EXPERIMENTAL BEHAVIOR OF STEEL BEAM-TO-COLUMN JOINTS: FULLY WELDED VS BOLTED CONNECTIONS

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SUMMARY

A wide experimental program on different types (welded and bolted) of beam-to-column connections has been carried out at the Material and Structures Test Laboratory of the Instituto Superior Técnico of Lisbon The experimental tests have been performed on specimens representative of frame structure beam-to-column joints close to the ones typical of European design practice. In this paper the results obtained from the experimental tests on two alternative connection solutions, namely top and seat with web angle (TSW) and fully welded connections (WW), designed for the same beam-to-column joints are presented. Six experimental tests have been executed on six (three welded and three bolted) different series of specimens, for a total of 36 tests. The test program was planned with the aim of assessing the comparative behaviour of bolted and welded connections, and for defining the effect of the column size and of the PZ design on the behaviour of the two types of connection, varying the applied loading history. In addition the accuracy of the *"component method"* of the Eurocode 3 Annex J in predicting stiffness and strength of the connections is evaluated through the comparison with experimental monotonic results.

INTRODUCTION

Following the Northridge (1994) and Hyogoken-nanbu (1995) earthquakes, the confidence of structural engineers in welded moment resisting connections was strongly compromised due to the extensive brittle damage detected in several frames. Staring form these observations, a great deal of theoretical and experimental research activity is presently being developed in USA, Japan and Europe on the cyclic behaviour of both welded and alternative configurations of beam-to-column connections. Since recently bolted connections, in particular top and seat with web angles connections, have not been considered appropriate in seismic applications, due to the partial strength and semirigidity characteristics. However, as pointed out by several researchers among which (Astaneh, 1995; Elnashai et Al., 1998), the dynamic behaviour semirigid frames can be particularly favourable due to the period elongation, related to the connection flexibility and to the damping increase, related to highly dissipative friction mechanism deriving from a proper "slip capacity design". Both these effects act as a sort of self-isolation of the frame structure, thus leading to remarkable reduction of the seismic actions.

It is worth to emphasise that also in the context of the SAC Steel Project, started immediately after the Northridge earthquake to address the specific problem of beam-to-column connections, a great interest in bolted configurations as alternative to the standard welded connections (Roeder, 1998), can be found.

In this research framework, a wide experimental program on different types (welded and bolted) of beam-to-column connections has been carried out at the Material and Structures Test Laboratory of the Instituto Superior Técnico of Lisbon. The experimental tests have been performed on specimens representative of frame structure beam-to-column joints close to the ones typical of European design practice. Some preliminary experimental results on the welded connections have been presented in (Mele et Al., 1997, 1999; Calado et Al., 1999). In this

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paper a comparative assessment of the cyclic behaviour of welded and bolted beam-to-column connections is provided.

OBJECT AND AIMS OF THE PAPER

In this paper the results obtained from the experimental tests on two alternative connection solutions, namely top and seat with web angle (TSW) and fully welded connections (WW), designed for the same beam-to-column joints are presented. Six experimental tests have been executed on six (three welded and three bolted) different series of specimens, for a total of 36 tests. The test program was planned with the aim of assessing the comparative behaviour of bolted and welded connections, and for defining the effect of the column size and of the PZ design on the behaviour of the two types of connection, varying the applied loading history. In addition the accuracy of the *"component method"* of the Eurocode 3 Annex J (CEN, 1997) in predicting stiffness and strength of the connections is evaluated through the comparison with experimental monotonic results, and the possibility of extrapolating such theoretical prediction to the cyclic range is examined.

