGS-4S	127.8	+ 460	+ 11	-450
KGS-5S	128	+ 470	+ 5.0	-420

approaches. The results are given in Table 5 for stiffened and unstiffened connections. As shown, the ratio of tension force/applied load in the upper bolts (B1 and B4) ranged between 82.5% to 85.7 with an average value of approximately 84% for the unstiffened connections (KGS-1, KGS-3, and KGS-6). On the other hand, for stiffened connections (KGS-2S, KGS-4S, and KGS-5S) this ratio ranged from 85.1% to 89.8% with an average value of 86.9%. With respect to bolts (B2 and B5) the corresponding tension force/applied load ratio ranged from 49.2% to 64.3% for unstiffened connections with an average value of 57.5%, while the ratio for stiffened connections ranged from 37% to 40.9% with an average of 39.5%. This indicates the effects of the column-stiffeners were slight on the upper bolts (B1 and B4) which showed an increase in the tension force in the bolts of 2.9%, whilst the effect showed on the tension force in bolts (B2 and B5) was a reduction of 18%.

Table 6 gives the measured loads and beam-strains obtained from strain gauges (G-2, G-3, and G-4) located at three different locations on the same beam section; upper flange, mid-web and lower flange, respectively. The measured strains were almost identical for stiffened and unstiffened frames. That is, the upper-flange strain (G-2) always indicated tensile strain (positive value) and the lower-flange strain (G-4) indicated compressive strain (negative value), while the mid-web strain (G-3) indicated a small value (approximately zero). The strain values were within ±25 m-strain and strain distributions were almost symmetrical about the neutral axis at the mid-web position of the beam. Representative graphs, Figs. 18 to 20 for frames KGS-3S, KGS-4S and KGS-5S, respectively, are presented to illustrate such

