STRENGTH AND DUCTILITY OF BOLTED T-STUB MACRO-COMPONENTS UNDER MONOTONIC AND CYCLIC LOADING

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Abstract. *In dissipative building frames, Mild Carbon Steel (MCS) is always used in members designed to undergo plastic deformations (e.g. beam and/or braces), while in truss members which have to remain predominantly elastic, such as columns, High Strength Steel (HSS) grade stub might be used [3]. In such a case within Moment Resisting (MR) Joints, T-stub macro-components made of two steel grades are obtained. Due to the fact, usually real yield strength in MCS beams is significantly higher than nominal value used in design, the T-stub macro-components, together with the column web panel become very important for strength and ductility of joints. However, web panel contribution is limited by EN 1998-1 at 30% from total plastic rotation capacity and, consequently, the main ductility demand goes for T-stub component. Bolted extended-end-plate full strength MR joints cannot be usually obtained without outer stiffeners or/and haunches. In such case it is difficult to obtain ductile failure modes for the relevant T-stub components. In order to observe and characterize this phenomenon, an extensive testing and numerical simulation program was undertaken at CEMSIG Centre. Present paper summarizes the main results of this investigation.*

1 INTRODUCTION

Seismic resistant building frames designed as dissipative structures must allow for plastic deformations to develop in specific members, whose behavior has to be predicted by proper design. Members designed to remain predominantly elastic during earthquake, such as columns, are responsible for robustness of the structure and prevention the collapse, being characterized by high strength demands. Consequently a framing solution obtained by combining High Strength Steel - HSS in non-dissipative members as columns provided with adequate over-strength, and Mild Carbon Steel – MCS in dissipative members, working as fuses, as beams, links or braces seems to be logical. The robustness of structures to severe seismic action is ensured by their global performance, in terms of ductility, stiffness and strength, e.g. the "plastic" members of MCS – (S235 to S355) will dissipate the seismic energy, while the "elastic" members (HSS - S460 to S690) by higher resistance of material and appropriate size of sections, will have the capacity to carry the supplementary stresses, following the redistribution of forces, after appearance of plastic hinges. Such a structure is termed Dual-Steels Structure – DS.

DS concept is extended to connections, too, on the same philosophy related to ductile and brittle components, in order to achieve both ductility and robustness criteria. In fact, when connect MCS beams to HSS columns will result a DS beam-to-column joint.

Starting from the above considerations, a large experimental research program was carried out at the "Politehnica" University of Timisoara, CEMSIG Research Centre (http://cemsig.ct.upt.ro) in order to study the performance of dual-steel configuration for beam-to-column joints under monotonic and cyclic loading. Joint specimens, T- stub and weld detail specimens have been tested [5], [6, [7].

When HSS is used in members designed to remain predominantly elastic, as columns or in end-plates of bolted joints, T-stub macro-components made of two steel grades are obtained. The performances of DS bolted T-stub specimens as strength and ductility under monotonic and cyclic loading are analyzed in present paper. Similar tests on MCS and DS bolted T-stubs, unstiffened and one-side stiffened were realized by [8], under monotonic loading and stiffener on the end-plate, and by [9], which applied cyclic loading on MCS unstiffened T-stubs.

According to seismic design provisions [2], Moment Resisting Frames (MRF) comprise full strength/rigid joints, which are demanding a minimum plastic rotation capacity ϕ_{pl}=0.035rad, and the overstrength of moment capacity of the joint of, at least 1.375 times the plastic bending moment of the beam, for partial resistant/semi-rigid joints the plastic rotation capacity $\phi_{pl}>\phi_{pl,necessary}$.

It is well-known that T-stub macro-component is falling down by 3 types of failure mode, named 1, 2 and 3 (see Table 4). After developing the experimental program and starting from previous considerations it was clear that failure mode 2 would be preferable in order to answer both criteria full strength and rotation capacity. Present paper is summarizing the results of this research, where starting from experimental results, authors are developing, starting from real joint configurations, some numerical studies in order to establish the borders for T-stub macro-component failure mode 2→1 and 2→3, and to verify their classification and behavior in between; after that we are going back to the joints to verify also their classification and behavior as failure mode in connection with the T-stub.

