ENGINEERING MEASUREMENTS IN SPHERE OF BUILDING CONSTRUCTIONS AND MATERIALS TESTING

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ABSTRACT

Selected realized technological solutions of special engineering geodesy problems in the field of testing the building constructions and materials are presented in the article. The technological procedure of bridge supporting piers tilt precise measurement at Jezernice viaduct by means of the vault supports horizontal shift measurement in course of the loading test is described. Further the technology of deformation measurement of small specially strengthened vaults in course of destructive static loading tests, and also the methodology of long time deformation surveying of specially strengthened panel elements and the HEA steel profiles deformation surveying are presented, the results of which supply the data for the qualitative analyses by strength testing.

1. INTRODUCTION

In recent time there are more frequently appearing the tendencies to reduce, or eliminate the use of geodetic methods for deformation surveying also in the field of building constructions and materials testing. Constructing specialists prefer the employment of e.g. tensometers, clinometers, laser levels, dial gauges etc. for determination of mutual relations inside the constructions. On the other side, the geodetic methods are advantageous from the costs point of view, and also by rather fast and reliable evaluation of the relative and absolute constructional geometric changes. The paper presents the selected realized technological solutions of special engineering geodesy measurements in the field of the constructions and materials testing.

2. MEASUREMENTS DURING THE JEZERNICE VIADUCT RECONSTRUCTION

The Jezernice Brick Viaduct (JBV) is a part of the high speed railway line Přerov – Hranice na Moravě, in km 203,000 of its kilometrage. It is a historic arched bridge built in 1842, of 411 m length, with 42 spans (Fig. 1). The bridge has continuous arch structure superimposed on stonework pillars with shallow basements. Clear width of larger vaults is 7,6 m, the clearance of smaller vaults located at the combined pillars is 5,7 m. The JBV is parallel to the Jezernice Stone Viaduct (JSV) built in 1873. Both the viaducts were in turn reconstructed in years 1998 - 2001. In course of reconstruction the damaged brick vaults were taken down to tops of the stone pillars. The arches were rebuilt as reinforced concrete

arches with brick veneer which retained the original character of the viaduct. The filling above arcches was realized by light concrete filler. Above it the reinforced spreading concrete slab was built up with connection to reinforcement of the previously reconstructed neighboring JSV, the waterproofing and the bridge superstructure.

This technologically new way of reconstruction using the lightweight concrete filler caused the need for monitoring the construction during realization and after completion, within the checking static loading test before the reconstructed viaduct could be put into service. Measurements on selected vaults during the check loading test covered the vertical shifts of vault heads and supports, the vertical shifts of pillars, and the horizontal span changes of vault supports. The vertical shifts were measured by high precise leveling method and by precise trigonometric method. Horizontal span changes were measured by technology of horizontally suspended invar wire. The invar wire was suspended horizontally (Fig. 2) by means of special fixtures (Fig. 3) on the measuring markers at vault supports.

Fig. 1. – Jezernice Viaduct.

The determination of change of a horizontal dimension is based on principle of sag change of a horizontally pendent wire caused by the change of fastening points distance (Fig. 4). It is possible to measure the sag change in vertical direction with help of dial gauge on accuracy level 0,01 mm, by means of a vertical wire suspended from the center of the horizontal wire. It is possible to fasten a scale to the vertical wire, and to independently measure the vertical deflection change geodetically by high precise leveling in connection to the control measuring marker outside the deformation zone.

