DUCTILITY REQUIREMENTS FOR THE DESIGN OF BOLTED SHEAR CONNECTIONS

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ABSTRACT

The resistance of shear bolted connections is traditionally evaluated by considering an equal distribution of internal forces amongst the bolts. In fact, such an assumption may only be seen as the result of a plastic redistribution of the internal forces, what requires shear ductility in the vicinity of the bolts. In the present paper, ductility requirements are proposed. They have been derived by the first author during a two-years stay at Liège University. For more details about this work; the interested reader is requested to refer to the Henriques thesis [5].

SHEAR BOLTED CONNECTIONS

A connection can be classified as Shear Bolted Connection when the forces transferred between the elements induce pure shear in the bolts. Two types of shear connections, also called lap connections, may be found: single and double overlap connections. The difference consists in the number of shear planes that cross the bolt shanks.
In Shear Bolted Connections, two different elements may be distinguished: connectors (bolts) and connected elements (plates). The term plate is used to refer to column flanges, beam flanges, beam webs, splice plates, etc.
When a bolted connection is submitted to shear, forces are transferred from one plate to the other (others) by plate-to-bolt contact. Neglecting the small friction developed between plates and negligible bending of the bolt, four different resistance and deformation modes should be considered:

- Bearing of the plate and/or bolt;
- Shear in the plates;
- Tension in the plates;
- Shear in the bolt shanks.

From these, the behaviour of a shear bolted connection can be defined by the response of two different parts: bolt zone, where bearing and shear forces develop; and the plate between holes where direct forces develop in the plate. The work presented in this article focuses on the bolt zone; so the failure of the connection by excess of tension in the connected pates is here not considered.

In these types of connections, the load to be transferred between the plates is distributed non-uniformly amongst the bolt-rows (Figure 1-a), Ju et al. [8]. If sufficient deformation is provided around each connector, a full plastic redistribution of forces may be noticed, otherwise failure is reached by lack of ductility and the maximum external force to be transferred is lower than the full plastic distribution. Schematically, the different stages of forces distribution in a shear bolted connection may be represented as in Figure 1.

