UNA NUOVA PROPOSTA PER LA PREVISIONE DELLA RESISTENZA A RIFOLLAMENTO DI COLLEGAMENTI BULLONATI DI ELEMENTI TUBOLARI PER SCAFFALATURA

A NEW PROPOSAL FOR PREDICTING THE BEARING RESISTANCE OF BOLTED CONNECTIONS FOR TUBULAR RACKING STRUCTURES

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ABSTRACT

In the field of shelving structures for the storage of goods realized adopting tubular members, an interesting connection typology to be used to fasten the vertical columns to the beams is represented by bolted connections with long bolts in shear. With reference to this kind of connections, the authors have already performed a wide experimental and numerical investigation highlighting the occurrence, in this specific case, of local buckling phenomena. In this paper, the attention is focused on the influence of local buckling on the connection resistance, which is neglected by the current version of EC3 in the prediction of the resistance of shear connections of plates. In particular, starting from the results of the experimental and numerical analyses previously performed, in the present paper a formulation for predicting the bearing resistance of the examined connection able to account for the phenomenon of local instability is set up.

SOMMARIO

Nel campo delle strutture di scaffalature per lo stoccaggio di merci realizzate con elementi tubolari, una tipologia interessante per collegare i profili verticali con le travi è rappresentata da collegamenti bullonati con bulloni lunghi passanti a taglio. Con riferimento a questa tipologia di connessioni, gli

stessi autori hanno già effettuato un'ampia indagine sperimentale e numerica che ha evidenziato la presenza, nel caso specifico, del fenomeno di instabilità locale. Pertanto, in questo lavoro, l'attenzione è focalizzata sull'influenza dell'instabilità locale nella previsione della resistenza del collegamento, non considerata nelle formulazioni di previsione della resistenza di unioni a taglio tra piatti metallici contenute nella versione corrente dell'EC3. In particolare, a partire dai risultati delle analisi sperimentali e numeriche precedentemente eseguite, nel presente lavoro viene messa a punto una formulazione per prevedere la resistenza del collegamento in esame in grado di portare in conto il fenomeno dell'instabilità locale.

1 INTRODUCTION

With reference to tubular structures of shelving for storage of goods, a particular system of connections, easy to realize and economic, consists of innovative thin profiles coupled with long bolts.
As demonstrated by other works of the same authors, due to the use of reduced thickness tubes and steel with a high elastic limit, these connections are usually characterized by a bearing failure with the contemporary occurrence of local buckling phenomena (fig.1). Nevertheless, even though these connections are largely applied in practice, Eurocode 3 part 1-8 does not suggest specific rules able to account for these phenomena. In fact, Eurocode 3 provides only the equations able to evaluate the shear strength of common connections, consisting in coupling two or more plates in direct contact with the head of the bolt or nut which, as a difference with respect to shear connections made with tubes and long bolts, are able to create a sort of "confinement effect" in the area of the plate in direct contact with the bolts, avoiding buckling effects. Therefore, in order to extend the EC3 approach to the new type of connection a campaign of tests has been already performed by the same authors at the laboratory Mecanique des Materiaux et Structures - ArGEnCO University of Liege in order to study the influence of the main parameters that govern the behaviour of such connections [1].