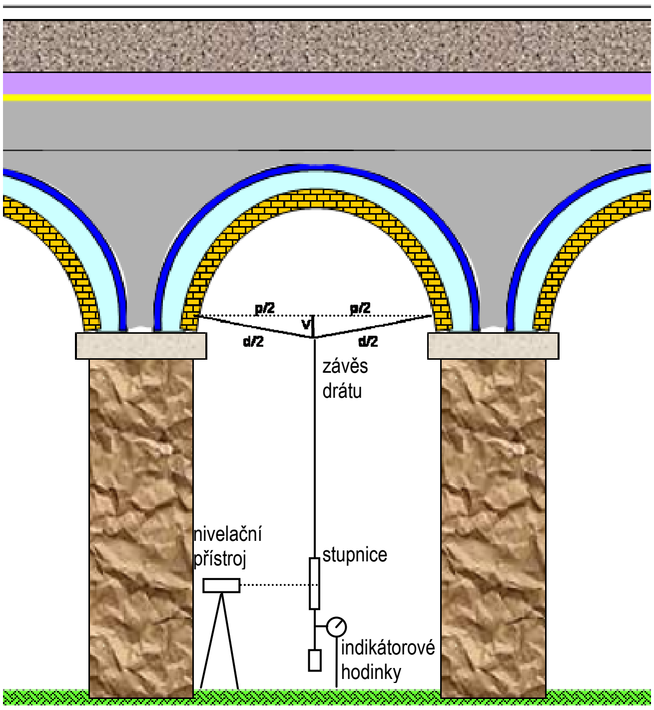


Fig. 2. – principle of measuring technology.

Fig. 3. – combined measuring mark (upper) and mark for invar wire (lower).

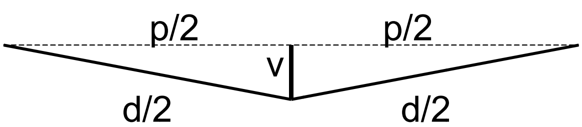


Fig. 4. – geometrical principle of horizontal shift measurement.

Change *dp* of hypotenuse *p* caused by change of sagitta *v* in a triangle is given by relation

$$\left(\frac{p}{2}\right)^2 = \left(\frac{d}{2}\right)^2 - v^2 \qquad , \tag{1}$$

then the change *dp* in relation to the change *dv* is

$$dp = -\frac{4v}{p}dv = -kdv \tag{2}$$

where the ratio $\frac{4v}{p} = k$ is constant for each particular suspension and is depending upon the magnitude of wire deflection. The coefficients can be chosen so that k = 0,1 , i.e. that the horizontal change manifest itself ten times in vertical direction – it can be measured with help of dial gauge or by precise leveling.
Accuracy of the horizontal change determination is given by standard deviation

$$\sigma_{dp} = \frac{4v}{p}\sigma_{dv} \tag{3}$$

The accuracy in determining the deflection change (in vertical direction) from 0,05 to 0,1 mm corresponds to the accuracy of 0,005 to 0,01 mm in the horizontal direction. It is necessary to introduce the correction of temperature difference, and correction of vertical shifts of the wire suspension points. Considering these influence the accuracy will be slightly lower at level from 0,01 to 0,03 mm. Maximum detected change in the horizontal span of the vault supports during the static loading test was 0,14 mm (Bureš et al., 2001).

3. DEFORMATION SURVEYING DURING THE STRUCTURAL ELEMENTS TESTING

In connection with reconstructions of older building constructions which already do not fulfill the criteria of carrying capacity and are faulted by cracks the problems of their effective rehabilitation is solved. One possible way of rehabilitation of such structures is the applying of externally bonded reinforcement which ensures hardening of the carrying elements. New hardening technologies follow up the properties of composite materials, i.e. materials composed at last of two components of very different properties which if combined ensure the properties outside the possibilities of the traditional materials. The disadvantage of these materials is their higher cost and therefore they are used only exceptionally in special cases. The research subject is focused on the effective applications of various hardening materials to the carrying elements and subsequent monitoring of their behavior during long term loading. One example of tested building elements constituted the hollow reinforced concrete not pre-stressed ceiling panels (Fig. 5). The elements tested

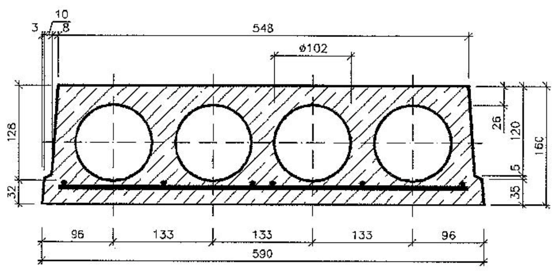


Fig. 5. – PREFA ceiling panel.