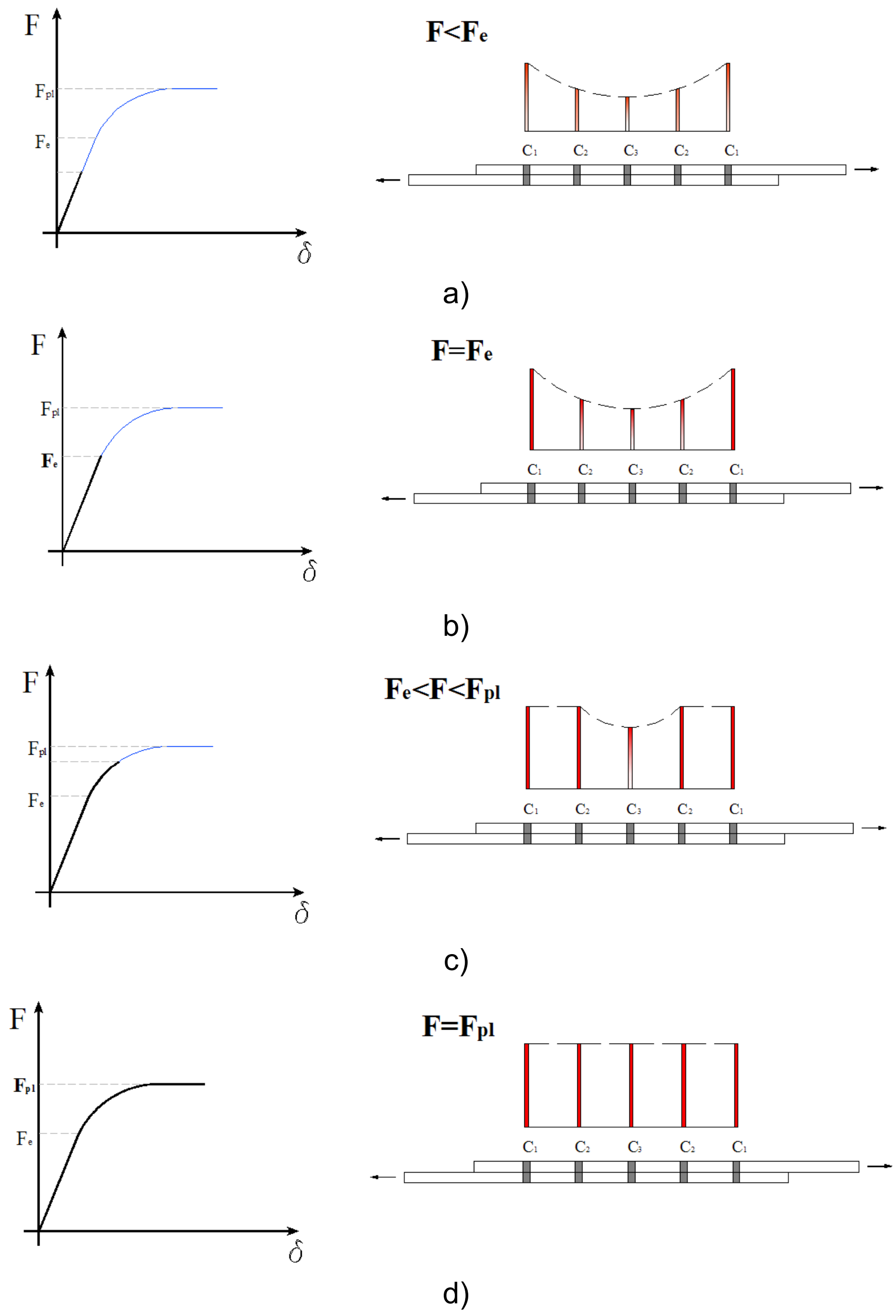


Figure 1 – a) None of the bolt rows yield; b) outsider bolt rows yield (elastic resistance of the connection); c) the following bolt rows yield; d) the remaining bolt row yield (full plastic resistance of the connection).

In the same study, Ju el al. [8] showed that in the nonlinear range the maximum load achieved by the connection is almost linearly proportional to the bolt number arranged in the connection. In part 1-8 of Eurocode 3 [1], a full plastic distribution of forces can be assumed as long as the connection length is limited.

Pietrapetrosa el al. [13] approached the subject by only considering fitted bolts. Their study showed that, inside the limits given by the code and by practical guidance, sufficient ductility to achieve a full plastic distribution of internal forces is available. However, the common practice is the use of non fitted bolts and the presence of imperfections is also a reality. Consequently, the lack-of-fit will increase the demands of ductility as some bolts bear before the others, as verified by Wald et al. [14]. They showed that for certain values of gap in some bolt rows, failure was first attained in the extreme bolts and therefore a full plastic resistance was not reached.

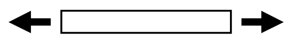
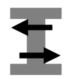
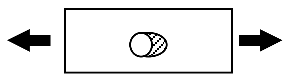
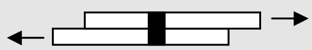
EUROCODE 3 DESIGN PROCEDURE FOR BOLTED CONNECTIONS IN SHEAR

According to the classification system for joints in Eurocode 3 part 1.8 [1], the connections considered here belong to category A: Shear Bolted Connections – Bearing Type. These ones resist by transferring forces through plate/bolt contact and bolt shearing. Non preloaded bolts are used and the small friction resistance between the contact surfaces is neglected.

Part 1.8 of Eurocode 3 [1] is dedicated to the design of joints in steel structures; it prescribes the so-called component approach for the evaluation of the mechanical properties of the joints. The analysis of shear bolted connections is not specifically treated. But the code gives recommendations for the evaluation of the stiffness and resistance properties of several individual components; the engineer has then to identify the involved components and to assemble them so as to finally predict the response of the whole connection.

Amongst the individual components presented in Table 6.1 of EC 3 part 1.8 [1], the following ones should be here considered: bolt in shear, plate/bolt in bearing and plate in tension. Furthermore it is then assumed that the failure mode of a bolt zone (i.e. a zone where a shear force is locally transferred from one plate to another) is associated to that of the weakest component. Through this procedure, the resistance and stiffness properties of the bolt zone may so be evaluated; however, no information is given for the deformation capacity. Table 1 summarises this procedure.

Table 1 – Eurocode 3 expressions to evaluate the characteristic resistance (R_c) and the stiffness (S_c) of the basic components.

		S_c	R_c
Plate in tension		$S_{pl} = EA/p_b$	$R_{pl}=min(A\ f_y;\ 0,9\ A_{net}\ f_u)$
Bolt in shear		$S_b = 8\ d^2\ f_{ub}/d_{M16}$	$R_b = \alpha_v f_{ub}\ A_b$
Plate in bearing		$S_p = 12\ k_b\ k_t\ d\ f_u$	$R_p = k_1 \alpha_b\ f_u\ d\ t$
Equivalent component		$S_{eq} = (S_b^{-1} + S_{p1}^{-1} + S_{p2}^{-1})^{-1}$	$R_{eq} = min(R_b, R_{p1}, R_{p2})$

