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Theoretical and Experimental Analysis of Column Web in Compression

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Abstract: Advanced analysis of steel structures requires an accurate modelling of beam-to-column joints to be used for complete non linear analyses including both mechanical non-linearity and geometrical non linearity. Therefore, the prediction of the rotational behaviour of beam-to-column joints is necessary. To this scope the component approach can be applied. Within this framework, in this paper the attention is focused on the behaviour of the column web in compression, which represents an important joint component influencing significantly the rotational response of unstiffened connections where continuity plates are omitted. In particular, in order to analyse the whole force-displacement behaviour of this component, an experimental program has been planned and performed on twelve specimens represented by standard HEA and HEB series whose web has been subjected to transversal compression, under displacement control, allowing the investigation of the post-buckling behaviour.

INTRODUCTION

Analysis and design of steel structures is often conducted by assuming that beam-to-column joints behave according to two extreme models: pinned or fixed. The first model is related to joint details characterized by high rotation capacity and poor flexural resistant so that design can be pratically carried out by assuming that joints are unable to transmit moments and they permit free rotations. As a consequence, these joints are designed in order to transmit shear forces only.

The second model is related to joint details characterized by high rotational stiffness, so that all the ends of the members converging in the joint are subjected to the same rotation. These joints are able to transmit both bending moments and shear forces.

However, the above models are only two ideal extreme cases, because, depending on the joint detail, the rotational behaviour is always intermediate between two extreme models of pinned joints and fixed joints.

Therefore, the actual behaviour of beam-to-column joints has to be properly modelled accounting for all the sources of deformation and resistance. To this scope the component method is nowadays widely adopted. This approach is based on the identification of active basic components, depending on the joint typology. Each component is characterized by means of its strength, stiffness and deformation capacity (i.e. by means of its force-displacement constitutive law), and contribute to the behaviour of the whole joint.

The application of the component method requires the identification of the active components, the evaluation of the mechanical properties of each individual basic component and the assembling of components into a mechanical model in order to evaluate the moment versus rotation response of the whole joint.

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In this framework, the component method can be regarded as analogous to the finite element method, because the basic joint components can be viewed as "finite elements" collected into a library of elements, so that different combinations of elements into different mechanical models allow the analysis of different structural details and/or different beam-to-column joint typologies [1].

The active components belonging to traditional steel joints have been deeply studied and recommendations for modelling their force-displacement constitutive law are given in part 1-8 of the (Eurocode 3) [2]. In particular, for each component the formulations for computing the initial stiffness and design strength are provided. Eurocode 3 also provides a combination procedure to obtain, for a wide variety of joint configurations, the whole joint response in terms of initial rotational stiffness and flexural resistance.

Unfortunately, the current available formulations simply regard initial stiffness and strength of beam-to-columns joints, but there are no generally accepted procedures for evaluating rotation capacity. Eurocode 3 gives a limited amount of information, suggesting only the situations where rotation capacity supply is believed to be sufficient to sustain imposed rotations, without a specific control of the rotation capacity of the joint.

Conversely, the possibility of evaluating the plastic rotation supply is of primary importance in designing steel connections so that the designers can be aware about the use of plastic analysis in case of ductile connections, and the use of elastic analysis in case of brittle connections.

In particular, the knowledge of the plastic rotation supply is of primary importance in the case of partial strength connections, because in this case yielding occurs in the connecting elements. Therefore, the setting up of a simple method for evaluating the rotation capacity of partial strength joints is of primary importance for everyday design practice.

In addition, regarding seismic design of steel structures, even though the plastic deformation supply under cyclic loads is clearly affected by low-cycle fatigue phenomena, so

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that it is less than the one occurring under monotonic loads, it should be noted that the knowledge of the plastic deformation supply under monotonic loads is the starting point for estimating the collapse condition under cyclic loads by means of damage functionals [3].

Recent seismic codes, such as (Eurocode 8) [4], have opened the door to the use of partial strength joints in designing ductile moment-resisting frames. In this case the earthquake input energy dissipation is provided by significant yielding of some properly selected connecting elements. This means that it is necessary to know the deformation capacity of connections and, in particular, an appropriate joint semirigid design is required to lead to a plastic rotation supply compatible with the plastic rotation demand under seismic motion. To this scope the designer has to be aware that the overall joint ductility is provided by the contribution of all the components engaged in plastic range and, in addition, that the premature collapse of brittle components has to be absolutely avoided. Therefore, an adeguate overstrength level has to be assumed in the design of fastening elements, such as bolts and welds, which do not exhibit a ductile behaviour.

According to the component approach, the evaluation of joint ductility requires the plastic deformation capacity of the basic joint components. In fact, if the beam-to-column joint had a single joint component engaged in plastic range, its plastic deformation supply would be coincident with the ratio between the ultimate displacement of that component and the lever arm. In general, there is one component engaged up to the achievement of its ultimate plastic displacement and other components partially engaged in plastic range, so that a procedure to combine the contribution of each component is necessary [5].