EXPERIMENTAL PROGRAM

The specimens

Two series of full scale specimens have been designed and tested, namely a WW specimen series (BCC5, BCC6 and BCC8) and a TSW specimen series (BCC9, BCC7 and BCC10). The specimens of the two series are T-shaped beam-column subassemblages, consisting of a 1000 mm long beam and a 1800 mm long column. The material used for the columns, beams, and angles is steel S235 JR, with nominal values of yield and ultimate stress respectively equal to f_y=235 MPa and f_u=360 MPa. In each series the cross section of the beam is the same (IPE300), while the column cross section has been varied, being respectively HE160B for the BCC5 (WW) and BCC9 (TSW) specimens, HE200B for the BCC6 (WW) and BCC7 (TSW) specimens, and HE240B for the BCC8 (WW) and BCC10 (TSW) specimens. In both the series, the continuity of the connection through the column has been ensured by horizontal 10 mm thick plate stiffeners, fillet welded to the column web and flanges.

WW specimens

In the WW specimens, the beam flanges have been connected to the column flange by means of complete joint penetration (CJP) groove welds, while fillet welds have been applied between both sides of the beam web and the column flange. The flexural strengths of the beam, column and panel zone, have been computed on the basis of the nominal and actual yield stress and are reported in table 1.

Table 1: moment capacities (in kNm) of the WW specimens (nominal and actual values)

		M_{pb}	M_{pc}	$M_{p,PZ}$
BCC5	Nominal values	147.6	83.2	91.1
	Actual values	173	114	149.8
BCC6	Nominal values	147.6	151.1	132.4
	Actual values	175	201	220.7
BCC8	Nominal values	147.6	247.5	182.9
	Actual values	183	316	239.7

From the simple comparison among the nominal plastic moments reported in table 1, it can be observed that in the three WW specimens the weakest component of the joint configuration is respectively: the column for the BCC5 specimen, the panel zone for the BCC6 specimen, the beam for the BCC8 specimen.

TSW specimens

In the BCC9, BCC7 and BCC10 (TSW) specimens, 120x120x10 angles have been adopted. Two rows of bolts are placed on each leg of the flange angles, while on the legs of the web angles there is only one row of two bolts. The bolts are M16 grade 8.8 (yield stress f_{yb}=640 MPa, ultimate stress f_{ub}=800 MPa, A_s=157 mm^2), preloaded according to the EC3 provisions, i.e. at $F_{P,CD}$= 0.7 f_{ub} A_s = 87.9 kN.

It is well known that two major phenomena characterise the behaviour of the TSW connection: the slippage of bolts and the yielding of the tension angle. For the TSW specimens herein described the bending moment corresponding to bolt slippage and angle yielding are reported in table 2, together with the beam and column moment capacities. From the comparison between the bending moments corresponding to bolt slippage and angle yielding, it derives that the specimens are "slip critical" connections, since slippage of top and seat angle bolts occurs at a load level higher than the one corresponding to yielding of the tension angle.

Table 2: threshold moment capacities (in kNm) of the TSW specimen (nominal and actual values)

		M_{slip}	$M_{y,angle}$	M_{pb}	M_{pc}
BCC9	Nominal values	32 - 47.5	23.3	147.6	83.2
	Actual values		28.1	198.2	123.4
BCC7	Nominal values	32 - 47.5	23.3	147.6	151.1
	Actual values		28.1	198.2	194.4
BCC10	Nominal values	32 - 47.5	23.3	147.6	247.5
	Actual values		28.1	198.2	305.7

Loading histories

For each of the six specimen series, the experimental program consisted of six tests, one monotonic test and five cyclic tests, for a total of 36 tests. The cyclic tests have been carried out by applying both constant and step-wise increasing amplitude displacement histories. This latter test type has been carried out according to the basic loading history recommended in (ECCS, 1986). In this paper only the increasing rotation tests is reported.

