

**SELECTED METHODS OF MONITORING AND BACK ANALYSIS
OF PILE AND DIAPHRAGM WALLS**

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Abstract

Results of precise monitoring provided in two parallel lines using the sliding micrometre were used for back-analysis to determine moment, shear force and superimposed lateral stresses distribution acting on the wall. The method of back analysis will be presented in the paper together with the recent development of the application of BOTDA (Brillouin Optical Time Domain Analysis) for geotechnical structures to innovate the way of above instrumentation. The present trial version of the developed software for back analysis brings benefits in detailed knowledge of distribution of actual internal forces in the structure on the construction site, which can be compared with those calculated at the design stage.

Key words: retaining structures, geotechnical monitoring, internal forces, back analysis, fibre optics, BOTDA

1 INTRODUCTION

In case of large-scale excavation works, application of analytical methods is general for safety control of earth retaining wall. Real behaviour of these structures is controlled by geotechnical monitoring with use of different instruments. The measured displacements and strut or anchor forces are usually compared with the values stated at the design stage.

The usual design of retaining walls is based on soil-structure interaction. The structure is considered as flexible and it interacts with the changing lateral stresses during staged construction process. On the other hand, the high stiffness of the structure results in significant changes of bending moment and shear force distribution, when deflection occurs, both in case of design and in-situ behaviour.

Comparison of measured data to those before computed or derived at the design stage gives not a picture of the distribution of internal forces in the wall. Although monitoring results are used for assessment of structural behaviour, the internal forces are not under direct control. This is the main reason for development of different methods of back-analyses, which were published by e.g. Machida, Omote & Yanagihara (1999), Likar & Vukadin (2003), Lee et al. (2004), Puzrin & Schmid (2007), Dey et al. (2011) and Zalesky et al. (2011).

2 METHODS OF BACK ANALYSIS

Flexible earth retaining walls with tiebacks or struts are often used as a supporting system for ground excavations. At the design stage, there are used different analytical approaches describing the relationships of soil – structure interaction (Lee et al. 2004). For staged constructions, development of contact stresses (on the rear side and or on the excavation side) shall be calculated according to construction stages. The stage as partial excavation followed by strut or anchor application offering

support is repeated in cycles till the final excavation level is reached. This process affects development of the soil – structure interaction. The diagram of pressure – displacement curve is modified with respect to construction stages and shall be used according to the Figure 1.

This type of calculation following construction stages is reliable to stress and displacement development history. Proper calculation offers data which can be controlled by geotechnical monitoring, as deflection curves of the wall and anchor forces at every construction stage.

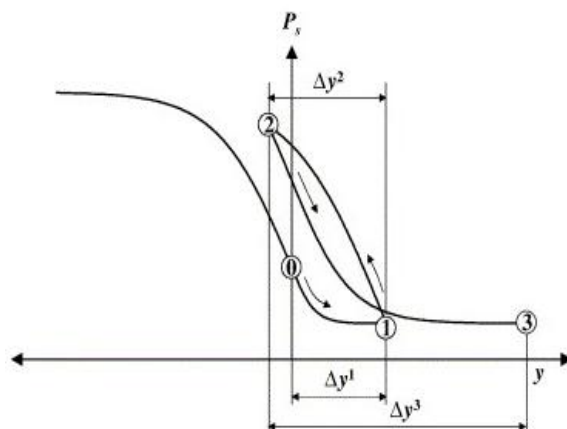


Fig. 1 Non-linear $P - y$ curve for staged excavations (in Lee et al. 2004)

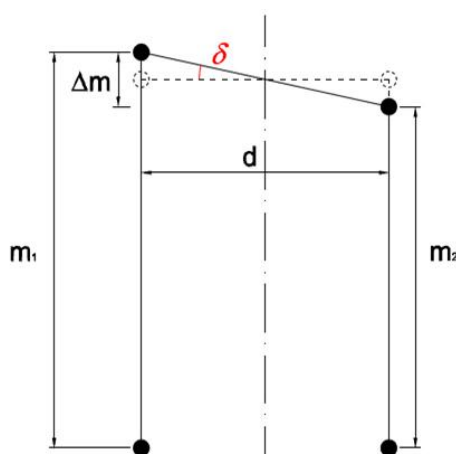


Fig. 2 Measured strain in the wall body; Legend: d - distance of axes of measuring casings or sensing optical fibres, m_1 , m_2 – distances between measuring marks, Δm – axial change of measuring marks distance in both casings, δ – change of inclination of the wall body central line

Monitoring tools together with methods of calculation of anchored retaining walls are under development. Commonly used practise is to monitor anchor forces with use of different load cells and displacements of anchor heads by total stations. This is the case of point-wise displacement measurement. Stresses in the wall body can be controlled locally in the reinforcement cage with the use of added so called sister bars (instrumented with strain sensors). Inclinometers can be used to

measure wall deflection if the wall is really flexible and or if the wall inclination is expected in order of mm/m at least.

Line-wise monitoring of the wall body can be made by sliding micrometre in pairs of casings or by the use of fibre optics application e.g. chains of Fibre Bragg Gratings (FBG) sensors or sensing cables for Brillouin Optic Time Domain Analysis (BOTDA). The monitoring offers pairs of strain distribution along the whole structure in the section. Measured data could be used for calculation of inclination changes along monitored structure or for deflection curve calculation according to Fig. 2. This approach results in time dependent deflection curve which follows construction stages.

Application of the monitored data at higher level needs back analysis to demonstrate development of bending moment and shear force distribution in the structure, which are important in judgement of safety of the structural design. Relationship presented in Fig. 1 is rather difficult to verify other way than by modelling and back-analysis. Direct control by monitoring of total stresses on the contact of the wall and soil sequence is not efficient in giving reliable data. This is due to installation problems together with technology of construction of retaining structures as pile / diaphragm walls. Pore-water pressure is monitored in stratified soil sequences and if cluster of pore pressure sensors is used, reliable data can be gathered.