Fig. 6. – strengthening by carbon lamellas.

were the standard products of PREFA Brno, 5.380 mm long with modified interior reinforcement. The concrete strength class according to ČSN 73 1201-78 is defined by grade B20. State of reduced carrying capacity of samples of new building elements had been simulated by partial pre-destruction by short intensive loading so that there appeared cracks in tensioned concrete areas. Their deflections were measured in course of the pre-destruction, and the elastic modulus values were derived from the deflections and magnitude of loading. The elements then were sorted into six groups – each group represented one type of strengthening. The lamellas Sika CarboDur S 512 along with the cleaner Sika and the adhesive SikaDur 30 were used for the strengthening (Fig. 6).

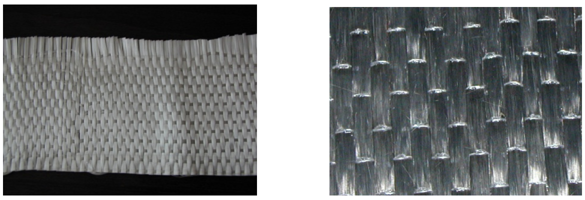


Fig. 7. – detail of carbon-based fabric.

Fig. 8. – strengthening by carbon fabric.

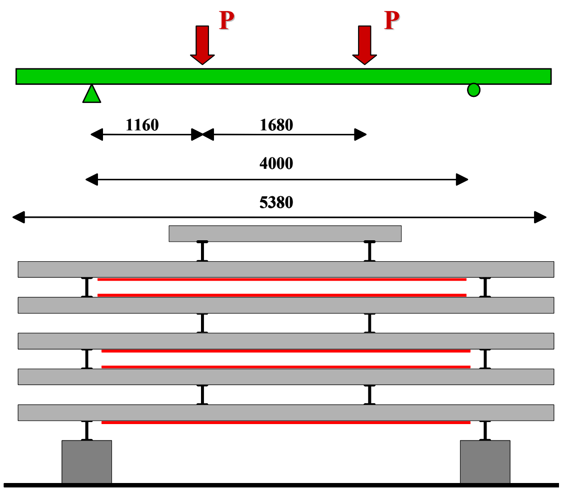


Fig. 9. – ordering of carrying elements.

Second strengthening technology used was the carbon fabric Sika Wrap (Fig. 7) along with the adhesive SikaDur 330 (Fig. 8). The mechanical-physical properties of the applied fabric are similar to pre-stressing. Strengthening material had been applied to one side of the element which is tension strained. The panel then were composed into groups of 5 pieces (Fig. 9) so that the uniform strain of all the group elements was ensured. The steel I-profiles were used for strain transmission. The element group was loaded at the top by a weight which along with resting panels loaded the lowest panel up to 60% of its carrying capacity. The single panels in group were then exposed to different loadings according to weights above. The strengthening is illustrated by thick line. The representing sample counted 6 panel groups (Fojtl et al., 2004), (Dibelka et. al., 2003), (Horák et. al., 2007).

Subjects of measurement of the panel groups were the relative deformation of interior and exterior reinforcement (measured by resistance strain gauges - Fig. 10), the relative flexure changes of neighboring panels in a group (measured by insert deformmeters - Fig. 11), the strengthening slippages caused by loading or by different thermal expansivity (measured by insert deformmeters - Fig. 12), the relative deformations of shear, tension and compression zones – development of cracks in concrete (measured by Hollan´s dilatometer - Fig. 13). The vertical deflections of two lowest levels of the panel groups, and the subsidence of the strip foundations carrying the panels. Were measured geodetically by high precise leveling using the precise optical level Zeiss NI 005A and digital level Leica DNA03 (Fig. 15). The results evaluated were the vertical shifts and the deformations of two