Experimental set-up and specimen instrumentation

The test set-up, mainly consists in a foundation, a supporting girder, a reaction r.c. wall, a power jackscrew and a lateral frame. The power jackscrew (capacity 1000 kN, stroke ± 400mm) is attached to a specific frame, pre-stressed against the reaction wall and designed to accommodate the screw backward movement. The specimen is connected to the supporting girder through two steel elements. The supporting girder is fastened to the reaction wall and to the foundation by means of pre-stressed bars. An automatic testing technique was developed to allow computerised control of the power jackscrew, of the displacement and of all the transducers used to monitor the specimens during the testing process. Specimens have been instrumented with electrical displacement transducers (LVDTs), which record the displacement histories at several points in order to obtain a careful documentation of the various phenomena occurring during the tests. The same arrangement of LVDTs has been adopted for the three WW and the three TSW specimen series.

CYCLIC TESTS

Premise

In the following the experimental results obtained in the test program are provided. In particular the cyclic behaviour and the failure modes observed for the six sets of specimens are described, and the moment rotation hysteresis loops obtained in the increasing amplitude tests are provided. In the moment rotation hysteresis loops hereafter presented, reference is made to three different values of rotation, namely: (1) the "*unprocessed*" total rotation given by the applied interstory drift angle d/H; (2) the beam rotation Φ_b and (3) the panel zone rotation Φ_{PZ}, both obtained through the measured LVTDs displacements of the specimens. Correspondingly, in the M-d/H and M-Φ_{PZ} experimental curves the moment is evaluated at the column centreline, while in the M_b-Φ_b curves the moment is evaluated at the column face.

WW specimens

In figure 1 (a) the moment - total rotation (M-d/H) experimental curves resulting from the BCC5C, BCC6C and BCC8D tests (cyclic increasing stepwise amplitude) are plotted, while in figure 1 (b) both the corresponding moment – beam plastic rotation and the moment - panel zone rotation curves are plotted. The beam plastic rotation has been obtained through the measured displacements at the beam instrumented section by subtracting the contributions of the beam and column elastic rotations as well as of the panel zone distortion.