Regarding the component method, in this paper the attention is focused on the ultimate behaviour and the plastic deformation capacity of the column web in compression, which represents an important joint component characterized by its ability in providing significant deformation capacity, provided that post-buckling behaviour can be exploited. In fact, in the case of welded connections the beam-to-column joint detail is often simplified by omitting the continuity plates commonly located at the beam flange level to stiffen the column web, so that the joint deformability cannot be neglected and it is necessary to model the structure as a semirigid frame.

In particular, the column web in compression is subjected to transverse compression (single or double symmetrical, depending on the joint configuration, i.e. external or internal joints) transmitted by the beam flanges, and can fail due to crushing or buckling.

The horizontal compression forces transmitted by the compressed flanges of connected beams produce in the panel zone horizontal normal stresses interacting with the shear stresses and the vertical normal stresses due to the axial load and the bending moment acting at the column end. The interaction between these local stresses affect not only the local crushing resistance, but also the buckling resistance of the column web [5]. An exhaustive understanding of the behaviour of the column web in compression is also complicated by geometrical and mechanical non-linearities.

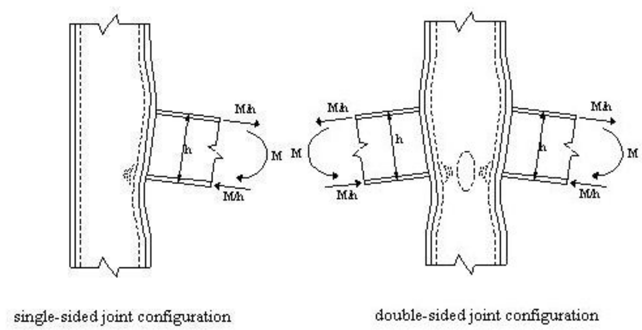

Fig. (1). Deformation of joints subjected to bending moments.

A significant research effort devoted to the interpretation of the behaviour of the column web in compression has been developed by Aribert *et al*. [6,7] which, on the basis of their experimental results, have underlined as the collapse of the column web in compression is accompanied by the development of a kinematic mechanism with the formation of plastic hinges concentrated along particular zones of the element, allowing the identification of a yield line model representing the collapse mechanism exhibited during experimental tests. Conversely, attention has not been point to the characterization of the ductility of the member, being the experimental tests executed under force control.

Regarding the investigation of the buckling resistance, several researchers [1,7,8,9] suggested the use of the Winter formula, as successively adopted by Eurocode 3.

However, the simple use of the Winter formula leads in some cases to the overestimation of the resistance of the column web compression, so that Faella *et al.* [5] proposed to compute the buckling resistance starting from an effective width of the column web, specific for buckling, accounting for the influence of the column section geometrical properties. In particular, Faella *et al.* formulation was derived modelling the column flange as a beam elastically supported by springs that represent the restraining action due to the column web, according to the classical Winkler model.

Recently, the kinematic approach, based on the yield line model, proposed by Aribert and Moheissen has been extended by Catenazzo and Piluso [10] to the problem of predicting the plastic deformation supply of the column web in compression. In particular, the kinematic theorem of plastic collapse is applied to obtain not only the collapse load, but also the equilibrium curve of the mechanism describing the post-buckling behaviour of the column web in compression. The knowledge of the mechanism equilibrium curve allows the prediction of the plastic deformation supply as the displacement value occurring when the corresponding force value of the softening equilibrium curve is equal to the crushing design resistance of the column web.

However, the comparison between the theoretical model and the experimental results has been limited to the maximum load only, because the experimental tests available in technical literature were conducted under force control. As a consequence, it was not possible to estimate the accuracy of the model in terms of post-buckling response and, consequently, in terms of its ability to estimate the ductility of the column web in compression.

The outcome of the analysis of previous research efforts is the lack of experimental data regarding the ductility of the column web in compression.

In this paper, the attention is focused on the ultimate behaviour and the plastic deformation capacity of the column web in compression, by investigating the whole force-displacement behaviour of this component by means of an experimental program planned and performed on twelve specimens represented by standard HEA and HEB series. The tests have been performed under displacement control applying symmetrical transverse loads to the member web, allowing the investigation of the post-buckling behaviour. Starting from these results, the analytical formulations, available in the technical literature, for predicting the stiffness, resistance (crushing or buckling) and ductility are analysed. Finally, the comparison between experimental and analytical results is also presented.

THE TEST PROGRAM

The post-buckling behaviour of the column web in compression (Fig. **1**) can be investigated by means of a simple testing procedure provided that the interaction with shear stress and axial force in the member is neglected. This is the case of the present work where the above interaction is out of the scope. In particular, by applying to specimens a symmetrical transverse load by means of a knife action of a stiffened plate, the testing scheme represented in Fig. (**2**) is obtained.

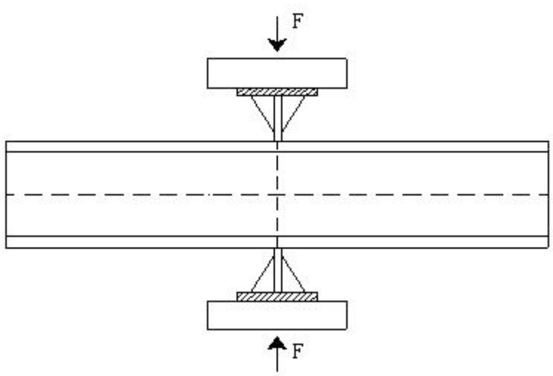

Fig. (2). Tested specimen.