From the experimental program, a FEM model was settled for T-stub macro-component. Authors had the idea to start from some real rigid full-resistant joints, to settle the dimensions and steel grade of end plate in order to obtain the borders of type 2 failure mechanism, to make a numerical analysis on extracted T-stubs and compare the results with the theoretical ones and finally to come back to the joints and verify their behavior and failure mode.

2 TESTING PROGRAM

2.1 Summary of testing program

The objective of the whole experimental program was to study the performance of welded and bolted end-plate beam to column joints realized from two different steel grades. The experimental program consisted in tests on materials, welded components, T-stub components, and beam to column joints. This paragraph describes only the investigations performed and T-stub components. Previous papers by the same authors already summarized the results on materials, welded components, weld details and beam-to-column joints [5], [6], [7].

T-stubs are basic components of the component method used in EN 1993-1.8 [1] for evaluation of strength and stiffness of bolted end-plate beam to column joints. Both monotonic and alternating cyclic tests were performed on T-stub components obtained by welding S235 web plates to S235, S460 and S690 end-plates, using K beveled full-penetration welds (Table 1). MAG welding was used, with G3Si1 (EN 440) electrodes for S235 to S235 welds, and ER 100S-G/AWS A5.28 (LNM Moniva) for S235 to S460 and S690 welds. T-stubs were connected using M20 gr. 8.8 bolts. EN 1993-1.8 was used to obtain the design strength of T-stubs and failure modes. Thickness of end-plates was determined so that the unstiffened T-stub (type C) would fail in mode 1 (end-plate) and mode 2 (combined failure through end-plate bending and bolt fracture). The same end-plate thickness was then used for the stiffened T-stubs (type B and A), see Table 1.

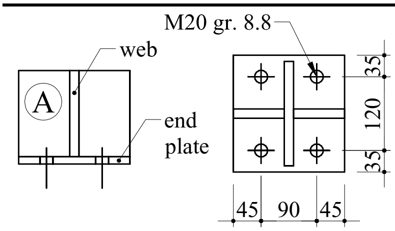
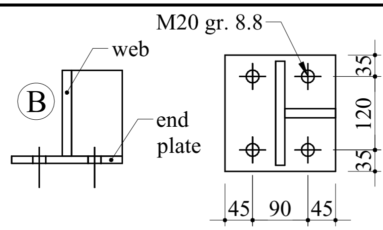
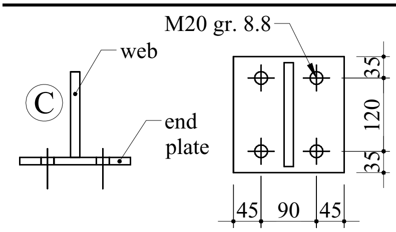
2.2 Dual-steel bolted T-stubs: monotonic and cyclic loading performance

Flat materials used for T-stub and welds details were supplied by UNIONOCEL, Czech Republic. Table 2 shows the measured average values of yield stress f_y, tensile strength f_u and elongation at rupture A. It has to be recognized that the value of elongation for S460 is surprisingly large. Bolts were tested in tension as well, showing an average ultimate strength of 862.6 N/mm^2.

Loading was applied in displacement control under tension and force control under compression. Compressive force was chosen so as to prevent buckling of the specimen.

For specimens of types B and C, it was not possible to have full reversible cycles due to the buckling. A good ductility was observed, in general; however, thicker end-plate specimens, even of S235, do not show the best ductility. It seems that the choice of thickness associated with steel grade is important in the conception of a proper connection, in order to obtain a good balance between strength, stiffness and ductility of components.

