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Postelastic static behavior of certain beams with composed cross-section made of thin-walled steel profiles

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Summary

The paperwork presents the behavior of a pair of composed cross-section beams made "C" class thin-walled profiles, subjected to gravitational loads. The profiles are made of 3.5mm thick, cold-rolled plates of FeE 320G steel.

The specimens were tested on the existent stand from the laboratory of the Structural Mechanics Department of the Faculty of Civil Engineering and Architecture of Iaşi. The beams of 5,0 m length were mounted at 1,0m distance. The load input from the hydraulic jacks has been transmitted by the means of some steel profiles.

The system response was obtained by the measurement of the deflections of every beam at the midspan and quarter of span and the strain measurements at midspans. Inductive transducers were used to measure the transverse deflections and the strains were measured with the resistive electrical transducers placed at the bottom flanges of the beam. The loading has been carried out by the means of hydraulic jacks up to a level of 10000daN per beam, when local buckling occurred.

The tests leaded to the following conclusions:

The necessity to eliminate the profile deformations due to the manufacturing, packaging and transportation processes, etc. because the buckling phenomena can be initiated at lower values than the than the design loads;

The structural elements (KB) must be provided with additional elements in order to assure the overall stability to bending and torsion effects;

The computational checking of the structural elements (beams, columns) must be also performed to avoid the stability loss by bending-torsion..

KEYWORDS: postelastic behavior, thin-walled steel profiles, KB450, local buckling

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1. INTRODUCTION

The testing of the elements was performed in order to establish their behavior when are subjected to gravitational loads. This was because the elements are classified as cold-rolled thin-gauge profiles as stated in NP 012-92 (EC 3 parts 1-3). Under these circumstances the elements behave different as the usual rolled profiles due to the fact that local buckling can occur, correlated with the profile shape and the sheet thickness.

This class of profiles (used for pillars and beams), are provided with some supplementary connections that lead to an improved behavior.

All these reasons prove the need for mechanical tests. The experimental research offers the quantizing of possible supplies for carrying capacity of these profile types in the structural system where are part of.

2. THE KB 450-3.5 BEAM TEST

The KB450-3.5 specimen was tested after the arrangement of the existent stand from the laboratory of the Structural Mechanics Department. The profiles are made of 3.5mm thick cold-rolled plates of FeE 320G steel. The yielding strength of the basic material is $f_{yb} = f_y = 320$ N/mm^2 and the ultimate strength of the basic material is $f_u = 390$ N/mm^2 (the ratio $f_u/ f_y = 390/320 = 1,22 > 1,2$). These elements are components of the cold-rolled thin-walled profiles, according to the NP 012-92 (EC3 parts 1-3) and behave differently as the usual rolled profiles, as a consequence of the local buckling phenomena, which are related to the cross-section shape. In the Fig. No. 1 is presented the stand mounted inside the laboratory.

The testing process has been designed in such a way that the KB profiles are not connected to any element that could alter the behavior. In fact, two profiles were used in the test. They were mounted in parallel positions and they were simply supported at ends (see Fig. No. 2). The load input from the hydraulic jacks has been transmitted by the means of some KB profiles. The loads then have been transmitted to the specimens by the means of some rubber rolled supports. The supports provide a big vertical stiffness, while the horizontal stiffness is insignificant. In this way the profiles are able move freely in case of lateral buckling.

Dynamometers were used in order to measure the applied force, equipped with electrical inductive transducers and connected to the data acquisition and processing system (see Fig. No.2).

