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High strength steel tension splices with one or two bolts

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ABSTRACT

The paper presents experimental research of 38 tension splices with bolts in double shear. Steel grade S690 was used. The objective of the research was to determine ductility and resistance of such connections. The experimental resistance is compared to Eurocode provisions and a new bearing resistance formula is developed. The reliability assessment is given according to EN 1990, Annex D.

1. Introduction

In constructional practice across Europe, structural high strength steels (HSSs) are considered as steels with specified minimum yield strength equal to or higher than 420 MPa. Such steels can reach a yield stress up to 1300 MPa. For successful application of higher strength steels, design and fabrication standards need to be accepted by the authorities responsible for safety and by industry. Eurocode 3, standard for the design of steel structures, is divided into several parts. Part EN 1993-1-1 [1] includes steel grades up to S460. Additional rules for steel grades up to S700 are presented in EN 1993-1-12 [2] and were the lattermost addition to Eurocode 3. HSSs are as a structural material competitive, mostly due to lower overall economy and structural efficiency. Nevertheless, HSSs undoubtedly have lower ductility than mild steels in terms of standard measures of ductility, such as ultimate-to-yield strength ratio, uniform strain and elongation at fracture. The values for high strength steels S690 are typically much lower than those for mild steels: ultimate-to-yield strength ratio $f_u/f_y = 1.05$, maximum uniform strain $\varepsilon_u = 0.05$ and elongation after fracture $A^c = 15\%$. Another measure of ductility is reduction of area at fracture Z, which is obtained by comparing the cross-sectional area after fracture. For high strength steel this measure indicates that the fractures can be characterized as ductile with large inelastic deformations. Local ductility in terms of plastic deformations is especially important in tension splices with bolts in shear, where contacts between bolts and plates are in general not established simultaneously for all bolts. For that reason, the local ductility of the connection has to be sufficient in order to ensure such bolthole elongation, that provides the transfer of loading through all bolts. Otherwise, the stress concentration would cause rupture of the steel plate or shear fracture of the bolt. Such inelastic behaviour is hidden in most of the nominally elastic resistance checks of steel structures, which is why sufficient local ductility has to be ensured. Tension splices are often referred to as shear connections. This term is adopted in what follows.

In previous researches on bolted shear connections made of higher steel grades, one- and two-bolt connections were studied. Kim and Yura [3] investigated shear connections with one or two bolts placed parallel to the loading. Among others they used steel with nominal yield strength $f_y = 430$ N/mm^2 with ratio $f_u/f_y = 1.13$. The experimental resistance was compared to bearing resistance according to American AISC standard and to Eurocode standard; the latter was found to be conservative. Their work was continued by Aalberg and Larsen [4,5]. They duplicated the tests by Kim and Yura, using steel grades S690 and S1100. The value of ultimate tensile-to-yield ratio was equal to $f_u/f_y = 1.05$ for both steel grades. The local ductility of connections was not decreased due to the low f_u/f_y ratio. Puthli and Fleisher [6] focused on the shear connections made of steel grade S460 ($f_u/f_y = 1.23$) with two bolts placed perpendicular to loading. They compared experimental resistances to resistance according to EN 1993-1-8 [7]. The focus was set on minimum end and edge distances and the result was the suggestion for the reduction of minimum distances and a modified bearing resistance formula. More recent research was published by Rex and Easterling [8]. The research on the behaviour of a bolt bearing on a single plate was part of a larger investigation of the behaviour of partially restrained steel and composite connections. A 6.5 mm thick plate of different high strength steel grades (ultimate strength from 665 to 752 MPa) was tested on bolt bearing. Due to small plate thickness and large end distance e_1, several curling failures were observed.

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Statistical evaluation and reliability assessment of proposed design resistances were not carried out by any of the authors. Može, Beg and Lopatič [9] previously studied net cross-section failures of members in tension made of steel S690. The study included 43 tests. The result was design net cross-section resistance of symmetrically and unsymmetrically connected members in tension, which was determined by statistical evaluation. All failures showed local ductility by necking of net cross-section or by bolt hole elongation. Even a small reduction in gross section (e.g. 10%) did not result in its yielding and consequently to large plastic deformations of the whole member; thus global ductility was limited. According to these results the proposed design net cross-section resistance in EN 1993-1-12 was changed to be equal to that in EN 1993-1-1:

$$F_{t,Rd} = \frac{0.9 A_{\mathrm{net}} f_u}{\gamma_{M2}}. \tag{1}$$

In EN 1993-1-8 the design bearing resistance per bolt is defined as:

$$F_{t,Rd} = \frac{k_1 \alpha_b f_u \mathrm{d}t}{\gamma_{M2}} \tag{2}$$

where f_u is nominal ultimate tensile strength of the plate, d is bolt diameter, t is plate thickness and partial factor γ_{M2} with the recommended value of 1.25. Parameters α_b and k_1 are defined as follows and take into account mainly geometrical parameters:

- in the direction of load transfer

$$\alpha_b = \min\left(\alpha_d; \frac{f_{ub}}{f_u}; 1\right) \tag{3}$$

$$\alpha_d = \frac{e_1}{3d_0} \quad \text{for end bolts} \tag{4}$$

$$\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4} \quad \text{for inner bolts} \tag{5}$$