lowest panels at half and quarters of span between the supports. Besides the standard precise invar leveling staffs also the special pendent short invar staffs (Fig. 14). Layout scheme of geodetic measuring markers is shown in Fig. 16. The vertical shifts were computed as differences of markers heights in particular measuring epochs determined by LS adjustment in respect to the control system marked on surrounding stable building objects. Vertical accuracy was derived from the LS adjustment. Relative accuracy was 0,05 mm, absolute accuracy in respect to the control system was 0,05 - 0,2 mm. Deformations of the measured panels was determined as the difference of the total vertical shift of the unit foundation + panel group and the vertical shift of the strip foundations. Considering the unequal subsidence of the strip foundations it was necessary to eliminate this influence individually for each panel group by linear interpolation using the vertical shifts of 4 markers on the strip foundations. The monitoring had been continued over three years period, in 20 measuring epochs.

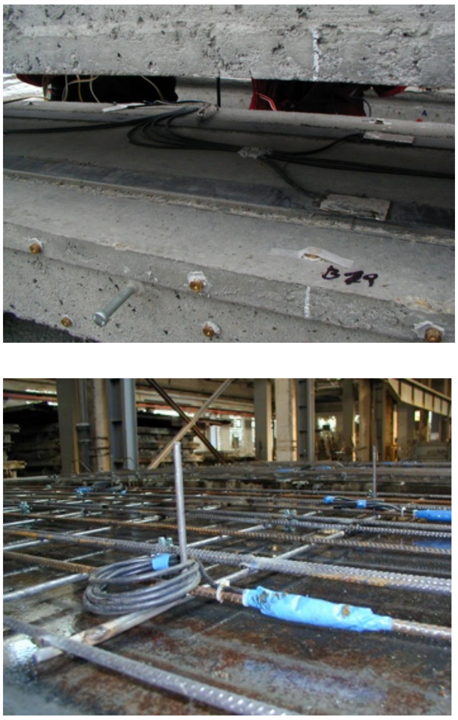


Fig. 10. – resistance strain gauge.

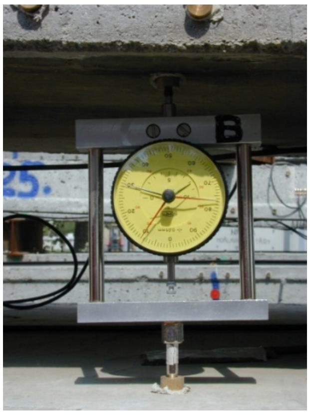


Fig. 11 – measurement of deflection changes

Fig. 12. – slippage measurement.

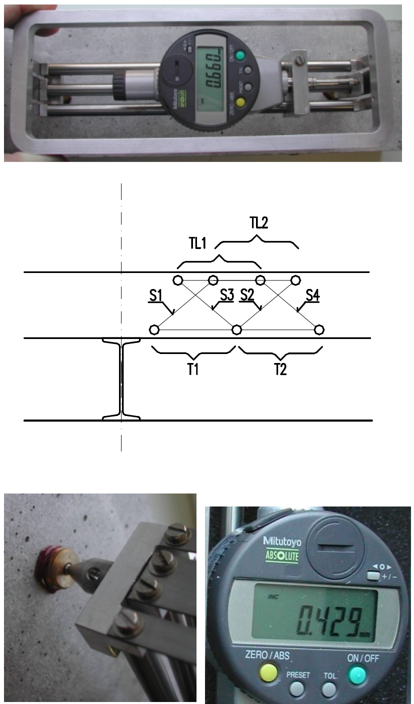


Fig. 13. – measurement of tension and pressure zones.

Deformations between epochs were in range from tenths to several millimeters, total deformations of the constructional elements after three years were up to 35 mm. The foundations subsidence was in range of a few millimeters depending upon the external

Fig. 14. – pendent staffs.

Fig. 15. – precise leveling.