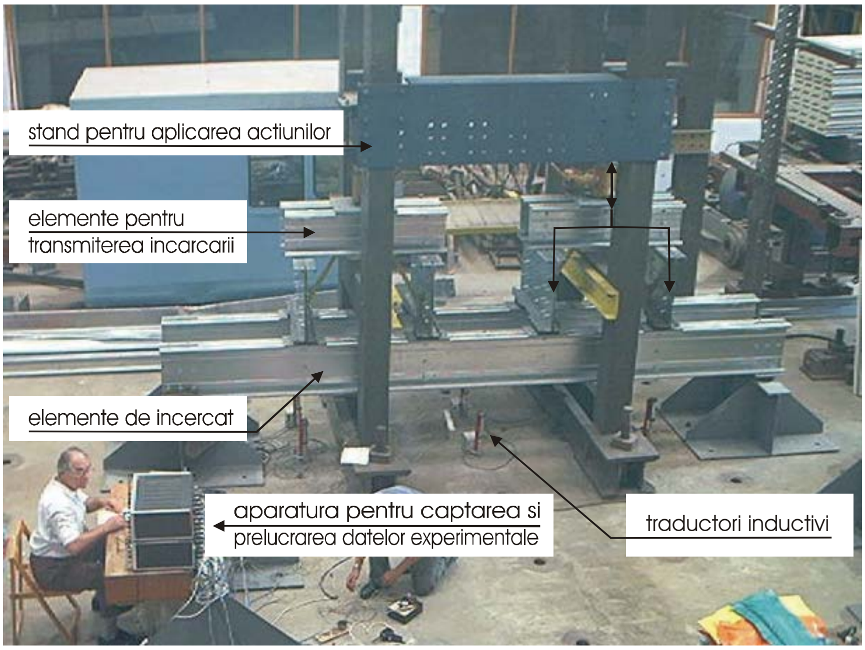

Fig. 1 The testing stand inside the laboratory.

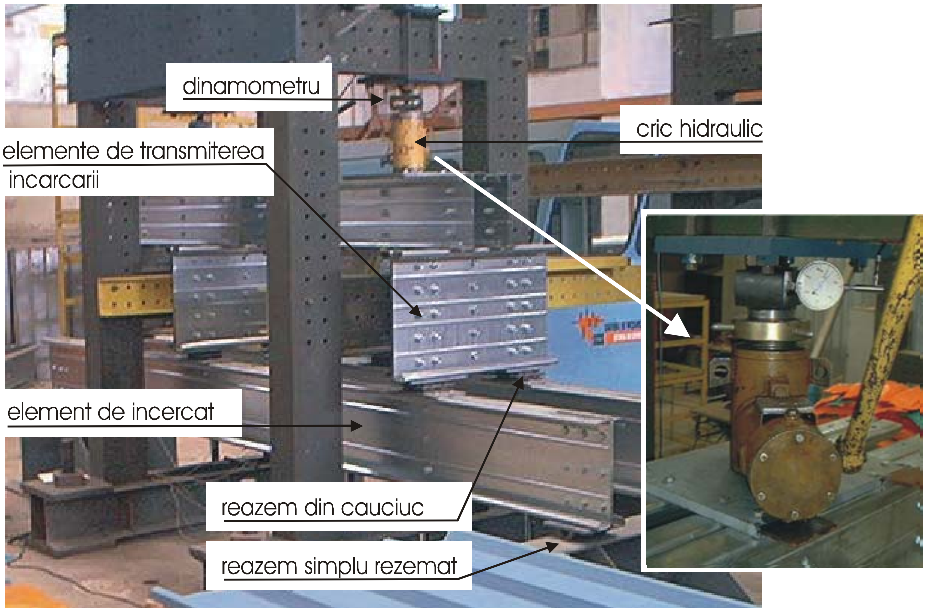

Fig. 2 The KB specimens mounted on the testing stand.

The system response has been determined by the means of both inductive and resistive electrical transducers. The positions of the transducers it is presented in the Fig. No.3.