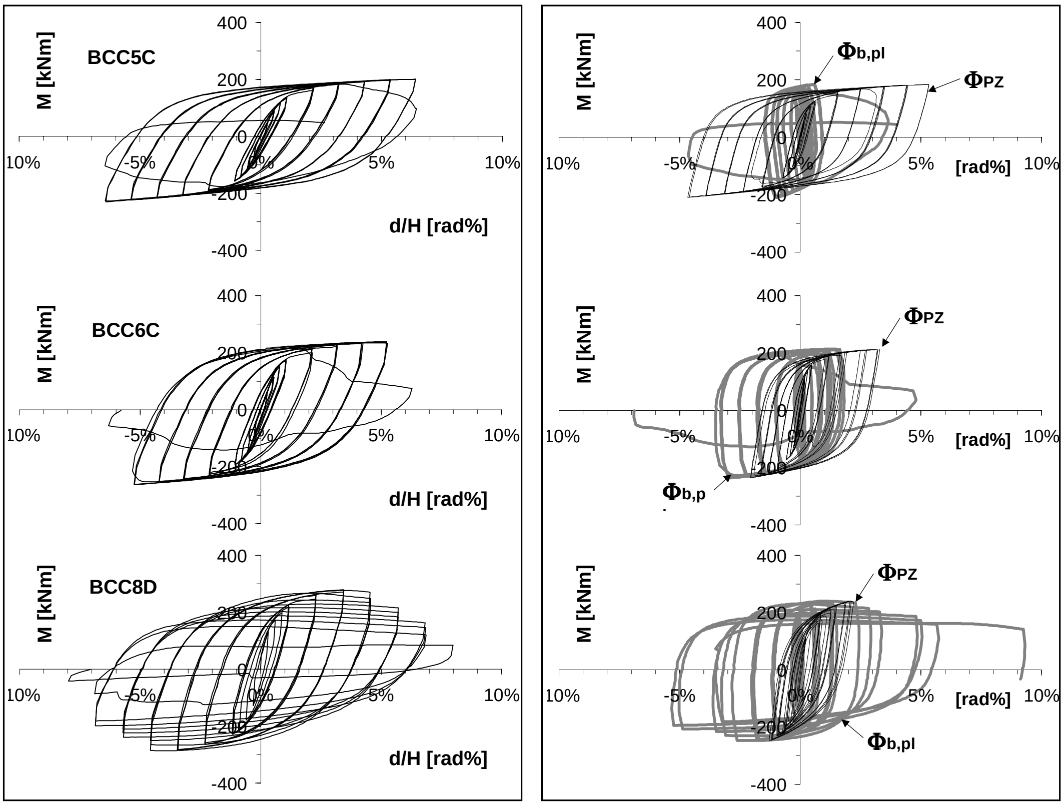

Figure 1 WW specimens: (a) moment-global rotation curves (b) moment-beam plastic rotation and moment-panel rotation curves

BCC5

As can be derived from the curves reported in figure 1 (a) and (b), and as demonstrated also throughout the experimental program, the cyclic behaviour of the specimen BCC5 is characterised by a great regularity and stability of the hysteresis loops up to failure, with no deterioration of stiffness and strength properties. In the very last cycle the specimen has collapsed with a sudden and sharp reduction of strength, due to fracture initiated in the beam flange and propagated also in the web. During the tests, significant distortion of the joint panel zone has been observed, while not remarkable plastic deformation in the beam occurred.

BCC6

Throughout the test program, two different kinds of cyclic behaviour have been observed for the BCC6 specimens. In some cases the behaviour of the specimens is close to the behaviour observed for the BCC5 type, with almost no deterioration of the mechanical properties up to the last cycle, during which the collapse occurred. For the other tests a gradual reduction of the peak moment at increasing number of cycles is evident. In these cases, starting from the very first plastic cycles, local buckling of the beam flanges occurred, and a well defined plastic hinge has formed in the beam. The contribution of the panel zone deformation has not been as significant as in the BCC5 specimen type.

BCC8

The hysteresis loops obtained from the tests on the BCC8 specimens show a gradual reduction of the peak moment starting from the second cycle, where the maximum value of the applied moment has been usually registered. This deterioration of the flexural strength of the connection is related to occurrence and spreading of local buckling in the beam flanges and web. A well defined plastic hinge in the beam has formed in all the tested specimens. In the specimens BCC8 the panel zone deformation has not been remarkable, and the plastic deformation mainly took place in the beam.

TSW specimens

In figure 2 (a) the moment - total rotation (M-d/H) experimental curves resulting from the BCC9D, BCC7C and BCC10C tests (cyclic increasing stepwise amplitude) are plotted, while in figure 2 (b) both the corresponding moment – beam rotation and the moment - panel zone rotation curves are plotted. As can be derived from the curves reported in figure 2, the shape of hysteresis loops of the three TSW specimens is very similar. The cyclic behaviour, the phenomena observed during the tests and the collapse modes are the same for the three specimen series, thus the following unique paragraph is devoted to describe the above issues for the three specimens.

