Doctoral Thesis

Development, analysis and applications of an 'inclinodeformeter' device for earth pressure measurements

Author(s):
Schwager, Markus V.

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Development, analysis and applications of an ‘inclinodeformeter’ device for earth pressure measurements

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presented by
Markus Viktor Schwager
MSc ETH in Civil Engineering
born May 25, 1983
citizen of Bichelsee-Balterswil TG

accepted on the recommendation of
Prof. Dr. Alexander Puzrin
Prof. Dr. Lyesse Laloui

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Abstract

Lateral earth pressures are a key factor for many practical problems in geotechnical engineering. Measuring the earth pressures, however, remains one of the most challenging problems in geotechnical monitoring. The limited ability to measure lateral earth pressures is a source of uncertainty. Reliable information about such pressures can improve geotechnical design and analysis.

The inclinodeformeter (IDM) is a novel device for measuring changes in lateral earth pressure. The device makes use of the existing and widely used technology of inclinometer measurements. A change in earth pressures leads to changes in the shape and dimensions of the inclinometer pipe. If these changes are carefully measured, the pressure increment can be back-calculated from the solution of a boundary value problem with properly described constitutive behaviors of the pipe, the grout and the surrounding soil. The precision of the device is within the range of 0.1 kPa to 0.7 kPa depending on the stiffness of the soil and the grout surrounding the pipe.

The influence of the mechanical properties of the materials involved (i.e. the pipe, the grout and the soil) on the back-calculation is investigated. The time-dependent deformation characteristics of the inclinometer pipe are studied in laboratory tests and numerical simulations. The viscoelastic four-parameter model is found to be appropriate to take this influence into account. The mechanical behavior of the grout used to refill the borehole is studied in laboratory tests on different mixtures. Recommendations are provided for setting-up the optimal composition of the grout mixture. The back-calculation procedure allows the effects of the stiffness of the involved materials to be taken into account. The back-calculation is successfully validated against full-scale laboratory tests.

Change in shape of an inclinometer pipe does not only occur because of earth pressure changes. As the pipe is bent longitudinally, its cross-section flattens into an oval shape. The influence of longitudinal bending can be corrected by applying an analytical solution calibrated using numerical simulations. For many applications, the effects of longitudinal bending can be neglected, especially for measurements in the sliding layer of a creeping landslide.
Inclinodeformeter pressure measurements have been carried out in three creeping landslides in Switzerland. These landslides are similar in size but have different boundary conditions and displacement fields: a) the St Moritz landslide is slowing downhill towards a rock outcrop at the bottom; b) the Braunwald landslide is accelerating downhill towards the vertical rock wall falling into a valley; c) the Ganter landslide is moving uniformly downhill towards a river bed. IDM pressure measurements allow identification of compression and extension zones in creeping landslides. Reasonably small pressure increments can be reliably backcalculated by applying an analytical solution which takes the time-dependency of the pipe material into account. By combining the IDM measurements with measurements of relative displacements, the tangent stiffness of the soil can also be clearly identified. Thus, the backcalculation procedure becomes independent of the measurement of stiffness of soil. Therefore, IDM pressure measurements are shown to provide a reliable tool for analysis and monitoring of creeping landslides.

In addition, IDM pressure measurements have been carried out behind a retaining wall of an excavation. For each soil layer, a separate boundary value problem has been solved in a horizontal cross-section. Using numerical analysis, the lateral earth pressure changes and the soil stiffness have been back-calculated based on IDM and local displacement measurements. The back-calculated soil stiffness has been successfully validated for each soil layer against the in-situ measurements obtained by CPT and Marchetti dilatometer. The strut loads, which were observed to be considerably higher than expected in the design, can be explained with the back-calculated pressure changes. The IDM device is shown to be reliable and effective in monitoring of lateral earth pressure changes behind a retaining wall.

The IDM pressure measurements may bring benefits for designers and contractors in order to provide reliable and cost efficient structures. The additional information may also contribute to a better safety assessment of slopes and structures, supporting the public and authorities in decisions concerning their safety.
Kurzfassung


Rohrverformungen in der Querschnittsebene treten nicht nur auf Grund von Erddruckänderungen auf: Erfährt ein Rohr entlang seiner Längsachse Biegung, so flacht der Querschnitt ab zu einer ovalen Form. Die IDM-Messungen können um diese Formänderung korrigiert werden mit Hilfe einer analytischen Lösung, welche an numerischen Simulationen kalibriert wurde. Bei vielen Anwendungen, so auch bei Erddruckmessungen in der Rutschmasse eines
Kriechhanges, kann jedoch auf die Korrektur des Einflusses der Längsbiegung verzichtet werden, da dieser vernachlässigbar ist.


IDM-Erd druckmessungen sind ein wertvolles Werkzeug zur Beurteilung der Verlässlichkeit und Effizienz von grundbaulichen Strukturen. Speziell im Fall von Kriechhängen und Baugruben kann die zusätzlich gewonnene Information zu einem erheblich verbesserten Verständnis der Sicherheit führen. Behörden werden dadurch unterstützt in ihren Entscheidungen bei geotechnischen Problemstellungen.
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1 Introduction

1.1 Rationale of the thesis

Lateral earth pressures are a key factor for many practical problems in geotechnical engineering. Measuring these earth pressures, however, remains one of the most challenging problems in geotechnical monitoring. The earth pressure measurements are particularly important in creeping landslides and at excavations:

- Earth pressure changes in the sliding layer of a creeping landslide are critical for analysis and stabilization of such landslides. Standard monitoring techniques such as geodetic, inclinometer and piezometer measurements cannot obtain sufficient information. For inverse analysis of creeping landslides and its validation, additional information about the lateral earth pressures is required (Puzrin and Schmid, 2011; Puzrin and Schmid, 2012). Of even greater importance, however, is information about the earth pressure changes for constrained landslides, where pressures in the compression zone could reach the passive pressure and lead to catastrophic failure. Measurements of the increase in lateral pressure in the compression zone could prevent danger.

- For the design of retaining walls in excavations, the real acting pressures on the wall and their distribution are very difficult to predict. Designers could gain essential knowledge from reliable measurements of earth pressures next to the retaining wall during construction. Especially when using observational method, information about the earth pressures becomes critical.

Conventionally, earth pressure measurements can be carried out by dilatometers or pressure cells:

- Dilatometer tests are designed to measure stiffness of soil through inflating a membrane against the soil. A probe with the membrane is either pressed directly into the soil or lowered into a borehole. In addition to the stiffness, the in-situ lateral earth pressures can be obtained by empirical correlations (Marchetti, 1980; Marchetti, 1997). Earth pressure measurements taken by dilatometer tests are influenced by the dis-
turbance of the stress state caused either by the drilling of the borehole (Gibson and Anderson, 1961) or by the penetration of the measuring probe (Smith, 1993). The disturbance of the stress state can be reduced by using a self-boring dilatometer (Jefferies, 1988), with application limited to fine-grained soils. Dilatometer tests do not allow the development of earth pressures to be traced over time.

Pressure cells are a conventional method to measure pressures in soil. Embedment earth pressure cells are installed surrounded by soil, for example in a borehole. Contact earth pressure cells are attached to a structure and measure the stresses acting on that structure. The method of embedment earth pressure cells has been proven inefficient depending on the ground conditions, mainly owing to effects of arching and relative stiffness, and disturbance of the free-field stress (Selig, 1964; Weiler and Kulhawy, 1982; Miura et al., 2003; Labuz and Theroux, 2005; Tesarik et al., 2006). Fifteen major factors affecting earth pressure cell measurements are reviewed by Weiler and Kulhawy (1982). “In summary, the many factors that affect measurements can result in substantial errors, so that measurements with embedment earth pressure cells can rarely be made with high accuracy” (Dunnicliff, 1988, 1993). Contact earth pressure cells are observed to perform better. Nevertheless, it is recommended to determine stresses acting on a structure by use of load cells and strain gauges within the structure (Dunnicliff, 1988, 1993). Earth pressure cells allow for a measurement at one point. The pressure cell and the installation, for example the borehole that may be required, are rather cost intensive, especially when considering that several pressure cells have to be installed in order to obtain a profile of lateral pressures with depth.

The application of conventional methods to landslides and retaining walls is reported in the literature:

- Araiba and Suemine (1998) and Sève et al. (1999) measured the lateral pressure in landslides using conventional methods such as pressure cells and dilatometers. The results obtained have been found to be of limited reliability. Hada et al. (1988) used deformable panels equipped with strain gauges to measure lateral earth pressures in a landslide. This approach is rather cost intensive and therefore hardly applicable for monitoring of large landslide areas.

- Experimental research on earth pressures acting on retaining walls has been extensively reported in the literature (Terzaghi, 1943; Rowe and Peaker, 1965; Tsagareli, 1965; Narain et al., 1969; Rehnman and Broms, 1972; Matsuo et al., 1978; Fukuoka et
al., 1981; Sherif et al., 1984; Fang and Ishibashi, 1986; Fang et al., 1997; Tweedie et al., 1998). The usual method is to make discrete measurements directly on the wall using contact earth pressure cells. Pressure measurements distant from the wall are not reported. Self-boring dilatometer tests next to a retaining wall were performed and analysed by Clarke and Wroth (1984). The earth pressure changes during excavation could be observed by drilling several boreholes. Although Clarke and Wroth applied the self-boring technique, the measured pressures were found to be influenced by the disturbance due to the penetration of the probe.