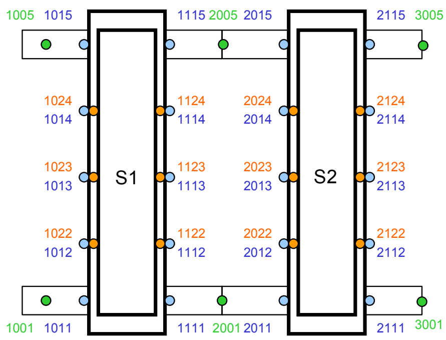


Fig. 16. – geodetic markers layout scheme.

conditions because the strip foundations were concreted directly upon the stabilized ground not under the freezing level. From the point of view of evaluation of the “pure” building element deformations the unequal shifts of the strip foundations turned out to be not a negligible factor which had to be eliminated from the total computed deformation. A deformation evaluation example of the unit foundation plus lowest panel is in Fig. 17 in form of isolines, the graph illustrates the increase of deflections over time.

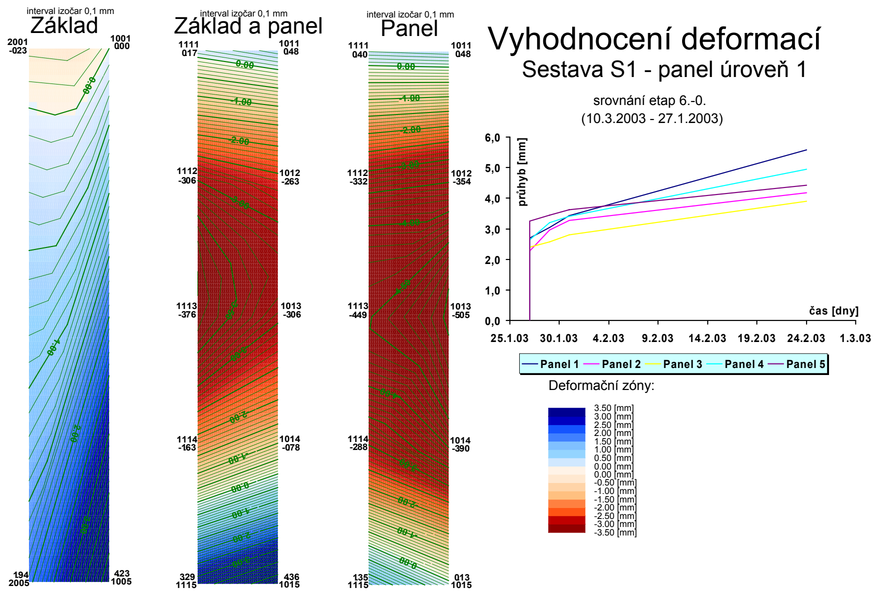


Fig. 17. – deformation evaluation of footings + panel group array.

Another research theme was the Helifix technology employed by strengthening of brick vaults. The vaults with and without the additional constructional strengthening had been tested. The span of the vaults tested was 2,6 m, the sagitta in vault head was 0,75 m, and the width was 0,90 m. The vaults were made from bricks with mortar binder. The vaults were subjected to a crushing test by pressing machine during which their spatial deformations were measured. The vault measuring markers were set up at head, at quarters of span, and at support places so that it would be possible to measure its spatial changes. The vertical changes were measured by means of suspended steel tapes with scales in millimeters, plummet loaded (Fig. 18). The horizontal changes were measured with electronic total station using the reflection foils Sokkia. With respect to the character of the vault geometry deformations caused by loading, the geodetically measured values were the vertical, axial and lateral changes. The loading increased step by step until the vault destruction stage.

Fig. 18. - tested vault.