2.1 Analytical techniques

Various analytical techniques proposed in back analysis for earth retaining walls have problems with confrontation of measured values of displacements of the wall and calculated ones by analyses. Machida, Omote & Yanagihara (1999) used in back analysis extended Kalman filter algorithm, and proposed the new establishment method of strut spring constants obtained from the relation of the measured displacements of wall to axial forces of struts in back analysis.

Puzrin & Schmid (2007) referred about back analysis made on unstable slope, where thickness of sliding soil mass was determined by monitoring. The analysis was based on analytical approach using complementary data of deflection of the stabilizing wall, Fig. 3.

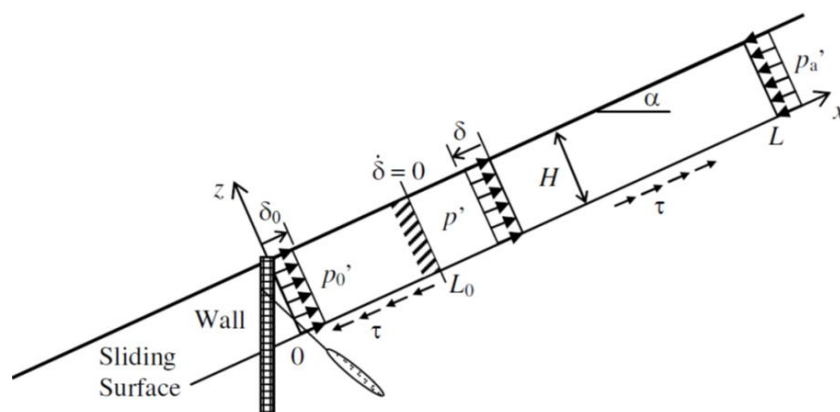


Fig.3 Schematic layout of the constrained landslide model (Puzrin & Schmid 2007)

The solution was based on equilibrium conditions and known kinematics of the slide. It was extended to the time dependency of the slide activity. It is suitable for estimation of the action on the wall due to the sliding mass, but not for calculation of moment and shear force distribution in the structure.

Selected results of monitoring of an anchored pile wall together with results and problems of back analyses using different interpretation techniques of deflection curves were presented by Zalesky et al. (2011). The use of instruments of line-wise monitoring like sliding micrometre casings in the pile wall was approved by our own installation.

2.2 Finite Element Method application

A multi-anchored pile retaining wall was constructed to protect the cut made for the cut and cover section of the twin Trojane Tunnel. Deformation monitoring surveys were conducted and measurements of movements were carried out throughout the construction cycle of the wall and beyond. In order to determine design parameters for the soil strata embedded in a complex geological sequence, the soil-wall interaction was back analysed using the finite element method. Soil parameters were carefully assessed from the laboratory data and adjusted by numerical back analyses. The parameters were verified on the neighbouring cut and cover sections of the same tunnel and later used in routine design. Generally, a good agreement between predicted and observed behaviour was achieved suggesting the adequate determination of the geotechnical model and the soil parameters, (Likar & Vukadin 2003).

Dey et al (2011) referred about a full scale experimental study on a free flexible retaining wall subjected to seismic loading. Embedded retaining wall in cohesive soil had a free height of 6m and was composed of two rows of reinforced concrete piles 800mm in diameter. Deflection curve and the wall inclination were monitored with use of conventional Inclinometers.

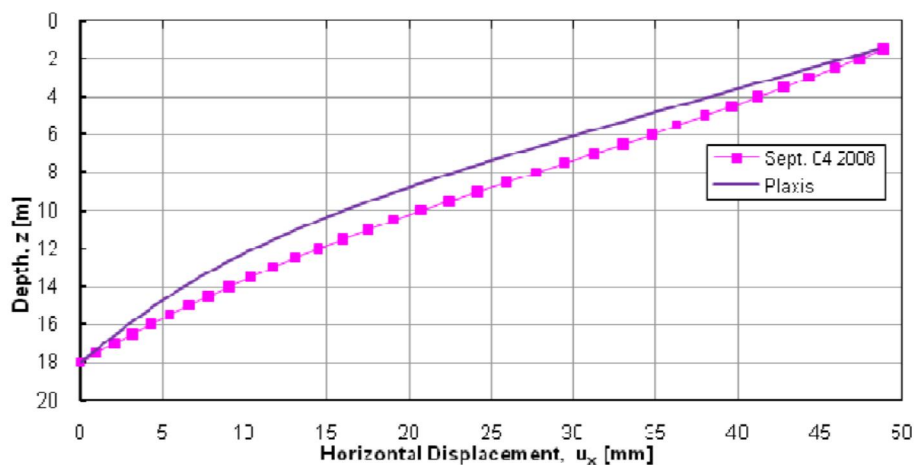


Fig. 4 Comparison between the first measurement and the numerical analyses results, (Dey et al. 2001)

The study was focused on interpretation of the monitoring results with use of numerical tools describing the wall behaviour and interpretation of bending moment and shear force distribution in the structure. Comparison between computed deflection curve and measured one is presented in the Fig. 4. The plot shows a reasonable agreement in the wall inclination. The deflection curves have different curvatures, especially in the wall embedded part resulting in overestimating of the computed bending moment. In fact the wall demonstrates behaviour with higher stiffness.

The above mentioned back analyses are long-term activities due to monitoring, laboratory and field testing of soils and in case of FEM time consuming computations by model calibration and parameter studies provided to obtain an acceptable fit to measured data. Direct interpretation of measured data

for determination of internal forces in the wall body will bring a new benefit in determination of the distribution of lateral stresses along the wall, if the primary data describing strain distribution along the section through the structure is line-wise monitored with high accuracy. The analytic approach can be very effective and reliable.

The role of underground water, especially in unstable areas, where different walls are used to stop the soil mass movement, is not discussed. The water changes load diagrams significantly in cases of independent water horizons and it is rather difficult to monitor.