Table 1: T-stub characteristics

T-stub type	Label	Web	End-plate	Design failure mode
	TST-12A-S235	S235 t=15 mm	S235 t = 12 mm	2
	TST-20A-S235		S235 t = 20 mm	2 → 3
	TST-10A-S460		S460 t = 10 mm	2
	TST-16A-S460		S460 t = 16 mm	2 → 3
	TST-8A-S690		S690 t = 8 mm	2
	TST-12A-S690		S690 t = 12 mm	2 → 3
	TST-12B-S235	S235 t=15 mm	S235 t = 12 mm	1 / 2
	TST-20B-S235		S235 t = 20 mm	2 / 2 → 3
	TST-10B-S460		S460 t = 10 mm	1 / 2
	TST-16B-S460		S460 t = 16 mm	2 / 2 → 3
	TST-8B-S690		S690 t = 8 mm	1 / 2
	TST-12B-S690		S690 t = 12 mm	2 / 2 → 3
	TST-12C-S235	S235 t=15 mm	S235 t = 12 mm	1
	TST-20C-S235		S235 t = 20 mm	2
	TST-10C-S460		S460 t = 10 mm	1
	TST-16C-S460		S460 t = 16 mm	2
	TST-8C-S690		S690 t = 8 mm	1
	TST-12C-S690		S690 t = 12 mm	2

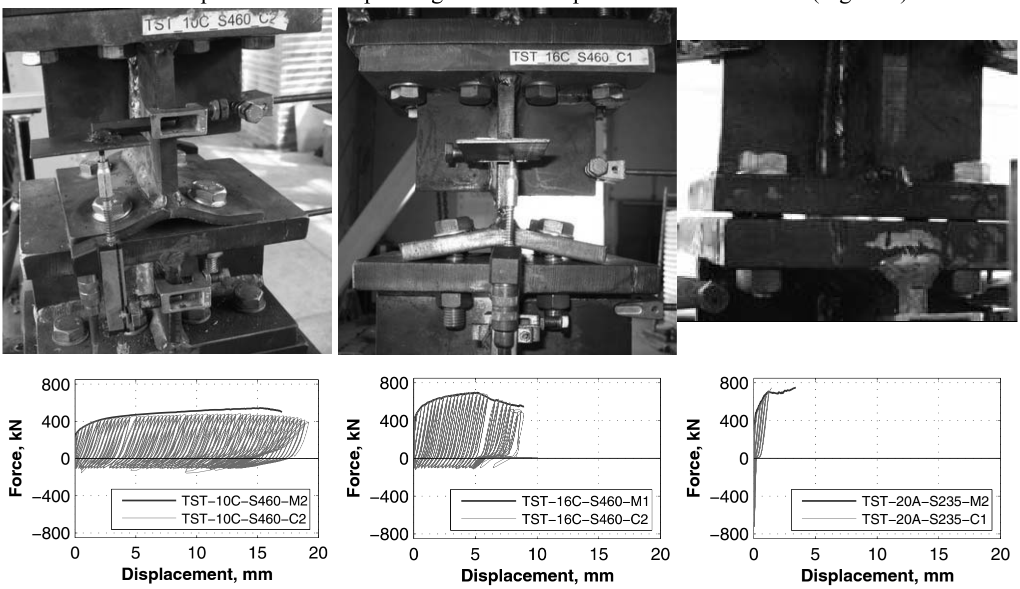
Note 1: One monotonic and two cyclic tests have been performed for each specimen type.
Note 2: Design failure mode 1 / 2 means failure mode 1 for unstiffened part and failure mode 2 for the stiffened one.

Table 2: Material properties

Nominal steel grade	f_y, N/mm^2	f_u, N/mm^2	A, %	Actual steel grade
S235	266	414	38	S235
S460	458	545	25	S460
S690	831	859	13	S690

Figure 1 shows examples with the 3 types of observed failure modes, together with the corresponding force-displacement relationships of T-stub specimens. There were no significant differences in force values between failure modes of monotonic and cyclic specimens, both generally agreeing with analytical predictions by EN 1993-1.8.

For the T-stub specimens, the following parameters were determined for each experimental test: initial stiffness K_{ini}, maximum force F_{max}, yield force F_y, and ultimate deformation, D_y. The initial stiffness was obtained by fitting a linear polynomial to the force-displacement curve between 0 and 25% of the maximum force. The yield force was determined at the intersection of the initial stiffness and tangent stiffness line, where the tangent stiffness was obtained by fitting a linear polynomial to force-displacement curve between 75% and 100% of the maximum force. The ultimate deformation was determined as the displacement corresponding to a 10% drop of the maximum force (Figure 2).