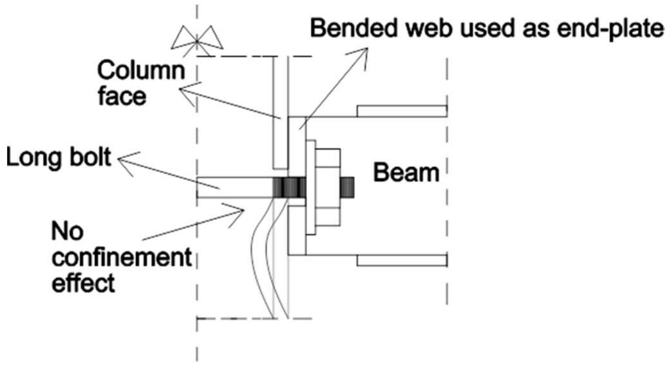


Fig. 1. Tube-to-plate shear connections

In the experimental tests carried out, the occurrence of local buckling in the compression zone of the tube loaded in shear by the bolt was highlighted. To better study this problem, a parametric analysis has been performed by the authors modelling the connection in Abaqus and validating the model by mean of the results of experimental tests [2]. The analyses, made by varying the thickness of the tube, have shown that there is a reduction of the maximum resistance when the phenomenon of local instability arises. In order to propose an improvement of the current formulation for bearing resistance of Eurocode, able to account for local instability, a simplified finite element model consisting on a simple thin plate has been developed in the present paper for the identification of the critical load of elastic buckling. To take into account that the instability does not occur in the elastic field and to take into account the geometric imperfections of the plate, effective width approach and Winter's formulation were applied. Finally, by means of a parametric analysis, a new coefficient for the reduction of resistance due to the crisis for local instability, to be applied to the formulation present in EC3 for the bearing resistance, is proposed.

2 EXPERIMENTAL CAMPAIGN AND FEM SIMULATION

The experimental campaign developed at University of Liege [1,3,4] was performed on 24 specimens with one tubular section fastened to two plates with one bolt (Fig. 2). The specimens are classified into six groups; each one is composed by four specimens nominally equal. The bolts have not been preloaded. Two different materials were tested HX420LAD and S235. In Table 1 the material grades, the dimensions of all the specimens and the experimental value of the resistance of the connections are given. The values of the geometrical parameters are: *L*, the length of the tube, *B*, the width of the tube, *d*, the diameter of the holes, *t*, the thickness of the tube, e_1, the distance between the hole and the edge of the tube, e_2, the distance between the hole and the lateral edge of the plate composing the tube section. In particular, in the experimental program, it has been observed the formation of buckling bumps in tests on specimens with steel HX420LAD and thickness ranging between 2 mm to 2.5 mm.

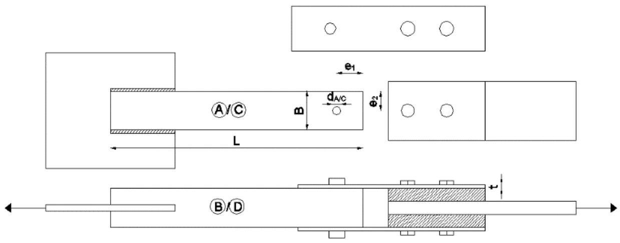


Fig. 2. Specimen typologies

Table 1. *Specimens' geometrical and mechanical properties*

	TUBE		*L* [mm]	*B* [mm]	d_N [mm]	*Holing procedure*	d_A [mm]	d_C [mm]	*T* [mm]	e_1 [mm]	e_2 [mm]	$F_{max,exp}$ [kN]
HX-2-M16	**HX420LAD**	1	399	60.15	16.5	Punched	16.59	16.66	2.04	50.26	29.67	69.32
	Thickness	2	398.8	60.16	16.5	Punched	16.50	16.51	2.11	48.66	29.68	80.78
	2mm, bolts	3	399.5	60.00	16.5	Punched	16.50	16.53	2.06	49.02	29.72	81.95
	M16	4	399.3	59.80	16.5	Punched	16.53	16.50	2.07	49.11	29.76	90.96
HX-2-M12	**HX420LAD**	5	389.9	60.00	12.5	Punched	13.28	12.9	2.03	39.27	29.92	67.64
	Thickness	6	390.2	59.80	12.5	Punched	12.89	13.05	2.06	40.00	30.21	70.75
	2mm,bolts	7	390.1	59.90	12.5	Punched	12.84	12.93	2.09	39.93	30.15	76.62
	M12	8	389.6	59.90	12.5	Punched	12.97	12.93	2.04	39.86	30.15	69.84
HX-2.5-	**HX420LAD**	9	391.2	60.00	12.5	Punched	12.55	12.64	2.57	39.60	29.84	89.62
	Thickness	10	388.7	60.10	12.5	Punched	12.46	12.56	2.55	40.00	29.99	90.25
	2.5mm,bolts	11	390.4	60.00	12.5	Punched	12.46	12.85	2.51	41.94	29.70	87.36
	M12	12	390.3	60.10	12.5	Punched	12.66	12.30	2.55	40.82	29.69	90.76
HX-2.5-	**HX420LAD**	13	399.0	59.90	16.5	Punched	16.59	16.49	2.57	49.58	29.93	121.60
	Thickness	14	399.0	60.10	16.5	Punched	16.98	16.45	2.54	51.67	30.02	120.04
	2.5mm,bolts	15	398.9	60.10	16.5	Punched	16.99	16.50	2.62	50.87	30.00	117.83
	M16	16	399.1	60.00	16.5	Punched	16.76	16.45	2.55	49.45	30.00	104.37
S235-4-	**S235**	1	391.7	60.50	12.5	Drilled	12.39	12.53	3.85	33.17	30.30	103.06
	Thickness	2	391.3	60.40	12.5	Drilled	12.41	12.51	3.87	33.36	30.39	100.39
	4mm,bolts	3	391.6	60.40	12.5	Drilled	12.57	12.56	3.85	33.05	30.49	97.24
	M12	4	391.5	60.40	12.5	Drilled	12.46	12.53	3.79	34.07	30.35	100.18
S 2	**S235**	5	391.4	60.30	16.5	Drilled	17.24	17.29	3.8	41.48	30.39	160.38
		6	391.5	60.60	16.5	Drilled	17.27	17.40	3.8	37.61	30.40	164.08

