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Reuse of Steel Structural Elements with Bolted Connections

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Abstract. New economy concept – a circular economy as an alternative to the traditional linear economy is an important topic today. Steel structural elements have excellent circular economy credentials in comparison with other building products. Reuse of steel elements offers greater advantages than recycling because the energy, used to remelt the element, is saved. Today numerous studies are being carried out and problems regarding the reuse of steel elements have been identified. The aim of the experimental part of this research is to determine the behaviour of reused structural elements with renewed bolted joints (beam to column) under cyclic and ultimate loading. For this purpose bolts of different classes and sizes were used. Special attention was paid to discovering the changing of connection properties due to residual plastic deformations and strain hardening of joint components. Test results were compared with those produced by the equivalent T-stub method (hand calculation method) and FEM software programme and the analysis of the results are presented.

Keywords: circular economy concept, moment resisting connections, test results, theoretical modelling.

Introduction

Steel is a very widely used material in construction, although its production is very energy intensive. Reduction of energy consumption always is an important issue. There are several ways to achieve this goal. First, it can be achieved by increasig the efficiency or utilization rate. Besides, the recycling of steel at the end of the life of a structure also saves energy, reduces the quantity of natural raw materials and it can be produced into a functionally equivalent product. Reuse of steel elements offers even greater advantages than recycling because the energy used to remelt the element is saved. This leads to a new economy concept – a circular economy as an alternative to the traditional linear economy. Nevertheless, there are many barriers, that hamper the using of the given benefits – technical, logistical problems, cost implications, steel recertification etc. The background of making and exploitation conditions of steel elements, loading type (static, dynamic, cyclic) and level are also very important.

Now problems regarding the reuse of steel elements are identified and numerous studies have been carried out, however the studies devoted to the behaviour of reused steel elements subjected to considerable loads, particularly with bolted joints are insufficient. In this article test results with specimens modelling beam-to-column moment connections are presented. For this purpose, the experimental and theoretical study, described in [1], was continued.

Initially, three series of steel specimens were made and tested once. Most of the test results showed a failure of tensioned bolts with insignificant plastic deformations of connection members - end plates and column flanges. In continuation a lot of tests were performed with these specimens with new bolts to discover the bahaviour of connecting components in the case of cyclic and ultimate loading. Numerical analysis, using the so called component method, adopted in EC 1993-1-8 [2] was carried out and FEM software method was used to evaluate and compare the obtained results.

The aim of this research is to discover the changing of joint properties, if steel elements are reused, and assess the influence of these changes in comparison with the expected theoretical results. The main objectives are: to acquire the relationships by performing the tests, to assess the stiffnesses of the connections, to determine the load bearing capacity and to give some recommendations and suggestions for further research.

Experimental research

The test specimens for modelling the behaviour of beam-to-column bolted connections were made from wide flange beams HE 140 B – as columns and double-sided I beams IPE 180. End-plates were welded to the beams. Properties of welding metal are compatible with parent metal S235JR.

All column-beam models can be divided into three series (Fig.1). Each series consists of equivalent specimens to obtain more reliable test results. It should be noted that series I and series II differ only regarding the location of the bolt rows. Series III specimens have an additional bolt row at the extended part of end plate and column. The effective throat thickness was adopted 4 mm for all welds made during fabrication of specimens. To increase the column web resistance in compression and tension, double sided stiffeners were provided. All materials purchased from producers had adequate certificates for production.

Initial tests [1] with hexagon bolts M12, class 8.8U SB resulted with bolts failure and small plastic deformations of connecting parts (end plates and column flanges). Ignoring these deformations, the specimens were reused and the test programme was expanded for further research. For one part of the specimens – higher strength bolts M12, class 10.9 HV with adequate nuts and washers were used. For another part of the specimens – bolts M12 were replaced with bolts M16, class 8.8 SB and class 10.9 HV. Cyclic loading was performed with reloading up to 50-60% to study the behaviour of the specimens under short time variable actions. Finally, ultimate loading was performed.