E Young Modulus	d diameter of the bolt
A gross area of the plate	d_0 diameter of the bolt hole
A_{net} net area of the plate	d_{M16} nominal diameter of a M16 bolt
p_b pitch distance (// to load transfer)	e_2 edge distance ($\perp$ to load transfer)
e_b end distance (// to load transfer)	p_2 pitch distance ($\perp$ to load transfer)
f_y yield strength of the plate	$k_b = min(k_{b1};k_{b2})$
f_u ultimate strength of the plate	$k_{b1} = 0,25\ e_b/d + 0,5$ but $k_{b1} \leq 1,25$
t thickness of the plate	$k_{b2} = 0,25\ p_b/d + 0,375$ but $k_{b2} \leq 1,25$
A_b shear area of the bolt (nominal or stress area)	$k_t = 1,5\ t\ /\ d_{M16}$ but $k_t < 2,5$
f_{ub} ultimate strength of the bolt	$\alpha_v = 0,5$ or $0,6$
	$\alpha_b = min(e_b/3d_0;\ p_b/3d_0 - 0,25\ ;\ f_{ub}/f_u;\ 1,0)$
	$k_1 = min(2,8\ e_2/d_0 - 1,7;\ 1,4\ p_2/d_0 - 1,7;\ 2,5)$

The application of the component method to evaluate the response of the whole shear bolted connection (Figure 2-a) requires now to consider the mechanical model shown in Figure 2-b. Here, each individual component is modelled through extensional springs. In the bolt zone, one observes that three springs act in series and therefore their behaviour may be assembled into an equivalent one (describing the bolt zone response). Thus, a simplified model is obtained where the components at the bolt zone are represented by a so-called equivalent bolt zone component, Figure 2-c.

For shear connections with more than one bolt zone “in length”, two recommendations given by the code are relevant. One is related to the resistance of connections with a limited number of bolt zones “in length”; this one is obtained as follows:

$$\begin{cases} if\ \ F_{v,Rd,i} \geq F_{b,Rd,i}\ \ \forall i \Rightarrow F_{Rd} = \sum F_{b,Rd,i} \\ if\ \ not\ \ F_{Rd} = n \min\left\{F_{Rd,i}\right\}\quad with\, F_{Rd,i} = \min(F_{v,Rd,i};F_{b,Rd,i}) \end{cases} \tag{1}$$

Where:

- F_{Rd} is the resistance of the whole connection;
- n is the number of bolt zones “in length;
- i indicates the bolt zone number;
- $F_{b,Rd,i}$ and $F_{v,Rd,i}$ are respectively the bearing and shear resistances of bolt zone I.

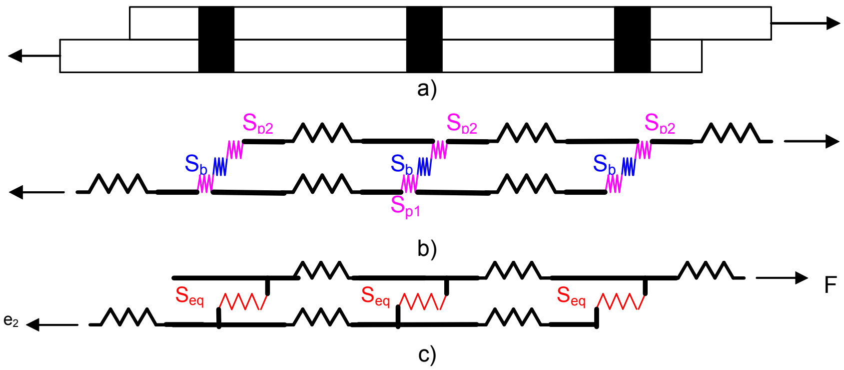


Figure 2 –a) Shear connection with three bolts; b) Real mechanical model; c) Simplified mechanical model.

The second rule is related to long joints where the shear resistance should be reduced if the connection length (L_j) exceeds 15d. In this case the following reduction factor should be applied to the connection resistance initially evaluated through Formula (1):

$$\beta_{Lf} = 1 - \frac{L_j - 15d}{200d} \; but \; 0.75 \leq \beta_{LF} \leq 1.0 \qquad (2)$$

EVALUATION OF IMPERFECTIONS/LACK OF FIT

As in every construction type, imperfections related to fabrication have to be considered in steel structures. As far as the response of shear connections is concerned, the discrepancy between the nominal and the real values of bolt diameters, hole diameters and positions (pitches and end distances) may affect the behaviour of the connections as the imperfections will lead to a non simultaneous transfer of forces between the bolts, as it would be the case for "perfect" connections (for instance, connections with fitted bolts).