Thickness 4mm,bolts M16	7	391.0	60.50	16.5	Drilled	17.31	17.37	3.8	40.61	30.49	161.67
	8	390.1	60.60	16.5	Drilled	17.26	17.28	3.8	41.27	30.25	163.04

Conversely, the problem of buckling was not observed in a clear manner in specimens made from mild steel.
In [2] the influence of the thickness of the tube on the bearing resistance has been analyzed by means of a parametric study performed on a FE model calibrated in Abaqus. In particular the model was calibrated on the base of the experimental tests performed on the series 9-12 with tubes made with high strength steel (fig. 3). The thickness of these tubes is 2.5 mm and the bolts are M16, 8.8 steel grade. As shown in fig.3, the simulation provided by the “initial” model, able to account also for the hole imperfection, is in good agreement with the experimental results highlighting only a slight underestimation of the resistance but a finer accuracy in the prediction of the stiffness of the connection with respect to “ideal model” which neglects the initial imperfection. The calibrated FE Model has been used in the present paper in order to widen the ensemble of available data by varying the thickness of the tubes and investigating the influence of the thickness on the bearing resistance of the connection. The results of these simulations are shown in fig.4.

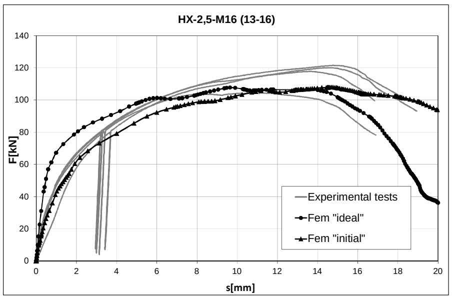


Fig. 3. Comparison between experimental test and models

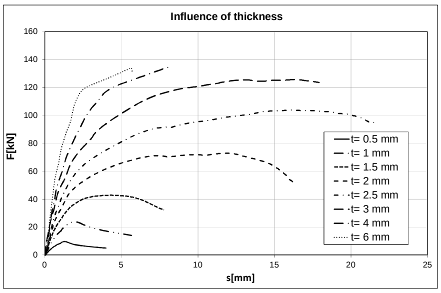


Fig. 4. Influence of thickness

The ultimate resistance achieved in the tests with tube thickness equal or greater than 3 mm is about 134 kN. EC3 gives for the bolt in shear the following value:

$$2F_{v,Rd} = 2\ 0.6 f_{ub}\mathrm{A} = \ 150{,}72\ \mathrm{kN} \tag{1}$$

The FEM simulations show that in cases of the tube thickness equal to 4 mm and 6 mm the bolt failure arises. In the case of tube thickness equal or less than 2,5mm, the connection resistance progressively reduces due to the bearing failure mode. EC3 provides the following formulation for the prediction of the bearing resistance:

$$F_{b,Rd} = 2{,}5 f_u d t \tag{2}$$

Table 2. *Comparison between resistance from EC's formulas and FEM simulations*

Thickness [mm]	0.5	1	1.5	2	2.5	3	4	6
FEM simulations [kN]	9.71	23.8	42.8	72.9	103.9	125.6	134.4	134.1
Bolt Failure [kN]	150.72	150.7	150.7	150.7	150.7	150.7	150.7	150.7
Bearing failure [kN]	20.4	40.8	61.1	81.5	101.9	122.3	163.0	244.6

