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EXPERIMENTAL STUDIES OF THE BACK-TO-BACK CONNECTED COLD FORMED STEEL PROFILE BOLTED JOINTS

BY

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Abstract. This paper presents the experimental test results of joints connecting the cold formed thin-walled steel profiles assembled in a pair, back-to-back cross section. The tests were carried on at the Faculty of Civil Engineering and Building Services from Jassy. In the first testing stage there were tested the joints of a frame structural model connected with high strength friction bolts. In the second stage it was advocated the increase of the profile bearing capacity by introducing some supplementary elements at the flanges; the maximum stresses were expected there and occurred. The specimen is built and modeled as a simply supported beam. In the midspan it is constructed the joint that focuses the research interest. There the load is applied as a concentrated force. The tests were carried on in quasi-static regime in several load-unloading cycles; in the last stage the load was increased until the model collapsed. The strengthening of the flange has not increased significantly the model stiffness, but the bearing capacity was increased with 20. . . 35% depending on the profile type.

Key Words: Thin-Walled Steel Profiles; Steel Joint Design; Experimental Tests; Strengthening.

1. Introduction

The thin walled cold formed steel profiles used for the joint assemblage are produced by the Kontirom manufacturer and are so-called the KB profiles; they have a cassette shape and are used as linear elements of the structural resistant frames. A frame element is typically made of two KB profiles positioned back-to-back and fixed together with a thick steel connector, as in the Fig. 1.

They can be used for main portal frames with fragmented girder at the ridge. The steel frame KB members (columns, beams, girders, counterbraces, etc.) made of twin thin-walled profiles are connected at the joints and fixed with bolts, allowing an easy and quickly assemblage at the building site.

The twin-profiled linear elements are connected through a steel plate web at the joint. Thus the joint is primarily consisting of webs and flanges made of steel welded plates, usually of 10 mm thickness.

The carbon steel strip of the profiles is protected by immersion into a zincamid bath and is made of FeE320G according to EN 10147 Product Norm (Euro Norm).

The mechanical characteristics of the material are:

a) the yielding strength of the basic material, $f_{yb} = f_y = 320$ N/mm^2;
b) the ultimate strength of the basic material, $f_u = 390$ N/mm^2;
c) the ratio $f_u/f_y = 390/320 = 1.22 > 1.2$.

Taking into account that the KB profiles height is relatively big, they required intermediate stiffeners on the web in order to prevent early buckling at very small loads and in order to discharge the loads on the entire plane surface. The intermediate and edge stiffeners, positioned as in Fig. 1, must satisfy the following condition:

$$I_{\min} = 3.66t^4\sqrt{\left(\frac{b_p}{t}\right)^2 - 144} \geq 18.4t^4, \text{ for the intermediate stiffeners.} \tag{1}$$

The edge stiffeners are active if the following relationship is satisfied:

$$I_{\min} = 1.83t^4\sqrt{\left(\frac{b_p}{t}\right)^2 - 144} \geq 9.2t^4. \tag{2}$$

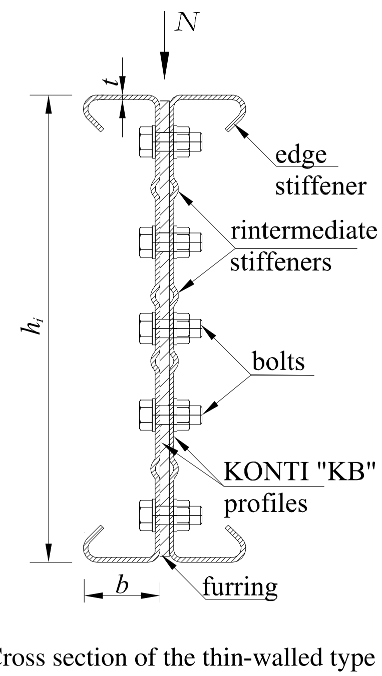


Fig. 1. – Cross section of the thin-walled type KB profiles.