The lateral earth pressure is a critical parameter which often cannot be traced by conventional geotechnical monitoring. The limited ability to measure such pressures is a source of uncertainty. Reliable information about lateral earth pressures can improve geotechnical design and analysis.
1.2 Main objectives of the thesis

The ultimate goal of the present work is to develop a technique which allows reliable and precise measurement of changes in lateral earth pressures. Earth pressure changes with time will be observed along a profile in depth.

The method being developed is to be validated in a wide range of soil and stress conditions. This includes full-scale laboratory tests, field experiments, and numerical and analytical modeling. Within this framework, the focal point is the appropriate description and understanding of the mechanical interaction between the soil and the monitoring tools.

Application to creeping landslides provides a better understanding of the landslide mechanisms. Creeping landslides with different boundary conditions are studied. Measuring close to an excavation provides insight into the distribution and development of earth pressure changes during the excavation process.

The technique is intended to be cost efficient. Therefore, it is based on the widely used inclinometer technology, which is briefly described in the next section. The method is to be designed to make use of installations which are already in the field.

In general, the present investigations should allow a better understanding of lateral earth pressures in practical engineering problems. The deeper knowledge and information about earth pressures may improve geotechnical design and analysis. This offers benefits for designers and contractors who need to provide reliable and cost efficient structures. The additional information may also contribute to a better safety assessment of slopes and structures, supporting the public and authorities in decisions concerning their safety.
1.3 Introduction to the conventional inclinometer

The developed device is based on conventional inclinometer technology. The device also makes use of the same installation in the field. Therefore, a short introduction is provided to explain the principles of conventional inclinometers.

“An inclinometer is a device for monitoring the onset and continuation of deformation normal to the axis of the borehole casing by passing a probe along the casing” (Dunnicliff, 1988, 1993; Stark and Choi, 2008). Thus, an inclinometer provides a profile of subsurface horizontal displacements.

Inclinometer measurements require installations on site. Therefore, a vertical borehole is drilled into the ground. An inclinometer pipe made of several connected casing elements is installed in the borehole. The annular space in the borehole around the inclinometer pipe is refilled.

To take inclinometer measurements, the probe is introduced into the pipe (Figure 1-1). The probe is guided by its wheels rolling inside the channels of the pipe. First, the probe is lowered to the bottom of the borehole. Accelerometers on the probe allow a reading to be taken of the inclination $\theta$ of the probe. The probe is raised incrementally, taking additional readings. The horizontal onset is calculated for each increment from the inclination $\theta$ of the probe and the distance $L$ between successive measurements. Summing up the increments provides the longitudinal shape of the pipe with respect to the true vertical. Horizontal soil displacements are obtained from the difference between the initial shape and subsequent measurements, provided the absolute displacement of one point along the borehole is known. Usually, the bottom end of the inclinometer pipe is assumed to be in stable ground and not to experience any displacement.
The inclinometer commonly used today became commercially available in the late 1950s (Stark and Choi, 2008). In those days, the device was also presented in the literature by Wilson and Hancock (1959), and by Wilson (1962). Nowadays, inclinometers are widely used, e.g. for monitoring of landslides, slopes, excavations, retaining walls, foundations, dams, embankments and tunneling. Inclinometers have become one of the most common methods in geotechnical monitoring.
1.4 Structure of the thesis

This thesis is structured in six chapters (chapters 2 to 7), each of them expected to make a valuable contribution towards the objectives noted above.

After the introduction (chapter 1), the novel inclinodeformeter device (IDM) and the experimental setup for validation (IDM box) are introduced (chapter 2).

Chapters 3, 4 and 5 deal with a detailed understanding of the mechanical interaction between the soil and the monitoring tools. Several mechanical effects are investigated separately as having a potential influence on the earth pressure measurements. Methods are presented to consider these effects in the framework of the developed device. The proposed methods of chapter 3 and 4 are validated against full-scale laboratory tests performed in the IDM box. Chapter 3 deals with effects of time-dependency on the earth pressure measurements. The time-dependent deformation characteristics of the inclinometer pipe are investigated. The influence of the mechanical behavior of the grout used to refill the borehole is studied in chapter 4. Laboratory tests were performed on different grout mixtures. General rules are developed for composing grout mixtures. The results derived in chapter 5 allow the effects of longitudinal bending of the inclinometer pipe to be taken into account.

Chapters 6 and 7 contribute to particular applications of the developed method. Earth pressure measurements are presented for three creeping landslides in Switzerland (chapter 6) with different boundary conditions and displacement fields: a) the St Moritz landslide, which is slowing downhill towards a rock outcrop at the bottom; b) the Braunwald landslide, which is accelerating downhill towards a vertical rock wall falling into a valley; c) the Ganter landslide, which is moving uniformly downhill towards a river bed. An analytical solution of the corresponding boundary value problem is developed in order to obtain the earth pressure changes. Another field of application is introduced in chapter 7, presenting earth pressure measurements taken behind a retaining wall next to an excavation. Development and distribution of earth pressure changes are observed during the jet-grouting phase and the excavation phase. The inclinodeformeter measurements are used for back-calculation of mechanical properties of the soil.

Finally the conclusions and ideas for future research are summarized in chapter 8.
The appendix (chapter 9) provides a detailed description of the device and of the method. A proposed method of 11 steps is formulated for application in monitoring practice. The appendix is based on a filed patent application (Puzrin et al., 2013).

REFERENCES


2 Introduction of the inclinodeformeter device for back-calculation of earth pressure changes

2.1 Abstract

The inclinodeformeter (IDM) is a novel device for measuring changes in earth pressure. The device makes use of the existing and widely used technology of inclinometer measurements. An advantage of the inclinodeformeter is that it does not require any additional infrastructure than standard inclinometer pipes, even long after they have been sheared and become unsuitable for inclinometer measurements. Changes in earth pressure lead to changes in the inclinometer pipe shape and dimensions. If these changes are carefully measured, the pressure increment can be back-calculated from the solution of a boundary value problem with properly described constitutive behaviors of the pipe, the grout and the surrounding soil. The precision of the device is within the range of 0.1 kPa to 0.7 kPa, depending on the stiffness of the soil and the grout surrounding the pipe. Full-scale laboratory tests performed in a 2 m high calibration chamber demonstrate that simple constitutive models can be used for back-calculation as a first approximation.
Earth pressure changes in a sliding layer of a creeping landslide are critical for understanding, analysis and stabilization of such landslides. Combining measurements of the displacements and the changes in pressure in two sections of a long thin sliding layer (Figure 2-1) allows back-calculation of the average shear strength on the sliding surface and the average lateral stiffness of the soil in the sliding layer.

\[

deltat_1^* = \frac{H}{L} (\Delta p_2 - \Delta p_1) \\
E = \frac{\Delta p}{\Delta \varepsilon} \approx \frac{(\Delta p_1 + \Delta p_2) / 2}{(\Delta \delta_1 - \Delta \delta_2) / L}
\]

Figure 2-1: Back-calculation of the average shear strength on the sliding surface and average lateral stiffness of soil.

Of even greater importance is information about the earth pressure changes and the stiffness for constrained landslides, where the pressures in the compression zone could reach the passive pressure and lead to catastrophic failure (Puzrin and Sterba, 2006). The measurement of the change in earth pressure in a section close to the constrained boundary allows back-calculation of the lateral stiffness (Figure 2-2).

Unfortunately, measuring the earth pressures is also one of the most challenging problems in geotechnical monitoring.
This chapter introduces a novel device called the inclinodeformeter (IDM) which allows back-calculation of the changes of earth pressure. In the first step, the IDM measures the change in dimensions of an inclinometer pipe in the sliding layer. The change in shape is assumed to be caused by the changes in the surrounding stress field. In the second step, the measured deformations are used to back-calculate the change in pressure via inverse analysis of the boundary value problem of a plastic pipe surrounded by soil under a changing stress state.

This chapter explains the IDM design and the required correction procedure for the deformation measurements, as well as the procedure for back-calculation of the pressure changes in the surrounding soil. These procedures are then validated in full-scale calibration chamber tests.
2.3 Measuring the pipe deformations

2.3.1 IDM design

The inclinodeformeter (IDM) makes use of the existing and widely used technology of inclinometer measurements. The IDM probe is lowered down the pipe on three wheels, guided along the channels (Figure 2-3). Continuous diameter measurements in two perpendicular directions can be taken, as described below. The upper and the lower wheels roll in the same channel. These wheels are fixed to the probe. The middle wheel is connected via a lever with two springs, so that it can be pressed against the opposite channel. A change in the diameter of the pipe leads to a change of the position of the middle wheel with respect to the probe. There are two tilt sensors (see section 2.3.2) which detect the relative inclination between the probe and the lever of the middle wheel. One sensor is located on the top of the probe, another on the middle wheel (Figure 2-3).