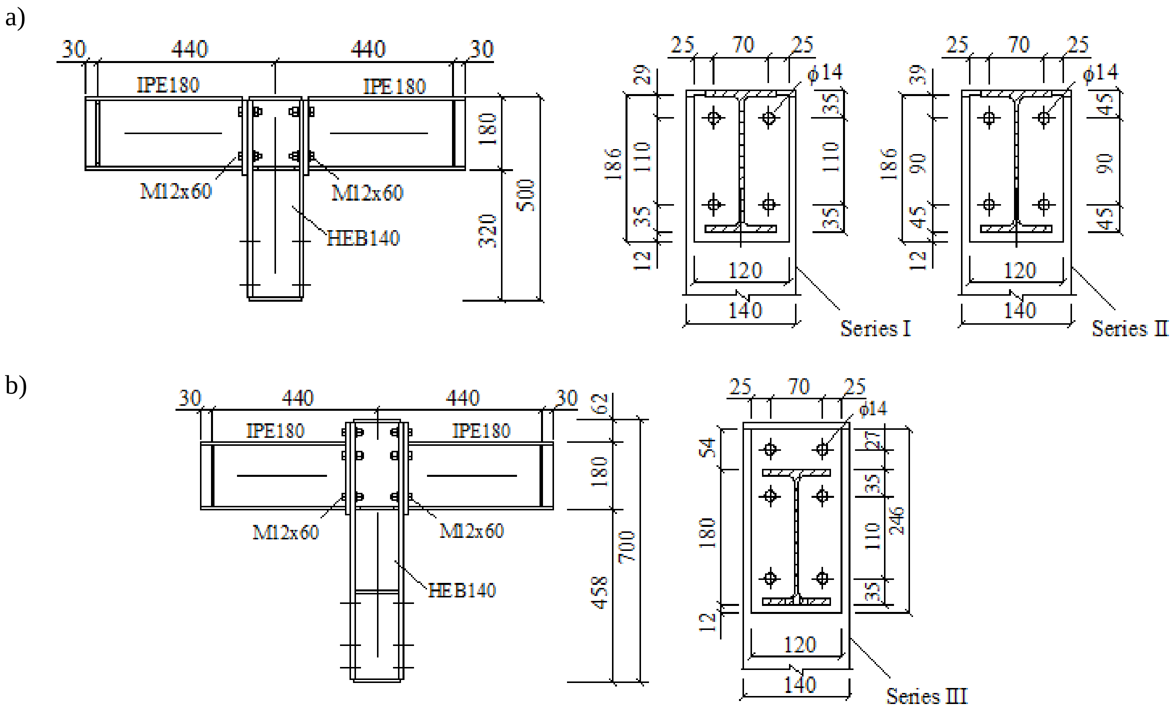

Fig. 1. Drawings of specimens for experimental (initial) tests: a - series I , series II ; b – series III

Bolt torque value for tightening (for a non-slip resistant connection) was calculated according to EN 1090-2:2008 [3]. It is stated that the design tension resistance of the bolt:

$$F_{t.Rd} = f_{yd}\, A_s$$

and bolt design preload:

$$F_{p.Cd} = 0{,}7\, f_{ub}\, A_s / \gamma_{M7}$$

Torque reference value for bolt tightening

$$M_r = k_m\, d\, F_{p.Cd}$$

where $\gamma_{M2} = 1{,}25$;
$\gamma_{M7} = 1{,}1$;
$f_{yd} = f_{yb} / \gamma_{M2}$;
$k_m = 0{,}2$ – for typical steel.