3 ANALYSES OF MEASURED DATA

The method of determination of acting loads and internal forces distribution described below can be used particularly for “slender” structures which react to external actions by deflections. The distribution of deformations along the structure should be measured with sufficient accuracy, so that it can be used as input data for the proposed back analysis. Examples of suitable measurement instruments or methods are: Inclinator, Sliding micrometer, Trivec or fibre optics BOTDA method. In the following text an application of anchored pile wall instrumentation for high precision deformation measurements is presented.

The method is valid only for determination of such a loading which induces bending of the structure. In case there is additional shift or inclination of the structure, an extra analysis is necessary.

3.1 Anchored pile wall and its instrumentation

The site, near Usti nad Labem in the Czech Republic, is located on the north-east side slope of an excavated brown coal open-pit mine. The highly exploited tertiary brown coal basin is mostly composed of clays, sedimentary / volcanic complexes, coal and quaternary overburden – loess, gravels or sands and regionally metamorphosed rocks under the tertiary fill and coal seams. Slope movements not only due to mining activities but also due to natural slope structure and site conditions often occur in this part of the coal basin. The Rabenov hill above the side slope is of volcanic origin. The northern slope of the hill has stratified soil structure. There are sequences soils of various grain sizes with sandy to gravelly layers (mostly pyroclastics), which are bounded by fine grained deposits. This subsoil profile contributes to high pore-water pressure in the area. Randomly distributed preferential water paths result in locally appearing springs, which form confined artesian aquifers. The majority of small spatially limited slides together with deep and large ones are initiated by high pore water pressure. Those were the reasons for design and construction of the pile wall and stabilizing embankment below. Two piles of a retaining system were selected for instrumentation. Two parallel measuring casings were installed in both piles. A scheme of one instrumented pile is presented in the Fig. 5. The measurements have been performed since 2007 with Sliding micrometer and high precision Inclinator. The Sliding micrometer is used for axial deformation measurements with resolution $\pm 1 \mu\text{m/m}$ and accuracy $\pm 3 \mu\text{m/m}$ in a combined casing. The high precision Inclinator gives horizontal deformation values (perpendicular to the casing axis) with resolution of 0,02 mm/m. The measuring casings are placed in the piles in the distance d (Fig. 2) in the plane close to the pile wall deflection.

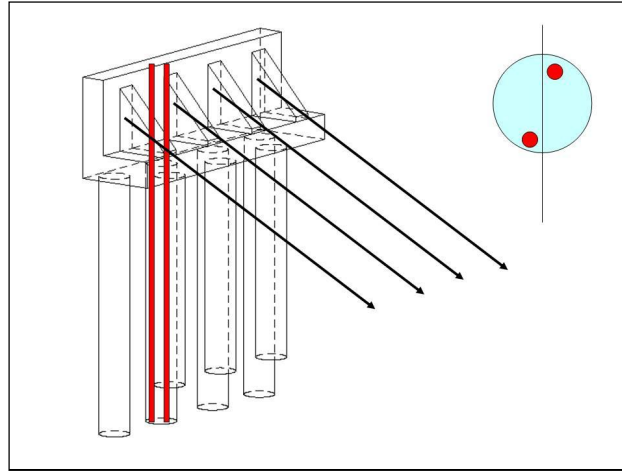


Fig. 5 Schematic view of an element of the anchored pile wall with cross section of instrumented pile and location of an anchor

Changes in the inclination of pile axis are calculated from discrete measured deformations (according to the Fig. 2). In the presented example, Fig. 5, the sets of paired measurements obtained by the sliding micrometer were used.

The results of measurement in the form of changes in the length base line can be considered as the second derivative average of the deflection curve along this section (multiplied by the distance of the axis of the measuring casing from the neutral axis):

$$m_{1,2}(x_i) - m_0(x_i) = \pm \int_{x_{i-1}}^{x_i} w''(x) \frac{d}{2} \quad (1)$$

Consequently it is possible to determine a change in the inclination of the directive of the central line of the cross section of the structure between the nearest measuring marks, respectively the change of the pile cross section rotation:

$$\delta_i = \frac{\Delta m_i}{d} = \int_{x_{i-1}}^{x_i} w'' dx = w'(x_i) - w'(x_{i-1}) \quad (2)$$

where Δm_i is the difference of measuring marks distances in positions x_i, x_{i-1} in both casings,

d is the distance of casings (in the bending direction),

x_i, x_{i-1} are positions of measuring marks in the casing,

$w''(x_i)$ is the second derivative of horizontal displacements of the pile (in the plane of bending) in the position of the measuring mark “i”,

$w'(x_i)$ is the first derivative of horizontal displacements of the pile (in the bending direction) in the position of the measuring mark “i”.

Therefore it is possible to calculate the rotation of the centre line of the pile in the measuring mark position:

$$w'(x_i) = \sum_{j=1}^i \int_{x_{i-1}}^{x_i} w'' dx + w'(x_0) = \sum_{j=1}^i \frac{\Delta m_j}{d} + w'(x_0) \quad (3)$$

These data, along with known stiffness of the structure, are the input data for the analysis of the distributions of the bending moment, shear forces and horizontal loading of the structure. The data files gathered by monitoring were analysed in several ways.

A possibility of fitting the measured values to a higher order polynomial was tested in one of the approaches. The static boundary conditions of the structure (conditionally suitable for a structure anchored at one level) were taken into account as an assumption: a retaining wall anchored at one level. The differential equation of equilibrium used for this procedure takes the form

$$(EIw'')'' = f + \sum_k P_k \delta_{xk} \quad (4)$$

where EI is the stiffness of the structure,

f is the external load from the subsoil (soil, pore-water pressure),

P_k is the measured anchor force,

δ_{xk} is the Dirac measure located at the anchor point.