Values of tolerances are given in European Standard for the Execution of Steel Structures and Aluminium Structures, pre-EN 1090-2 [2], in ISO/DIS 4759-1 [6] and in ISO286-2 [7]. Based on these values the lack of fit in bolted connection may be quantified. However, due to the multiple parameters involved, this task is complex. In order to simplify, and have in consideration the evaluation of the maximum required deformation in a bolt zone, some assumptions are established in order to get the "worst situation" (i.e. the one for which the highest demand in terms of ductility is required from a bolt zone):

- Possibility to have different values of real hole diameters in every plate;
- Possibility to have different hole deviations in every plate, and consequently different values of pitch and end distances in every plate;
- The bolt initially in contact with the plates is one of the outer bolts (henceforth this bolt will be designated as FBW [First Bolt Working], while the notation RB [Rest of the Bolts] will be used for all the others), this allows to maximise the requested deformation capacity for the FWB bolts;

- ➢ The “worst situation” results from the combination of all these possibilities. Even if this is not the more realistic pattern, it could anyway happen; and for sure it is the one leading to the highest request in terms of ductility.

Using the standards values for tolerances and the previous assumptions, several connection layouts may be drawn to identify the “worst case”, as illustrated in Figure 3.

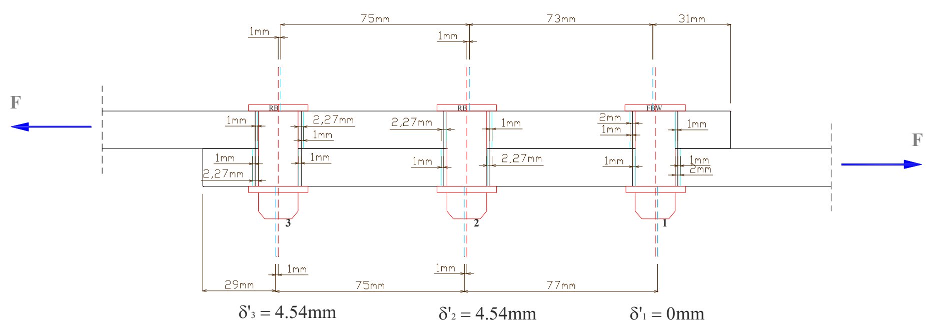


Figure 3 – Connection layout considering the presence of imperfections.

Analysing several situations, as different bolt diameters, one obtains the gaps to be considered in a bolted connection which follows the previous assumptions. Table 2 presents maximum gaps that may observed in a connection layout according to the bolt diameter used.

Table 2 – Gaps in bolted connections

Bolts	**2, 3 or more Bolts**		
	FBW gap	**RB gap**	**Max. Gap**
M12-M14	0.00	3.08	3.08
M16	0.00	4.54	4.54
M18-M24	0.00	4.66	4.66
M27	1.00	5.66	4.66
over	1.00	5.78	4.78

The main factors which distinguish the different values obtained are the hole clearance and the tolerances allowed by standards.

RESPONSE OF THE INDIVIDUAL COMPONENTS

As mentioned before, two different individual components interact in the bolt zone: the bolt in shear and the plate/bolt in bearing. And in order to analyze shear bolted connections, the behaviour of these components has first to be predicted. Hereafter, code recommendations and results of former investigations are used to achieve it.

Bolt in Shear

In Moscow, Karmalin et al. [10] have performed numerous experimental tests on bolts in shear. Resistance, stiffness and deformation capacity of bolts subjected to shear have been measured for M16, M20 and M24 with grades 5.8, 8.8 and also for bolts with a minimum tensile strength equal to 1100MPa (high strength). The tested specimens consisted of single bolted connections with two-shear planes.

In Table 3 are presented the test results.

Table 3 – Moscow test results

Bolts Grade	$R_{u,b}$ [kN]			$\delta_{u,b}$ [mm]		
	M16	M20	M24	M16	M20	M24
5.8	63 – 72	97 – 110	137 – 150	2.9 – 3.4	3.4 – 3.8	4.1 – 4.4
8.8	81 – 93	124 - 141	175 – 193	2.2 – 2.5	2.6 – 3.0	3.1 – 3.5
High-strength	126 – 150	195 - 220	275 - 308	1.6 – 2.0	1.8 – 2.2	2.1 – 2.7

Based on the EC3 part 1.8 [1] expressions (see Table 1) and on these experimental results, expressions to determine the ultimate deformation capacity, ultimate resistance and strain-hardening stiffness of bolts in shear have been derived. With the aim to refer explicitly to Eurocodes, the here-above listed parameters are expressed as a function of the initial stiffness (S_b) and of the nominal resistance (R_b), the values of which are given in Eurocode 3 (see Table 1). Table 4 presents these expressions.

Table 4 – Ultimate resistance, ultimate deformation capacity and strain-hardening stiffness for the "bolt in shear" component.