If preload is required only for execution purposes, then the level of preload can be specified in the National Annex, or ~ 50% $F_{p.Cd}$. Then the calculated torque value $M_{r(non\ slip)}$ when there is no slip resistance (Table 1):

TABLE 1

Preload of bolts according to EN 1090-2:2008

Bolt	Bolt class	A_s mm^2	$F_{t.Rd}$ kN	$M_{r(non\ slip)}$ N m
M12	8.8	84.3	43.2	51.5
M12	10.9	84.3	60.7	64.4
M16	8.8	157	80.4	127.9
M16	10.9	157	113	175.8

This torque value was provided by torque wrench tool. The tightening of bolts was restored after each reloading.

Prepared specimens and loading performance is shown in Fig.2. The tests were carried out using the hydraulic equipment Zwick-Roell and software programme TestXpertII. To define the upper value of the cyclic loading (up to the plastic limit) the results of the first tests were taken into account. To exclude the initial slip of joining members, the preload of 5 kN was applied. As it was noted, tests with bolts M12 class 8.8 SB showed the weakness of these bolts (see the dotted curves 1 in Fig.3 a, Fig.4 a, Fig.5 a). Nonlinear deformations were caused mainly due to the plastic elongations of the bolts.

a) b)

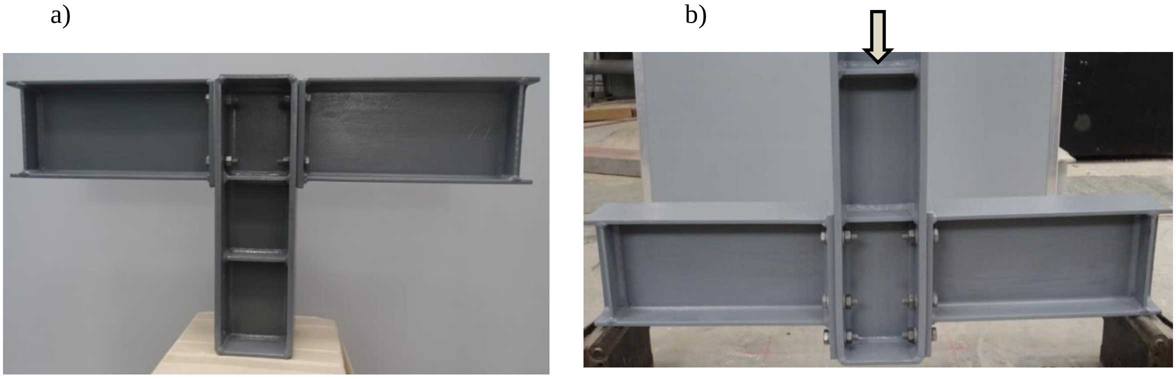

Fig. 2. Column-beam specimens (a) and loading performance during the tests (b)

a) b) c)

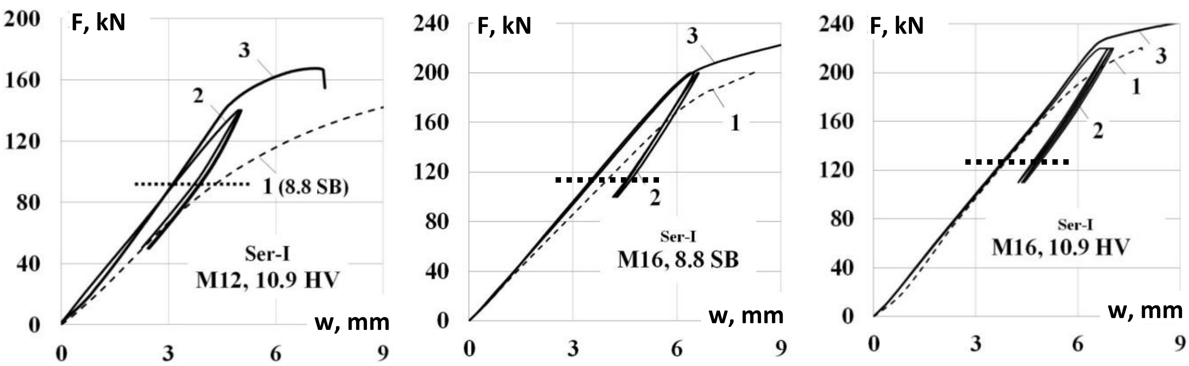

Fig. 3. Test results: specimens of Series I under static and cyclic loading (F – loading; w – displacement)

a) b) c)