2. The Joint Structure

The testing study and program was started after some phenomena were noticed during the frame assemblage process, especially at nodes. The node skeleton consisting of welded steel plates is made at the factory. In the end the plates are sometimes polished, painted for protection against the corrosion, or even other treatments are applied. This surface treatment can lead to a decrease of the joint bearing capacity. This is because some rotation of the profiles in the node may be also present.

The following type of joints can be met on a typical transverse steel frame:

a) column-foundation node, at the base of the column;
b) beam to column node, gutter node;
c) beam to beam node, ridge node.

- Beam to beam node, ridge node.

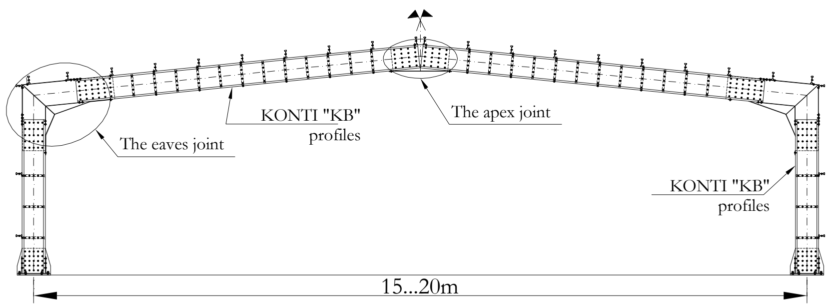


Fig. 2. – The structural system.

It was thought another kind of joint that should carry on the occurring loads, with a good bearing capacity of the bending moment. The KB profiles are stiffened at the flanges with some profiles, in order to prevent the local buckling. The shear capacity can be increased if a box shape joint is used. If a good ratio joint material / KB twin profiles and material disposal all over the joint, also the number of the bolts may be diminished. The bold disposal is also very important and can lead to an improved bearing capacity. In this stage of the experimental research program it was designed a joint without pre-stressed bolts (with normal behavior).

Then, in order to check the behavior of the new joint the beam members were assembled of pairs of KB 600-5.0 and KB 450-3.5 cold formed thin walled steel profiles, *i.e.* two classes of beam specimens were primarily built and tested.

Next, the beam elements were constructed with longitudinal flange stiffeners and without stiffeners. For the next analysis we denominated by *R* the joint with stiffeners and *N* (normal) the joint without the rigid profiles at flanges. As it follows, the denominations of the tested girders were

a) *N*-KB600-5.0;
b) *R*-KB600-5.0;
c) *N*-KB450-3.5;
d) *R*-KB450-3.5.

For both models *N*-KB, as well for the second system with stiffeners, *R*-KB, the simply supported beam with a middle span joint was selected as the structure experimental model (see the dimensions in the Fig. 3).

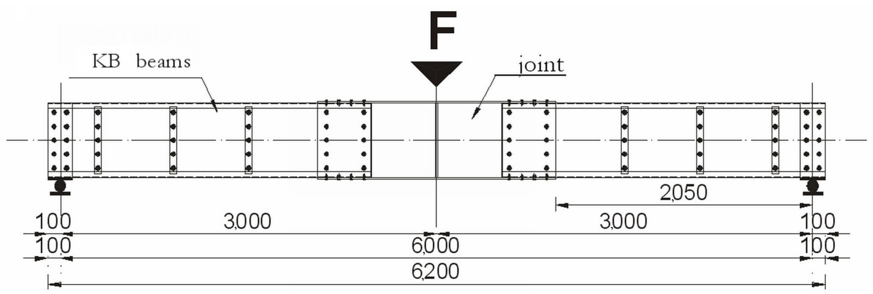


Fig. 3. – The dimensions of the proposed specimens.

In the Fig. 4 there are presented the stiffening elements fixed at the flange level that were used in the second stage of the research (for the *R* beams/joints).