In addition to the two tilt sensors in the plane of the measured diameter, there is another tilt sensor which measures in the perpendicular direction out of this plane. This sensor is used for correction of the measurements due to the out-of-plane inclination of the pipe (see section 2.3.3). Above the top wheel there is a temperature sensor and a pressure cell to measure the water pressure in the inclinometer pipe.

At the top of the borehole, the cable on which the device is hanging goes around a wheel (Figure 2-3d). An incremental rotation sensor measures the wheel rotation, which determines the depth position of the probe in the inclinometer pipe. As the probe is lowered down into the inclinometer pipe, all the sensor measurements are saved on the computer for the corresponding depth position in the pipe.

The IDM is built to fit inside the two most common diameters of inclinometer pipes in Switzerland: 71 and 84 mm. The device can be easily switched between the different inclinometer pipe diameters. A detailed description of the design of the IDM device is provided in appendix 9.4.
Figure 2-3: The inclinodeformeter: a) The complete probe, b) The probe without the front panel, c) Picture of the probe, d) Installation at the top of the borehole.
2.3.2 Performance of the tilt sensors

The tilt sensors are of the type of MEMS accelerometers described e.g. by Sellers and Taylor (2008). This kind of sensor is advantageous for geotechnical monitoring because of the high repeatability and long-term stability of the measurement (Sheahan et al., 2008), as well as extremely high shock resistance. The tilt sensors built into the IDM provide a resolution in terms of the pipe diameter change of 0.1 micrometer. There are two major disadvantages of these tilt sensors. First, the measurements are not independent of the inclination out of the plane. This can be resolved by an external correction (see section 2.3.3). Second, the tilt measurement is dependent on temperature. The measured deformation of the pipe could be affected by +/- 0.5 micrometers per °C in the worst case of accumulation of the errors of both sensors. Measuring the temperature of the ground water in the pipe allows this influence to be corrected for most of the applications.

2.3.3 Correction for the pipe out-of-plane inclination

The measured diameter $D$ inside of the pipe is a function of the two angles $\alpha_L$ and $\alpha_P$ measured at the lever of the middle wheel and at the probe:

$$
D = d + X + Y \sin(\alpha_L - \alpha_P)
$$

(2-1)

where $X$, $Y$ and $d$ are constants depending on geometry (see appendix 9.4.1).

The measurements of $\alpha_L$ and $\alpha_P$ are not independent of the inclination $\beta$ of the device out of the plane. Assuming that the sensors give the true $(\alpha_L - \alpha_P)$ value at $\beta = 0^\circ$, there is an error occurring in the tilt measurements when $\beta$ is different from $0^\circ$. Because the diameter is just a function of the difference $(\alpha_L - \alpha_P)$, it is sufficient to describe the error $\Delta$ affecting this difference. This error can be found by calibration measurements on a biaxial inclinable table (Figure 2-4a).

The error due to the device inclination out of the plane can be described as a function of $\alpha_L$, $\alpha_P$ and $\beta$ as follows:

$$
\Delta = (A_1\alpha_L + A_2\alpha_P + A_3)\beta^2 + (C_1\alpha_L + C_2\alpha_P + C_3)\beta
$$

(2-2)
where $A_1, A_2, A_3, C_1, C_2$ and $C_3$ are constants derived by a regression analysis of the calibration measurements. Correcting the difference $(\alpha_L - \alpha_P)$ by the error function leads to the corrected diameter $D_{\text{cor}}$:

$$D_{\text{cor}} = d + X + Y \sin(\alpha_L - \alpha_P - \Delta)$$  \hspace{1cm} (2-3)

By using the error function from Equation (2-2), the corrected measurements of the diameter $D$ reach a precision of $+/- 0.5$ micrometers within $\beta = +/- 4^\circ$, compared with the measurement at $\beta = 0^\circ$ (Figure 2-4b).

Figure 2-4: a) The inclinodeformeter on a biaxial inclinable table for calibration measurements, b) The precision of the diameter $D$ compared with the measurement at $\beta = 0^\circ$. 
2.3.4 Ovalization of the inclinometer pipe

The aim of IDM is to obtain measurements of deformations of the inclinometer pipes over a period of several years. It is therefore important to avoid the influence of a possible shift of the device reference. This issue is resolved by describing the shape of the pipe in terms of the difference of two measured diameters. For field applications (as presented in chapters 6 and 7), the ovalization value \( \Omega \) is introduced in the form

\[
\Omega = \frac{D_A - D_B}{R}
\]  

(2-4)

where \( D_A \) and \( D_B \) are the two inner diameters of the inclinometer pipe, and \( R \) is the nominal outer radius of the pipe (Figure 2-5). The outer radius \( R \) is equal to 35.4 mm and 42 mm for the two most common inclinometer pipes in Switzerland. A more general definition of the ovalization value is provided in appendix 9.5.

![Figure 2-5: Geometry of the inclinometer pipe.](image)

Hence, the change in shape of the pipe can be described by the change in ovalization value

\[
\Delta \Omega = \Omega - \Omega_0 = \frac{\Delta D_A - \Delta D_B}{R}
\]  

(2-5)

where \( \Omega_0 \) is the ovalization value at the zero measurement.

The definition of the ovalization value allows for effective reduction of errors affecting both diameter measurements (e.g. influence of the long-term stability of the probe; influence of
the actual field conditions: temperature, humidity, water pressures inside and outside the pipe).

2.3.5 Precision of IDM measurements

The precision of a single IDM diameter measurement for a length section of 35 cm of a vertical inclinometer pipe is +/- 1.5 micrometers. Because the tilt sensors on the probe are not independent of the inclination out of the plane, additional scattering is taken into account. Calibration measurements show a remaining error of +/- 0.5 micrometers after the correction of the measurements applying a calibrated error function (section 2.3.3). Hence, the precision of a single diameter reading can be assumed to be around +/- 2 micrometers by combining the variance.

Owing to the definition of the ovalization value (see section 2.3.4), earth pressure changes in field applications are obtained as a function of the change in the difference of the two measured diameters. Taking the difference of several diameter measurements increases the scattering. Nevertheless, the repeatability in field measurements is +/- 2 micrometers as the measurements are averaged over a larger pipe section of 211 cm (section 6.3; appendix 9.5 IX). Therefore, the precision of the IDM diameter measurements $[\Delta D_B - \Delta D_A]_{\min}$ is considered to be 2 micrometers, which corresponds to a precision in terms of ovalization value $\Delta \Omega_{\min}$ of $4.8 \times 10^{-5}$ or $5.6 \times 10^{-5}$ for the different pipes.

2.4 Back-calculating pressures

2.4.1 Boundary value problem

The pressure changes can be back-calculated from a solution of an inverse generalized plane stress boundary value problem (Figure 2-6), from the measured changes in pipe diameters, provided the stiffness of the soil, the stiffness of the grout and the stiffness of the pipe in this range of stresses are known. The boundary conditions in a general case are static: the two principal earth pressures. The major principal stress direction is assumed to coincide with the direction of displacement vector which is known from the conventional inclinometer measurements.
The measured diameters are not only affected by the earth pressure changes. The bending of the inclinometer pipe produced by the movement of the landslide also causes changes in diameter. This correction has to be carried out before modeling in the generalized plane stress problem. The issue of diameter changes due to bending is addressed in chapter 5. An approach allowing for correction of the measurements is provided.

### 2.4.2 Effects of stiffness

The stiffness of the pipe, the grout and the soil affect the result of the back-calculation significantly. Therefore, it is very important to describe the constitutive behavior of the pipe, the grout annulus and the surrounding soil in an appropriate way. Within this chapter, simplifying assumptions will help to introduce the back-calculation of earth pressures. The influence of the deformation characteristics of the involved materials on the back-calculation is investigated in the following chapters. Chapter 3 deals with the time-dependent behavior of the PVC pipe. The influence of stiffness of the grout is addressed in chapter 4.
Stiffness of the inclinometer pipe

The short-term Young’s modulus for fast loading of the inclinometer pipe was determined by compression tests. The pipe was loaded by a linear distributed load in a purpose-built test apparatus (Figure 2-7a). The deformations were measured for several angles between the direction of the force and the direction of the channels in the pipe.

![Compression test on the inclinometer pipe](a)

![Stiffness $k$ as a function of the direction of force](b)

Figure 2-7: a) Compression test on the inclinometer pipe, b) Stiffness $k$ of the pipe as a function of the direction of force.

The tangent stiffness of the pipe $k$ is defined as the ratio between the increment of force $f$ divided by the increment of displacement $u$ (or, more precisely, half of the displacement, owing to the symmetry of the setup):
The stiffness is strongly dependent on the angle between the direction of force and the direction of the channels in the pipe, since they soften the pipe cross-section. The highest stiffness is achieved when there is an angle of 45° between the force direction and the channels (Figure 2-7b). In this configuration, there is hardly any bending moment acting in the area of the channels, where the bending stiffness is reduced significantly.