TST-10C-S460-C2 Failure mode 1 | TST-16C-S460-C2 Failure mode 2 | TST-20A-S235-C1 Failure mode 3

Figure 1: Examples of failure modes of T-stub specimens

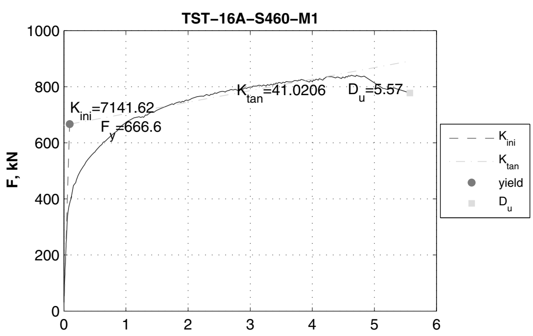


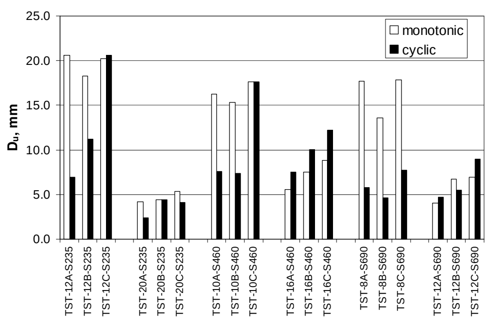
Figure 2: Experimental characteristics of T-stub specimens

Cyclic loading reduced the maximum force of the T-stub specimens, though the reduction was not significant. The ductility of the T-stub specimens was quantified through the ultimate displacement D_u. Under monotonic loading, ultimate displacement was smaller for specimens with thicker end-plates that failed in modes 2 and 3 involving bolt failure (Figure 3a). Cyclic loading reduced significantly ultimate displacement of specimens with thinner end-plates that failed in mode 1. This behavior is attributed to low-cycle fatigue that generated cracks in the HAZ near the welds, along yield lines. On the other hand, cyclic loading did not affect much ultimate displacement for specimens with thicker end-plates that failed in modes 2 and 3, governed by bolt response. It is interesting to note that specimens realized from high-strength end plates (S460 and S690, with lower elongation at rupture), had a ductility comparable with the one of specimens realized from mild carbon steel (S235). The parameters governing the ductility of T-stubs were type of loading (mono-tonic / cyclic) and failure mode (end-plate or bolts).

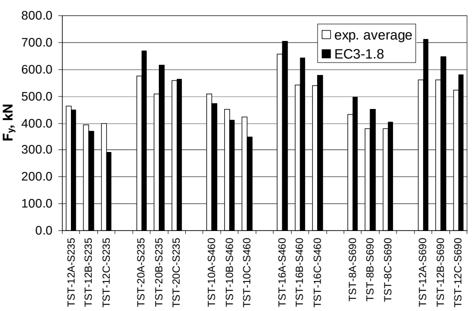
A comparison between test and theoretical results was made (Table 3 and Figure 3). Theoretical characteristics were evaluated by component method from EN1993-1-8. It may be remarked that, with some exceptions, the procedure from EN1993-1-8, including specimens of type A is confirmed; the exceptions can be covered by safety coefficients.