Bolts Grade	$\delta_{u,b}$			$S_{st,b}$	$R_{u,b}$
	M16	M20	M24		
5.8	4.7 R_b/S_b	5.5 R_b/S_b	6.7 R_b/S_b	$S_b/2.5$	1.58 R_b
8.8	3.0 R_b/S_b	3.5 R_b/S_b	4.2 R_b/S_b	$S_b/7.0$	1.05 R_b
High-strength	2.6 R_b/S_b	2.9 R_b/S_b	3.4 R_b/S_b	$S_b/1.5$	1.44 R_b

Plate/bolt in bearing

During the research period, numerical works have been achieved. The main goal was to develop a numerical model for the simulation of bearing phenomena. Bearing problems are complex as they deal with contact between two bodies consequently the number of tools available to reproduce the contact problems is reduced. In the present investigations, the Lagamine code [11], software developed at the ArGEnCo Department at the University of Liège, has been used.

As it had not been planned to carry out experimental tests in Liège, available tests made in others universities are used to calibrate the numerical model. Tests made on shear bolted connections at the University of Ljubljana [12] and at the Technical University of Delft are used [4].

One of the main objectives was to be able to model bearing failure; this goal was not completely achieved at the end of the research period. Further related investigations are therefore still needed.
As a consequence, the characterization of the plate/bolt in bearing behaviour is based hereafter on the existent knowledge: the elastic stiffness and the nominal resistance re determined using code recommendations, see Table 1, while, for the other parameters (strain-hardening stiffness, ultimate resistance and ultimate deformation), expressions from previous works ([5], [9] and [13]) are used.

$$S_{sb,p,b} = \frac{S_{p,b}}{40} \quad (3)$$

$$R_{u,p,b} = 1.25 R_{p,b} \quad (4)$$

$$\delta_{u,p,b} = 11 \frac{R_{p,b}}{S_{p,b}} \quad (5)$$

Plate in tension

Although present research work focuses on the bolt zone and on its capability to redistribute forces, the deformability of a plate in tension has an important influence on the distribution of forces amongst the bolts. The stiffness of the plate in tension has therefore to be predicted too; an expression is provided in Table 1.

Assembly of the basic components

In this part, the individual basic components are assembled with the objective to derive the available ductility of the equivalent bolt zone components and the ductility required to allow a full redistribution of internal forces in shear bolted connections.

Available deformation capacity of the equivalent bolt zone component

The deformation available in the equivalent bolt zone component is obtained through the “association” of the two basic components: the bolt in shear and the plate/bolt in bearing. Each basic component is characterized and the deformation capacity evaluated according to the knowledge presented in the previous sections. Subsequently an assembly is done according to their resistance and deformability. The complete behaviour of the equivalent bolt zone component is then obtained.

The derivation of formulae to determine the available deformation capacity of the equivalent component depends on several factors such as: single or double overlap connections, plates with equal or different behaviour (different thickness, different steel properties), and the relation between the resistances of the individual components. So, many cases may be obtained. In Figure 4 is exemplified one of these cases and in Table 5 are presented a list of expressions for several common cases.

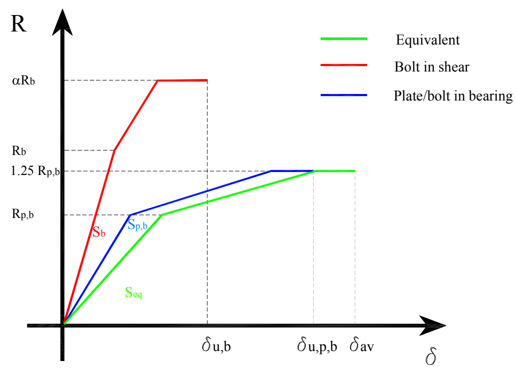


Figure 4 – Assemblage of the individual components behaviour.

Table 5 – Derived expressions to determine the available deformation capacity.