Then, force and moment equilibrium conditions of the structure apply in an ideal case, and, therefore, the static boundary conditions are met. The limiting condition for the sought minimum values of deflections of calculated and measured deformations in the form of static boundary conditions, however, may distort the solution, particularly in peripheral parts of the structure. This is the reason why a possibility of a discrete solution of relation (4) was analysed using the formula

$$(K_{\varphi\varphi} - K_{\varphi w} K_{ww}^{-1} K_{w\varphi})^* r_{\varphi}^* = f_M - K_{\varphi w} K_{ww}^{-1} f_w \quad (5)$$

where K_{ww} , $K_{w\varphi}$, $K_{\varphi w}$, $K_{\varphi\varphi}$ are submatrices of the pile stiffness matrix K

$$K = \begin{bmatrix} K_{ww} & K_{w\varphi} \\ K_{\varphi w} & K_{\varphi\varphi} \end{bmatrix}$$

r_w is the column vector of horizontal deformations of pile joints,

r_{φ} is the column vector of pile joints' rotations,

and both are subvectors to the pile deformation vector $r = \begin{Bmatrix} r_w \\ r_{\varphi} \end{Bmatrix}$

f_w is the column vector of transformed loads at joints in

the direction of horizontal displacements of the deflected pile,

f_M is the column vector of transformed loads at pile joints due to bending moments.

The asterisks used with individual variables denote their formal adjustments allowing the solvability of the problem.

Subsequently, a suitable load distribution may be searched so that it meets the static boundary conditions, and thus the force and moment equilibrium conditions to which such a vector r_{φ} is designated, which best corresponds to the measured values.

Partial results of the fitting of measured deformation values are presented in Fig. 6 together with calculated actions on the pile wall. The deformation was measured in one metre distances along the pile which corresponds to the nodes in the calculations. The results of calculation of superimposed stresses are influenced by oscillations caused by inaccuracies of measured data and the method of calculation. In the Fig. 6 are presented two trials in oscillation dumping. Application of direct stiffness method on measured deflection curve results in unreliable oscillations similar to results of multiple derivatives (blue curve in the Fig. 6). Suitability of approximation of deflection curve was controlled

by direct stiffness method (DSM) so that approximated displacements and rotations of nodes of the structure cause only small bending moment loading in nodes. Horizontal forces were similar as result of DSM considering and not considering zero bending moment loading nodes. Violet curve in the Figure represents results of DCM with non-zero bending moments in nodes. Because of a simple shape of deflection curve of the pile wall the deflection curve was fitted by higher degree polynomial by the method of least squares according to boundary loading conditions. Results of derivatives are presented by the green curve. The polynomial fit may not be used in the case of multilevel support of the wall.

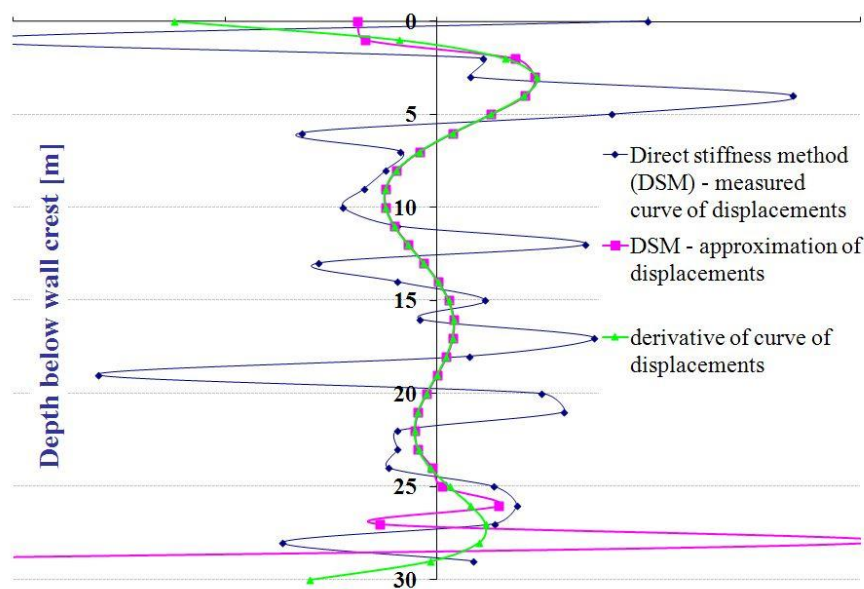


Fig. 6 Loading of pile wall computed from measured and approximated deflection curve using DSM (zero bending moment loads in nodes) and derivatives of pile deflection curve

A “general” solution was developed in cooperation with the Department of Mathematics and it is based on optional method of damping the oscillations of the derivatives of approximated curves of measured data. The functions of the superimposed load acting on the retaining wall are assumed linear in their parts, and they may be discontinuous at given points. This applies to loading with both the solid and liquid phase of soils. An advantage of this solution is the adjustable elimination of the influence of random measuring errors from the point of view of statistics and physics. This requires interventions by an informed user during the calculation, but it allows the evaluation of measured data even in a highly heterogeneous environment and for multilevel support of retaining structures (Pultar et al. 2013).

Input data and additional information required for the calculation:

- dimensions and cross-sectional characteristics of the structure in order to determine its bending stiffness
- boundary conditions (generally static)

- in case of anchored walls: anchors levels and anchor forces (in order to fulfil the equilibrium conditions)
- identification of non-force effects which influence measured deformations
- the locations of discontinuities in the superimposed actions (according to site investigation results) – solid particles and underground water separated

It is possible to insert discontinuities into the superimposed load distribution under calculation according to layers of geological profile or according to artesian ground water appearance. The informed user can adjust the settings of mathematical solution so close fit to known site conditions and measured deflection curve is obtained. The bending stiffness of the examined structure can be set in several intervals according to the changes of cross section parameters.

During the structural analysis it is necessary to determine additional effects as temperature changes which are included in measured values describing structure deformation. From this point of view there is an advantage of the measurement by optical fibres because these are able to provide information about the deformation and temperature distribution.