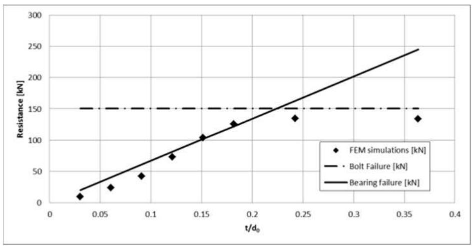


Fig.5. Comparison between EC3 formulations and FEM simulations

where f_u is the ultimate stress of the steel tube, *d* is the bolt diameter and *t* is the tube thickness.
In table 2, the comparison between the maximum loads provided by the FEM simulation and those provided by the EC3 formulation is given.
In Fig. 5, the same comparison is represented in terms of t/d_0 ratio. It can be observed that the thickness of the tube influences significantly the resistance of the connections and the collapse mode. In particular, Fig. 5 shows that for a value of the ratio t/d_0 greater than 0,23 the resistance of the connection is governed by the failure of the bolts in shear. On the contrary, for values of the ratio t/d_0 less than 0,23, the bearing resistance is lower than the shear resistance of the bolts. However, for values of the ratio t/d_0 less than 0,146, fig.5 shows that the bearing resistance provided by Eq. (2) are not on the safe side. This occurs when local buckling of the tube becomes more significant due to the high local slenderness of the plate. Therefore, in correspondence of low values of the tube thickness, the resistance of the connection can be governed by the buckling of the plate which can anticipate the bearing failure. For this reason, it appears essential to improve the current formulation of the code for the evaluation of the resistance to bearing failure. To this scope, it is necessary to further investigate the buckling resistance of the plate constituting the tube aiming to calibrate an appropriate coefficient to be introduced in the code approach for the evaluation of the bearing resistance able to take into account the limitation due to the buckling resistance of the connection.

4 BUCKLING ANALYSIS

4.1 Simplified model

In order to analyse the local buckling of the plate of the tube in contact with the bolt, a simplified 2-D FE model in SAP2000, able to account for all the interactions between the plates composing the tube, has been implemented. In particular, the modelling has been limited to the part of the tube where buckling occurs, i.e. the part of plate in between the bolt and the end of the tube (Fig.6), whose dimensions are the width of the tube and distance between the edge of the hole and the free edge of the tube. The load on the plate due to the compression of the bolt has been considered as uniformly distributed along the width of the diameter of the bolt.