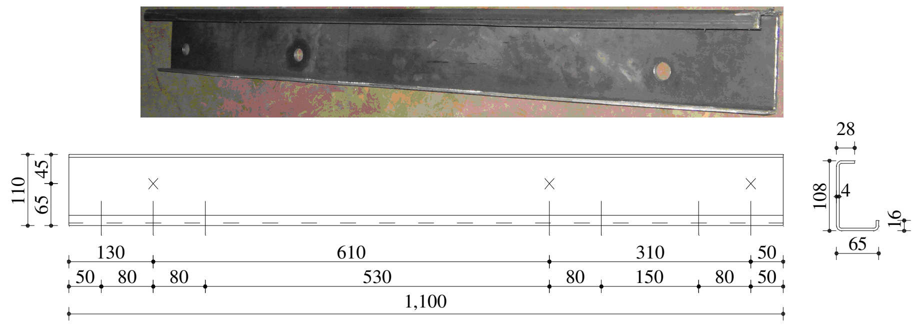


Fig. 4. – The stiffening elements.

The additional elements are made of 5 mm thick steel plates; they are made of OL52 cold steel sheet and are riveted on the profile flange, as it can be noticed from the Fig. 5. The strengthening elements are 1,100 mm long. They were fixed with the bolts from the cassette flange, as presented in Fig. 6.

The cross-sectional characteristics of the stiffeners are: $A = 7.9$ cm^2, $Iy = 127.3504$ cm^4, $Iz = 35.2691$ cm^4.

The cross-section of the two twin profile beams (the *N* and the *R* beams) is depicted in the figure bellow:

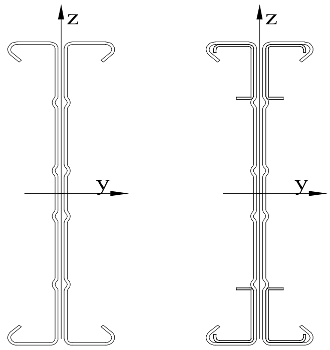
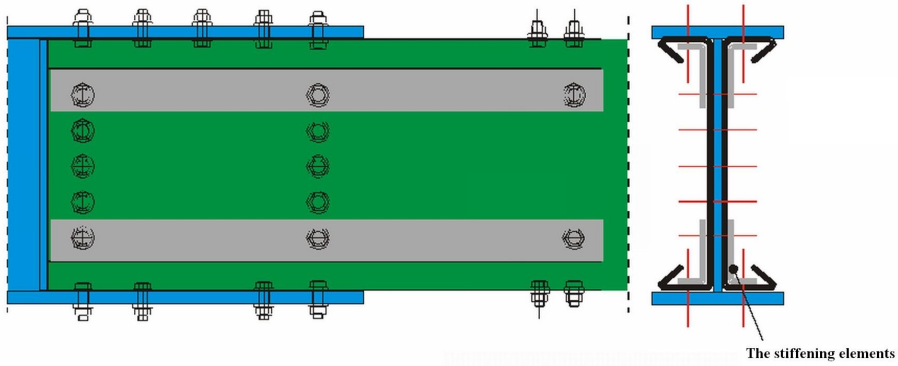


Fig. 5. – Cross sections of beams.



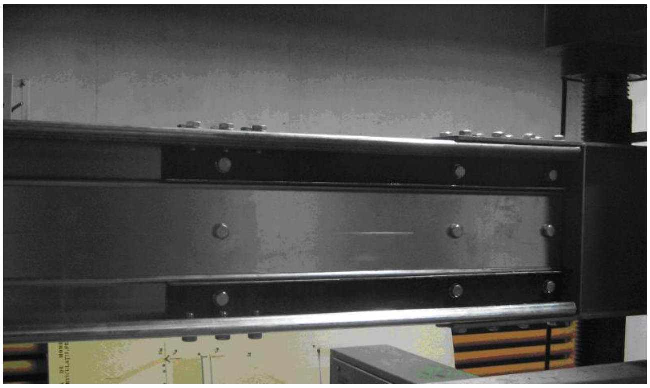


Fig. 6. – The positions of the strengthening elements R-KB 450-3.5.