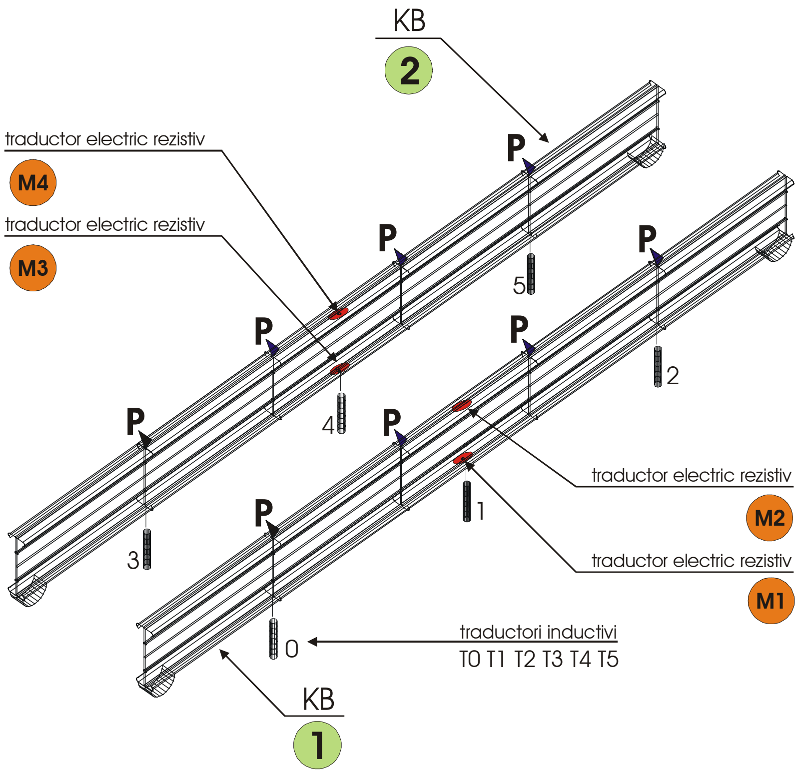

Fig. 3 The specimens, the load and transducer positions.

At the beginning, the test should have consisted of two stages. In the first stage the load should have reached the equivalent level of the steel design resistance, the equivalent of the ultimate limit state of strength and stability (P_c = 5376 daN $\Rightarrow$ 4 P_c = 21504 daN, w_c = 15.6 mm). In the second testing stage the loading level should have reached the steel yielding strength (P_y = 5913 daN $\Rightarrow$ 4 P_y = 23652 daN, w_y = 17.2 mm).

In the first stage the load was only of 3870 daN, that means 72% of the expected loading level, when some incipient buckling phenomena have been detected at the loading system. Under these circumstances the loading was stopped and the unloading phase has been carried out. The Figure No. 4 presents the incipient buckling phenomena of the KB profile No. 1.

The load-deflection and load-strain relationships are presented in the Fig. No.5 … 9. Thus, in the Figure No.5 that presents the force-displacement curve for the central transducers one may notice that after the 10000 daN of load there is a trend of changing the linear shape, a fact that certifies the buckling phenomenon.

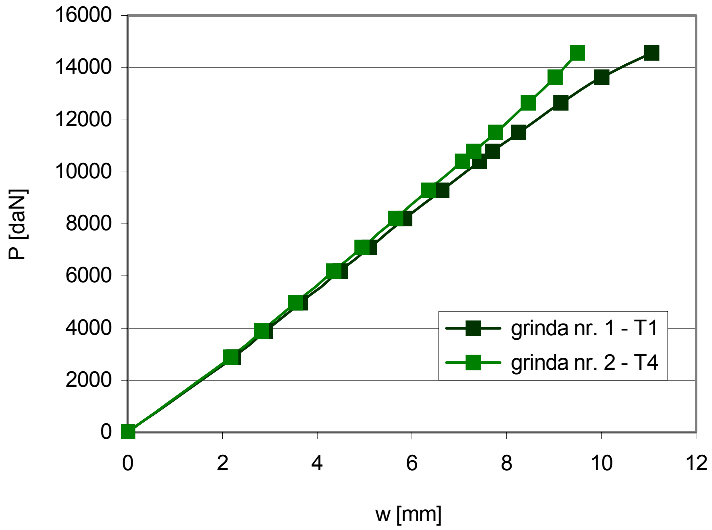

Fig. 5 The load-deflection relationship at midspans.