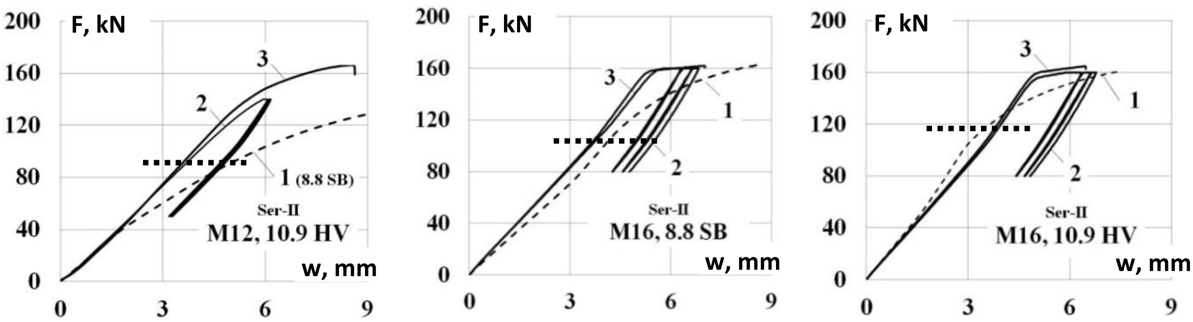

Fig. 4. Test results: specimens of Series II under static and cyclic loading (F – loading; w – displacement)

a) b) c)

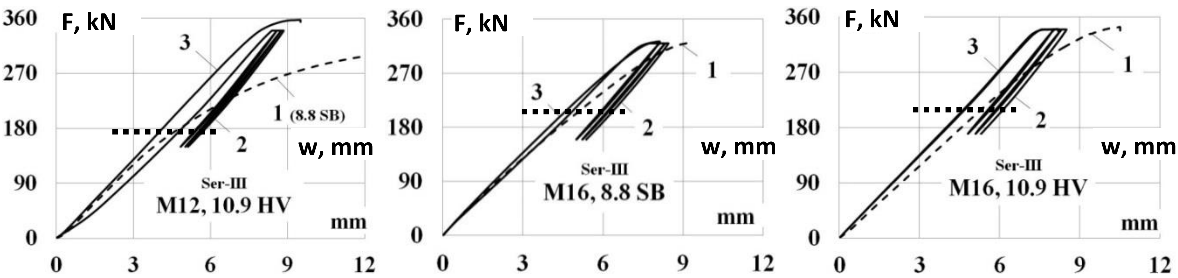

Fig. 5. Test results: specimens of Series III under static and cyclic loading (F – loading; w – displacement)

- Test results with bolts M12 class 10.9 HV instead of the bolts M12 class 8.8 SB:

- stabilization of deformations can be observed after 3-5 times of cyclic loading (see curves **2**);
- ultimate loading (see curves **3**) leads to failure of bolts M12; in case of series III specimens, considerable deformations of the extended parts of the end plates were fixed;
- the tests for identical samples showed practically equal graphs;
- comparing the effectiveness of bolts M12 class 10.9 for series I, II and III the distance effect can be assessed between the rows and the number of bolts in joining. Reducing the distance between the bolt rows (only 10 mm for each row) leads to increasing of the ductility and decreasing of stiffness (see Fig.3 a and Fig.4 a). On the contrary – the increase of the load capacity of joining was obtained by an additional bolt row in the extended part of the end plate (Fig.5 a).