- perpendicular to the direction of load transfer

$$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7;\ 1.4\frac{p_2}{d_0} - 1.7; 2.5\right) \quad \text{for edge bolts} \tag{6}$$

$$k_1 = \min\left(1.4\frac{p_2}{d_0} - 1.7;\ 2.5\right) \quad \text{for inner bolts.} \tag{7}$$

In the previous definitions f_{ub} is nominal ultimate tensile strength of the bolt and d_0 is a bolt hole diameter. End distance e_1, edge distance e_2 and pitches p_1 and p_2 are defined in Fig. 1. Maximum and minimum end and edge distances and pitches are also limited in EN 1993-1-8. Maximum dimensions are related to the exposure of steel to corrosion and local buckling of plate in compression. Minimum dimensions set lower limit for the resistance and are indirectly related also to execution tolerances. Minimum distances are: end distance $e_1 \geq 1.2d_0$, edge distance $e_2 \geq 1.2d_0$, pitch $p_1 \geq 2.2d_0$ and pitch $p_2 \geq 2.4d_0$. Standard EN 1993-1-12 gives no additions to any of these rules.

American standard ANS/AISC 360-05 [10] defines bearing resistance separately in serviceability limit state (SLS) and in ultimate limit state (ULS). In SLS the bearing strength is defined by hole elongation equal to 6.35 mm (1/4 in.) and in ULS an upper bound anticipates hole elongation at maximum bearing strength. As opposed to that other hand, Eurocode does not prescribe the limit for hole elongation. In Background documentation to Eurocode 3 [11] it is stated that it is necessary to give a limit for mean bearing stress to control deformation. This limit stress was originally set to the average bearing stress of $3f_y$ and not to a specified deformation.

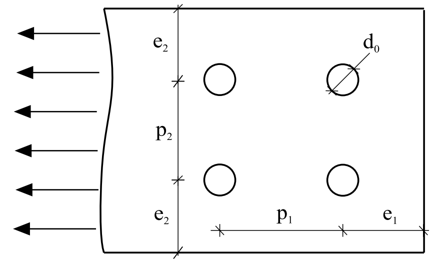

Fig. 1. Definition of end e_1, edge e_2 distance and pitches p_1, p_2.

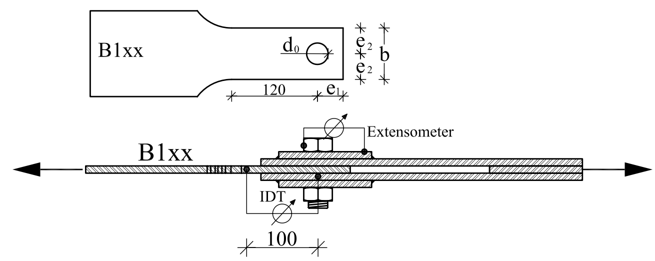

Fig. 2. Specimen type B1.

For group of bolts the following statement is given in 3.7(1), EN 1993-1-8: "*The design resistance of a group of fasteners may be taken as the sum of the design bearing resistances $F_{b,Rd}$ of the individual fasteners provided that the design shear resistance $F_{v,Rd}$ of each individual fastener is greater than or equal to the design bearing resistance $F_{b,Rd}$. Otherwise the design resistance of a group of fasteners should be taken as the number of fasteners multiplied by the smallest design resistance of any of the individual fasteners*". As stated in [12]: "*This statement is meant to persuade the designer to choose a balanced bolt pattern and to avoid having a relatively small end distance in combination with a relatively large pitch. A wrong design may lead to premature failure of the end bolts before the inner bolts reach their capacities. The capacity of the group of bolts will be overestimated in such cases*". In order to correctly predict a balanced bolt pattern and consequently the desired ductility and failure, the bearing resistance formula should accurately describe the phenomena and should also be supported by experimental results.

In this paper a total of 38 tests on connections with bolts in double shear are discussed. Quenched and tempered steel S690 QL was used. The aim of this investigation was to study the effect of connection geometry on strength and ductility and to revaluate bearing resistance according to EN 1993-1-8.

2. Experimental program

The specimens were fabricated from a single steel plate, grade S690 QL (nominal material strengths $f_y = 690$ N/mm^2, $f_u = 770$ N/mm^2). The nominal plate thickness was $t = 10$ mm. The actual material strengths of the plate were determined by standard tensile tests according to EN 10002-1 [13]. The test pieces were prepared according to EN ISO 377 [14]. The average obtained yield and tensile strengths were $f_{y,\mathrm{act}} = 847$ N/mm^2 and $f_{u,\mathrm{act}} = 885$ N/mm^2, respectively. The average elongation after fracture was $A = 14.4\%$ and the average percentage reduction of area $Z = 58.4\%$.

Tension splices with one or two bolts in double shear were studied. The B1 test series included 25 single bolt shear connections (Fig. 2). Bolts M27 an M22 were used in drilled standard size holes $d_0 = 30$ mm and 24 mm (see Table 1). Some of the holes were positioned eccentrically to loading axis. The B2 test series included 13