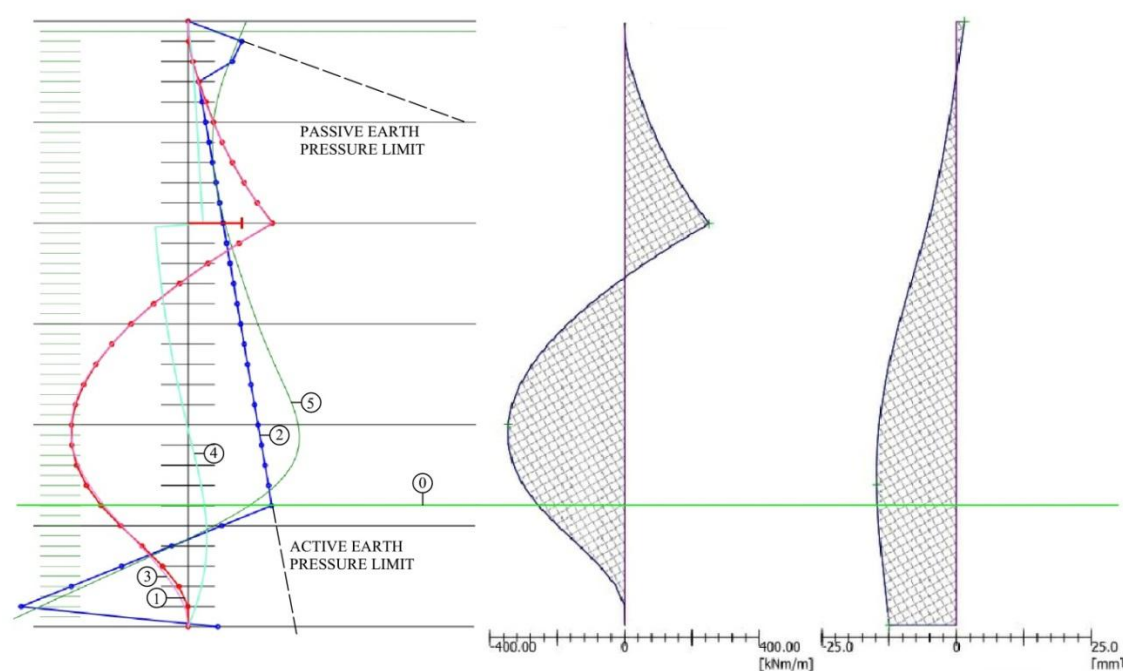


Fig. 7_Results of GEO5 computations and analyses by the new software. On the left: 0-excavation bottom, 1- bending moment distribution (GEO5), 2-pressure acting on the wall (GEO5), 3-bending moment distribution (analysis), 4-shear force distribution (analysis), 5-pressure acting on the wall (analysis). Cross hatched diagrams: Bending moment and horizontal displacement (GEO5)

The presented test example was computed by the GEO5 software for analysis of staged retaining structures, Fig. 7. A diaphragm wall with the thickness of 0,60 m was considered in the non-cohesive soil of $\gamma' = 20 \text{ kN/m}^3$, $\phi' = 36^\circ$, $c' = 0 \text{ kPa}$. Anchor was placed at 5,0 m below the original ground level and horizontal component of the force was 350 kN/m in the final excavation stage. The soil in the

foundation pit was excavated to the depth of 12,0 m. The depth of the wall toe is 3,0 m below the excavation. The staged computation was done and the final stage was subjected to back analyses with the new software.

4 APPLICATION OF BOTDA MEASUREMENT SYSTEM

The main advantage of Brillouin Optical Time Domain Analyses (BOTDA) is, compared to the common means of geotechnical monitoring, that the location of deformation or temperature changes along the installed measurement cable can be detected by the analyser and not-known in advance. The resolution and accuracy of deformation or temperature measurements depends on the length of measurement optical fibre and on the properties of the analyser. The resolution of BOTDA analysers is given by the producer and varies between $0,1\mu\epsilon$ and $20\mu\epsilon$ and is dependent on the type and length of the sensing cable.

Stimulated Brillouin effect, also called stimulated Brillouin scattering (SBS), arises from the interaction of glass fibre molecules and transmitted light. Distributed Strain and Temperature Sensors are used for tracking changes of Brillouin scattering in the time domain. This measurement method is known as Brillouin Optical Time Domain Reflectometry (BOTDR). Similar method, BOTDA, provides better results because two sources of light are used: the "pump" at the beginning of the fibre and the "probe" at the end of the fibre. A scheme of the measurement principle is presented in the Fig. 8. The shift of Brillouin frequency corresponds to the deformation or temperature change. The location of the frequency change along the optical fibre is calculated from the travel time of the backscattered light.

The analysers can determine absolute or relative values of deformation and temperature. The absolute values are derived from the specific Brillouin frequency measured on a free (unloaded) optical fibre. The relative values of strain or temperature are calculated from comparison of actual value with the baseline measurement.

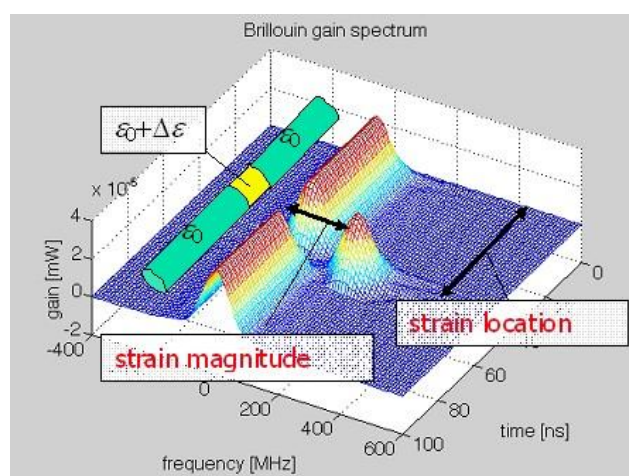


Fig. 8 Scheme of BOTDA measurement principle (Neubrex, 2009)

The BOTDA monitoring of the pile wall is expected to be performed by the sensing cables used in pairs according to the Fig. 5. There are two possibilities of sensing cables configuration. The first one

is to use two separated sensing cables deployed close to excavation and rear side of the retaining structure and the other one is to use one cable in a closed loop crossing the bottom part of the structure, e.g. from excavation to rear side.