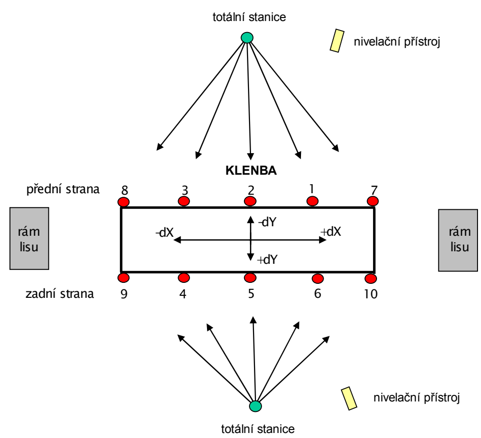


Fig. 19. - vault measuring scheme.

The vertical shifts were measured employing the high precise optical leveling methodics and instrumentation. The vertical changes were read on scales of pendent tapes using the instrumental optical micrometers, with accuracy of 0,05 mm. Two independent readings were recorded for each deformation stage. The optical leveling instruments Zeiss NI005A

and Zeiss NI007 were used. The changes in horizontal plane (in axial and lateral vault directions) were measured by polar method using the electronic total stations Topcon GTS - 300 and Topcon GTS-6A (angle accuracy 0,5 mgon). The configuration of mutual placement of the instrument and the measured vault (Fig. 19) enabled to determine the axial vault changes with accuracy expressed by standard deviation = 0,1 mm, and in lateral vault direction by standard deviation = 0,5 mm, with respect to the need of distance measurement.

Supporting constructions are often structured from steel profiles of I, H, U, T, O forms. Their carrying capacity is influenced i.e. by accuracy of the geometrical shape. During extreme loading there appear the first local failure in place of the greatest geometrical imperfection of the carrying element. Subjects of deformation surveying were the steel profiles HEA 140 of 3 000 mm length and the steel columns TR 150 of 3 000 mm length.

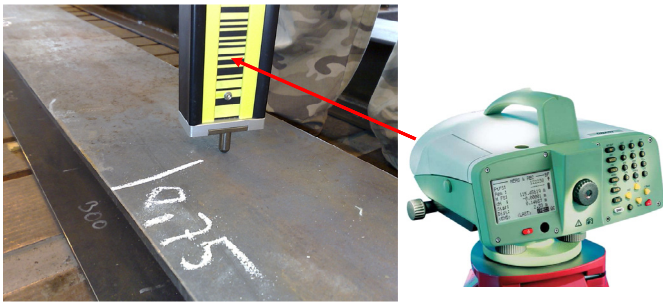


Fig. 20. – measurement of HEA profiles.

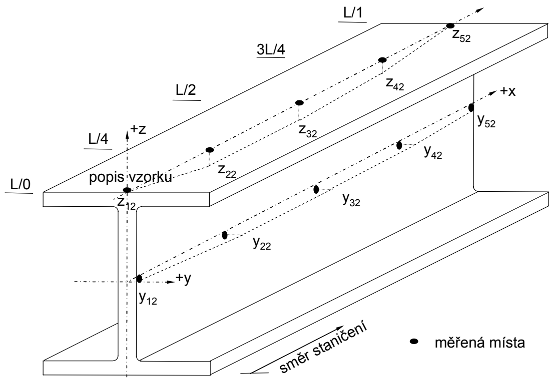


Fig. 21. – axial deformations.

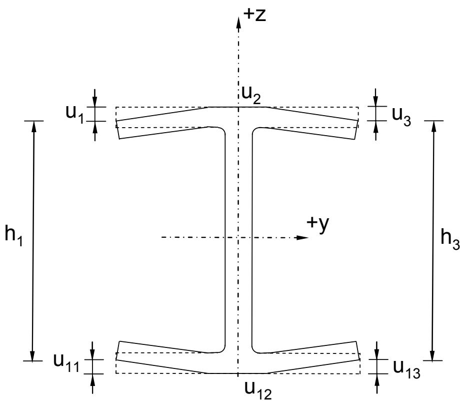


Fig. 22. – deviation of flange inclination.