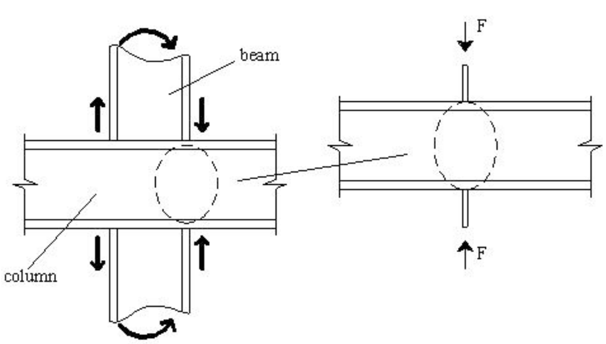

Fig. (3). Model scheme.

The main scope of the test is to simulate the local compression action of opposite beam flanges of a simple beam-to-column joint (Fig. **3**). This loading condition occurs in the case of internal joints of frames subjected to vertical loads only.

The experimental program regards twelve specimens represented by standard HEA and HEB series, according to the sections 200, 220, 240, 260, 280 and 300, characterised by several lengths and made of S275 steel grade. The length of the specimens has been selected according to the relationship L=4h, where h is equal to the section height, proposed in [7]. This distance assures that at the member ends the effects due to the load application can be neglected.

For each particular section, geometric properties, such as web (t_w), flange (t_f) and knife-plate (t_{fb}) thickness, and mechanical properties (yield stress of web f_{yw} and flange f_{yf}) are presented in Table **1**.

Table 1. Tested Specimens

Specimen	b [mm]	H [mm]	t_w [mm]	t_f [mm]
HE200A	200,0	200,0	9,50	15,00
HE200B	199,0	203,5	10,00	14,50
HE220A	220,0	215,0	7,50	11,50
HE220B	220,0	220,0	10,50	16,40
HE240A	240,0	234,0	8,30	12,00
HE240B	240,0	245,0	11,50	16,70
HE260A	261,0	253,0	8,10	13,10
HE260B	260,0	262,0	10,90	17,30
HE280A	280,0	274,0	9,30	12,50
HE280B	283,0	285,0	10,70	18,30
HE300A	299,0	295,0	10,00	14,40
HE300B	340,0	340,0	11,50	19,40
HE200A	18	25	335,771	278,262
HE200B	18	25	284,309	272,763
HE220A	18	25	335,712	321,291
HE220B	18	25	283,060	267,262
HE240A	21	25	396,656	341,403
HE240B	21	25	354,829	-
HE260A	24	25	318,143	280,458
HE260B	24	25	265,156	269,039
HE280A	24	25	319,201	331,583
HE280B	24	25	270,667	262,293
HE300A	27	25	306,824	313,452
HE300B	27	25	323,563	275,768

The application of the transverse load to the knife-plate is obtained by means of an hydraulic actuator, Schenck RBS 4000 (maximum test load 4000 kN, piston stroke ± 100 mm), equipped with an automatic displacement control system guided by means of electric transducers directly located on the specimen. The load process is continuously recorded obtaining the load-displacement curves, which are provided in Figs. (**4,5**) for every specimen of the considered series (δ_{exp} is the total displacement in the web plane due to the deformation of both the specimen and the load transmitting system).

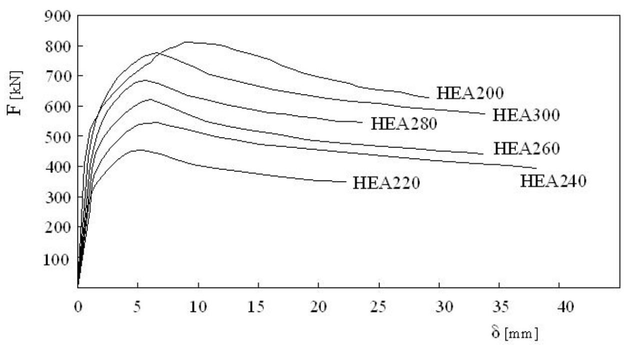

Fig. (4). Load-displacement curves for HEA series.

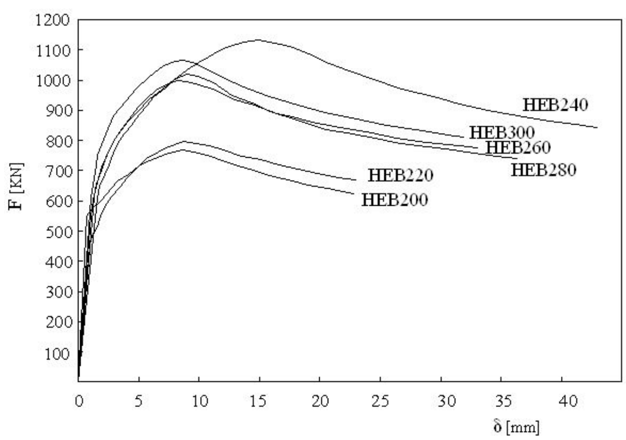

Fig. (5). Load-displacement curves for HEB series.