The first case needs splicing of a pair of sensing fibres to obtain loops for BOTDA strain sensing, the second one is more sensitive to damage by the installation of reinforcement cage and concreting of the structure.

In the next part of the paper are presented selected results of BOTDA measurements performed within a R&D project: laboratory tests of the spliced connection of strain sensing cable and test measurements in a tunnel with construction traffic and different ways of fibre optics deployment. There were made two parallel tests of the strain sensing cables simulating cable concreted in the structure or subsequent deployment of the sensing cable on the completed body of a wall.

4.1 Laboratory testing

The aim of the laboratory testing was measurement of changes of length of the measurement base by BOTDA method together with independent deformation measurement method. There are important parameters checked by the laboratory testing prior to in situ deployment: the stability of measurement in time, the hysteresis of the sensing cable and consequently the development of optimal system of attachment of sensing cables to monitored structure.

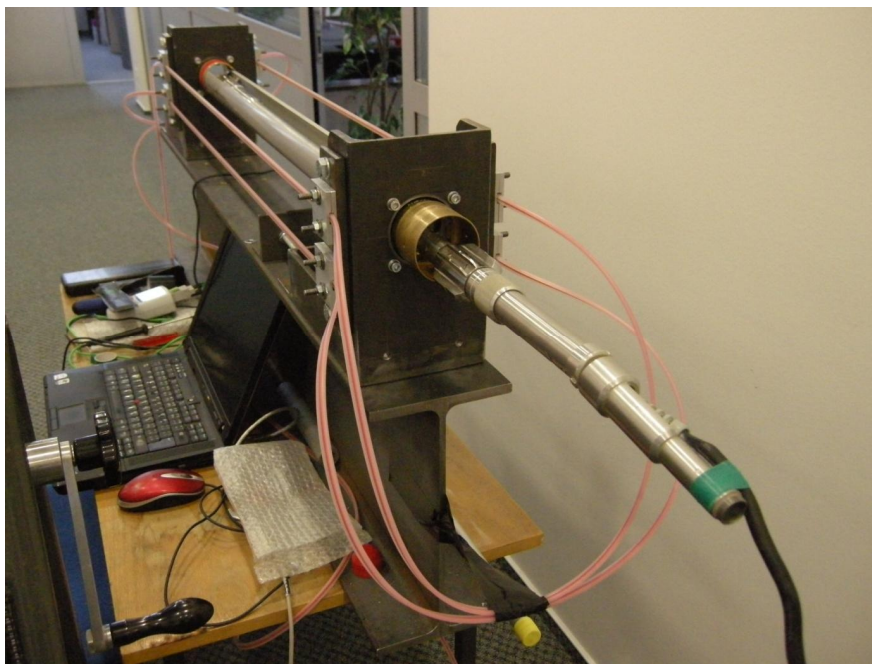


Fig. 9 Configuration of measuring cables and independent deformation measurement by Sliding micrometer, by J. Zalesky

The cable was attached to the rigid testing beam in four measuring sections – one above the other on both sides of the beam, Fig. 9. There are two possibilities of the fibre termination – either the splicing

of fibres into a loop or use of special terminal element. In case of splicing, the fibre is measured in the loop and therefore measured deformation distribution is mirrored along the point of splice, Fig. 10.

Smartprofile sensing cable was subjected to laboratory testing. This type of sensing cable contains four fibres – two of them are used for strain sensing and in the centre of the cable there are two loose temperature sensing fibres embedded in gel, Fig. 10.

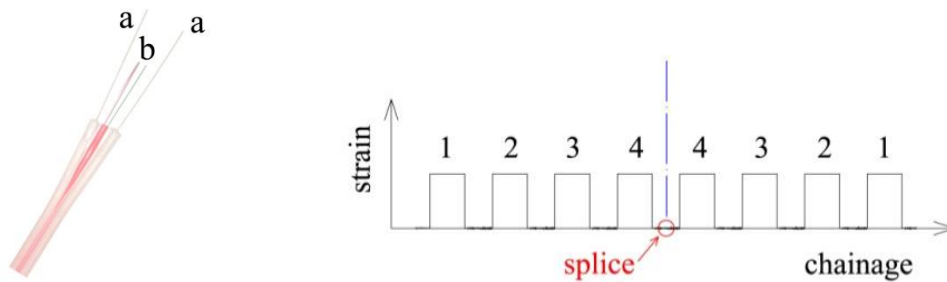
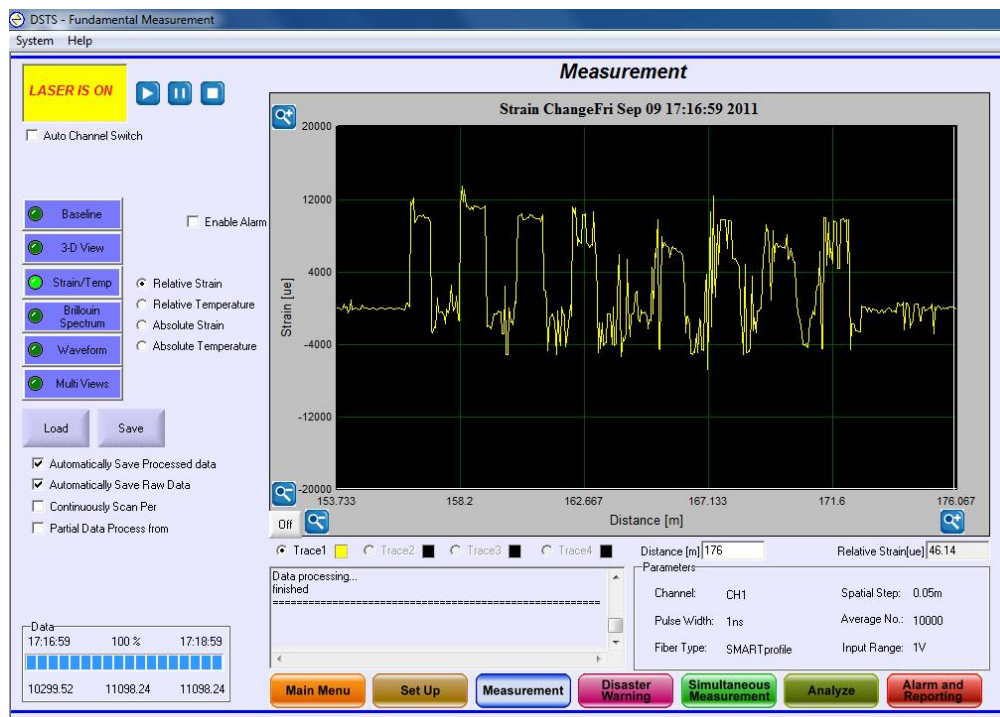