Plates with equal mechanical and geometrical properties	
Single Overlap Connections	*Double Overlap Connections*
Case: $R_b > 1.25R_{p,b}$	
$\delta_{av} = \frac{R_{eq}}{S_{eq}} + 2*(\delta_{u,p,b} - \frac{R_{eq}}{S_{p,b}}) + \frac{0.25R_{eq}}{S_b}$	$\delta_{av} = \frac{R_{eq}}{S_{eq}} + (\delta_{u,p,b} - \frac{R_{eq}}{S_{p,b}}) + \frac{0.125R_{eq}}{S_b} + \frac{0.125R_{eq}}{S_{p,b}}$
Case: $R_{p,b} > \alpha R_b$	
$\delta_{av} = \frac{R_{eq}}{S_{eq}} + (\delta_{u,b} - \frac{R_{eq}}{S_b}) + 2*\frac{(\alpha-1)R_{eq}}{S_{p,b}}$	$\delta_{av} = \frac{R_{eq}}{S_{eq}} + (\delta_{u,b} - \frac{R_{eq}/2}{S_b}) + \frac{(\alpha-1)R_{eq}}{S_{p,b}} + \frac{(\alpha-1)R_{eq}/2}{S_{p,b}}$
Case: $1 \le R_b/R_{p,b} \le 1.25$	
$\delta_{av} = \frac{R_{eq}}{S_{eq}} + 2*(\delta_{u,p,b} - \frac{R_{eq}}{S_{p,b}}) + \frac{R_{e,b} - R_{eq}}{S_b} + \frac{(1.25R_{eq} - R_{e,b})}{S_b/\beta}$	$\delta_{av} = \frac{R_{eq}}{S_{eq}} + (\delta_{u,p,b} - \frac{R_{eq}}{S_{p,b}}) + \frac{0.125R_{eq}}{S_{p,b}} + \frac{R_{e,b} - 0.5R_{eq}}{S_b} + \frac{(0.625R_{eq} - R_{e,b})}{S_b/\beta}$
Case: $1 \le R_{p,b}/R_b \le \alpha$	
$\delta_{av} = \frac{R_{eq}}{S_{eq}} + (\delta_{u,b} - \frac{R_{eq}}{S_b}) + 2*\frac{R_{p,b} - R_{eq}}{S_{p,b}} + 2*\frac{(1.25R_{eq} - R_{p,b})}{S_{p,b}/40}$	$\delta_{av} = \frac{R_{eq}}{S_{eq}} + (\delta_{u,b} - \frac{R_{eq}/2}{S_b}) + \frac{R_{p,b} - 2R_{eq}}{S_{p,b}} + \frac{(2\alpha R_{eq} - R_{p,b})}{S_{p,b}/40} + \frac{\alpha * R_{eq}/2}{S_{p,b}}$

Required deformation capacity in actual shear bolted connections

The required deformation capacity is the deformation which should be reached in the most loaded bolt zone in order to reach a full plastic redistribution of efforts in the connection.

In the work done by Pietrapertosa el al. [13] expressions to determine the required deformation of the equivalent bolt zone component for fitted bolts have been proposed. Based in this study, similar expressions for actual connections, taken into account the presence of imperfections, are proposed.

The derived expressions should consider the most demanding situation that has been assumed before; i.e. the case where one of the extreme bolts is in contact while the others are not. Several cases have been analysed and it has been concluded that the most demanding case is obtained when the middle bolt zone (or middle bolt zones in the case of even number of bolt rows) is (are) the last one(s) to reach its (their) maximum resistance. Figure 5 shows the deformed shape and the distribution of internal forces for a connection with 5 bolt rows.

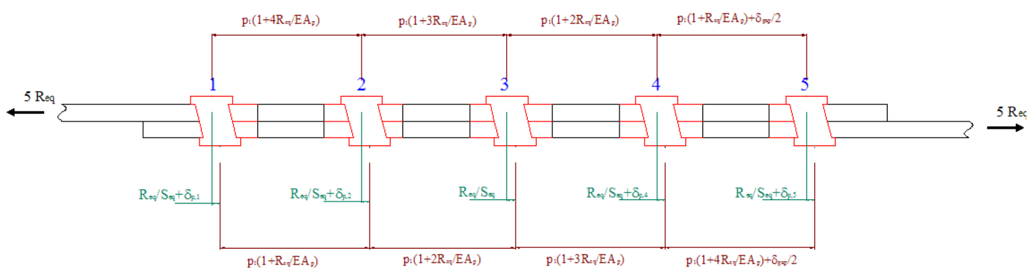


Figure 5 – Connection with 5 bolt rows.

This analysis has been extended to other cases (different number of bolt rows) and the following general expression has been obtained:

$$\delta_{req} = R_{eq}\left(\frac{1}{S_{eq}} + \rho\frac{p_1}{EA_p}\right) + \delta_{gap}$$

with

$$\rho = \sum_{i=1}^{n_1/2}(n_1 - 2i) \quad \text{for an even value of } n$$

$$\rho = \sum_{i=1}^{(n_1-1)/2}(n_1 - 2i) \quad \text{for an odd value of } n \qquad (6)$$

Additionally, a numerical model based on the use of the Liège home-made nonlinear FEM software FINELG [3] allowed validating all the analytical results.

Ductility requirements for shear bolted connections

In order to determine ductility requirements that a connection should satisfy so as to ensure a full plastic redistribution of the internal forces amongst the bolt zones, reference will obviously be made to the expressions derived before for the available and required ductility in bolt zone components; hence, such ductility requirements are for sure dependent on all the geometrical and mechanical parameters that influence the two previously mentioned values of ductility:

- Steel grade of the plate;
- Bolt grade;
- Geometrical properties of the connection [t, b, e_1, e_2, p_1, p_2, d, d_0];
- Number of bolt rows (n_1 – in the direction of loading) and number of bolt lines (n_2 – in the perpendicular direction of loading).