EXPERIMENTAL BEHAVIOUR OF SPECIMENS

The experimental program has been performed for monotonic displacement histories, so that the initial stiffness, ultimate resistance and also the whole post-buckling behaviour are obtained for any tested specimen.

The transducer arrangement detailed in Fig. (**6**) has been used to measure both the transverse displacements in the plane of the applied load and the web out-of-plane displacements occurring during the post-buckling behaviour.

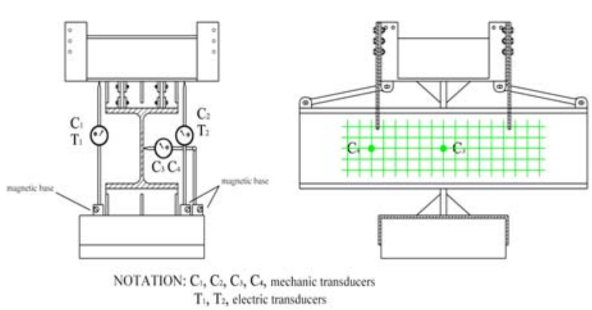

Fig. (6). Transducers arrangement.

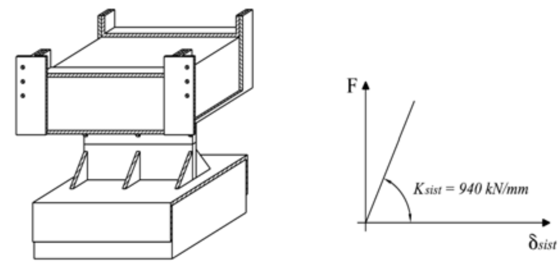

Fig. (7). Load system test.

In particular, transducers C1, C2, T1 and T2 provide the displacements between the stiffened plates, while transducers C3 and C4 provide the out-of-plane deformation of the web. It can be noted that C1, C2, T1 and T2 transducers follow the displacement of the load transmitting stiffened plates, so that the obtained measures include also the deformations of the load transmitting system.

Therefore, a preliminary characterization of the load transmitting system has been carried out by means of a monotonic test aimed at the evaluation of its stiffness K_{sist}. This test has been led under force control until a load level greater than the maximum value of the crushing resistance estimated for the considered specimen series (Fig. **7**).

The stiffness value of the load transmitting system (K_{sist}=940 kN/mm) has been successively used to correct the displacements of the experimental load-displacement curves according to the following analytical expression:

$$\delta = \delta_{exp} - \delta_{sist} = \delta_{exp} - \frac{F}{K_{sist}} \qquad (1)$$

Regarding the experimental behaviour of tested specimens, it is characterized by an initial linear response with no perceptible elastic deformation. The leaving from the linear response occurs with the appearance of the first yield line on the web profile and the inflection of the loaded flanges. As far as the controlled displacement increases, a considerable plastic engagement of the web panel adjacent to the loaded flanges occurs with a progressive extension of the yielded zones (Figs. **8,9**). The occurrence of the maximum resistance is followed by the complete development of a kinematic mechanism characterising the post-buckling behaviour (Fig. **10**). The post-buckling behaviour governs the softening branch of the load-displacement curve.

Fig. (8). Plastic engagement of the web panel.

Fig. (9). Plastic deformation measure.

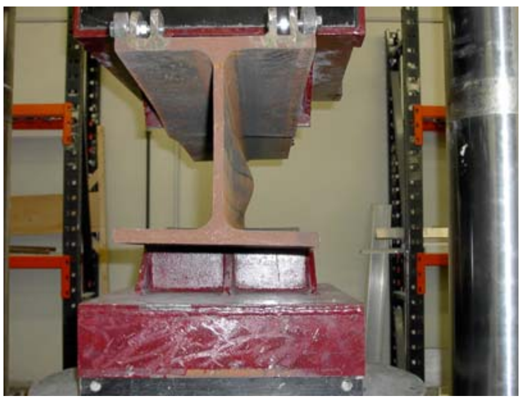

Fig. (10). Post-buckling kinematic mechanism.

TESTING RESULTS AND COMPARISON WITH AVAILABLE FORMULATIONS

Starting from the experimental results, a comparison is herein presented with the formulations for predicting the initial stiffness, the ultimate strength and the deformation capacity, available in Eurocode 3 and/or in the technical literature. In particular, three comparisons are separately presented with reference to the initial stiffness, the resistance (crushing or buckling) and the plastic deformation capacity.

It has to be mentioned that the available formulations have been applied considering the measured values of the material mechanical properties.

Initial Stiffness

According to Eurocode 3 formulation, the column web in compression is modelled by means of an extensional spring characterised by the following stiffness:

$$K_{cwc} = \frac{E\ B\ t_{wc}}{d_{wc}} \qquad (2)$$

where t_{wc} is the web thickness, d_{wc} is the clear depth of the web and B is the effective width.

In particular, B is given by the following equation [9]:

$$B = 0{,}7 b_{eff,cwc} = 0{,}7\left(t_{fb} + 2a_b\sqrt{2} + 5k \right) \qquad (3)$$

where t_{fb} is the beam flange thickness, a_b is the throat thickness of the beam flange-to-column flange weld and $k=t_{fc}+r_c$ for rolled sections, being t_{fc} the column flange thickness and r_c the web toe fillet.