Fig. 10 SmartProfile on the left, after SmarTec SA, idealized scheme of the measurement result on the test beam; Legend: a – strain measurement fibre, b – temperature measurement fibres, 1 to 4 – numbers of intervals, where the measurement cable is subjected to strain

The example of results from the testing with measuring cable is shown at Fig. 11 – it is a screenshot from the software of the analysing unit. There are clearly visible four loaded parts of the fibre with nearly linear strain distribution. The other four peaks with non-linear strain distribution were formed because of the defect in splicing of two parts of the fibre together with strong signal noise.

Fig. 11 Screenshot of strain distribution during testing with BOTDA analyser OZ Optics



Additional unexpected peaks arise in the beginning of the first two sections. These peaks may be formed due to the attachment system and rapid changes of strain applied periodically to the sensing cable. The testing proved the ability of identification of fibre deformations and corresponding chainage when the fibre is fixed to the structure in points. The testing proved quite high sensitivity from the point of view of the magnitude of strain and chainage along the measuring fibre (the ability of distinction of the area along the axis of the fibre where the change of strain is applied). The move from unloaded to loaded part of the fibre is well defined.

One part of the testing was also step wise increase of deformation of fibres up to 1,5% elongation with consequent shortening back to the initial value. The cable response was not completely elastic.

4.2 Field instrumentation

The instrumentation of BOTDR fibres took place in summer 2011. The fibre optic was used for monitoring of a position and a magnitude of deformations and temperature changes. The testing was performed in an underground gallery Marketa which had been made in the middle between future rail tunnels of an extension of the Prague metro line A. One of those rail tunnels (from Motol station to Petriny station, accessible from Vypich construction pit) was bored close to installed optic fibre monitoring system by EPB Shield technology (Earth Pressure Balanced). For the boring process two tunnelling shields were used – S-609 named Tonda and S-610 Adela (produced by Herrenknecht, Germany). Tonda machine was boring the rail tunnel in vicinity of optic fibre monitoring system from 5th to 11th July 2011.

The sensing cable was installed into the lining of the gallery in two different manners. Approximately a half of the length of monitoring cable in a line was put into a groove 15 x 20 mm which had been cut into the lining. The groove with the cable was filled with fast hardening cement grout. The other half of the sensing cable was attached to the lining by couples of duralumin plates (50 x 50 x 5 mm) in distances of one meter approximately. These plates were connected to the lining by pairs of 6 mm screw bolts.

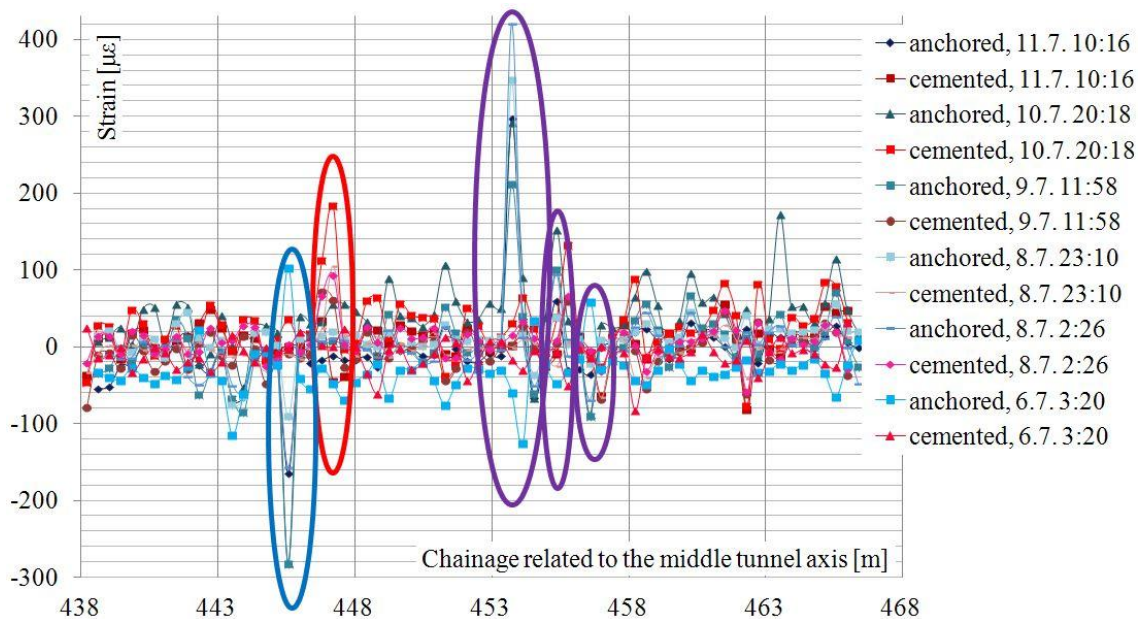


Fig. 12 Summary of strain distribution along the fibre from 6.7.2011 to 11.7.2011

Results from optic fibre monitoring were compared with the movement of the cutting head of shield Tonda and this comparison is presented in Fig. 12. Selected data fields were used in the Figure to describe deformation development in time. Blue lines represent the behaviour of the sensing cable attached to the lining with a help of duralumin plates (anchored cable) and red lines represent the behaviour of the sensing cable in a groove (cemented cable).