- Test results with bolts M16 SB class 8.8 and M16 HV class 10.9 instead of the bolts M12 (Fig.3 b,c; Fig.4 b,c; Fig.5 b,c):

- the dotted curves **1** show first loading results using bolts M16. It serves as an attempt to determine the real plastic stage of the joining and for the cyclic load level acceptance;
- obviously the bolts M16 are useful for the specimens of series I. Ultimate loading (see curves **3**) shows elastic-plastic stage bahaviour without brittle collapsing;
- due to the plastic deformations of the end plates and the column flanges the joinings of II and III series become ductile. In this case increasing the bolt size does not increase the load capacity of joining; particularly it is visible in test results with specimens of series II. The cyclic loading and ultimate loading (see curves **2** and **3**) show plastic stage behaviour without reserves of load bearing capacity;
- comparing the curves of first loading and last loading (until constant plastic deformations) the strain hardening effect is visible. It should be applied to all components of the joint, but especially on behaviour of the end plates and column flanges. As a result - the yielding stress of the steel increases and stiffness of the joining changes. Many studies have been devoted to the effects of strain hardening under the cyclic loading [5], [6], but they are insufficient regarding reused structural elements.

Theoretical modelling

Numerical analysis, using the component method, adopted in EC 1993-1-8 [2] was used. The behaviour of the components have been studied considering that the joint can be sub-divided. One of the main components is the equivalent T-stub in tension modelling column flange and beam end-plate in bending. For theoretical analysis the MathCad programme is used to calculate tensioned equivalent T-stub flange resistances and perform the necessary checks. The results are given in Table 2.

TABLE 2

Calculated T-stub flange resistances

Series	Diameter of the Bolt		Bolt class	Column flange			Beam end-plate	
				$F_{t1Rd.c}$ (Mode 1)	$F_{t2Rd.c}$ (Mode 2)	$F_{t3Rd.c}$ (Mode 3)	$F_{t1Rd.ep}$ (Mode 1)	$F_{t2Rd.ep}$ (Mode 2)
1	2		3	4	5	6	7	8
I	M12		8.8	244.5	102.2	97.1	212.2	92.0
			10.9	244.5	115.1	121.4	211.7	103.2
	M16		8.8	263.4	146.8	180.9	226.1	131.1
			10.9	263.4	170.9	226.1	227.2	152.2
II	M12		8.8	244.5	98.1	97.1	200.8	89.5
			10.9	244.5	111.0	121.4	200.8	100.8
II	M16		8.8	263.4	142.7	180.9	213.4	128.4
			10.9	263.4	166.8	226.1	213.8	149.5
III	M12	row1	8.8	244.5	100.0	97.1	113.1	73.7
		row2		233.4	94.8	97.1	215.4	92.7
		row1	10.9	244.5	113.0	121.4	113.1	87.2
		row2		233.4	107.8	121.4	215.4	104.0
	M16	row1	8.8	263.4	144.7	180.9	122.1	120.2
		row2		251.4	139.5	180.9	228.4	86.5*
		row1	10.9	263.4	168.8	226.1	122.1	145.3
		row2		251.4	163.6	226.1	228.4	84.6*

* - classified as beam flange collapse in compression

Following the conditions given in [2], [4] the design tension resistances of T-stub flange are obtained for each bolt row separately. Significant impact on the results is created by the location of bolt rows and stiffening of column web and flanges. Practically the method is based on the correct determination of the effective length of yielding line, named as circular or non circular pattern, i.e., designer need to choose the correct values of ℓ_{eff} from Tables 6.4 – 6.6 [2].

Theoretically it is obvious, that the value of the resistance of the end plate in bending is the main limiting factor. Actually, mode 2 (bolt failure with yielding of the flange of T-stub) prevailed in tests.