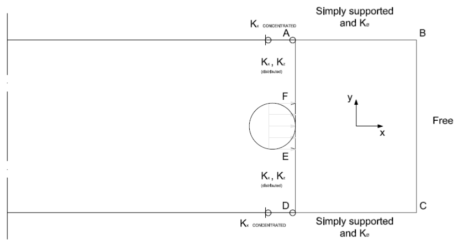


Fig. 6. - Boundary conditions

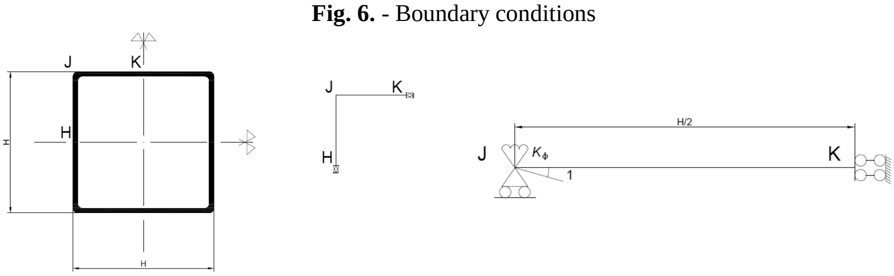


Fig. 7. - Diagram for rotational springs

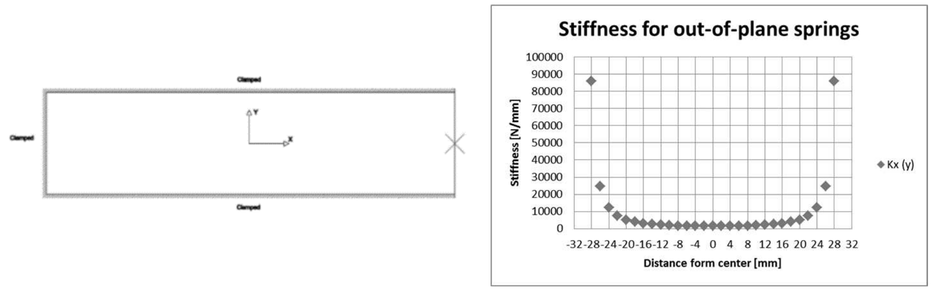
On the sides of the plate A-B and D-C of Fig. 6, in order to account for the restraining action due to the other plates composing the tube, rotational springs that prevent the translational movement of the plate were applied. The stiffness of these springs was assessed by adopting the hypothesis of double symmetry of the profile as shown in Fig. 7.
Therefore, the following value of the spring stiffness has been assumed:

$$K_{\varphi} = \frac{2EI}{H} \tag{3}$$

The side B-C is free, while on the side EF, in order to account for the presence of the remaining part of the tube, extensional springs acting in X direction have been applied. In particular, at points A and D of the model two springs were applied with extensional stiffness in the X direction calculated as:

$$K_{X,conc} = \frac{E(A/2)}{L} \tag{4}$$

where *L* is the length of the tube, *E* is the Young's Modulus and *A* is the area of the lateral wall of tube. In addition, from points A to F and from points D to E, distributed extensional springs in x direction have been applied. Along the same edges, in order to restrict the movement out of the plane of the plate, a distributed extensional spring in Z direction have been considered. The stiffness of these springs has been preliminary evaluated by modelling the residual part of the plate between the section A-B and the restrained edge of the tube according to the model showed in Fig. 8a.
By applying, in the free side of this plate, a concentrated force and, by moving the force from the centre to the end of the edge, the trend of the stiffness along the free side of the plate has been evaluated (Fig. 8b).



(a) (b)

Fig. 8. - Grashof's model and stiffness for out-of-plane springs

It can be observed that, the trend of the stiffness is almost constant along the whole plate. Only near to the edge of the plate the stiffness asymptotically tends to infinity, due to the presence of clamped edges. In order to maintain a simple structure of the model, taking into account that the application of springs with variable stiffness along the edge don't provided appreciable differences with respect to the model with springs of constant stiffness, uniform stiffness has been assumed.
The plate restrained as previously described was modelled in SAP2000 with a shell element considering the small thickness of the tube, the shell thin plate element has been assumed, neglecting, according to Kirchhoff's Theory [5,6], the transverse shear deformation.

4.2 Inelastic buckling load

First of all, by means of the above model, the elastic buckling load has been calculated (Fig. 9). In table 3 the values of the elastic critical load provided by the FE model are given for different values of the plate thickness.

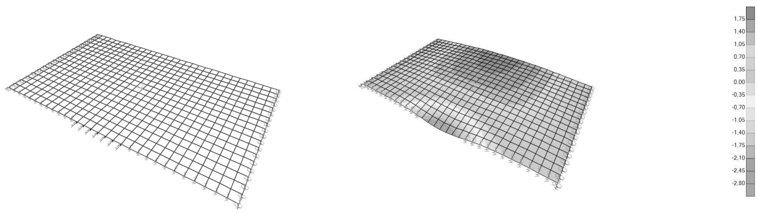


Fig. 9 - Model for plate in SAP2000 and first buckling mode.

In order to account for the influence of the imperfections and the buckling in the elastic-plastic range, the Winter formula and the effective width approach has been adopted. According to the Winter approach [7], the critical load of the individual plate $N_{cr,Rd}$, affected by parametric imperfections, is given by:

$$N_{cr,Rd} = N_{pl}\left[\left(1 - \frac{0{,}22}{\overline{\lambda_p}}\right)\frac{1}{\overline{\lambda_p}}\right] \leq N_{pl} \tag{5}$$

where λ_p is the slenderness provided by:

$$\overline{\lambda_p} = \left(\frac{N_{pl}}{F_{cr}}\right)^{1/2} \tag{6}$$

In eq. (6) F_{cr} is the critical load in elastic range and N_{pl} is the plastic load evaluated by means of the effective width on which uniform yield stress is assumed. According to the stress distribution in plastic range assumed by EC3, an effective width equal to $3{,}5d_0$ has been adopted for the sake of simplicity (Fig.10). Therefore, the plastic load has been evaluated by means of the following formulation:

$$N_{pl} = 3.5\ d_0 t f_y \tag{7}$$

In tab. 3 the values of N_{pl}, Winter's reduction coefficient and inelastic buckling load are given. In order to analyse the influence of local buckling on the prediction of the connection resistance, in Fig. 11 the values of the maximum loads provided by the FE model simulations are compared with those provided by eq.(1) for the bolt shear resistance and eq.(2) for bearing resistance. It can be seen in fig. 11 that, taking into account the proposed formulation for the evaluation of the instability, a conservative estimation of the resistant load is reached also in the zone of lower values of thickness, when the formulation of bearing resistance don't provide a prediction on safe side.

5. IMPROVEMENT OF EC3 APPROACH

The resistance of shear connections fastening plate elements by means of one bolt is provided in EC3 by the following relationship which accounts for, simultaneously, the plate bearing failure, the plate shear failure and the plate failure in tension:

$$F_{b,RD} = \ k_1 \alpha_b\ f_U\ d\ t \tag{8}$$

where α_b is the smallest of $\frac{e_1}{3d_0}, \frac{f_{ub}}{f_u}$ or 1,0, and k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7\ or\ 2{,}5$.
Aiming to adopt the same approach and to account for the effect of buckling, an additional factor in the evaluation of coefficient α_b of Eq. (8) can be introduced. To this scope, adopting the same structure of Eq. (8) and considering the resistance provided by Eq. (5) and Eq. (7), a new factor $\alpha_{b,st}$ that can take into account the occurrence of buckling phenomena on the plate has been defined as:

$$\frac{F_{b,RD}}{N_{cr,RD}} = \frac{2{,}5\alpha_{b,st}\ \ d_0 t f_u}{3{,}5 d_0 t f_y \beta} = 1 \tag{9}$$

and, therefore the new coefficient to insert in the classical bearing resistance formula, is:

$$\alpha_{b,st} = 1.4\frac{f_y}{f_u}\beta \tag{10}$$

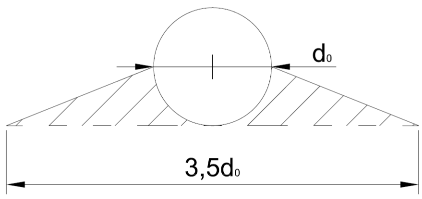


Fig. 1. - Evaluation of the width for stress diffusion in the plate

Table 1 - Evaluation of the critical load for the plate

Thickness [mm]	t/d	f_y [N/mm^2]	d_0 [mm]	Elastic Buckling Load [kN]	$N_{plastic\ load}$ [kN]	λ_p	Winter's reduction	$N_{pl,red}$ [kN]
0.5	0.030303	443	16	0.47	12.03	5.048	0.189	2.279
1	0.060606	443	16	8.67	24.06	1.666	0.521	12.534
1.5	0.090909	443	16	29.26	36.08	1.110	0.722	26.057
2	0.121212	443	16	69.36	48.11	0.833	0.884	42.508
2.5	0.151515	443	16	135.47	60.14	0.673	1.000	60.150
3	0.181818	443	16	217.97	72.17	0.673	1.000	72.180

.

where $\beta = \left(1 - \frac{0,22}{\bar{\lambda}_p}\right)\frac{1}{\bar{\lambda}_p}$ evaluated according to Winter's formula.

In order to calibrate the parameter β, a parametric study using the plate model developed in SAP2000 has been performed by varying the ratio between the thickness of the plate and the diameter of the hole. As shown in fig.12 the data series follow a trend almost linear. By means of linear regression analysis, the following relationship has been obtained for the β coefficient:

$$\beta = 5,24\frac{t}{d_0} + 0,13 \tag{11}$$

Therefore, accounting Eq.(11), the following new coefficient able to take into account the buckling phenomena has been derived:

$$\alpha_{b,st} = 1,4\frac{f_y}{f_u}\left[5,24\left(\frac{t}{d_0}\right) + 0.13\right] \tag{12}$$

and, as a consequence, the approach of EC3 for evaluating the resistance of shear connections expressed by Eq.(8) can be extended to the case of shear connections fastening plates to hollow section provided that the coefficient α_b of Eq. (8) is evaluated according to the following relation:

$$\alpha_{b,st} = \min\left(\frac{e_1}{3d_0};\ \frac{f_{ub}}{f_u};\ 1;\ 1,4\frac{f_y}{f_u}\left[5,24\left(\frac{t}{d_0}\right) + 0.13\right]\right) \tag{13}$$