In Fig. 12 there are marked spots in the graph indicating clear peaks – red mark is for peak on the cemented cable only (chainage 447,191 m), blue mark is for peak on the anchored cable only (chainage 445,557 m) and violet mark is for peak on both cemented and anchored cables (chainage 454,130 m, 455,767 m and 456,992 m).

By comparison with the date – chainage diagram of the shield Tonda movement it was found that those peaks more or less correspond to the time when the shield was passing by the sensing cable (either the cutting head of the shield or any other part of it). One of these strain distributions is shown together with a movement of the shield Tonda for corresponding cemented and anchored cables in Fig. 13.

There is a quite thick layer of the lining between the middle tunnel and the bored tunnel; deformations due to the movement of the shield were not expected to be large, the monitoring system was installed more for the possible indication of cracks or technological joints (intervals of concreting) during the shield movement.

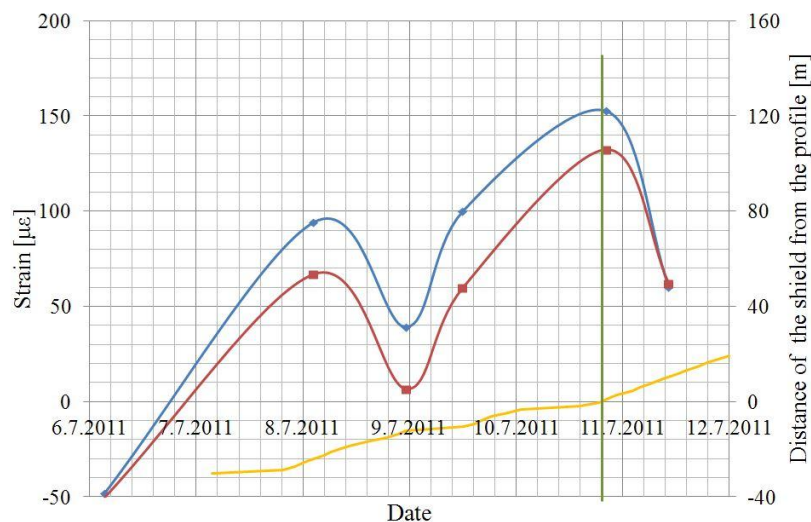


Fig. 13 Strain development at chainage 455,767 m; blue-anchored, red-cemented, yellow-shield Tonda advance

The anchored cable is strongly influenced by the environment inside the Marketa gallery. The signal can be disturbed for example by changing temperature, dynamic effects of heavy traffic or splashing dirty water – particles of mud remain and dry on the cable. The temperature changes effect can be compensated because the fibre measures also temperature simultaneously with strain. In case of the cable in a groove these effects are partially suppressed. On the other hand, the main advantage of the fixing the cable by duralumin plates is a fast cable deployment and easy fitting on the rough surface. The installation of the cable into a groove is time consuming; cutting the groove, putting the cable in

and fixing it by fast hardening cement. The most convenient option of the fixing system should be the sensing cable attached by stainless steel plates and protection by a cable sleeve.

A software tool which is able to process a huge amount of gathered data to prepare them for quick preview by the user and also to transfer the data into txt format has been created. It is also necessary to gain more knowledge about the background signal noise and about the unexpected data appearance and afterwards to implement a sorting tool into this software.

5 CONCLUSIONS

In order to provide a back analysis of a pile or diaphragm wall based on deflection curve it is necessary to have monitoring data as precise as possible. There are several options how to obtain these data. High resolution Inclinator is a suitable instrument in case of large deformations. If the retaining structure has high stiffness the Inclinator can be replaced by Trivec in a single measuring casing. Monitoring by Sliding micrometer / Trivec where a pair of measuring casings is installed into the pile (forming a plane close to the pile wall deflection) brings pairs of line-wise axial deformations. When the measurement is carried out in a stiff structure and therefore the deformations of the wall are very small, this method is able to resolve them (Sliding micrometer resolution is $1\mu\text{m/m}$) but measurement inaccuracies result in troubles in back analysing. Another advanced monitoring tool is the use of BOTDA measurement system. The BOTDA monitoring has one important advantage – it is possible to measure the strain distribution simultaneously with the temperature distribution both on the excavated and on the rear side of the wall - the temperature changes affect the strain distribution as well.

The back analysis based on monitoring results and particularly on determination of retaining structure deflection curve is affected by measurement inaccuracies. This was the reason for finding out of a method of damping of measurement induced oscillations during back analyses. The oscillation damping was implemented into the calculation and the preliminary version of software was invented. In this version there is the availability of flexible damping of oscillations step by step but the user has to be well informed.

Complementary information as geological soil/rock sequence is required because of implementation of stratified sequence in calculation. In more complicated hydrogeological conditions deformation monitoring methods are not sufficient, it is necessary to have hypothesis about the pore water pressure distribution and indication of artesian water appearance at least.

The analysis method used in the software is based on the deflection curve, therefore the software is capable to calculate bending moment and shear force distribution. Distribution of actions creating structural deflection can be determined. If there is any additional shift or rotation of the wall, an extra analysis is necessary. It is possible to utilize the behaviour of the excavated part of the wall as for calculation parameters adjustment in further studies.

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USED SOFTWARE

GEO5 – Sheeting check (student licence), Fine spol s r.o., version 5.15.17.0