The ductility criterion which is expressed below and which constitutes the main outcome of the study is based on an intensive parametrical study where all the above-listed geometrical and mechanical parameters have been considered, but for single overlap connections only (what is not really restrictive). As mentioned before, situations where tension plate failure is relevant have been omitted.

In order to define this criterion, two fundamental parameters defined below have been identified. Figure 6 illustrates the basis of the criterion.

The parameter on the vertical axis represents the ratio between the available and the required deformation capacities. This ratio reflects the sufficient or insufficient ductility exhibited by the equivalent bolt zone component. The second fundamental parameter represents the ratio between the nominal resistance of the plate/bolt in bearing component and the ultimate resistance of the bolt in shear component. These two parameters embody all the important mechanical and geometrical parameters listed before.

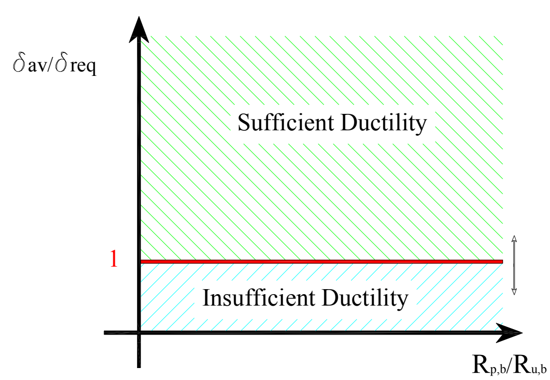


Figure 6 – Two fundamental parameters

Figure 7 presents the results of the parametrical analysis in which the following variation of the basic parameters have been considered:

- Steel grade: S235 and S355;
- Bolt diameters: M16, M20 and M24;
- Spacing, end and edge distances: max and min of e_1, e_2, p_1 and p_2;

- Width of the plates: max and min values e_2 and p_2 as well as max and min values of *b* taken into account;
- Thickness of the plate: the variation of *t* is made in order to cover the whole ranges of $R_{p,b}/R_{u,b}$;
- Finally, the number of bolt rows and lines varies: n_1, from 2 to 10, and n_2, from 1 to 5.

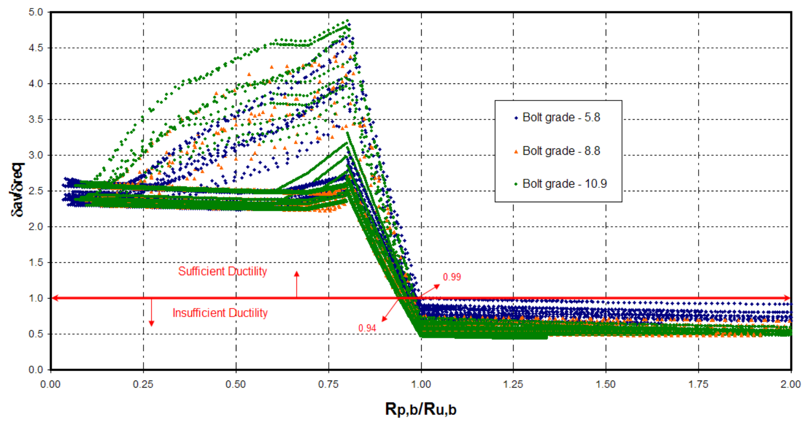


Figure 7 – Parametric analysis results.

One can observe that the variation of the fundamental parameter $R_{p,b}/R_{u,b}$, close to the boundary between sufficient and insufficient ductility ($\delta_{av}/\delta_{req}=1$) is small, from 0.94 to 0.99. So, a safe and simplified ductility criterion may be suggested as follows:

$$\begin{cases} If\ \dfrac{R_{p,b}}{R_{u,b}} \le 0.94 \Rightarrow F_{r,c} = n_1 n_2 R_{eq} \left(plastic\ distribution\ of\ internal\ forces\ allowed\right) \\ If\ \dfrac{R_{p,b}}{R_{u,b}} > 0.94 \Rightarrow F_{r,c} < n_1 n_2 R_{eq} \left(plastic\ distribution\ of\ internal\ forces\ not\ allowed\right) \end{cases} \qquad (7)$$

In order to apply the criterion, some practical cases have been considered and the results have been compared with the present Eurocode 3 rules. This comparison considered two situations, one where the criterion is verified and another where it is not, as shown in Figure 8.