The Young’s modulus of the pipe $E_p$ is related to $k$ by an analytical elastic solution for a solid pipe (without channels) loaded by two opposite forces (e.g. Bouma (1993)):

$$E_p = \frac{2R_m k}{\pi A} + \frac{2R_m^3 k}{\pi I} \left(1/9 + 1/225 + 1/1225 + \ldots\right)$$

(2-7)

where $R_m =$ middle radius; $A =$ area of the pipe section; and $I =$ moment of inertia of the pipe section.

Using Equation (2-5) with the stiffness $k$ from the 45° force direction measurement (Figure 2-7b) leads to the immediate Young’s modulus of $E_p = 2850$ MPa for fast loading. For back-calculations of pressures in creeping landslides, however, the long-term modulus is of much greater concern than the immediate modulus for fast loading. The viscous behavior of the pipe has to be considered. Within the scope of this chapter, this aspect is not further discussed. More information on this topic is given in chapter 3. The Poisson’s ratio of the inclinometer pipe is specified by the producer of the pipe as 0.34.

**Stiffness of the grout surrounding the pipe**

In many cases, the inclinometer pipe is fixed in the borehole by grouting the annulus using a cement-bentonite mixture. The stiffness of the grout can be estimated from results of laboratory tests (e.g. unconfined compression tests). If the stiffness of the grout differs significantly from that of the surrounding soil, the grouted annulus has to be incorporated into the boundary value problem. In practice, however, it is common to use a grout mixture with a stiffness as close as possible to the surrounding soil. More information on this topic is given in chapter 4.
Stiffness of the surrounding soil

The stiffness of the surrounding soil can be determined from dilatometer tests performed in the same borehole as was drilled for the inclinometer. If no field measurements are available, the stiffness of the soil can be roughly estimated from the results of laboratory tests (e.g. consolidation tests). The loading history and the nonlinearity due to stress dependency have major effects on the soil stiffness, so it is essential to determine the stiffness both in loading and unloading–reloading modes within the stress range expected to be measured by the inclinodeformeter.

The stiffness of the soil may be found from the combination of IDM measurements with measurements of relative displacements. Both earth pressure changes and lateral stiffness can be back-calculated provided lateral strains are known. More information on this topic is given in chapters 6 and 7.

2.4.3 Inverse analysis

In general, the soil and the grout behavior have to be modeled as visco-elasto-plastic and that of the pipe as viscoelastic with geometric non-linearity due to the channels. Therefore, the forward boundary value problem is usually solved using numerical analysis. The backcalculation of pressures is performed in two steps: first, a finite element program computes the deformations caused by the trial stresses. An inverse analysis uses the Levenberg–Marquardt algorithm to solve the optimization problem by minimizing the objective function $F$ (the sum of squared errors between the measured and the computed pipe deformations) for the changing trial stresses. This approach based on numerical analysis is used in chapters 2, 3 and 4, regarding laboratory tests, and in chapter 7, regarding field measurements.

For certain simplifications of the boundary value problem, an analytical solution can be obtained (chapter 6). For this, the constitutive behavior of the soil is assumed to be linear elastic and the influence of the grout is neglected. Hence, the solution can be applied if the deformation characteristics of the sand can be assumed to be linear within the pressure increment and the stiffness of the grout is similar to the stiffness of the soil. No optimization problem has to be solved using the analytical solution for back-calculation. The analytical solution and its application to field measurements in creeping landslides are shown in chapter 6.
General case with static boundary conditions

In a general case the boundary conditions are given by the two trial principal stresses $\sigma_1$ and $\sigma_2$. Figure 2-8 shows the objective function $F$ as a function of $\sigma_1$ and $\sigma_2$. Both the soil and the pipe are assumed to be elastic, with Young’s moduli of the soil and the pipe being set to 50 MPa and 2850 MPa, respectively. The Poisson ratios were assumed to be 0.25 and 0.34, respectively. The minimum of the objective function is located at $\sigma_1 = 100$ kPa and $\sigma_2 = 60$ kPa.

![Figure 2-8: Objective function for inverse analysis with static boundary conditions.](image)

The low gradient along the objective function “valley” as seen in Figure 2-8 implies that a variety of stress states may lead to similar pipe deformations. This can be explained in that the bending moments cause much bigger deformations of the pipe than normal forces, and different combinations of the stress ratio $\sigma_2/\sigma_1$ and the average stress $(\sigma_1 + \sigma_2)/2$ can produce the same bending moments at different levels of compression of the pipe. From Figure 2-8 it follows that variation in $\sigma_2/\sigma_1$ at a fixed $(\sigma_1 + \sigma_2)/2$ produces larger pipe deformation than the other way round. This makes back-calculation of stress increments more challenging.

Particular case with constrained boundaries

For large creeping landslides with a wide moving front, plane strain conditions can be assumed in the vertical plane parallel to the slope gradient. This implies that there are no displacements perpendicular to that plane. In this particular case, we can solve a problem with kinematically constrained boundary conditions in the direction of the minor principal stress (Figure 2-9). This assumption makes the back-calculation problem well-posed, and the solution can be much more easily reached.
Figure 2-9: Generalized plane stress model in the case of constrained boundaries in direction of the minor principal stress.

Figure 2-10 shows the objective function $F$ as a function of $\sigma_1$ in this particular case for the same constitutive behavior as in Figure 2-8. Because the behavior of both the pipe and the soil is assumed to be elastic, the plot in Figure 2-10 is in principle a cross-section of the general surface in Figure 2-8 by the plane $\sigma_2/\sigma_1 = \nu_s$ (the Poisson ratio of the soil).

Figure 2-10: Objective function for inverse analysis with constrained boundary conditions to the sides.
Using the results of inverse analysis, it is possible to back-calculate the precision of the IDM stress measurements by taking the change in diameters of the pipe equal to the precision of the diameter measurements \([\Delta D_B - \Delta D_A]_{\text{min}}\) of 2 micrometers (section 2.3.5). The precision of the IDM for common stiffness of soil and grout is within the range of 0.1 kPa to 0.7 kPa (Figure 2-11). Softer soil leads to a higher precision. It seems that the right choice of grout and its stiffness improves the device capacity: the softer the grout, the higher becomes the precision. This rule is limited to grout stiffness higher than 20 MPa and to stiffness of the ground lower than about 500 MPa. More information on the influence of the grout is given in chapter 4.

This precision strongly depends on the model applied for the back-calculation, especially on its boundary conditions. All the three materials involved were assumed to be elastic. Based on chapter 3, the long-term modulus of the pipe is estimated at 2000 MPa in order to account approximately for viscous behavior. The Poisson’s ratios of the pipe, the soil and the grout were assumed to be 0.34, 0.25 and 0.2. The outer diameter of the grout annulus is set to a common drilling diameter of 131 mm. A friction coefficient of 0.5 is assumed within the interface surrounding the pipe.
2.5 Validation: IDM box

2.5.1 IDM box: Test setup

For validation of the back-calculation procedure, full-scale laboratory tests were performed in a 2 m high calibration chamber (IDM box) with a cross-section of 40 by 40 cm (Figure 2-12). Each of the four vertical walls of the chamber is equipped with a pressure membrane. The setup allows the application of two independent principal horizontal stresses. The chamber is filled with sand in absence of grout, and the inclinometer pipe is fixed in the middle of the chamber. An increase in principal stresses results in deformations in the pipe which are measured using the inclinodeformeter. More information on the IDM box tests is provided in chapter 4.

![Image](Figure 2-12: a) The IDM box, b) Sand surrounding the pipe in the opened IDM box.)

2.5.2 Comparison of applied and back-calculated pressures

The boundary conditions applied during the test are shown in Figure 2-9. The top boundary in the IDM box is constrained, and therefore we consider a boundary value problem with plane strain conditions in vertical direction. The major principal stress $\sigma_1$ was applied via a couple of two opposite pressure membranes. The other two pressure membranes were empty and opened to atmospheric pressure in such a way that the boundaries had zero displacements. The loading path was: loading, unloading, reloading and final unloading. The numerical model and its parameters for the back-calculation of pressures are presented in chapter 4. The comparison between applied pressures and pressures back-calculated for both pipe diameters is given in Figure 2-13. It can be seen that an elastic model can be used for back-calculation as a reasonable first approximation. Precision of the pressure measurement
corresponding to the precision of the diameter measurement (section 2.3.5) in initial loading is 0.2 kPa, while in unloading–reloading it is 0.6 kPa. As expected, softer soils provide higher precision for pressure measurements.

Validation using finite elements (Abaqus) showed that for the inclinometer pipe with diameter 84 mm, dimensions of 40 by 40 cm are sufficient to avoid the effect of the boundaries, when comparing with a free field solution. The error in back-calculated pressure for the example considered is within a range of 7%. The error in averaged pressures at the constrained boundaries is within a range of 10%.