3. The Testing Facilities and Conditions

A 300,000 daN hydraulic press was used for testing. In the Fig. 7 it is presented the transducer map on the specimens and the complementary elements used in the experiments. At this testing stage it was proposed the following instrumentation of specimens:

a) two displacement transducers mounted on the central joint from the midspan (*D*0, *D*1);

b) two displacement transducers mounted at the joint edge (*D*2, *D*4 - *D*3, *D*5);

c) two displacement transducers mounted on the KB profile at the joint vicinity (*D*6, *D*8 - *D*7, *D*9);

d) one force transducer to accomplish the automatic load recording.

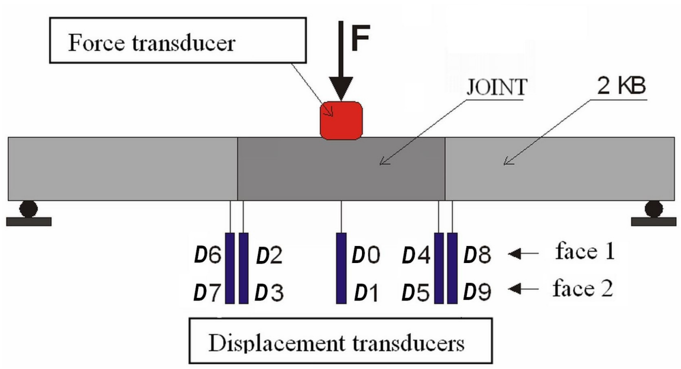


Fig. 7. – The transducer positions on the two sides of beams.

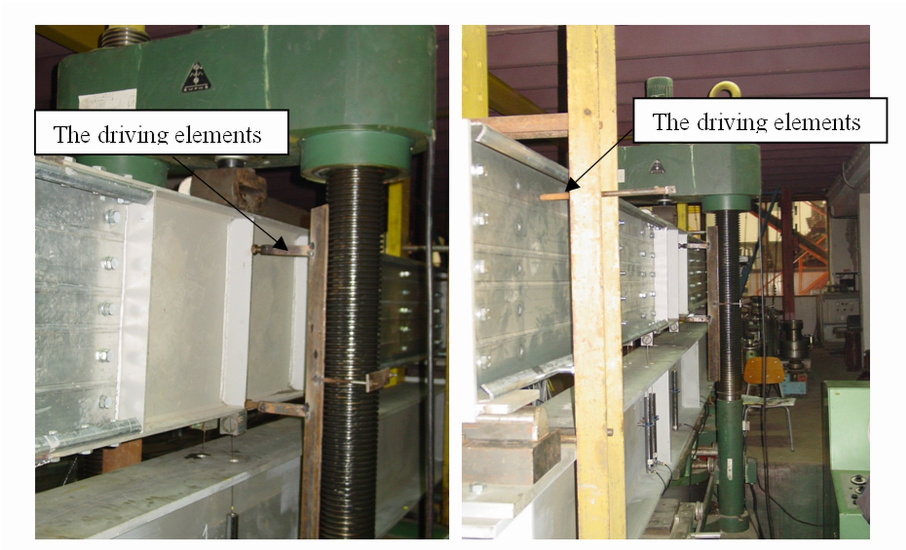


Fig. 8. – The driving elements mounted to prevent the lateral buckling.

In order to avoid the lateral buckling of the beams a driving system was thought and mounted at middle of each specimen edge, as it is depicted in the Fig. 8.

The signals received from all the transducers were amplified and introduced into an analog–digital converter system and processed numerically.

The experiment was realized in monotonic loading paths up to the levels of 100 kN, 200 kN, 300 kN and 350 kN. The last value coresponds to a stress of 2,900 daN/cm^2, *i.e.* the design strength.

4. The Test Results

The first step of the testing program scheduled in 2006 the experiments of the KB600-5.0 beams of *N* class, without stiffeners. In the Figs. 9 and 10 there are presented the force–displacement relationships for the *N*-KB600-5 beam.