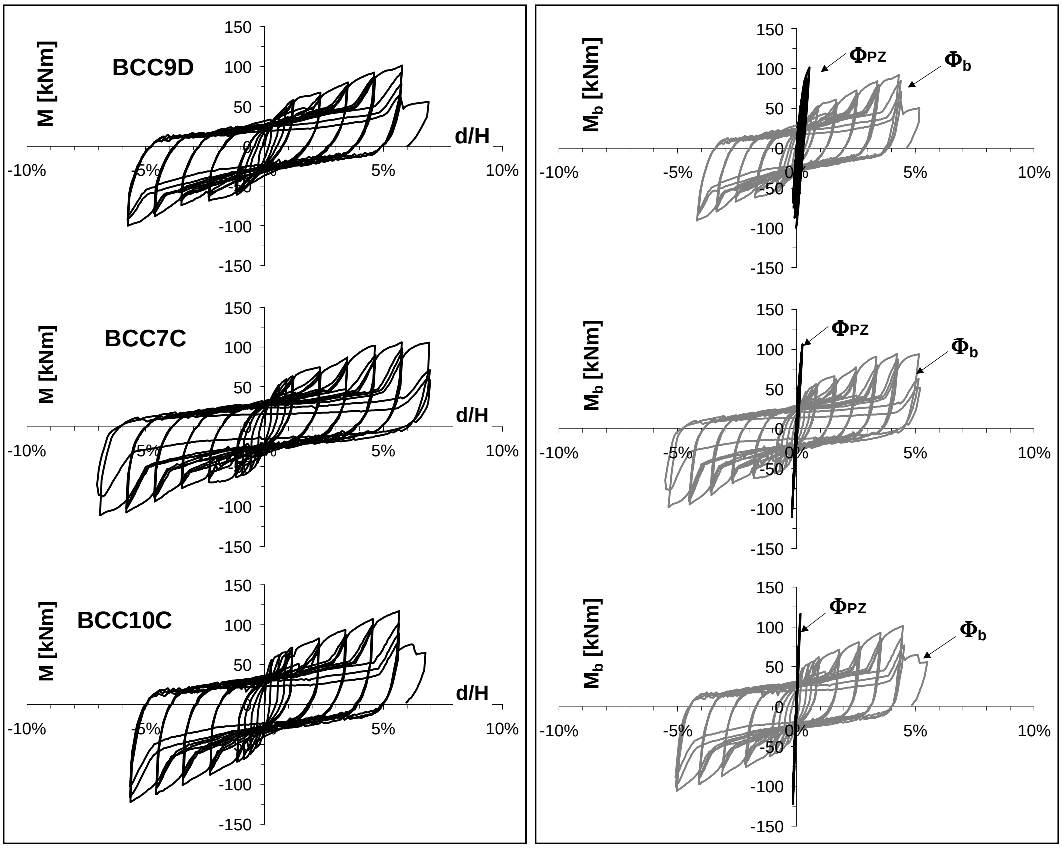

Figure 2 TSW specimens: (a) moment-global rotation curves, (b) moment-beam net rotation and moment - panel rotation curves

BCC9 / BCC7 / BCC10

The cyclic behaviour of the TSW connections is characterised by bolt slippage and yielding and spreading of plastic deformation in the top and bottom angles, cyclically subjected to tension. Plastic ovalization of the bolt holes have also been observed mainly in the leg of the angle adjacent to column flange. The experimental curves, typical of this type of connection, shows pinched hysteresis loops, with a large slip plateau (very low slope of the experimental curve) and subsequent sudden stiffening. In fact when the specimen position is at $d = 0$, due to the concomitant effects of bolt slippage, hole ovalization and the plastic deformation of the angle legs adjacent to the column flange, the beam is completely separated from the column (gap open). At large applied displacements, which impose large rotations to the connection, the contact of the compression angle and the beam web to the column flange (gap closure) give rise to sudden stiffening of the connection, which is evident in the experimental curves. No significant rotation of the column and distortion of the panel zone has been observed throughout the experimental tests carried out on the three specimens. At each step on the test, slight deterioration of the joint resistance in the three applied cycles can be observed in the experimental curves, mainly due to yielding and spreading of plastic deformation in the top and bottom angles, cyclically subjected to tension. In all the test carried out on the three specimen series, the collapse of the connection occurred due to fracture in the leg angle located on the beam flange, immediately after the fillet. Negligible scatters can be observed in the moment capacity of the three connection series, as it is expected, since the inelastic behaviour of the connection is governed by the angle. Also the maximum values of global rotation experienced by the specimens is the same for the BCC9 and BCC10 series, and slightly larger for the BCC7 one.