Some authors Tschemmernegg *et al.* [11], Faella *et al.* [5-9] have already pointed out that the above formulation overestimates the component stiffness; therefore an alternative formula has been proposed:

$$B = t_{fb} + 2a_b\sqrt{2} + 2\left(t_{fc} + r_c \right) \qquad (4)$$

With reference to the physical meaning of equation (2), it is useful to note that the testing scheme adopted in the present experimental work concerns an internal joint, so that, from the deformation point of view, the measured displacements are due to both the left and the right connections converging in the joint. Therefore, taking into account that equation (2) can be directly applied only for external joints, the system stiffness K is derived as:

$$\frac{1}{K} = \frac{1}{K_{cwc}} + \frac{1}{K_{cwc}} = \frac{2}{K_{cwc}} \qquad (5)$$

Considering also the above interpretation, the following empirical equation for characterising the initial stiffness of members under symmetrical transverse compression has been proposed by Aribert *et al.* [12].

$$K = 0{,}45\left(\frac{b_{fc}\, t_{fc}^{3} t_{wc}}{d_{wc}} \right)^{0{,}25} \frac{E}{2} \qquad (6)$$

In addition, the same research group has proposed a refinement of the above formulation in a successive work [13]:

$$K = 0{,}95\left(\frac{b_{fc}\, t_{fc}^{3} t^{2}{}_{wc}}{b_{eff}\, d_{wc}} \right)^{0{,}25} \frac{E}{2} \qquad (7)$$

The stiffness values obtained by means of equation (2), with the effective widths given by formulations (3) and (4), and by means of equations (6) and (7) are presented in Table **2**. The contribution of the beam flange-to-column flange weld is equal to zero, being a_b=0 in the experimental model. In order to investigate the accuracy of the mentioned formulations, in the same table the experimental values are also presented.

The comparison among the several formulations is quantitatively provided in Table **3** by means of the evaluation of the ratio between the theoretical and the experimental values. For the above ratio, the mean value and the standard deviation are also given.

From Table **3**, it can be observed that relationship [4], suggested by Tschemmernegg *et al.* [11] and by Faella *et al.* [5,9] leads to values closest to the experimental ones. It can be noted, moreover, that also the Aribert *et al.* formulations leads to a better prediction than Eurocode formulation, confirming its trend in overestimating the initial stiffness of the component under examination.

Resistance

The design resistance of the column web in compression is given by the minimum value between the crushing resistance and the buckling resistance of the web panel.

Table 2. Stiffness Values According to Different Formulations

Profile	B [mm] Ref. [2]	B [mm] Ref. [5]	K_{exp} [kN/mm]
HE200A	133,0	91,0	712
HE200B	131,3	90,0	824
HE220A	120,8	84,0	284
HE220B	137,9	93,8	456
HE240A	133,0	91,0	251
HE240B	133,0	100,4	383
HE260A	147,4	99,2	350
HE260B	162,1	107,6	550
HE280A	145,3	98,0	395
HE280B	165,6	109,6	465
HE300A	162,4	107,8	528
HE300B	179,9	117,8	576

Profile	K [kN/mm] Ref. [2]	K [kN/mm] Ref. [5]	K [kN/mm] Ref. [6]	K [kN/mm] Ref. [7]
HE200A	971	665	685	684
HE200B	976	669	670	680
HE220A	598	416	522	503
HE220B	986	671	747	757
HE240A	677	463	554	535
HE240B	929	701	769	782
HE260A	688	463	592	553
HE260B	1014	673	784	771
HE280A	692	467	584	568
HE280B	910	603	808	787
HE300A	788	523	664	638
HE300B	862	565	854	829

The crushing resistance is provided by the yield condition of the web panel zone adjacent to the loaded flange, while the buckling load is given by the well known Winter formula. The crushing resistance value can be obtained by the following relationship [2]:

$$F_{cwc} = \rho\ k_{cwc}\ f_{yw}\ t_{wc}\ b_{eff,cwc} \tag{8}$$

where ρ is a factor accounting for shear interaction (equal to 1 for the tested specimen), k_{cwc} is a factor accounting for the influence of the vertical normal stress (equal to 1 for tested specimen), f_{yw} is the yield stress of the column web, t_{wc} is the web thickness and $b_{eff,cwc}$ is the effective width of the column web in compression.

In particular, $b_{eff,cwc}$ is calculated according to the following equation:

$$b_{beff,cwc} = t_{fb} + 2a_b\sqrt{2} + 5(t_{fc} + r_c) \tag{9}$$

Table 3. Comparison Among Different Formulations

Profile	K/K_{exp}			
	Ref. [2]	Ref. [5]	Ref. [6]	Ref. [7]
HE200A	1,36	0,93	0,96	0,96
HE200B	1,18	0,81	0,81	0,83
HE220A	2,11	1,46	1,84	1,77
HE220B	2,16	1,47	1,64	1,66
HE240A	2,70	1,84	2,21	2,13
HE240B	2,43	1,83	2,01	2,04
HE260A	1,97	1,32	1,69	1,58
HE260B	1,84	1,22	1,43	1,40
HE280A	1,75	1,18	1,48	1,44
HE280B	1,96	1,30	1,74	1,69
HE300A	1,49	0,99	1,26	1,21
HE300B	1,50	0,98	1,48	1,44
Mean	1,87	1,28	1,54	1,51
std. dev.	0,18	0,10	0,15	0,14

where t_{fb} is the beam flange thickness, t_{fc} is the column flange thickness, r_c is the web toe fillet and a_b is the throat thickness of the beam flange-to-column flange weld.