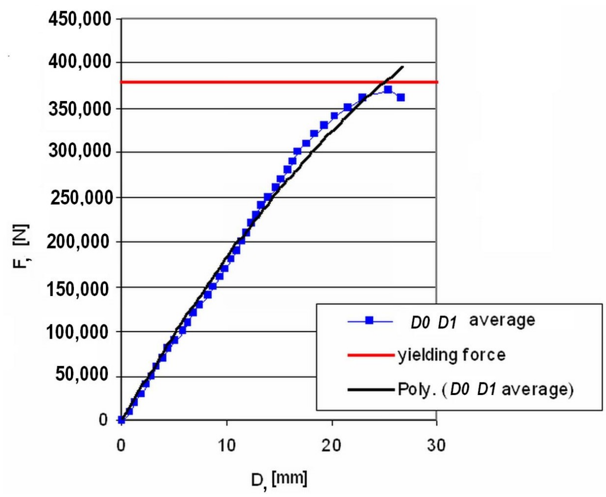


Fig. 9. – The force–deflection relationship at the midspan of the KB 600-5.0 beam.

After the analysis of the force–displacement relationship in the case of this type of beam it cannot be noticed a significant increase of the element stiffness (Fig. 11). Unlike the previous tests, this time the element was tested till the failure. Thus, it comes out that element ceased due to the local buckling at the boundary of the strengthening elements (Fig. 12).

The bearing capacity of the element is significantly increased, the buckling occurred at a force level of 485,200 N. Under these circumstances it results a force level increase greater than 22% when compared to the yielding level of the KB basic material.

The beams made of the KB450-3.5 profiles were tested under the same conditions as those consisting of pairs of KB600-5.0. The Figs. 13 and 14 depict the behavior of the *N*-KB450-3.5 beam until it reaches the yield limit stress.

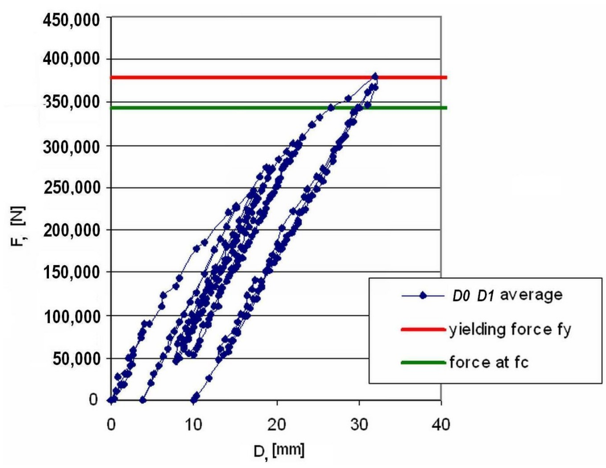


Fig. 10. – The force–deflection relationship at the midspan of the *N*-KB 600-5.0.

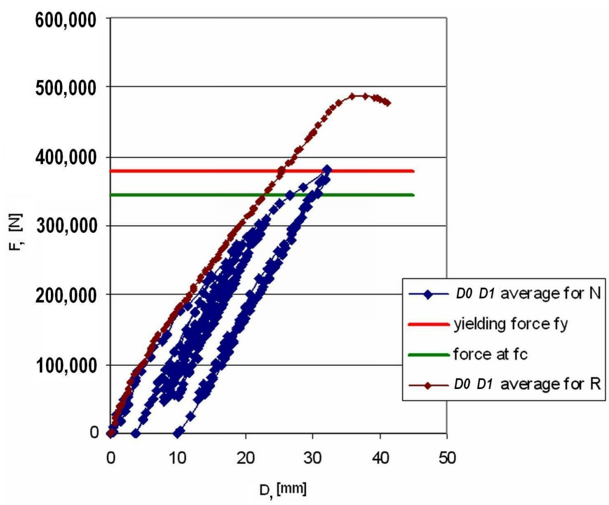


Fig. 11. – The force–deflection relationship for the *N*-KB 600-5.0 and *R*-KB 600-5.0 beams.