Table 3: Interpretation of monotonic tests

Specimen	$F_{y,exp,average}$ [kN]	$F_{yEC3-1.8}$ [kN]	$F_{y,EC3}/F_{y,exp}$	$F_{max,exp}$ [kN]	$D_{u,exp}$ [mm]	$K_{ini,exp}$ [kN/mm^2]	Failure mode
TST-12A-S235	463.9	425.4	0.92	705.6	20.6	4709.4	1
TST-12B-S235	395.0	357.8	0.91	559.0	18.3	4097.9	1
TST-12C-S235	397.8	290.3	0.73	582.6	20.2	4352.2	1
TST-20A-S235	576.4	645.6	1.12	760.8	4.2	5312.4	3
TST-20B-S235	509.0	589.1	1.16	744.2	9.0	5561.8	2→3
TST-20C-S235	559.5	532.6	0.95	758.3	5.4	6737.8	2
TST-10A-S460	508.3	440.9	0.87	688.7	16.2	3703.6	1
TST-10B-S460	451.7	383.8	0.85	606.4	15.3	3063.3	1
TST-10C-S460	423.8	326.6	0.77	550.2	17.6	5916.5	1
TST-16A-S460	656.8	658.4	1.00	832.8	5.5	6242.1	2
TST-16B-S460	541.2	598.1	1.11	745.9	7.5	5114.8	2
TST-16C-S460	538.6	537.8	1.00	687.5	8.8	5436.1	2
TST-8A-S690	432.0	446.1	1.03	618.4	17.7	2756.1	1
TST-8B-S690	380.5	392.4	1.03	511.3	13.6	2392.7	1
TST-8C-S690	379.6	338.7	0.89	474.2	17.9	5262.6	1
TST-12A-S690	560.7	626.8	1.12	799.5	4.0	3005.0	3
TST-12B-S690	561.8	575.8	1.02	771.0	6.7	4431.4	2
TST-12A-S235	463.9	425.4	0.92	705.6	20.6	4709.4	1



(a)



(b)

Figure 3: Interpretation of results

3 NUMERICAL ANALYSIS

During the experimental research, it was used for the end plate of T-stub macro-component, different steel grades as S355, S460 and S690. It is well known that the failure mode of a T-stub macro-component could be type 1, 2 or 3, which means ductile, semi-ductile and fragile (Table 4).

Table 4: Classification of joints according to T-stub failure mode

Failure mode	Ductility	Classification
Mode 1	Ductile	Partial-strength / Semi-rigid
Mode 2	Semi-ductile	Full strength / Rigid
Mode 3	Fragile	Full strength / Rigid

The numerical analysis started with two types of real rigid full-resistant joints from two multi-storey buildings of 21 and 16 stories, respectively, designed in two seismic loading circumstances, in Bucharest and Brasov (see Figure 4 and Table 5). The joint is not containing the beam component.

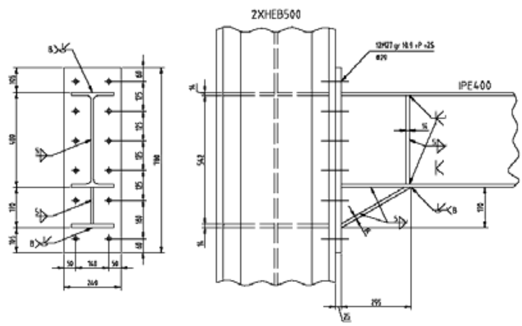


Figure 4: Brasov joint configuration

Table 5 Real joints configuration

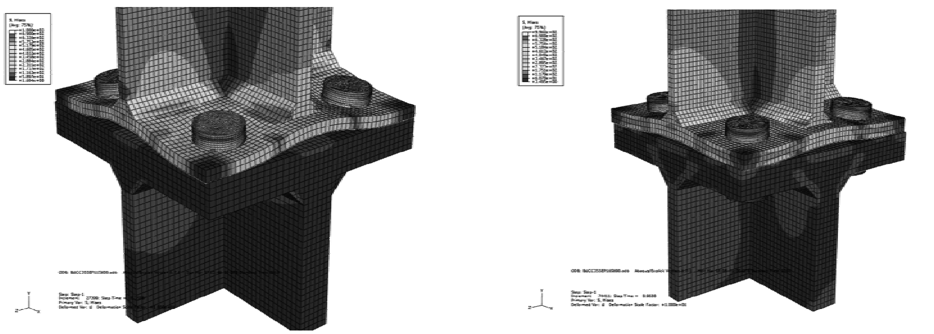
Joint	Column	Beam	Haunch	Bolts
Bucharest	HEB 800	IPE 500	200x400	12 M24 gr 10.9
Brasov	HEB 500	IPE 400	170x300	12 M20 gr 10.9