The buckling load, instead, is given by:

$$F'_{cwc} = F_{cwc}\left[\frac{1}{\bar{\lambda}}\left(1 - \frac{0,22}{\bar{\lambda}}\right)\right] \le F_{cwc} \tag{10}$$

where the slenderness $\bar{\lambda}$ is given by:

$$\bar{\lambda} = \left(\frac{b_{eff,cwc}\ t_{wc}\ f_{yw}}{F_{cr}}\right)^{1/2} \tag{11}$$

and

$$F_{cr} = \frac{\pi\ E\ t_{wc}^3}{3\left(1 - \nu^2\right)\ d_{wc}} \tag{12}$$

where υ is the Poisson ratio.

Another interpretation of the effective width of the column web in compression has been provided by Faella *et al.* [5] considering the column flange as a beam elastically supported by springs modelling the restraining action due to the column web. By means of this model, the effective width for buckling resistance is obtained by the following equation:

$$b^*_{eff,cwc} = t_{fb} + 2a_b\sqrt{2} + 2\left[\bar{\psi}\left(\frac{d_{wc}b_{fc}}{3t_{wc}t_{fc}}\right)^{1/4}\right]\left(t_{fc} + r_c\right) \tag{13}$$

where $\bar{\psi}$ represents a reduction coefficient depending on the actual restraining action provided by the column web

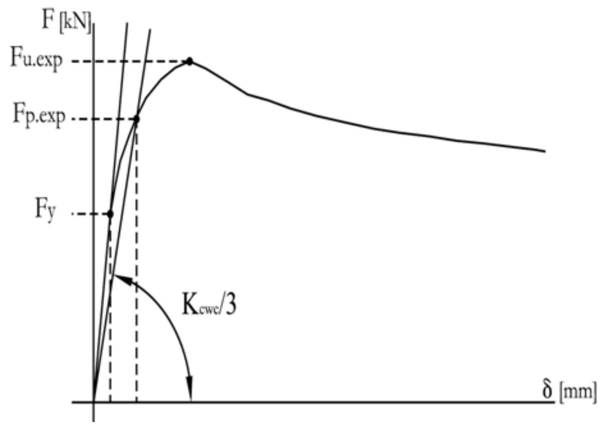

Fig. (11). Experimental resistance.

which was assumed equal to 0.75 on the basis of available experimental results.

An alternative approach has been proposed by Aribert *et al.* [7] and extended by Catenazzo and Piluso [10] to the prediction of the whole softening branch of the load-displacement curve. This approach consists in a yield line model representing the collapse mechanism exhibited during experimental tests. It is characterised by the interaction between flanges and web. In particular, the model provides the collapse load and the equilibrium curve of the mechanism describing the post-buckling behaviour of the column web in compression.

The accuracy of the above methods for evaluating the design resistance of the column web in compression, has been investigated by means of a comparison with the experimental results. The results of such comparison are given in the following tables. In particular, Table **4** contains the experimental values of the ultimate resistance, the plastic resistance corresponding to the knee of the experimental load-displacement curve, obtained by means of a secant stiffness given by $K_{cwc,s} = K_{cwc}/3$ (Fig. **11**), and the values of the slenderness λ. Table **5** provides the comparison with the predictions coming from Eurocode formulation, from Faella *et al.* approach, and also from the analytical results of the yield line approach [10]. In particular, the predicted ultimate resistance has been taken as the minimum value between crushing and buckling resistance.

Table 4. Experimental Results Concerning Resistance

Specimen	$F_{p.\,exp}$ [kN]	$F_{u.\,exp}$ [kN]	$\bar{\lambda}$
HE200A	628,6	812	0,63
HE200B	616,4	768,5	0,56
HE220A	450,3	454	0,82
HE220B	706,8	795,5	0,57
HE240A	546,5	546,5	0,88
HE240B	1007,1	1132,5	0,60
HE260A	616,8	620,5	0,88
HE260B	906	998,5	0,62
HE280A	681,3	684,5	0,80
HE280B	943,7	1020,5	0,69
HE300A	729,2	777,5	0,80
HE300B	987,7	1065,5	0,81

Moreover, for each tested specimen, Table **5** provides both the comparisons with the values of the ultimate resistance and with the values of the plastic resistance corresponding to the knee of the load-displacement curve.

In addition, the mean value and the standard deviation of the ratio between the theoretical prediction and the experimental value are also given.