2.6 Conclusions

The novel inclinodeformeter device looks promising, owing to its simplicity and accuracy of measurements. In addition, it does not require any additional infrastructure other than standard inclinometer pipes, which are regularly installed anyway for landslide monitoring. Furthermore, these pipes can be used for pressure change measurements in the sliding layer long after they were sheared along the slip surface and became unsuitable for inclinometer measurements. Back-calculation of pressures is a challenging task, which requires the study of such effects as viscosity of the pipe material, stiffness of grout and non-linearity of soil behavior. Nevertheless, full-scale laboratory tests performed in a 2 m high calibration chamber demonstrate that simple constitutive models can be used for back-calculation as a first approximation.
ACKNOWLEDGEMENTS

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REFERENCES


NOTATION

\( A \) area of the cross-section of the pipe
\( A_1 \) coefficient of the error function
\( A_2 \) coefficient of the error function
\( A_3 \) coefficient of the error function
\( C_1 \) coefficient of the error function
\( C_2 \) coefficient of the error function
\( C_3 \) coefficient of the error function
\( D \) inner diameter of the inclinometer pipe
\( D_A \) inner diameter of the pipe within the channels in direction A
\( D_B \) inner diameter of the pipe within the channels in direction B
\( D_{\text{cor}} \) corrected inner diameter of the inclinometer pipe
\( d \) diameter of the wheels of the probe
\( E \) average lateral modulus of the soil
\( E_g \) Young’s modulus of the grout
\( E_p \) Young’s modulus of the pipe
\( E_s \) Young’s modulus of the soil
\( F \) objective function
\( H \) thickness of the sliding layer
\( I \) moment of inertia of the cross-section of the pipe
\( k \) tangent stiffness of the pipe
\( L \) distance between measurement sections
\( p \) lateral earth pressure
\( R_m \) mean radius of the pipe
\( X \) distance describing the location of the center of rotation of the lever
\( Y \) base length of the lever
\( \alpha_L \) \hspace{1cm} \text{inclination of the lever} \\
\( \alpha_P \) \hspace{1cm} \text{inclination of the probe} \\
\( \beta \) \hspace{1cm} \text{inclination of the probe out of the plane} \\
\( \Delta \) \hspace{1cm} \text{error function due to the inclination out of the plane} \\
\( \Delta D_A \) \hspace{1cm} \text{change in diameter in direction A} \\
\( \Delta D_B \) \hspace{1cm} \text{change in diameter in direction B} \\
\( \Delta f \) \hspace{1cm} \text{increment of force} \\
\( \Delta p \) \hspace{1cm} \text{lateral pressure increment} \\
\( \Delta u \) \hspace{1cm} \text{increment of radial displacement} \\
\( \Delta \delta \) \hspace{1cm} \text{displacement increment} \\
\( \Delta \varepsilon \) \hspace{1cm} \text{lateral strain increment} \\
\( \Delta \Omega \) \hspace{1cm} \text{change in ovalization value} \\
\( \dot{\delta} \) \hspace{1cm} \text{displacement rate} \\
\( \nu_g \) \hspace{1cm} \text{Poisson's ratio of the grout} \\
\( \nu_p \) \hspace{1cm} \text{Poisson's ratio of the pipe} \\
\( \nu_s \) \hspace{1cm} \text{Poisson's ratio of the soil} \\
\( \sigma_1 \) \hspace{1cm} \text{principal lateral earth pressure} \\
\( \sigma_2 \) \hspace{1cm} \text{principal lateral earth pressure} \\
\( \tau^* \) \hspace{1cm} \text{average shear strength on the sliding surface} \\
\( \Omega \) \hspace{1cm} \text{ovalization value} \\
\( \Omega_0 \) \hspace{1cm} \text{zero measurement of the ovalization value}
3 Effects of time-dependency on earth pressure measurements taken by inclinodeformeter

3.1 Abstract

The inclinodeformeter (IDM) is a novel device for measuring changes in earth pressures in a sliding layer of a creeping landslide. The change of earth pressures in the sliding layer leads to changes in the shape and dimensions of the inclinometer pipe. If these changes are carefully measured, the pressure increment can be back-calculated from the solution of a boundary value problem with properly described constitutive behavior of all the materials involved: i.e. the pipe, the grout and the soil.

This chapter focuses on time-dependent mechanical properties of the inclinometer pipe and their effect on the measured changes in pipe diameter. Viscoelastic parameters of the pipe material have been obtained from creep tests and loading tests at a constant stress rate. For validation, full-scale laboratory creep tests were performed in a 2 m high calibration chamber. A numerical model allows for a reasonably accurate back-calculation of the applied pressures at any time during the creep test.
3.2 Introduction

3.2.1 Inclinodeformeter

The inclinodeformeter (IDM) is a novel device for measuring changes in earth pressure in a sliding layer of a creeping landslide (chapter 2). The device makes use of the existing and widely used technology of inclinometer measurements. The change of lateral earth pressures in the sliding layer leads to changes in the shape and dimensions of the inclinometer pipe. If these changes are measured, the pressure increment can be back-calculated from the solution of a boundary value problem with properly described constitutive behaviors of all the materials involved: i.e. the pipe, the grout and the soil.

The deformation characteristics of the pipe are a crucial component of the boundary value problem. Thanks to its industrial production, the behavior of the pipe is less variable than the behavior of the grout and the soil. This chapter focuses on time-dependent mechanical properties of a plastic pipe, which is the most commonly used type of inclinometer pipe in Switzerland.

The pipe is made of unplasticized PVC, which is an amorphous thermoplastic polymer. For an ideal thermoplastic material, deformations at constant stress will continuously increase with time (e.g. Brinson & Brinson (2008)). Therefore, in order to be able to back-calculate changes in earth pressures at the time scale of creeping landslides, it is important to describe the long-term stiffness of the pipe in an appropriate way. Unplasticized PVC follows a linear viscoelastic behavior below the yield point and below the glass transition temperature \( T_g \) (Povolo et al., 1996), which was determined to lie within the range of 347 K to 353 K by Becker (1955), Povolo et al. (1996) and Domininghaus et al. (2008), i.e. considerably higher than the temperature in soil.

The viscous properties of the plastic pipes are normally derived from axial tests, because these are elementary tests with easy interpretation. In this study, viscoelastic parameters of the pipe material have been obtained in the longitudinal and transversal direction by calibrating the linear four-parameter model (Burgers model) against the results of creep tests and element tests at constant rate. The influence of stiffness anisotropy of the pipe material was found to be significant. The viscoelastic model was then incorporated into the finite ele-
ment analysis of the boundary value problem for the back-calculation of earth pressures and successfully validated against full-scale laboratory tests.

3.3 Pipe behavior in the longitudinal direction

3.3.1 Element test setup

To obtain the time-dependent mechanical properties of the pipe, compression tests were performed on samples in the axial direction. Three different type of compression tests were conducted: displacement controlled loading tests at high stress level to obtain strength (see section 3.3.2), creep tests at low stress level (see section 3.3.4), and loading tests at constant stress rate at low stress level (see section 3.3.5). The different tests were performed with different loading machines: fast loading tests at high stresses were conducted with a hydraulic press (Figure 3-1a). Short-term creep tests and loading tests at constant stress rate were controlled by a step motor (Figure 3-1b). The samples for long-term creep tests were loaded by weight (Figure 3-1c).

Figure 3-1: Element compression test: a) In a hydraulic press, b) Controlled by a step motor, c) By weight.

The samples have a diameter of 84 mm and a wall thickness of 6.2 mm. The height of the samples in the displacement controlled loading test at high stress level was 55 mm (section 3.3.2). All the experiments performed at low stress level were conducted on samples with a height of 120 mm (sections 3.3.4 and 3.3.5). Numerical calculations with finite elements showed that these experiments can be considered as element tests.
Different pipes were used in the two full-scale experiments which are shown in section 3.5.1. Although they are of the same type of pipe, these two pipes do not have exactly the same properties. Therefore the element tests were performed on samples of both pipes, which are denoted as pipe samples A and B respectively. The difference in properties may be due to different ageing or due to different conditions in production.

### 3.3.2 Stress-strain behavior

In order to obtain an indication of the stress level below which viscoelastic behavior can be assumed, the pipe was brought to failure. Figure 3-2 shows the stress–strain behavior of samples which were loaded in axial compression at constant strain rates. Uniaxial compressive strength is dependent on the strain rate. The compressive strength for fast loading is about 56 MPa, which is very close to the strength obtained by Pink (1976) and Povolo et al. (1996).

![Figure 3-2: Element compression tests at constant strain rates and 296 K.](image)

For the slow test the uniaxial compression strength is around 39 MPa. In both tests, the stress–strain behavior could be observed to be completely reversible up to at least 15 MPa. Further considerations within this chapter are focusing on long-term pipe deformations caused at stresses below 15 MPa, because this stress level seems to be sufficient for most of the applications of IDM. Within this range, in accordance with the results of Povolo et al. (1996), viscoelastic behavior was assumed.
3.3.3 The four-parameter model

The mechanical analog of the viscoelastic four-parameter model introduced by Burgers (1935) consists of a Maxwell and a Kelvin element connected in series (Figure 3-3).

![Four-parameter model diagram](image)

Figure 3-3: The four-parameter model.