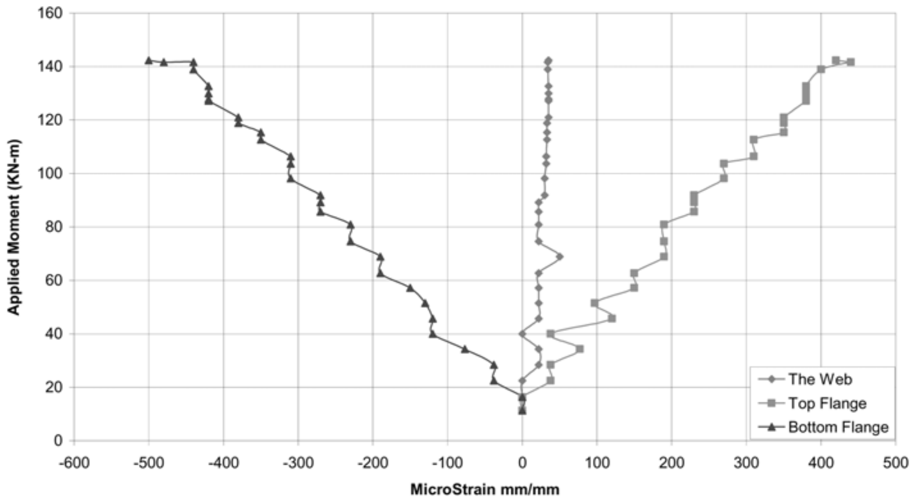


Fig. 18 Applied moment versus beam strain for connection KGS-3S

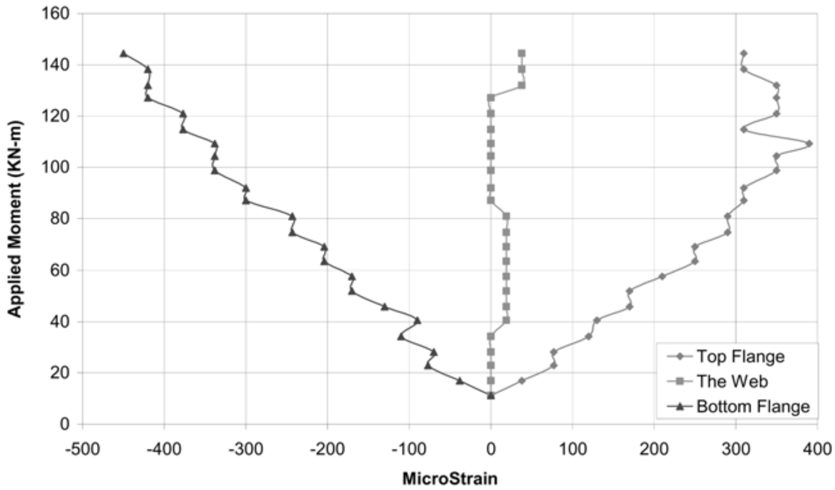


Fig. 19 Applied moment versus beam strain for connection KGS-4S

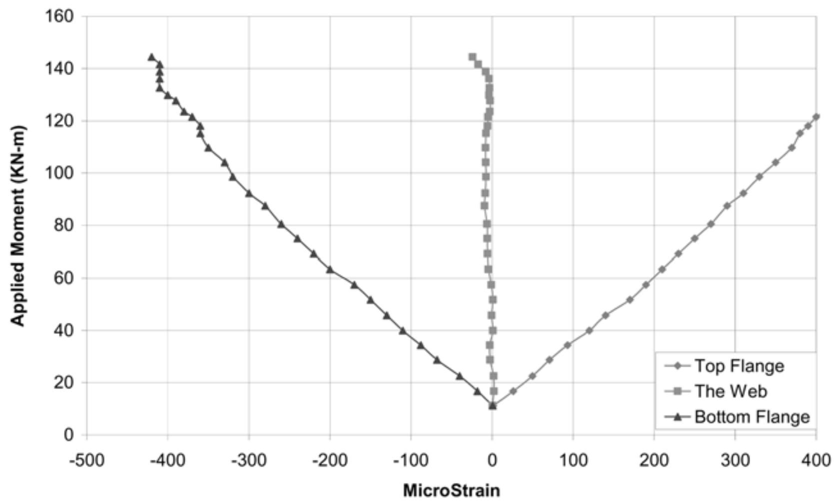


Fig. 20 Applied moment versus beam strains for connection KGS-5S

beam-strain behavior. The column-web strains were measured using a 45-degree rosette strain gauge (G-1) located at the web center between the stiffeners. Results of the applied moment versus column-strain for frame KGS-5S are given in Fig. 21. As indicated, the measured shear strains were much larger than the normal strains. It should be noted that the final deformed shape of all frames tested after removing the load, showed the permanent deformation retained in the structure due to plastic deformations.

6. Conclusions

In the present paper, several points have been investigated concerning stiffened and unstiffened beam-to-column connections such as: tension force in the bolt, extended end-plate behavior, moment-

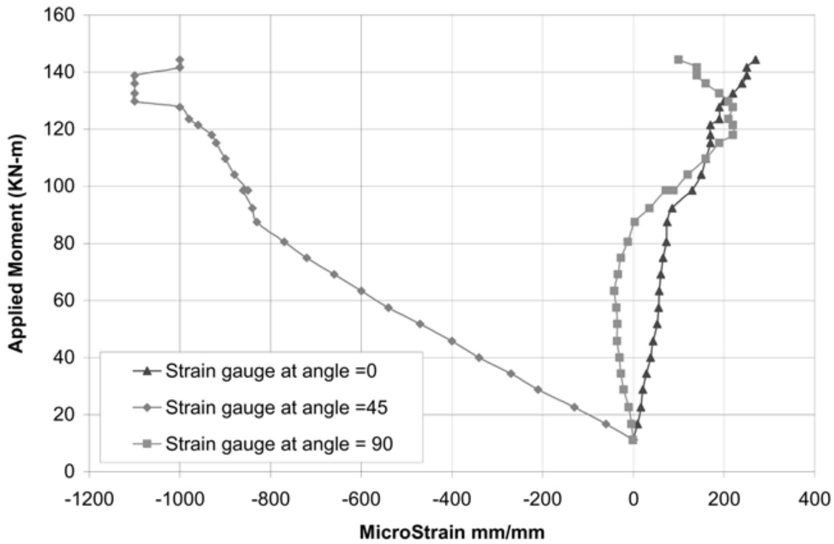


Fig. 21 Applied moment versus column strains for connection KGS-5S

rotation relationship, and beam and column strains. Six full scale joints have been tested and their results are compared with two theoretical approaches. From this study the following conclusions are reached:

- The tension forces in the upper bolts above the beam upper-flange can be reasonably determined using the equal distribution method. For this case, averaged maximum differences of -3.3% and -6.6% for unstiffened and stiffened connections, respectively, were obtained.
- The equal distribution method was found to be unsuitable for the upper bolts below the beam upper-flange, since it overestimated the bolt forces. However the linear distribution method underestimated them.
- The deformation behavior of all tested connections was characterized by separation of the column-flange from the extended end-plate almost down to the upper two bolts of the lower group. Below this point the two parts remained in full contact.
- Connections without stiffeners showed more separation than those with stiffeners and local buckling of the column-flange was observed in all tested frames within the separation (gap) region. This affected the distribution of force among the rows of bolts.
- The neutral axis of the deformed joint may be reasonably assumed to pass very close to the line joining the upper two bolts of the lower group.
- Smooth monotonic moment-rotation relations were observed for the all tested frames.
- The distribution of beam strains indicated almost symmetrical behavior for the top-flange and bottom-flange strains about the mid-depth axis of the beam whereas the web-strain was almost zero. With respect to column-strains, the axial and transverse normal strains were very small compared to the resulting shear strain in the diagonal direction.