MONOTONIC TESTS

The moment rotation curves obtained from the monotonic tests carried out on the six specimens are presented in figures 3 (a) and (b) which respectively report the results of the WW and TSW specimens. In these curves the moment is evaluated at column centreline and the rotation is given by the total interstory drift angle (*d/H*). In each figure also the moment panel zone rotation are reported. From the experimental results on the two series of specimen the effect both of the connection typology (TSW and WW) and of the column cross section (HE160B, HE200B, HE240B) can be derived.

By comparing the two series of experimental curves it must be noticed that the three WW specimens show significant differences in the initial stiffness, maximum strength and deformation capacity, thus confirming the strong effect of the column cross section size already observed in the cyclic tests. On the contrary, the three TSW specimens present quite close experimental responses. This difference between the behaviour of WW and TSW specimens is mainly related to the design of the specimens, since in the TSW connections the weakest component is the same in the three specimens (the angle in tension), thus the beam, column and panel zone strength ratios does not affect the response of the specimens. Slight scatters can be observed in the initial stiffness, due to the different column and panel zone deformability, but the nonlinear portion of the curve and the maximum bending moment are very similar.

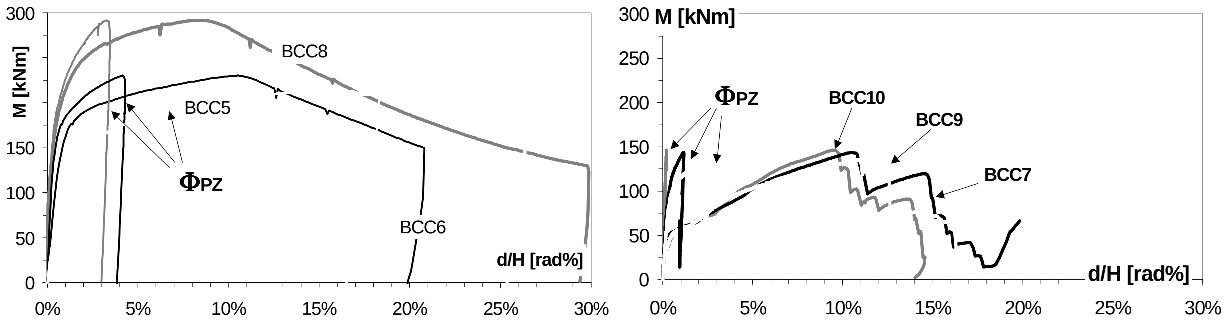

Figure 3 Monotonic experimental curves: (a) WW specimens; (b) TSW specimens

As already evidenced, the behaviour of the WW connections is affected by the column dimensions since the three combinations of beam and column framing in the joint give rise to panel zone strength values respectively: smaller than, approximately equal to and larger than the plastic moment of the beam, for the BCC5, BCC6 and BCC8 specimens. These observations are confirmed by analysing the main test data provided in table 3.

Table 3: main experimental data from the monotonic tests on the WW and TSW specimens

		M [kNm]	M_b [kNm]	d/H [%]	Φ_{PZ} [%]	$\Phi_{b,pl}$ - Φ_{PZ} [%]
WW	BCC5	214	196	21.4	16.5	0.66
	BCC6	231	207	10.5	4.5	4.9
	BCC8	292	251	8.3	3.3	5.3
		M [kNm]	**M_b [kNm]**	**d/H [%]**	**Φ_{PZ} [%]**	**Φ_{slip} [%]**
TSW	BCC9	135	122.5	10.8	2.4	5.2
	BCC7	144	127	10.5	1.15	6.8
	BCC10	146	126	9.4	0.19	4.8

In particular it can be noticed that while in the WW specimens the panel zone distortion Φ_{PZ} significantly contributes to the specimen global rotation d/H (at the maximum value of the bending moment registered in the relevant test), though at different extent in the three specimens, a completely different order of magnitude of this contribution is registered in the TSW connections. For the TSW specimens, instead, the rotations due to bolt slippage Φ_{slip} computed on the basis of the LVDTs measured displacements constitutes a major contribution to the total rotation d/H.