Its differential stress–strain relation is shown in Equation (3-1).

\[
\sigma + \left( \frac{\eta_0}{E_0} + \frac{\eta_1}{E_1} \right) \sigma + \left( \frac{\eta_0 \eta_1}{E_0 E_1} \right) \dot{\sigma} = \eta_0 \dot{\varepsilon} + \left( \frac{\eta_0 \eta_1}{E_1} \right) \ddot{\varepsilon}
\]  

(3-1)

The strain response for the case of creep with constant stress \(\sigma_0\) results in (e.g. Brinson & Brinson (2008)):

\[
\varepsilon(t) = \sigma_0 \left( \frac{1}{E_0} + \frac{1}{E_1} \right) \left( 1 - e^{-t / \eta_1} \right) + \frac{t}{\eta_0}
\]

(3-2)

The strain due to loading at constant stress rate \(\dot{\sigma}_0\) can be derived in a similar way:

\[
\varepsilon(t) = \dot{\sigma}_0 \left( -\frac{\eta_1}{E_1} \right) \left( 1 - e^{-t / \eta_1} \right) + \frac{1}{2\eta_0} t^2 + \left( \frac{1}{E_0} + \frac{1}{E_1} \right) t
\]

(3-3)

For the IDM device both cases of constant stress and of constant stress rate are of special interest: e.g., in Switzerland in some constrained creeping landslides the lateral earth pressure
stays constant over the fall and winter and increases linearly with time over the spring and summer (Puzrin and Schmid, 2011). Even if there is expected to be no change in the stress field, pipe deformations at constant stress should be predictable in order to check the hypothesis of no change in earth pressure.

### 3.3.4 Element creep test

The four-parameter model is assumed to describe the stiffness in uniaxial loading with time. In order to calibrate the four-parameter model, uniaxial element creep tests were performed on samples of both pipes at several stress levels between 4.7 MPa and 8.7 MPa. Figure 3-4 shows the measured strains with time normalized by the constant creep stress $\sigma_0$. Initial deformations when applying $\sigma_0$ are included. The parameters (Table 3-1) of the four-parameter model were obtained by adjusting the analytical solution in Equation (3-2) to the measured data in Figure 3-4.

![Figure 3-4: Element creep tests on pipe A and B and analytical creep functions of the four-parameter model. Creep tests at 293.2 K, $\sigma_0_{pipe,A} = 4.7$ MPa, $\sigma_0_{pipe,B} = 8.7$ MPa.](image)

The sample of pipe A shows considerably softer behavior. The main difference occurs in the very first day after loading. Therefore initial modulus $E_0$ is the most affected parameter of the model. To obtain viscoelastic properties for another pipe of the same type it would be sufficient to perform an element creep test of one day, with loss of minor precision.
Table 3-1: Parameters of the four-parameter model determined by longitudinal loading of the pipe.

<table>
<thead>
<tr>
<th>parameters</th>
<th>pipe A</th>
<th>pipe B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0\text{ axial}$ [MPa]</td>
<td>2500</td>
<td>2830</td>
</tr>
<tr>
<td>$\eta_0\text{ axial}$ [MPa×day]</td>
<td>$3.0\times10^6$</td>
<td>$3.4\times10^6$</td>
</tr>
<tr>
<td>$E_1\text{ axial}$ [MPa]</td>
<td>24000</td>
<td>26000</td>
</tr>
<tr>
<td>$\eta_1\text{ axial}$ [MPa×day]</td>
<td>$5.0\times10^4$</td>
<td>$5.5\times10^4$</td>
</tr>
</tbody>
</table>

3.3.5 Element test at constant stress rate

Element tests at constant stress rate were also performed, because of its relevance for the application of IDM in creeping landslides. Samples of both pipes were loaded in axial direction with 0.57 MPa/min up to 14 MPa. Figure 3-5 shows the comparison between the measured strain and the strain derived from the analytical function provided by Equation (3-3) and parameters derived from the creep tests and listed in Table 3-1.

![Graph showing element tests at constant stress rate on pipes A and B, and analytical creep functions of the four-parameter model. Element tests at 293.2 K, $\dot{\sigma}_0 = 0.57$ MPa/min.](image)

The relatively high stress rate and the short duration of the element tests are not representative for creeping landslides. Nevertheless these results may be meaningful with respect to long-term behavior, because the strain is expected to develop almost linearly with time ac-
According to the viscoelastic model. The term in Equation (3-3) that depends linearly on time is dominant for the derived parameters (Table 3-1). According to the model, strains should grow for about one year almost uniformly with time for any stress rate.

3.3.6 Influence of water

Inclinometer pipes often go down below the ground water table. Therefore it is essential to check the influence of the water on the stiffness of the material. Preliminary stress relaxation element tests showed that the viscoelastic properties of the pipe material are not affected by the presence of water. Even under water pressure of 500 kPa, the stress relaxation of the sample was identical to that in the test with air at atmospheric pressure surrounding the sample.

3.4 Pipe behavior in transversal direction

3.4.1 Stiffness anisotropy

The anisotropy of the pipe stiffness is a result of its production process. During the extrusion the polymer melt is strained in the flow direction, orientating the polymer molecules and producing stiffer behavior in the axial direction. After the extrusion the melt is cooled from outside. Internal stresses form because parts of the cross-section of the pipe become solid while other parts are still molten. These so-called thermal stresses can also produce anisotropy.

In order to detect this anisotropy, samples of the pipe material (Figure 3-6a) were heated within glycerin above the glass transition temperature $T_g$. In the rubber elastic state between $T_g = 353$ K and the melting temperature $T_m = 453$ K (Domininghaus et al., 2008) internal stresses are released, producing deformations (Figure 3-6b).

The samples expand in the transversal direction but shrink in the axial direction of the pipe (Figure 3-6c). The observed deformations are indications of the thermal stresses resulting in the stiffness anisotropy.
Figure 3-6: Heating experiment on pipe samples: a) Sample before heating, b) Deformed sample after heating to 433 K, c) Measured strains due to released internal stresses.
3.4.2 Transversal loading test setup

In order to quantify the influence of the observed anisotropy, loading tests were performed on the pipe in the transversal direction. A piece of pipe with a length of 55 mm was subjected to a load applied along a line (Figure 3-7a). For the creep tests, the load was kept constant with time at 200 N. For the constant rate tests, the load was continually increased with time up to 200 N. The load was controlled by a step motor; radial deformations were measured by an LVDT (linear variable differential transformer). On the inside of the pipe there are two pairs of channels, which are used to guide the IDM probe. Loading tests were conducted with the channels rotated by 0° and by 45° with respect to the direction of the force.

Figure 3-7: Loading test in transversal direction with the channels parallel to the force: a) Test setup, b) Finite element model.

3.4.3 Numerical model of transversal loading tests

The boundary value problem of the transversal loading test is solved numerically using finite elements (Abaqus) to be able to compare the observed transversal and axial stiffness on the material level. Because of symmetry, only half of the real length of the pipe has to be taken into account for the finite element model. The piece of pipe loaded on one side by a linear distributed load and supported on the other side along a line is considered as a three-dimensional boundary value problem. The finite element mesh with boundary conditions is shown in Figure 3-7b. Three-dimensional elements (C3D8) with linear shape function and
full integration scheme were chosen. Implementation of the viscoelastic constitutive model is described in section 3.5.3.

### 3.4.4 Transversal creep test

Transversal creep tests indicate considerably softer material behavior in the transversal direction than in the longitudinal direction. Anisotropy is introduced via the elastic components of the four-parameter model (Figure 3-3):

$$f_A = \frac{E_{\text{transversal}}}{E_{\text{axial}}} = \frac{E_{\text{transversal}}}{E_{\text{axial}}}$$  \hspace{1cm} (3-4)

The viscous components of the four-parameter model are assumed not to be affected by the anisotropy:

$$\eta_{\text{axial}} = \eta_{\text{transversal}}, \quad \eta_{\text{axial}} = \eta_{\text{transversal}}$$  \hspace{1cm} (3-5)

Good agreement between the measured and the calculated creep deformations (Figure 3-8) could be achieved with the parameters (Table 3-2) of the four-parameter model based on a rather reasonable value of $f_A = 0.88$.

![Figure 3-8: Creep loading tests in transversal direction on pipe A and B and calculated creep deformations using the four-parameter model considering anisotropy with $f_A = 0.88$. Creep tests at 293.2 K.](image)

The Poisson’s ratio of the pipe is assumed to be time independent. It is specified by the producer as 0.34.
Table 3-2: Parameters of the four-parameter model determined by transversal loading of the pipe.

<table>
<thead>
<tr>
<th>parameters</th>
<th>pipe A</th>
<th>pipe B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0$ transversal</td>
<td>[MPa]</td>
<td>2200</td>
</tr>
<tr>
<td>$\eta_0$ transversal</td>
<td>[MPa×day]</td>
<td>3.0×10^6</td>
</tr>
<tr>
<td>$E_1$ transversal</td>
<td>[MPa]</td>
<td>21120</td>
</tr>
<tr>
<td>$\eta_1$ transversal</td>
<td>[MPa×day]</td>
<td>5.0×10^4</td>
</tr>
</tbody>
</table>

3.4.5 Transversal test at constant rate

Transversal loading tests at constant rate were also performed, because of their relevance for the application of IDM in creeping landslides. In all the tests, the samples were loaded from 0 to 200 N. Figure 3-9 shows the comparison of the measured and calculated deformations for loading at constant rate over a period of 4 days. The factor of anisotropy $f_A = 0.88$ and the model parameters according to Table 3-2 can be confirmed.