Table 6 Joint properties and classification

Joint	$M_{j,Rd}$ [kNm]	$M_{j,Rd}/M_{b,Rd}$	$S_{j,ini,Rd}$ [kNm]	EC3-1.8 Classification	T-stub failure mode (predicted)
BUC_EP15_S355	1027	1.320	804972	Rigid/full-strength	$2\rightarrow1$
BUC_EP22_S355	1125	1.446	879040	Rigid/full-strength	$2\rightarrow3$
BUC_EP14_S460	1056	1.357	766240	Rigid/full-strength	$2\rightarrow1$
BUC_EP20_S460	1133	1.456	858772	Rigid/full-strength	$2\rightarrow3$
BUC_EP11_S690	1038	1.334	603458	Rigid/full-strength	$2\rightarrow1$
BUC_EP20_S460	1132	1.455	782508	Rigid/full-strength	$2\rightarrow3$
BV_EP13_S355	553	1.192	448862	Rigid/full-strength	$2\rightarrow1$
BV_EP20_S355	631	1.360	555632	Rigid/full-strength	$2\rightarrow3$
BV_EP12_S460	567	1.222	412836	Rigid/full-strength	$2\rightarrow1$
BV_EP16_S460	617	1.330	507110	Rigid/full-strength	$2\rightarrow3$
BV_EP10_S690	571	1.230	323794	Rigid/full-strength	$2\rightarrow1$
BV_EP12_S690	604	1.302	398500	Rigid/full-strength	$2\rightarrow3$

Using different steel grades (S355, S460, S690) and thickness for the end-plate we obtained the failure modes of interest for our study, mode $2\rightarrow1$ and $2\rightarrow3$. The numerical analysis was performed with

ABAQUS computer program [10]. These T-stubs configurations and classification are presented in Table 6. Figure 5 shows two examples of T-stubs behavior and failure modes obtained by numerical simulations for specimens derived from Bucharest joint, while Figure 6 describes the behavior curves for the T-subs corresponding to all the cases in Table 6.



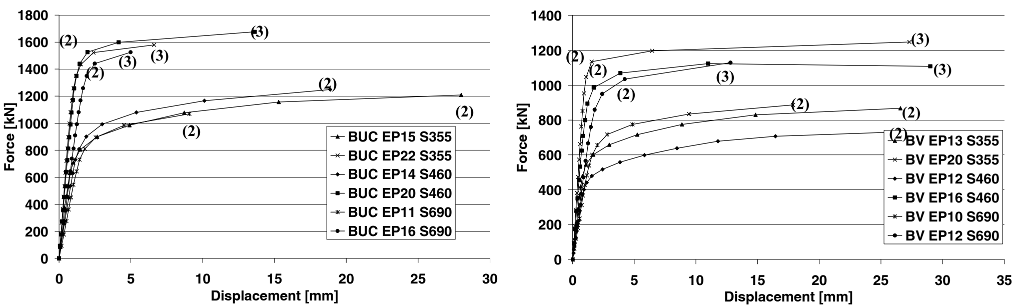
a) BUC_EP11_S690 (2→1) b) BUC_EP20_S460 (2→3)

Figure 5: T-stub behavior and failure mode according to numerical analysis

In failure mode (2), at the end, almost always, the bolt failure (3) might occur. In case of T-stubs designed for failure mode (2→1) (Figure 5.a), which are more ductile, first occurs the plasticization near the end-plate – beam flange junction, and starts the plasticization near the 1st and 2nd bolt rows, prior bolt fractures; in case of specimens of (2→3) failure mode (Figure 5.b), the second plasticization, usually does not occur, and bolt failure (3) arrives earlier.

In order to check the behavior of T-stubs in the MR joints, the response of two specimens of Table 6 has been simulated with ABAQUS, for monotonic loading only. The results, with a zoom of T-stub deformation mode are displayed in Figure 7.