The tests for the *R*-KB 450-3.5 girder were carried out in the same cycles as for the beam *N*-KB 450-3.5; after that the test continued up to the failure. In the Fig. 15 it is depicted the force–displacement relationship at the midpoint of the beam.

The beam failure occurred by the local buckling of the flange of one KB profile at the boundary area of the ”strengthening” elements (Fig. 16). The bearing capacity of the beam with ”strengthening” elements is increased by approximately 35%.

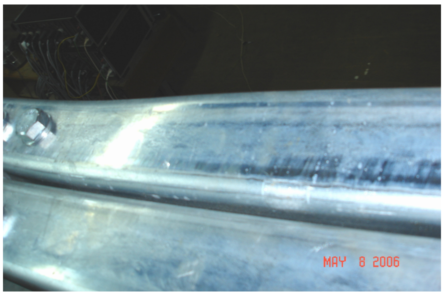


Fig. 12. – The local buckling of the KB element of the *R*-KB 600-5 beam.

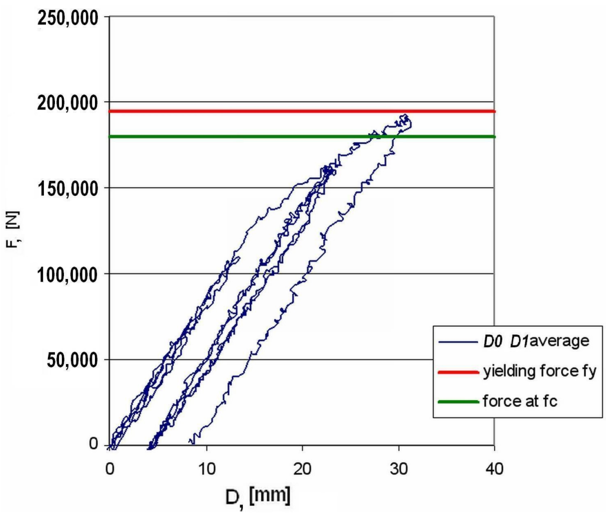


Fig. 13. – The force–deflection relationship at the midspan of the *N*-KB 450-3.5 beam.

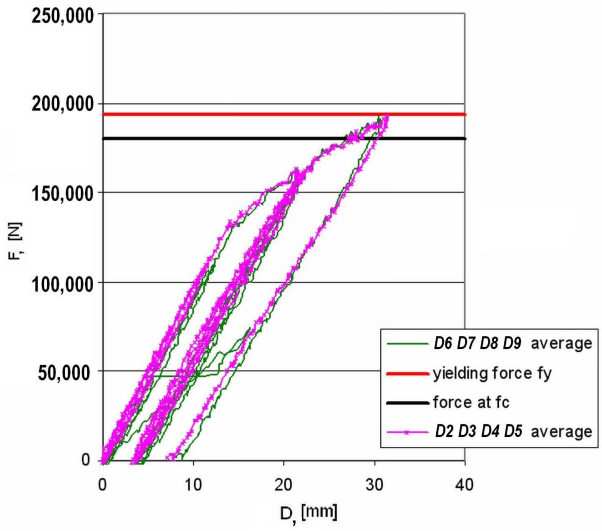


Fig. 14. – The force–deflection relationship at the joint ends for the *N*-KB 450-3.5 beam.