Figure 3-9: Comparison of measured and calculated deformations in loading tests in transversal direction at constant rate of 0.01 N/m/s: a) On pipe A, b) On pipe B. Constant rate tests at 293.2 K.

The loading rate in these tests is too high to be representative for creeping landslides. Nevertheless, the results may be meaningful with respect to long-term behavior. According to the viscoelastic model and the obtained parameters, the deformations should develop propor-
tionally to the loading rate and almost linearly with time. Verification in short-term laboratory tests both of proportionality to the loading rate and linearity with time may allow the proposed model to be assumed in creeping landslides. Therefore, the loading rate was changed in different tests across four orders of magnitude. Measured deformations are normalized by the loading rate (Figure 3-10).

![Figure 3-10: Comparison of measured and calculated deformations in loading tests in transversal direction at constant rate of 10 N/m/s, 1 N/m/s, 0.1 N/m/s, 0.01 N/m/s: a) On pipe A, b) On pipe B. Constant rate tests at 293.2 K.](image)

The good agreement with the numerical calculations confirms the proportional dependency of the deformations on the loading rate across four orders of magnitude. Also, the linearity with time can be observed between 10 seconds and about 4 days. These results may allow the proposed model to be assumed for long-term behavior.
3.5 Validation of the model

3.5.1 Full-scale laboratory creep test

For validation of the viscoelastic model, full-scale laboratory creep tests were performed in a 2 m high chamber (IDM box). The pipe embedded into a test chamber was subjected to an immediate change in earth pressure. Once the earth pressure reached 125 kPa it was kept constant for more than 300 days.

The calibration chamber has a cross-section of 40 cm by 40 cm. The pipe stands in the middle of the IDM box. In the first test setup, pipe A is surrounded by sand representing the soil; in the second test setup, pipe B is surrounded by a grouted annulus embedded in sand (Figure 3-11).

The earth pressure $\sigma_1$ in the principal direction is applied with pressure membranes. The boundary conditions in the minor principal direction are constrained. Deformation measurements of inner diameters are taken by the inclinodeformeter.

3.5.2 Numerical model

A horizontal, two dimensional cross-section of the IDM box can be considered as the boundary value problem to be solved using finite elements. The complex visco-elasto-plastic behavior of the grout and the sand is simplified in the numerical analysis in order to focus on effects due to viscoelasticity of the pipe. The stiffness of the sand was measured in element...
tests (chapter 4). The equivalent stiffness which is applied to the intact grout was determined in order to take the influence of the cracks into account (chapter 4). The derived elastic parameters are adjusted to the actual stress level of the performed creep test and listed in Table 3-3.

Table 3-3: Elastic parameters of the sand and of the grout.

<table>
<thead>
<tr>
<th></th>
<th>sand</th>
<th>grout</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ [MPa]</td>
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<td>4.5</td>
</tr>
<tr>
<td>$\nu$ [-]</td>
<td>0.24</td>
<td>0.2</td>
</tr>
</tbody>
</table>

3.5.3 Implementation of the viscoelastic model

The four-parameter model of Figure 3-3 is equivalent to a generalized Maxwell model consisting of two Maxwell elements connected in parallel (e.g. Bland and Lee (1956)), subject to the proper mapping between the corresponding sets of model parameters.

![Generalized Maxwell model with two elements.](image)

The four parameters of the generalized Maxwell model (Figure 3-12) are related to the original four parameters of the model in Figure 3-3 by a set of algebraic expressions. A generalized Maxwell model can be easily incorporated into common finite element codes using a Prony series. Parameters in Table 3-1 and Table 3-2 were therefore transferred into parameters of a generalized Maxwell model (Table 3-4) for implementation.
Table 3-4: Parameters of the generalized Maxwell model in axial and transversal directions of the pipe.

<table>
<thead>
<tr>
<th>parameters</th>
<th>pipe A</th>
<th>pipe B</th>
</tr>
</thead>
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<td>$3.4\times10^6$</td>
</tr>
<tr>
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<td>279</td>
</tr>
<tr>
<td>$\eta_{2,M \text{ axial}}$ [MPa×day]</td>
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<td>531</td>
</tr>
<tr>
<td>$E_{1,M \text{ transversal}}$ [MPa]</td>
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<td>2245</td>
</tr>
<tr>
<td>$\eta_{1,M \text{ transversal}}$ [MPa×day]</td>
<td>$3.0\times10^6$</td>
<td>$3.4\times10^6$</td>
</tr>
<tr>
<td>$E_{2,M \text{ transversal}}$ [MPa]</td>
<td>208</td>
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</tr>
<tr>
<td>$\eta_{2,M \text{ transversal}}$ [MPa×day]</td>
<td>446</td>
<td>531</td>
</tr>
</tbody>
</table>

3.5.4 Comparison

Comparisons of the measured and the calculated changes in pipe diameters during the creep tests in the IDM box are shown in Figure 3-13. The linear (viscous) term in the model (see Equation 3-2) seems to reproduce the observed behavior very well after 10 days. Rather good agreement has also been achieved in the period between 1 and 10 days, which is mainly controlled by the exponential (delayed elastic) term. The deformations occurring in the very first day of the creep tests are underestimated by about 11% by the numerical model based on the parameters derived from element tests in the pipe’s longitudinal direction.

Significantly better agreement can be achieved by using the parameters of the four-parameter model obtained in the transversal direction of the pipe to take account of the stiffness anisotropy (Figure 3-13).
3.5.5 Discussion

Even after taking anisotropy into account, the calculated initial deformations are still about 6% smaller than the measured deformations. One possible explanation is that the stiffness of the sand in the box is likely to be lower at low pressures than that obtained from the element tests owing to the better controlled preparation of the sample in the element tests.

In order to quantify this effect, the stiffness of the sand was adjusted to provide the best fit to the data in Figure 3-13. The resulting value of Young’s modulus taken for the sand, 5.8 MPa, appeared to be consistent with the stiffness values back-calculated from the previous box experiments with fast loading. This confirms that the lower stiffness of the sand is the likely reason for the remaining deviation (Figure 3-13).

3.6 Back-calculating pressures

The pressure changes can be back-calculated from the measured diameter changes using the solution of the inverse boundary value problem, provided the stiffness of the materials involved is known (see chapter 2).

For a time-dependent inverse analysis the pressure history has to be known or assumed. The applied pressure in the IDM box test could be back-calculated because both the time when the pressure was applied and the fact that it stayed constant are known. Based solely on the parameters derived from element tests, the back-calculated pressures for both tests are about
11% higher than the applied pressure. While for the majority of field applications this accuracy is reasonable, efforts to account for the pipe’s anisotropy can improve the accuracy of back-calculation of pressures. The error in the back-calculated pressures could be demonstrated to be as low as 6% when pipe anisotropy is taken into account.

3.7 Conclusions

Time-dependent deformations of an inclinometer pipe can be described by the viscoelastic four-parameter model. Normally, viscous properties of the plastic pipes are derived from axial tests, because these are elementary tests with easy interpretation. In this study, stiffness anisotropy between axial and transversal pipe directions was found to have a significant influence and could be effectively introduced into the model in a rather simple way. The model and its independently determined parameters could be successfully validated in full-scale laboratory experiments.

The viscoelastic model with parameters defined from the transversal direction tests is essential for field applications of IDM. It can provide reliable back-calculation of pressures both for instantaneous and continuously applied loads.

ACKNOWLEDGEMENTS

The contributions of P. Oberender of the ETH Zurich to this study are highly appreciated. This work has been partially supported by the ASTRA/VSS (grant no. VSS 2010/502) ‘Landslide-Road-Interaction: Applications’.

REFERENCES


**NOTATION**

\( E \) \hspace{1cm} Young’s modulus

\( E_0 \) \hspace{1cm} elastic parameter of the Burgers model (four-parameter model)

\( E_1 \) \hspace{1cm} elastic parameter of the Burgers model (four-parameter model)

\( E_{1,M} \) \hspace{1cm} elastic parameter of the generalized Maxwell model

\( E_{2,M} \) \hspace{1cm} elastic parameter of the generalized Maxwell model

\( f_A \) \hspace{1cm} factor of anisotropy for the elastic stiffness components

\( T_g \) \hspace{1cm} glass transition temperature

\( T_m \) \hspace{1cm} melting temperature

\( t \) \hspace{1cm} time

\( \varepsilon \) \hspace{1cm} strain

\( \eta_0 \) \hspace{1cm} viscous parameter of the Burgers model (four-parameter model)

\( \eta_1 \) \hspace{1cm} viscous parameter of the Burgers model (four-parameter model)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\eta_{1,M}$</td>
<td>viscous parameter of the generalized Maxwell model</td>
</tr>
<tr>
<td>$\eta_{2,M}$</td>
<td>viscous parameter of the generalized Maxwell model</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>stress</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>constant creep stress</td>
</tr>
<tr>
<td>$\dot{\sigma}_0$</td>
<td>constant stress rate</td>
</tr>
</tbody>
</table>
4 Influence of the grout on earth pressure measurements taken by inclinodeformeter

4.1 Abstract

The grout is used as a backfill of the free space in the borehole around the pipe. It transfers the earth pressure from the ground to the pipe, effecting its deformation. Therefore, understanding the influence of the mechanical behavior of the grout is important for earth pressure measurements taken by IDM. The mechanical properties of the grout can be chosen via its composition. Based on laboratory tests on different grout compounds, a procedure is developed to design the optimal grout composition according to the requirements of IDM measurements.