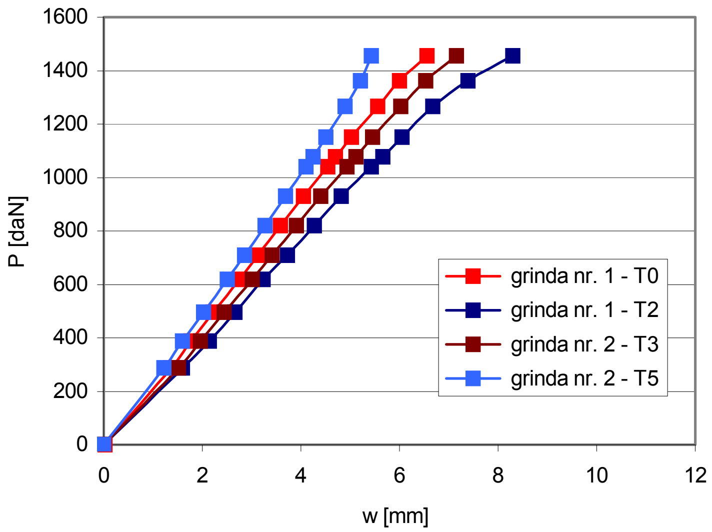

Fig. 6 The load-deflection relationship at 1m from support.

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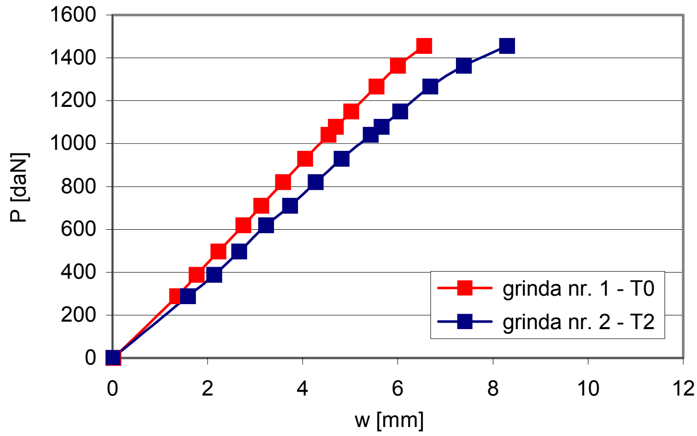

Fig. 7 The force-displacement relationship in the side sections at 1 m from support.

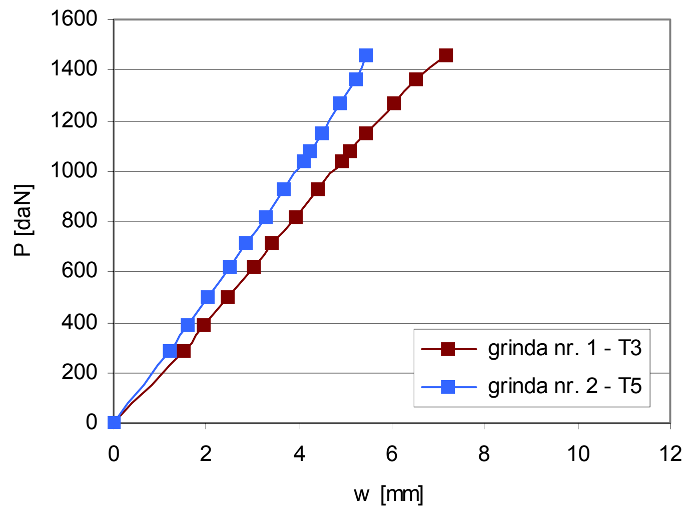

Fig. 8 The force-displacement relationship in the side sections at 1m from support.

After unloading, there occurred insignificant permanent deflections (0.763mm for the beam No. 1 and 0.522 mm for beam No. 2). The curve shape at the reloading indicates a trend of buckling, especially at the beam No. 2 (see Fig. No. 10).