It can be noted that the formulation suggested by Faella *et al.* leads, with respect to Eurocode 3, to a slight improvement in the prediction of the ultimate resistance of the column web in compression. A further and more significant improvement is obtained when the yield line approach is applied. However, it is important to underline that Faella *et al.* approach is aimed at the prediction of the knee of the load-displacement curve (design resistance) rather than the prediction of the ultimate resistance. Conversely, Eurocode 3 relationship was established for the ENV version and based on experimental ultimate resistances whose statistical interpretation was made according to Annex Z of ENV-1993.

Table 5. Comparison Among Different Formulation for Resistance Evaluation

Ref. [2]			
Specimen	F_{cwc} **[kN]**	$F_{cwc}/F_{p.exp}$	$F_{cwc}/F_{u.exp}$
HE200A	606,07	0,96	0,75
HE200B	533,08	0,86	0,69
HE220A	386,6	0,86	0,85
HE220B	585,51	0,83	0,74
HE240A	532,86	0,98	0,98
HE240B	775,3	0,77	0,68
HE260A	463,17	0,75	0,75
HE260B	669,08	0,74	0,67
HE280A	555,67	0,82	0,81
HE280B	677,41	0,72	0,66
HE300A	645,92	0,89	0,83
HE300B	860,04	0,87	0,81
mean		0,84	0,77
dev. std.		0,006	0,008

Table 5. Contd….

Ref. [5]			
Specimen	**F_{cwc} [kN]**	**$F_{cwc}/F_{p.exp}$**	**$F_{cwc}/F_{u.exp}$**
HE200A	606,07	0,96	0,75
HE200B	533,08	0,86	0,69
HE220A	390,67	0,87	0,86
HE220B	585,51	0,83	0,74
HE240A	539,76	0,99	0,99
HE240B	775,3	0,77	0,68
HE260A	475,01	0,77	0,77
HE260B	669,08	0,74	0,67
HE280A	578,61	0,85	0,85
HE280B	684,94	0,73	0,67
HE300A	663,4	0,91	0,85
HE300B	862,31	0,87	0,81
mean		0,85	0,78
dev. std.		0,006	0,009

Ref. [10]			
Specimen	**F_u [kN]**	**$F_u/F_{p.exp}$**	**$F_u/F_{u.exp}$**
HE200A	672	1,07	0,83
HE200B	669,5	1,09	0,87
HE220A	428	0,95	0,94
HE220B	679,5	0,96	0,85
HE240A	560	1,02	1,02
HE240B	-	-	-
HE260A	494,5	0,8	0,80
HE260B	729	0,8	0,73
HE280A	592,5	0,87	0,87
HE280B	718	0,76	0,70
HE300A	683	0,94	0,88
HE300B	915	0,93	0,86
mean	-	0,93	0,85
dev. std.	-	0,011	0,007

Plastic Deformation Capacity

The plastic deformation capacity of the column web in compression is defined following an approach similar to that used for determining the plastic deformation capacity of steel members, as suggested by Kuhlmann with reference to this joint component (Fig. **12**) [14].

With reference to (Fig. **11**), F_y is the elastic limit load, F_p is the plastic resistance defined consistently to Eurocode 3 and F_u is the ultimate resistance. The corresponding displacements are δ_y, δ_p, and δ_m, respectively. Finally, δ_u is the ultimate displacement defined as the maximum displacement before the resistance of the component falls down the plastic resistance.

The experimental results, according to the above definitions are given in Table **6**. From this table, it can be noted that, in some cases, the ultimate displacement, as previously defined, has not been attained during experimental tests.

Table 6. Experimental Values

Profile	**δ_y [mm]**	**δ_p [mm]**	**δ_m [mm]**	**δ_u [mm]**
HE200A	0,56	2,63	8,88	-
HE200B	0,65	2,21	8,61	-
HE220A	0,96	4,54	5,39	5,84
HE220B	0,98	4,68	8,70	18,33
HE240A	1,04	6,61	6,62	-
HE240B	1,57	8,15	14,91	24,48
HE260A	1,14	5,41	6,05	6,33
HE260B	0,91	4,85	8,23	15,33
HE280A	1,03	5,18	5,62	5,99
HE280B	1,31	6,08	9,01	13,04
HE300A	0,83	4,18	6,51	9,54
HE300B	1,07	5,19	8,67	13,16

Regarding the prediction of the plastic deformation capacity of the column web in compression, in the technical literature, to the best Author's knowledge, there is only one proposal by Beg *et al.* [15] who provided an empirical formulation for evaluating the displacement δ_m corresponding to the ultimate resistance. According to Beg *et al.* [15], the column web in compression is modelled by means of the simplified load-displacement relationship presented in Fig. (**13**).

The plastic deformation capacity is evaluated as the displacement level corresponding to the maximum resistance. Obviously, this is a safe side assumption, because the post-buckling behaviour is neglected.