Plastic moment resistances of the bolt rows were calculated from the lesser values of tension resistances. Considering double-sided beam-to-column joint configuration (Fig.1), the corresponding loads which mark the limit state are as shown in Table 3:

TABLE 3

Moment resistances and corresponding loads

Series	Bolt size	Moment resistances of bolted joints $M_{pl.Rd}$, kNm		Corresponding loading F_{Ed}, kN		Stiffness S_j kNm/rad	Stiffness S_{eksp} kNm/rad
		Bolt class 8.8	Bolt class 10.9	Bolt class 8.8	Bolt class 10.9		
I	M12	15.5	17.4	83.8	94.0	3705	2212
	M16	20.8	23.2	112.4	125.4	4620	2395
II	M12	14.7	16.7	79.5	90.3	3120	1910
	M16	20.0	21.9	108.4	118.4	3820	2064
III	M12	28.7	32.8	155.1	177.3	10110	3075
	M16	36.6	36.7	197.8	198.4	12340	3075

For comparing and verification of given results, the IDEA StatiCa Connection software programme (the programme uses component-based finite element model) was used. All the results obtained are comparable and the software programme discovered 6 – 10 % reserves of load capacity due to a more accurate modelling of the interaction of the joining elements.

Results and discussion

Test results with reused specimens under cyclic and ultimate loading show significant deviations from the expected theoretical volumes obtained for initial (first) loading conditions. As it can be seen in Figures 3, 4 and 5, reusing the specimens with renewed bolts lead to the changing of connection properties. The rotational stiffness values and design resistances of connections should be analysed.

Accordingly to the initial properties of the specimens, the boundary limits of stiffnesses are $S_{j.rig}$ = 50400 kNm/rad – if the rigid connection, and $S_{j.pin}$ = 3150 kNm/rad – if pinned connection, i.e., actually the models should be classified as semi-rigid (Table 3).

In addition to the experimental curves F-w, moment-rotation curves M-φ were determined from which the stiffness calculation is possible. Only the linear part of the curves from the last (ultimate) loading was taken into account. Numerical values of the stiffnesses S_{eksp} are shown in Table 3. Comparing the results with those from the initial conditions (S_j), it should be concluded, that all the reused specimens have become more ductile and the connections can be classified as pinned. This should be explained by the changing of the initial conditions.

As noted before, the plate elements of the connections (end plates, column flanges) due to small residual plastic deformations and strain hardening changed the interaction of joining elements. Therefore, the moment-rotation curves (linear part of them) reflect mainly the resistance of alone tensioned bolts until the full contact between the plates is restored. Following this, in the case of reusing the steel joining elements, it is important to take into consideration the contact conditions between the plate surfaces.

Assessing the strength of the model joint, the resistances are lesser then the moment resistance of the beam cross section (39.1 kNm), therefore the joint is classified as partial-strength joint.

The load bearing capacity of the reused specimens (see horizontal dotted lines in Fig.3, Fig.4 and Fig.5) is much higher than the expected design resistances. Along with the above reasons it can be noted that significant reserves may be caused also from the inadequacy of steel element properties (according to the Certificate of quality and quantity). For example - tensile strength f_u of bolts class 8.8 SB actually is declared up to 908 MPa and the resistance of steel S235JR plates actually is up to 304 MPa (yield strength).

Increasing of the load bearing capacity of joints by using the bolt class 10.9 HV instead of class 8.8 SB should be assessed in conjunction with more brittle failure of bolts class10.9 HV (in case of extreme loading). In this context, the use of more plastic bolts 8.8 SB increases the safety of joining and the whole structure.

Conclusions

In case of reusing of steel structural elements with bolted connections particular attention should be paid to the contact conditions between the joined elements. Due to incomplete surface contact the joining may become much more ductile.

The analysis of the given test results show, that the moment resisting connection with semi-rigid stiffness due to insignificant plastic deformations and strain hardening during the previous (initial) tests can be changed to pinned joining. Following this, the behaviour of the entire construction should be evaluated.

Significant reserves of the load bearing capacity discovered by the results of testing (30-40% in comparing with theoretical) show an influence of many factors, including inadequacy of steel element properties (according to the Certificate of quality and quantity). Using a professional software programme confirms this conclusion.

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