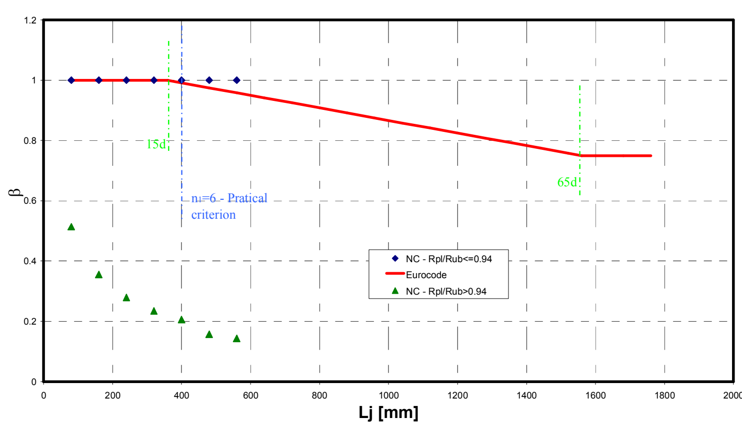


Figure 8 – Comparison with the Eurocode 3 criterion.

CONCLUSIONS

The present work proposes a criterion to check whether sufficient ductility for a full plastic redistribution of internal forces may be contemplated in actual shear connections with non preloaded bolts. It is based on the presence of imperfections in the connection layout which can lead to situations where some bolts bear before the others.

All the aspects inherent to shear bolted connections have been approached: the evaluation of imperfections according to the standards for tolerances; the characterization of the individual component response; the derivation of expressions to determine the available deformation capacity in the bolt zone component; the required deformation in the bolt zone component for a full plastic redistribution of forces.

The application of the proposed criterion showed considerable differences between the code criterion and the proposed one. This fact shows that imperfections may have a relevant effect in the connection behaviour if the bolt in shear component is the "weakest". Actually, in these cases the transferred force is considerably smaller than the one determined according to the code provisions, as observed in Figure 8. This situation should then be further investigated in future works. At the same time the evaluation of the imperfections in the connection layout should be better analysed. The values here obtained (based on the "worst" layout of imperfections) seem to be too severe for the case of "weak" bolts, as seen in Figure 8.

REFERENCES

[1] CEN, European Committee for Standardization (2005) – *"Eurocode 3: Design of steel structures – Part 1-8: Design of joints"*, Final Draft prEN 1993-1-8, December 2003, Brussels, Belgium.

[2] CEN, European Committee for Standardization (2005) – *"Execution of steel structures and aluminium structures – Part 2: Technical requirements for the execution of steel structures – Stage 34"*, Draft prEN 1090-2, April 2005, Brussels, Belgium.

[3] FineLg User's manual, V9.0 (2004) – University of Liège (M&S) / Design office Greisch, Belgium.

[4] Freitas, S. T. (2005) – *"Experimental research project on bolted connections in bearing for high strength steel"*, Final report of the framework of the Socrates-Erasmus program, June 2005.

[5] Henriques, J. (2008) – “Ductility requirements in shear bolted connections”, Master Thesis, Coimbra, Portugal, January 21.

[6] ISO (1999) – *“Tolérances des éléments de fixation – Partie 1: Vis, goujons et écrous – Grades A, B et C”*, Projet de Norme Internationale, ISO/DIS 4759-1.2.

[7] ISO (2006) – *“Système ISO de tolérance et d’ajustements – Partie 2: Tables des degrés de tolérance normalisés et écarts limites des alésages et des arbres” – Norme Base”*, ISO 286/DIN 267 – 2.

[8] Ju, S.-H.; Fan, C.-Y.; Wu, G. H. (2003) – *“Three-dimensional finite elements of steel bolted connections”*, Engineering Structures 26, 2004, 403-413.

[9] Jaspart, J. P. (1991) – *“Etude de la semi-rigidité des nœuds pouter-colonne et son influence sur la résistance et la stabilité des ossatures en acier”*, Ph. D. Thesis, M&S Departement, Liège University, Belgium.

[10] Karmalin, V. V. and Pavlov A. B. (1989) – *“Load capacity and deformability of bearing and friction-bearing connections”*, Proceedings of the International Colloqquium on Bolted and Special Structural Connections, USSR, Moscow, May 15-20, pp. 52-60.

[11] LAGAMINE code, Finite Element Software, ArGEnCO Department, Liège University, Belgium.

[12] Moze, P.; Beg, D.; Lopatic, J. (2006) – *“Ductility and Strength of bolted connections made of high strength steel”*, International conference in metal structures “*Steel – A new and traditional material for buildings*”, Poiana Brasov, Romania, Sptember 20-22, pp. 323-330.

[13] Pietrapertosa C.; Piraprez E.; Jaspart J.P. (2004) – *“Ductility requirements in shear bolted connections”*, ECCS/AISC Workshop: Connections in steel structures V: Behaviour, Strength and Design, Amsterdam, The Netherlands, June 3-4, pp. 335-345.

[14] Wald, F.; Sokol, Z.; Moal, M.; Mazura, V.; Muzeau, J. P. (2002) – “Stiffness of cover plate connections with slotted holes”, Eurosteel: Third European conference on Steel Structures, Coimbra, Portugal, pp. 1007-1016.