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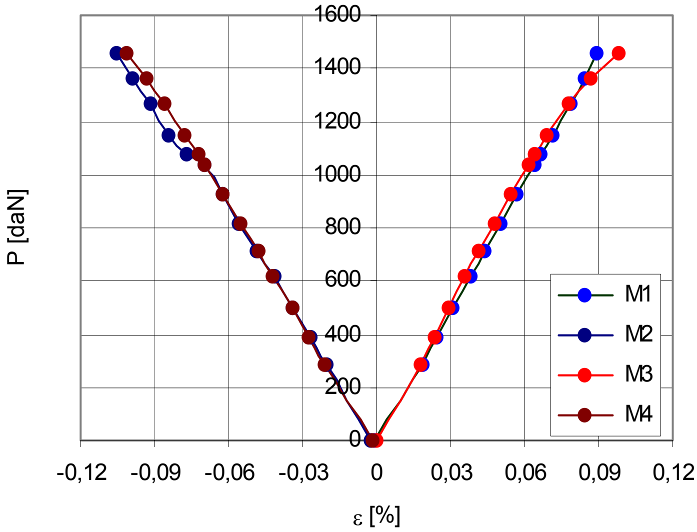

Fig. 9 The force-strain relationship in the middle span of beams.

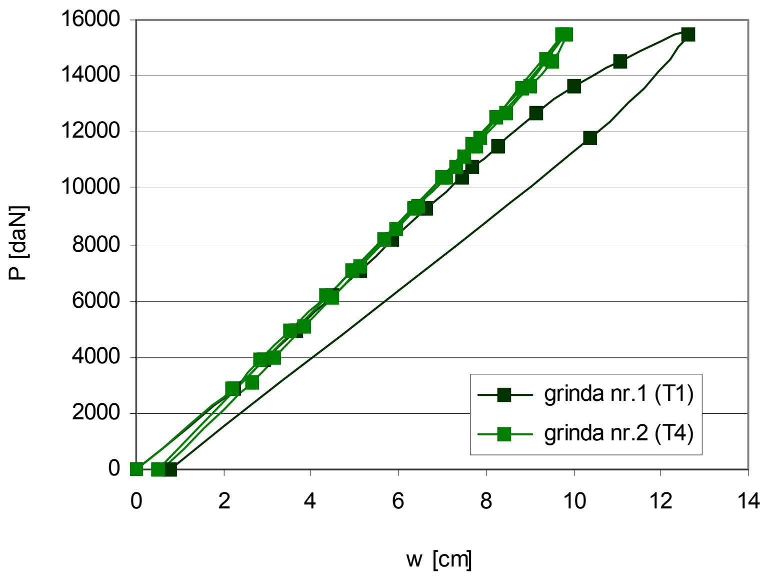

Fig. 10 The load-displacement relationship at the middle span of beams after unloading.

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When the problems correlated to the buckling of the loading system were fixed, the system was reloaded. On this occasion it was noticed that the curve shape is similar to the initial one, with some recovery of beam No. 1 (Fig. No.11). Next, the system was unloaded, (see Fig. No.12).

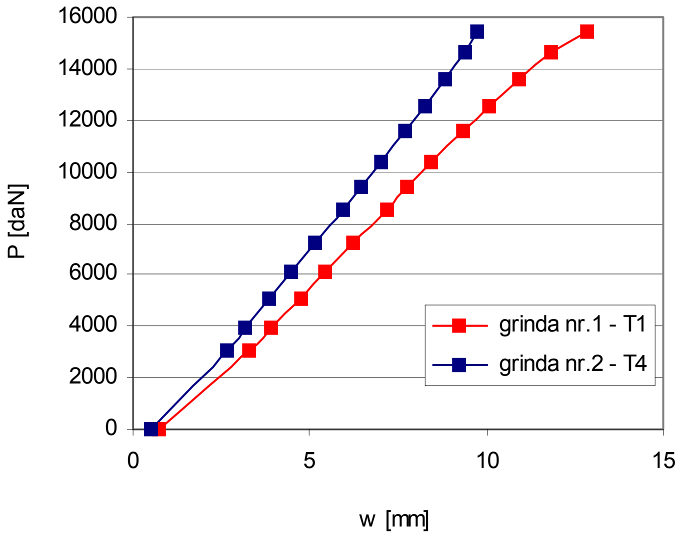

Fig.11 The load-displacement relationship at the midspan of beams after reloading.