PREDICTION OF THE MONOTONIC BEHAVIOUR

Comparing the monotonic and the cyclic curves both for the BCC5 WW specimen, and for the BCC9 TSW specimen, it can be observed that the monotonic curve perfectly envelope the cyclic one. Thus it seems of primary importance assessing the prediction capacity of the monotonic behaviour offered by the available numerical models.

In figures 4 and 5 the monotonic curves, respectively of the BCC5 and BCC9 specimens, are compared to theoretical moment-rotation curves. In figure 4 (a) and (b) the reference experimental curves report the moment at the column centreline, M, vs the different rotation values, i.e.: the global rotation d/H, the beam and the PZ rotations, Φ_b and Φ_{PZ}, obtained from the LVDT measured displacements, and depurated of: rigid rotation of the specimen supports, elastic rotations of the beam and column. The theoretical curve is the one obtained through the application of the "component method" as outlined in the EC3 Annex J. In the figure 5 (a) and (b) the reference experimental curve reports the moment at the column centreline, M, vs the global rotation d/H. The theoretical curves obtained through: the application of the EC3 Annex J (CEN, 1997) procedure, the model proposed by (Kishi & Chen, 1990) and the simplified approach proposed by (De Luca et Al., 1995) are also shown in the figure. Both the figure 4 (b) and 5 (b) provide the detail of the curves up to 1% rotation.

WW BCC5 specimen

In the case of welded connections stiffened by means of continuity plates as the ones described in this paper, the Annex J procedure suggests to account for only one component in the evaluation of the rotational stiffness of the joint, i.e. the column web panel in shear, while the flexural strength of the joint is given by the resistance, multiplied by the lever arm, of the weakest among five basic components, namely the panel in shear, the column web in tension and in compression, the column flange in bending and the beam flange and web in compression. For the BCC5 specimen the panel shear resistance resulted the minimum one, thus confirming the governing role of the PZ in the deformation and failure modes of the BCC5 specimen observed in the tests. The moment rotation curve obtained through the Annex J procedure, improved as suggested in (Faella et Al., 1995), shows a reasonable accuracy both in terms of strength (figure 4 (a)) and in terms of initial stiffness (figure 4 (b)). By observing that the monotonic curve of the BCC5 specimen envelopes the experimental curves obtained under cyclic loading, it seems that a reliable prediction of the joint monotonic behaviour is of major importance also for assessing the cyclic performance.