The stable part of the deformation capacity δ_m can be determined by means of the following equation:

$$\delta_m = \varepsilon_u \, d \tag{14}$$

where d is the clear depth of the web, given by:

$$d = h_c - 2\,r_c - 2\,t_{fc} \tag{15}$$

and ε_u can be regarded as the non-dimensional deformation capacity, evaluated as a function of the axial force and of the web slenderness $d/(t_w\varepsilon)$, with $\varepsilon = \sqrt{235/f_y}$. In particular, Beg *et al*. suggest the following equations:

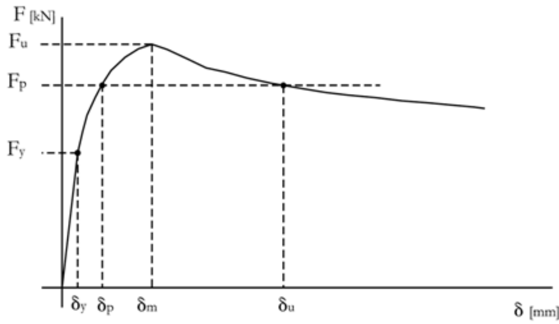

Fig. (12). Kuhlmann conventional procedure

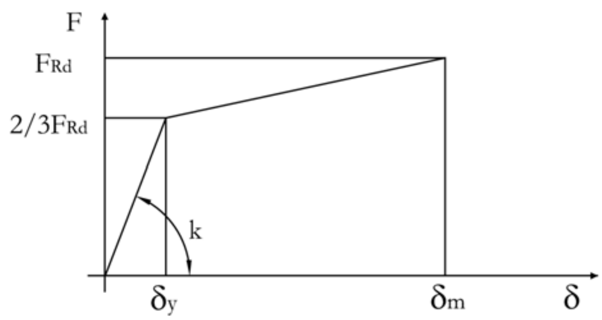

Fig. (13). Simplified force-displacement diagram.

$$\varepsilon_u = \begin{cases} 18{,}5 - 0{,}75 \cdot \dfrac{d_{wc}}{t_{wc}} \dfrac{1}{\varepsilon} & \text{for} \quad \dfrac{d_{wc}}{t_{wc}} \dfrac{1}{\varepsilon} < 20 \\ 5{,}7 - 0{,}11 \cdot \dfrac{d_{wc}}{t_{wc}} \dfrac{1}{\varepsilon} & \text{for} \quad 20 \le \dfrac{d_{wc}}{t_{wc}} \dfrac{1}{\varepsilon} < 33 \\ 2{,}07 & \text{for} \quad 33 \le \dfrac{d_{wc}}{t_{wc}} \dfrac{1}{\varepsilon} \end{cases} \qquad (16)$$

where ε_u is provided in percent (%). In addition, Beg *et al.* [15] provide also a similar relationship to be applied for columns subjected to axial forces.

Table **7** provides the δ_m values obtained by means of equation (16) for the tested specimens. In addition, the experimental values are also reported. Finally, the ratios between the value predicted by means of Beg *et al.* formulas and experimental value are also provided with the corresponding values of the mean and the standard deviation.

It can be noted that the mean value of this ratio is close to one with a very small standard deviation, so that it can be concluded that Beg *et al.* empirical formula leads to high degree of accuracy.

CONCLUSIONS

The results of an experimental program dealing with the ultimate behaviour of the column web in compression have been presented and discussed in this paper. The experimental results have been compared with the available formulations for predicting the initial stiffness, the resistance and the plastic deformation capacity.

Table 7. Comparison with Beg *et al.* Formulation

Specimen	$\varepsilon = \sqrt{\frac{235}{f_y}}$	$\frac{d}{t_w \varepsilon}$	ε_u	$\delta_{m,Beg}$ [mm]	$\delta_{m,exp}$ [mm]	$\delta_{m,Beg}/\delta_{m,exp}$
HE200A	0,8366	16,860	5,855	7,85	8,88	0,88
HE200B	0,9092	15,234	7,075	9,80	8,61	1,14
HE220A	0,8367	24,861	2,965	4,63	5,39	0,86
HE220B	0,9112	15,804	6,647	10,05	8,70	1,16
HE240A	0,7697	26,297	2,807	4,72	6,62	0,71
HE240B	0,8138	18,122	4,909	8,32	14,91	0,56
HE260A	0,8595	25,684	2,875	5,14	6,05	0,85
HE260B	0,9414	17,483	5,388	9,67	8,23	1,17
HE280A	0,8580	25,189	2,929	5,89	5,62	1,05
HE280B	0,9318	20,100	3,489	6,99	9,01	0,78
HE300A	0,8752	24,247	3,033	6,44	6,51	0,99
HE300B	0,8522	25,223	5,423	13,40	8,67	1,55
mean						0,97
dev. std.						0,06

Regarding the initial stiffness, the comparison between experimental and theoretical results shows that the formulation of Faella *et al.* [(5-9] leads to values closest to the experimental ones. In addition, the trend of Eurocode 3 formulation in overestimating the initial stiffness of such joint component has been confirmed by this experimental program.

Regarding the resistance, all the available formulations provide safe side results, because they are aimed at providing the plastic resistance, corresponding to the knee of the load-displacement curve, rather than the ultimate resistance.

Finally, regarding the plastic deformation capacity, it has been pointed out that the empirical formula proposed by Beg *et al.* is able to predict with high degree of accuracy the displacement corresponding to the maximum load carrying capacity, but post-buckling behaviour is neglected.

The forthcoming activity will be aimed at the experimental investigation of specimens including the influence of the column axial force. In addition, the setting up of a FEM model will allow the development of a parametric analysis.

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