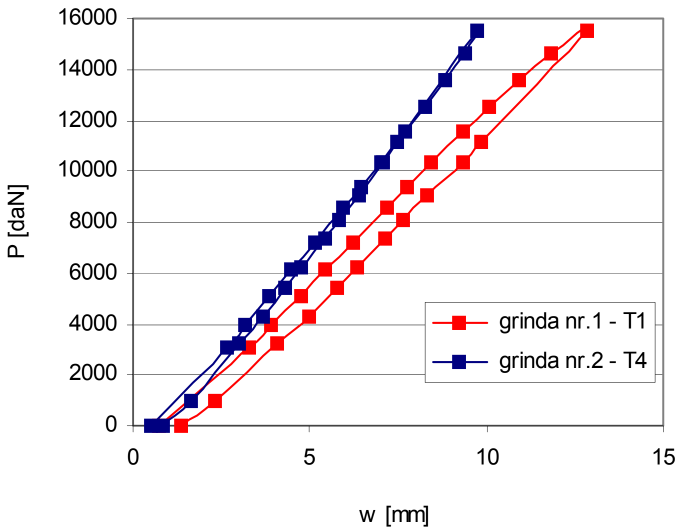

Fig. 12 The load-displacement relationship at the midspan when unloading (after reloading).

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The results indicated a better behavior for the beam No. 1 in comparison to the initial situation. The Figure No.13 presents the load-strain relationship for the beam No. 1; one can observe a symmetrical behavior of both flanges.

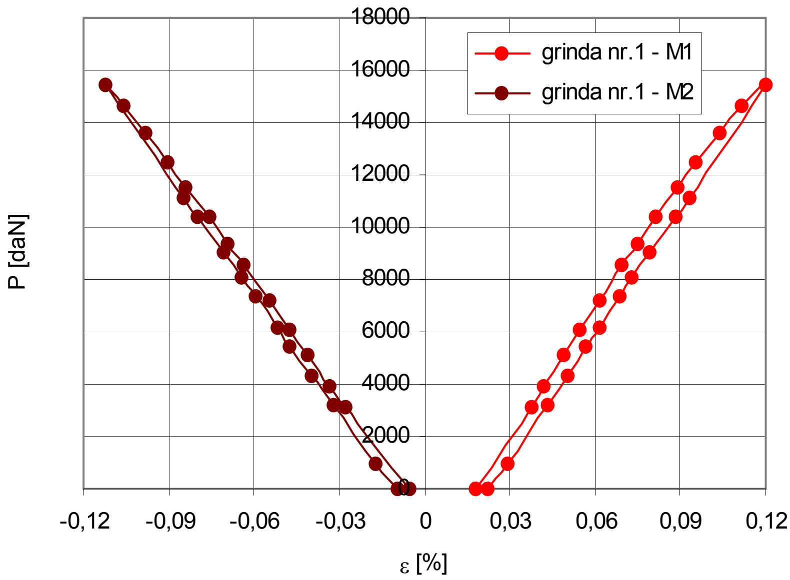

Fig. 13 The load-strain relationship for beam No. 1.

In the last stage the loading process continued until the elements collapsed. Figure No. 14 depicts the load-maximum displacement relationship for both beams. The relationship is disturbed because at the yielding point the forces that acted on the system could not be symmetrically controlled. The Figure No. 14 presents the average loads applied to the system. Even though, one can notice a smaller increase of the deflections for beam No. 1 and increased deflections for the beam No. 2, where the buckling phenomenon was obvious.

The Figure No.15 presents the start of the buckling process for both beams. Figures No. 16 and 17 show some views of the deformed specimens due to the local and general buckling.