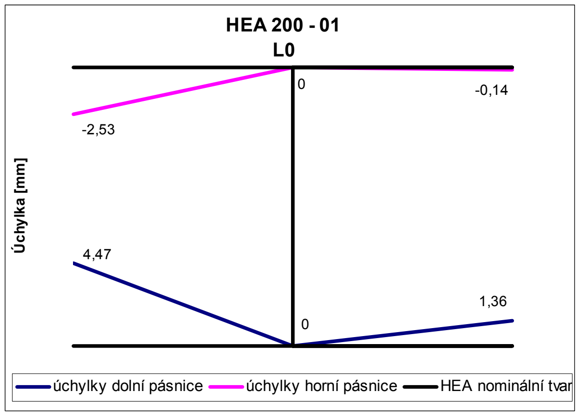
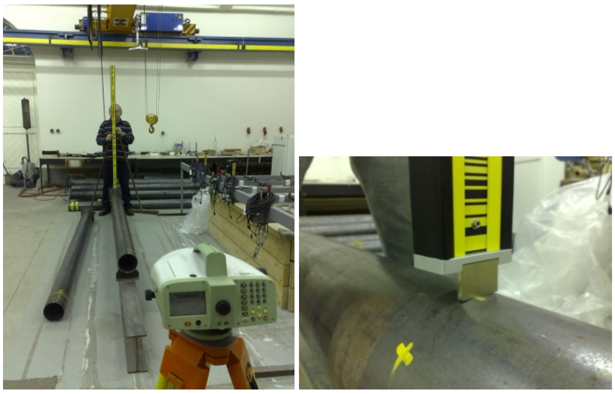


Fig. 23. – evaluation of flange inclination.

Measured were series of 9 samples of each profile type. The aim was to determine the maximal axial deformation of each particular sample.



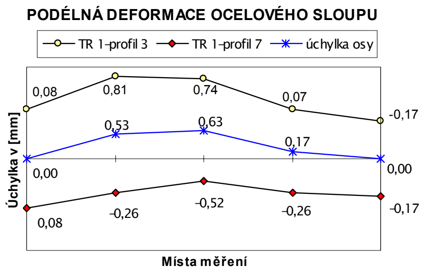


Fig. 24. - TR columns deformation measurement.

Fig. 25. - TR columns deformation evaluation.

The steel samples were then tested for buckling, and subsequently the flexure direction of the steel element was explored on base of the axial deformations.
The high precise leveling method (HPL) was used for the measurement of steel samples deformations using precise digital level Leica DNA03 and special staff GWCL with invar code scale designated for industrial surveys. The samples were measured in horizontal position. The HEA profiles had been measured laying on the concrete floor (Fig. 20), the columns had been measured on special steel bench equipped with bearings (Fig. 24) which enabled to rotate the sample tested. The measuring staff was equipped with special foot which enabled the exact contact of the staff and the measured sample. The measurement had been carried out in sample axial direction in places 0, L/4, L/2, 3L/4, L, where L is the sample length. The HEA profiles were measured axially in two mutually perpendicular directions, i.e. in web direction (coordinate axis Z) and in lateral direction (axis Y) (Fig. 21). Subjects of the measurements were also the flange inclinations (Fig. 22 and Fig. 23). On the columns the coating surface were measured in four mutually opposite places, and at both ends the exterior diameters were measured by a slide gauge. Subjects of evaluation were the coating deformations and the axis course deviations (Fig. 25). The values of deformations were recalculated relative to the endpoints of the steel profile.
The accuracy achieved was derived from the analysis of repeated measurements in form of standard deviation which amounted to = 0,03 mm for the axial deviation of the HEA profiles and the axial deviations of the column coatings The accuracy of column diameters determination was = 0,1 mm.

4. CONCLUSION

Non-geodetic measuring methods often give high precise relative values describing the mutual position within the measured construction which in no case can yield complex information about its behavior and about its general deformation in space and time. Therefore it is always necessary to include also the results of geodetic measuring methods into the interpretation which have character of absolute values. **This paper was prepared with support of the research project MSM 0021630519.**

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