In Fig. 13 (a) the comparison between the results of FEM simulations and the resistance predicted by mean of Eqs (8 and 13) is given. It can be observed the good accuracy of the proposed formulation also in the cases of low values of the plate thickness. Finally it has to be underlined that in case of design analyses in which partial safety factors have to be applied, because the check for instability in EC3 is characterized by the application of a partial safety factor γ_{m1} different from that used for the connections γ_{m2} in order to take into account this difference, Eq. (13) can be replaced as:

$$\alpha_{b,st} = \min\left(\frac{e_1}{3d_0};\ \frac{f_{ub}}{f_u};\ 1;\ 1,4\frac{f_y}{f_u}\left[5,24\left(\frac{t}{d_0}\right) + 0.13\right]\frac{\gamma_{m2}}{\gamma_{m1}}\right) \tag{14}$$

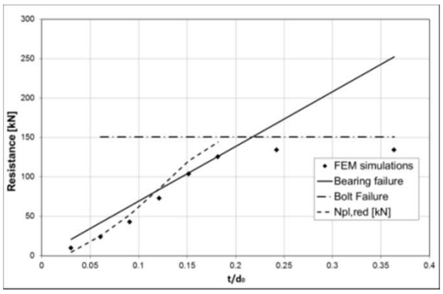


Fig. 2 - Connection resistance

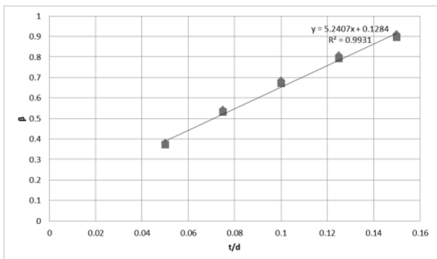
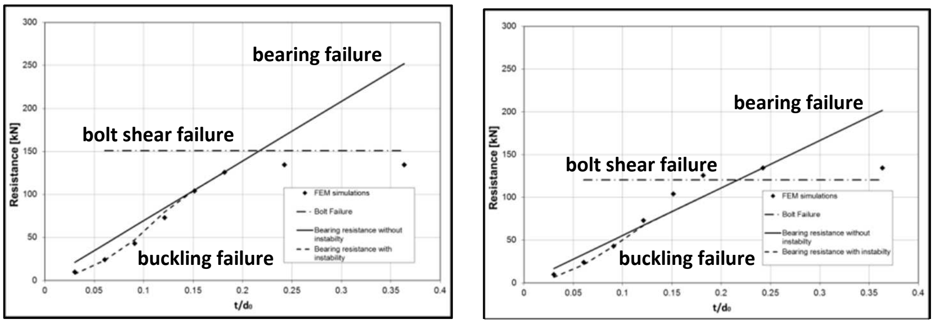


Fig. 12. - Parametric analysis varying *t/d* ratio

and, obviously, eq. (8) is replaced as:

$$F_{b,RD} = \frac{k_1 \alpha_b f_U d t}{\gamma_{m2}} \tag{15}$$

In Fig. 13 b, the FEM simulations have been represented together with the predicted resistance provided by Eqs (14) and (15).



(a) Resistance evaluated without γ_{m2} (b) Resistance evaluated with γ_{m2}

Fig. 3 - Comparison between proposed formulation and FEM simulation

6 CONCLUSIONS

In this work the behaviour of a new joint for the racking structures not covered by EC3 has been investigated. In particular, starting from an experimental and numerical analysis already performed by the authors, in the present paper, in order to take into account the influence of the buckling effects, a reduction factor of the crushing resistance of the plate has been obtained using the Winter's formula, and effective width approach. The good agreement between the developed simulations and the resistance analytically evaluated accounting for stability check in case of low values of *t/d* ratios testifies the good improvement provided by the proposed formulation.

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