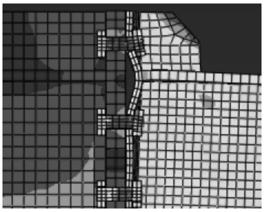
Going back from the T-stub to the joints, we analyzed numerically also with ABAQUS, two types of joints from the same family, e.g. Bucharest, but with T-stub configuration from the 2 borders of failure mode (2→1) and (2→3). In Figure 7 there is evident that both are confirming the way that they were designed.



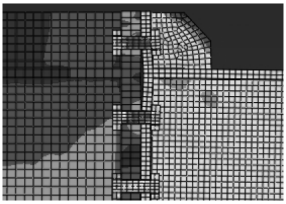
Bucharest Brasov

Figure 6: T-stub behavior according to numerical analysis

T-stub macro-component of the joint, which is falling down in mode 2 has sufficient rotation capacity and ductility (e.g. 40 mrad)to develop a plastic mechanism in joint if necessary, but only in mode (2→1) and not in mode (2→3), where the rotation capacity is quite poor like 15-20 mrad; on the hand, mode (2→3) is obviously stronger than mode (2→3).



BUC_EP11_S690 (2→1)



BUC_EP16_S690 (2→3)

Figure 7: Joint behavior

4 CONCLUSIONS

Seismic provisions [1] impose both minimum over-strength (1.373 $M_{j,Rd}$) and ductility (35 mrad) for beam-to-column joints. Since the column web panel contribution is limited by design, in case of bolted extended and stiffened end-plate beam-to-column joints, the main source of ductility is the end-plate, providing that its plastic failure mechanism is governed by mode 2.

Present paper demonstrates the end-plate can be sized by design (thickness & steel grade)to supply the ductility requested by code provisions.

REFERENCES

[1] EN 1993-1.8. 2003. *Eurocode 3. Design of steel structures. Part 1-8: Design of joints*, European standard.

[2] EN 1998-1. 2004. *Eurocode 8. Design of structures for earthquake resistance. General rules, seismic actions and rules for buildings*, European standard

[3] Dubina, D. and Dinu, F., "High strength steel for seismic resistant building frames", *Proc. of 6th Int. Conf. Steel and Aluminium Structures – ICSAS'07*, R. G. Beale (ed.), Oxford Brooks Univ., 133-140, 2007.

[4] Dubina, D., Stratan, A. and Dinu, F., "High Strength Steel EB Frames with Low Strength Bolted Links", *Proc. of Int. Conf. on Advances in Steel Structures – ICASS 2007,* Liew J.Y.R., Choo Y.S.(eds.), Research Publishing, Singapore, 249-255, 2007.

[5] Dubina, D., Stratan, A. Muntean, N. and Grecea, D., "Dual-steel T-stub behaviour under monotonic and cyclic loading", *ECCS/AISC Workshop: Connections in Steel Structures VI*, Chicago, Illinois, USA, 23-55, 2008.

[6] Dubina, D., Stratan, A. Muntean, N. and Dinu, F., "Experimental program for evaluation of Moment Beam-to-Column Joints of High Strength Steel Components", *ECCS/AISC Workshop: Connections in Steel Structures VI*, Chicago, Illinois, USA, June 23-55, 2008.

[7] Dubina, D., Muntean, N., Stratan, A., Grecea, D. and Zaharia, R., "Testing program to evaluate behaviour of dual steel connections under monotonic and cyclic loading", *Proc. of 5th European Conference on Steel and Composite Structures - Eurosteel 2008*, 3-5 September, Graz, Austria, 609-614, 2008.

[8] Girao Coelho, A.M., Bijlaard, F.S.K., Gresnigt, N. and Simoes da Silva, L., "Experimental assessment of the behaviour of bolted T-stub connections made up of welded plates", Journal of Constructional Steel Research 60, 269-311, 2004.

[9] Piluso, V. and Rizzano, G., "Experimental analysis and modelling of bolted T-stubs under cyclic loads", Journal of Constructional Steel Research, 10.1016/j.jcsr.2007.12.009, 2007.

[10] ABAQUS Inc. "ABAQUS analysis user manual", version 6.6, 2006.