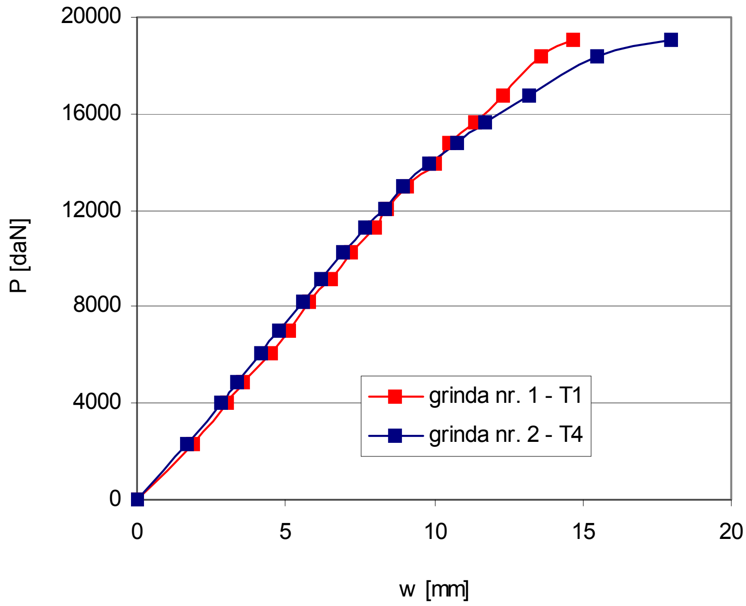

Fig. 14 The load-displacement relationship at the midspan of beams after reloading.

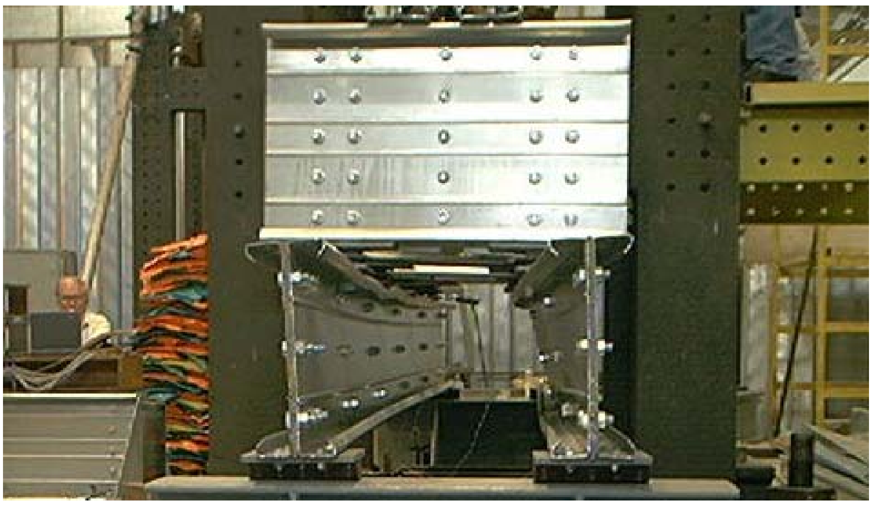

Fig. 15 The buckling phenomenon.

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Fig. 16 The overall buckling – detail.

Fig. 17 The local buckling – detail.

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3. CONCLUSIONS

- The buckling occurred before the loading level corresponding to the design resistance;
- The buckling occurred by a general effect, the further ballasting leaded to the local buckling effects;
- The elimination of the profile deformations due to the manufacturing, packaging and transportation processes, etc. because the buckling phenomena can be initiated at lower loads than the design values;
- The adequate make-up of the connecting elements at the beam ends to the supports because they can lead by deformation to the overall buckling;
- The structural elements (KB) must be provided with additional elements in order to assure the overall stability to bending and torsion effects;
- The computational checking of the structural elements (purlins, beams, columns) must be also performed to avoid the stability loss by bending-torsion.

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