Acknowledgements

The research reported in this paper was sponsored and funded by Dean of Research, Research No. 30/2000, Jordan University of Science and Technology, Irbid, Jordan.

References

Abel, M. and Murray, T. M. (1994), "Analytical and experimental investigation of the extended unstiffened moment end-plate connections with four at beam flange", Report No. CE/VPI-ST 93/8. Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

American Institute of Steel Construction. (2005), *AISC/LRFD Steel Construction Manual, 13 Edition.*

Baniotopoulos C. C. (1995), "On the separation problem in bolted steel splice plates", *J. Constr. Steel Res.*, **32**, 15-33.

Baniotopoulos, C. C., Mistakidis, E. S., Bisbos, C. D. and Panagiotopoulos, P. D. (1996), "A 2D numerical model for the analysis T-stub connections", C1/WG6/1996-09. Meeting No 6, Clermont-Ferrand.

British Standards Institution. (2001), *BS 5950: Part 1: Structural use of steelwork in building. London, BSI.*

Bursi, O. S. and Jaspart, J. P. (1997), "Benchmark for finite element modeling of bolted steel connections", *J. Constr. Steel Res.*, Elsevier, **43**(1-3), 17-42.

Chen, W. F. and Kishi, N. (1989), "Semi rigid steel beam-to-column connections: Data base and modeling", *J. Struct. Eng., ASCE,* **115**(1), 105-119.

Coelho, Ana M. Girão, Bijlaard, Frans S. K. and Simões da Silva, Luís (2004), "Experimental assessment of the ductility of extended end-plate connections", *J. Eng. Struct.*, **26**, 1185-1206.

Coelho, Ana M. Girão, Bijlaard, Frans S. K., Gresnigt, Nol and Simões da Silva, Luís (2004), "Experimental assessment of the behavior of bolted T-stub connections made up of welded plates", *J. Constr. Steel Res.*, **60**, 269-311.

European Committee for Standardization (CEN). PrEN 1993; (1-8): 2003, Eurocode 3: Design of steel structures. Part 1.8: Design of joints. Stage 49 draft, Brussels; 2003.

Gebbeken, N., Rothert, H. and Binder, B. (1994), "On the numerical analysis of end-plate connections", *J. Constr. Steel Res.*, Elsevier, **30**, 177-196.

Gomes, F. C., Neves, L. F., Sliva, L. and Simoes, R. (1995), "Numerical simulation of a T-stub", Document C1/WG6/1995-21. Meeting No 5, Graz.

Jaspart, J. P. (1994), "Numerical simulation of T-stub – Experimental data", Document C1/WG6/1994-09. Meeting No 2, London.

Jaspart, J. P. (1997), "Contributions to recent advance in the field of steel joints, column base and further configurations for beam-to-column joints and beam splices", Aggregation Thesis. University of Liege, Liege.

Jaspart, J. P., Mac, B. and Cerfontaine, F. (1999), "Strength of joints subject to combined action of bending moments and axial forces", *Eurosteel 1999- Second European Conference on Steel Structures*, Prague, Cezeh Republic.

Jolly, C. K. (1995), "Numerical simulation of a T-stub connection – Implications for more general simulations", Document C1/WG6/1995-13. Meeting No 5, Graz.

Meng, R. (1996), "Design of moment end-plate connections for seismic loading", Doctoral Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

Mistakidis, E. S., Baniotopoulos, C. C., Bisbos, C. D. and Panagiotopoulos, P. C. (1996), "Steel T-stub connection under static loading: an effective 2-D numerical model", *J. Constr. Steel Res.*, **44**(1-2), 51-67.

Ragupathy, P. and Virdi, K. S. (1995), "Numerical simulation of a T-stub connection using ANSYS", Document C1/WG6/1995-16. Meeting No 4, Trento.

Troup, S., Xiao, R. Y. and Moy, S. S. J. (1998), "Numerical modeling of bolted steel connections", *J. Constr. Steel Res.*, **46**(1-3): Paper No. 362.

Wald, F. and Baniotopoulos, C. H. (1998), "Numerical modeling of column base connection. COST C1, control of the semi-rigid behavior of civil engineering structural connections", *Proceeding of the International Conference Liege, 17-19 Septemper pp 497-506.*

Wald, F., Villanova, F. and Zuhilke, A. (1997), "2D and 3D modelling of a T-stub, the COST C1 example", Document C1/WG6/1997-3 No 7, Munchen.

Zoetemeijer, P. (1990), "Summery of the research on bolted beam-to-column connections", *Report 26-6-90-2.* Delft University of Technology. Faculty of Civil Engineering, Stevin Laboratory – Steel Structures.

CC