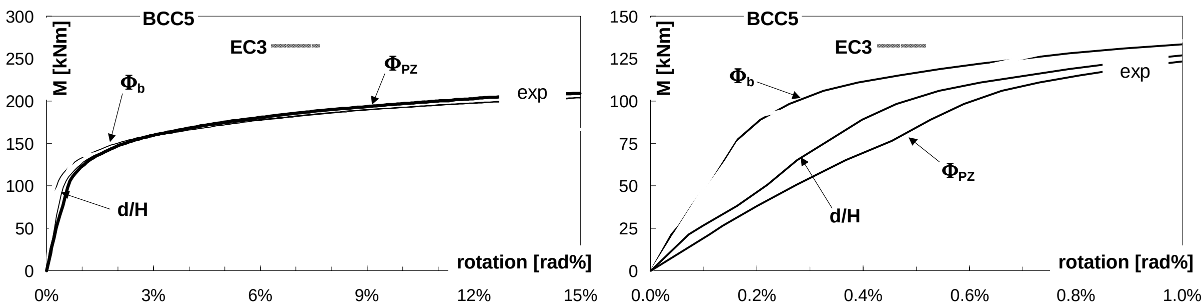

Figure 4 experimental vs theoretical curves: WW BCC5 specimen

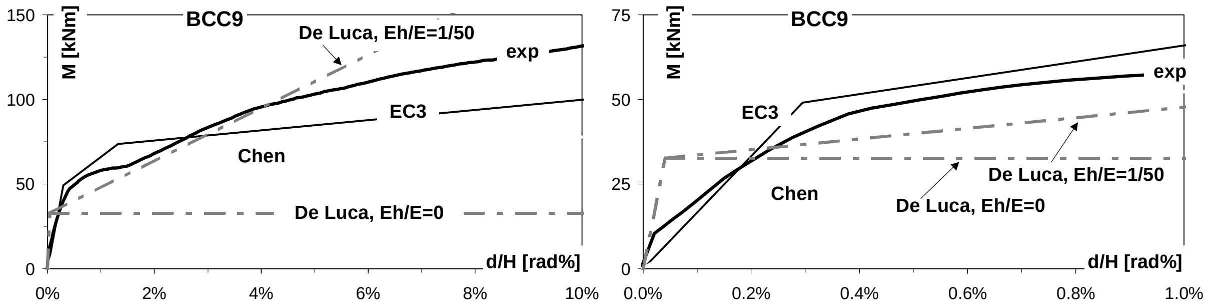

Figure 5 experimental vs theoretical curves: TSW BCC9 specimen

TSW BCC9 specimen

In the experimental curve of figure 5 (a) four major branches can be identified: an initial linear branch; a second, remarkably nonlinear branch, going approximately from to 35 to 55 kNm, where the both the slip of bolts and yielding of the angle occur; a third sensibly linear branch which arrives at 85 kN, characterised by a reduced slope, where hardening of the angle occurs up to the failure of the first row of bolts; and a final, always approximately linear branch but with a reduced slope, where the angle, also due to geometry variation related to the loss of a bolt row, is further stretched and carry the load in simple tension. It is worth to notice that the ratio of the slope of the third to the first branch of the curve is close to the ratio E_h/E derived from the tension tests on the angle coupon, while the maximum bending moment registered in the test at the column face is close to the moment corresponding to the pure tensile bearing capacity of the angle. Similar considerations still hold by observing the other TSW experimental curves.

The prediction models do not represent all these phenomena, since the ultimate bearing capacity developed by the connection is due to a deformation state which is not compatible with the global stability of the structure. Therefore the purpose of the models is the prediction of the initial stiffness, the plastic threshold and the consequent nonlinear branch up to a given deformation value, which should be compatible with the structure and checked against the deformation capacity of the connection. In this perspective, the EC3 model, applied with the improvements suggested by (Faella et Al., 1996), matches quite closely the initial stiffness, slightly overstimates the plastic threshold given by 2/3 of the resistance M_{jRd}, and provides a reasonable value of M_{jRd}. In addition, by considering a hardening slope equal to 1/55 and accounting for a strength ratio f_u/f_y=1.48, as derived from the angle coupon tests, also a portion of the fourth branch can be match. The model of Kishi and Chen also leads to a good estimation of strength, but does not match the initial stiffness, since it does not allow for bolt preloading. The simplified approach of (De Luca et Al., 1995) provides a range which includes the experimental curve. A deeper assessment of the prediction capacity of the different models is not possible here due to space limitations.

SUMMARY AND CONCLUSIVE REMARKS

The experimental results obtained in this research allow to define the collapse modes, the rotation capacity and the ultimate bending strength of bolted and welded beam-to-column connections. In this paper the major aspects governing the cyclic and the monotonic behaviour of bolted (TSW) and welded (WW) connections have been evidenced against experimental results. It has been shown that the panel zone does not affect the behaviour of the TSW connections, which instead is mainly related to the tension angle geometry and strength properties. On the contrary the panel zone has demonstrated to affect at large extent all the response parameters (stiffness, strength and deformation capacity) of welded connections. Finally the application of the EC3 Annex J model (with some modifications suggested by other Authors) to two single examples of each connection typology, has shown to provide reasonable results.

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