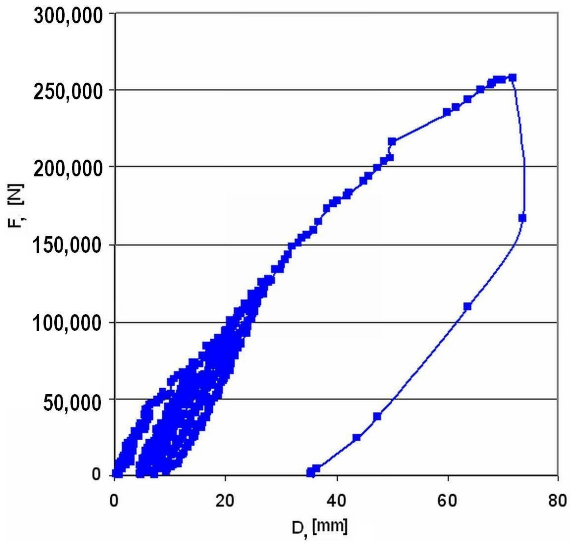


Fig. 15. – The force–deflection relationship (*D*0-*D*1 average) for the *R*-KB 450-3.5 beam.

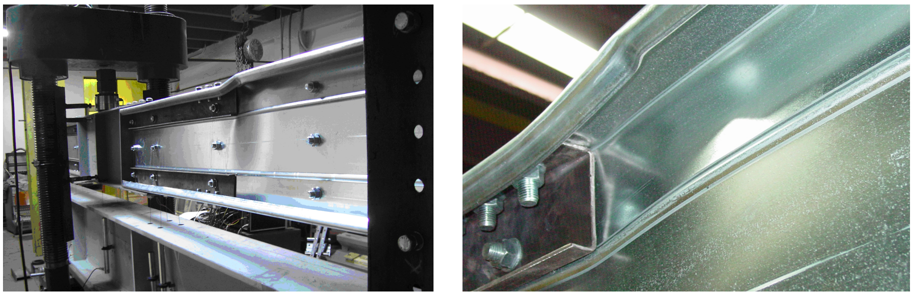


Fig. 16. – The local buckling of the KB element of the R-KB 600-5 beam.

5. Conclusions

The boundary bolted connections assure a good behavior between the KB profiles and the joint element.

The presence of a too significant gap between the joint carcass and the KB profile allows the rotation of the profile until all the bolts start working – this phenomenon is consumed during repeated cycles.

The mounting of the strengthening elements leads to an increase of the bearing capacity up to 30…35% with respect to the yield limit of the KB material.

The use of the strengthening elements leads to a safer cross-section of the compound KB profile when the bending moments may lead to the local buckling.

The usage of these stiffeners allows the optimum use of the KB profiles, thus leading to the material quantity reduction.

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STUDII EXPERIMENTALE ASUPRA UNOR ÎMBINĂRI CU ŞURUBURI ALE SECŢIUNILOR DUBLE REALIZATE DIN PROFILE CU PEREŢI SUBŢIRI

(Rezumat)

Se prezintă rezultatele experimentale obţinute în urma unui program de teste a unor noduri de structuri metalice realizate din profile cu pereţi subţiri laminate la rece. Elementele sunt alcătuite din câte două profile dispuse în tandem.

Testele au fost realizate la Facultatea de Construcţii şi Instalaţii din Iaşi. In prima etapă a programului experimental au fost încercate nodurile unor modele structurale asamblate cu şuruburi de înaltă rezistenţă pretensionate. În cea de a doua etapă s-au propus noduri a căror capacitate portantă să fie crescută prin introducerea unor elemete suplimentare (rigidizări) la tălpile elementelor. Acolo au fost anticipate tensiunile maxime care s-au produs.

Modelele experimentale au fost de tip grindă simplu rezemată. La mijlocul grinzilor a fost construită îmbinarea asupra căreia s-au concentrat studiile şi observaţiile. Acţiunea a fost aplicată tot la mijlocul grinzii sub forma unei forţe concentrate.

Incercarile au fost de tip cvasi-static, monoton crescător şi în câteva cicluri încărcare–descărcare; în ultima etapă de încărcare modelele au fost încărcate până la avariere. Consolidarea tălpilor profilelor grinzilor cu pereţi subţiri nu a condus la o creştere semnificativă a rigidităţii structurale, dar capacitatea portantă a crescut cu 20…35% în funcţie de profilele utilizate.