In this chapter, the effects of the grout on the back-calculation of lateral earth pressures are investigated using physical and numerical modeling. In the analysis, the linear elastic model is used for the grout and the soil; the linear viscoelastic four-parameter model is used for the pipe. The required material parameters are derived from independent element tests. The back-calculation is successfully validated against full-scale laboratory tests. The parameters obtained from the element material tests allowed a precise back-calculation of the applied pressures. The back-calculation of pressure increments is shown to be reliable and almost independent of initial conditions. The smallest detectable pressure increment is as small as 0.2 kPa.

Cracks in the cement-bentonite grout ring can reduce its stiffness significantly. Fortunately, the back-calculation of pressures is not sensitive to the stiffness of the grout ring. Less than a twofold increase in the back-calculated stress increments is caused by a tenfold increase in the stiffness of the grout. Below the ground water table, the grout is likely to be intact. In dry soils, where the cement-bentonite grout is prone to shrink and crack, sand may be used as an alternative to refill the borehole.
4.2 Introduction

The inclinodeformeter (IDM) is a novel device for back-calculation of changes in lateral earth pressure. A change in lateral earth pressure leads to changes in shape and dimensions of the inclinometer pipe. Provided that these changes are measured, the pressure increment can be back-calculated from the solution of the corresponding boundary value problem.

The grout is used as a backfill of the free space in the borehole around the pipe. It transfers the earth pressure from the ground to the pipe, causing its deformation. Therefore, understanding the influence of the mechanical behavior of the grout is important for earth pressure measurements taken by IDM.

Cement-bentonite grout is used most commonly to refill boreholes. Guidelines for the composition of the mixture and for the mixing process are provided by Mikkelsen (2002) regarding application to borehole instruments. If the grout is used for inclinometers, it should satisfy the criteria of maximum and minimum strength according to Dunnicliff (1988, 1993).

Choosing the composition of the grout mixture provides the opportunity to control the mechanical properties of the grout. The general rule is to mimic the strength and deformation characteristics of the surrounding soil (Mikkelsen, 2002). The stiffness of the grout is of major concern for IDM. The stiffness of the grout affects the measured deformations of the pipe. Therefore, the potential for back-calculation of earth pressures is influenced by the stiffness of the grout. Based on laboratory tests on different grout compounds, a procedure is developed to design the optimal grout composition according to the requirements of IDM measurements.

In this chapter, the effects of the grout on the back-calculation of lateral earth pressures are investigated using physical and numerical modeling. In the analysis, the linear elastic model is used for the grout and the soil; the linear viscoelastic four-parameter model is used for the pipe. The required material parameters are derived from independent element tests. The back-calculation is successfully validated against full-scale laboratory tests.
Chapter 4: Influence of the grout on earth pressure measurements taken by inclinometer

4.3 Full-scale laboratory tests

4.3.1 Test setup

Full-scale laboratory tests were carried out in a 2 m high calibration chamber (IDM box, Figure 4-1) to study the influence of the mechanical behavior of the grout. Within the chamber, the pipe surrounded by a grouted annulus surrounded by soil is subjected to earth pressure changes. For comparison, the experiment is also performed in the absence of grout with only soil surrounding the pipe. A well-graded sand is used for the soil.

![Figure 4-1: The IDM box: a) View from outside, b) Sand surrounding the pipe in the opened IDM box, c) Dimensions of the cross-section of the IDM box.](image)

The calibration chamber has a cross-section of 40 by 40 cm² and is equipped with pressure membranes on each of the four inner walls (Figure 4-1c). Two independent principal horizontal stresses can be applied. The inclinometer pipe is fixed in the middle of the box. It is surrounded by either sand alone or a grout annulus surrounded by sand (Figure 4-2).

![Figure 4-2: Boundary conditions in IDM box loading tests: a) Experiment without grout, b) Experiment with grout surrounding the pipe.](image)
The pressure membranes are generally inflated by compressed air. The membranes in the major principal direction (direction 1) are inflated, applying pressure on the soil. The membranes in the minor principal direction (direction 2) are opened to atmospheric pressure, providing constrained boundary conditions. The pressure membranes can also be operated with water. Closing the water-filled membranes in the minor principal direction provides the opportunity to measure the minor principal stress at the constrained boundary. The kinematically constrained boundary conditions to the sides in the direction of the minor principal stress are representative for many problems in geotechnical engineering. In particular, for large landslides with a wide moving front, plane strain conditions can be assumed in the vertical plane parallel to the slope. The IDM box is closed at its top with a steel plate. Therefore, no deformation can occur in the vertical and third principal direction. The deformations of the pipe are measured. The IDM device is used to take readings of the inner diameters in direction 1 and direction 2.

Numerical calculations showed that the differences in pressures and deformations due to the dimensions of the IDM box are small compared with a model with larger dimensions representing a free field solution.

4.3.2 Composition of the grout

A cement-bentonite grout is used for the experiments, because this type of grout is widely used in geotechnical practice. The grout mixture is made of water, type 1 Portland cement and sodium bentonite. The grout compound was chosen to fulfill requirements of workability (feasibility to pump), sedimentation (no bleeding), strength and stiffness (see section 4.8). The chosen grout compound has a weight ratio of water : cement : bentonite equal to 2.6 : 1 : 0.4.

The water and the cement are mixed first, as described by Mikkelsen (2002). The bentonite is added later. This mixing procedure allows control of the water–cement ratio of the grout, which has a strong influence on its properties. Hence, the strength and the stiffness of the grout are better controlled. The water–cement paste and the cement–bentonite grout were both mixed for five minutes in a planetary paddle mixer as described by Contreras et al. (2008).
4.3.3 Experimental results

The soil in the calibration chamber was subjected to loading, unloading, reloading and eventually full unloading (Figure 4-3). Therefore, the pressure $\sigma_1$ at the static boundary condition in direction 1 was increased to 200 kPa, then reduced to 75 kPa, increased to 200 kPa and eventually released to zero pressure. Diameter 1 in direction of the applied stress becomes smaller with increasing pressure $\sigma_1$, whereas diameter 2 in the perpendicular direction becomes larger. The measurements at low pressures were ignored before the membranes opened overcoming the earth pressure at rest. Owing to leaking of the membranes, the experiment in absence of grout could only be conducted up to 194 kPa.

Non-reversible deformations are observed in both experiments. In both experiments, deformations due to unloading or reloading seem to be reversible provided the pressure does not drop too much in the unloading phase. Pipe deformations are observed to depend almost linearly on the pressures in the loading phase and in the reversible part of the unloading phase. The behavior is even more linear in the experiment with grout than in the experiment without grout.

The pressure $\sigma_2$ at the boundary in the minor principal direction was measured when applying the pressure $\sigma_1$ at the boundary in the major principal direction (Figure 4-4). The ratio $\sigma_2/\sigma_1$ is denoted as $K_0$ which is practically constant for both configurations. The ratio $K_0$ is found to be the same in both experiments.
4.3.4 Discussion of experimental results

Large pressure increments are applied in the full-scale laboratory experiments in order to study the behavior of the grout across a large range of pressures. However, the pressure increments in field applications may be considerably smaller. Therefore, the precision of IDM diameter measurements of ±2 micrometers (see section 2.3.5) is considered in order to assess the smallest pressure increment that can still be detected by IDM. The precision in terms of pressure is obtained by comparing the measured deformations with the precision of the IDM diameter measurement. For the loading phase, the precision in terms of pressure is 0.2 kPa. For the reloading phase, the precision is 0.6 kPa in the experiment without grout and 0.5 kPa in the experiment with grout.

The relation between earth pressure changes and pipe deformation becomes more linear due to the grout (Figure 4-3). Therefore, the back-calculation procedure will become more reliable and less dependent on the initial stress state.
4.4 Tests on the materials used in the full-scale laboratory experiment

The applied pressure increments can be back-calculated provided that the stiffness of all the materials is known and described in an appropriate way. Therefore, laboratory tests are performed on the material of the pipe, the grout and the soil in order to measure their deformation characteristics. The linear elastic model is assumed for the constitutive behavior of the soil and the grout for simplification, although the real material behavior is visco-elasto-plastic. The pipe is considered to be viscoelastic. Parameters are derived for all three materials at the range of stress of the full-scale experiment. The parameters are used for the validation of the numerical model.

4.4.1 Stiffness of the pipe

Two different pipes are used in the two full-scale experiments presented in this chapter. Although they are of the same type, the two pipes do not have exactly the same properties. Therefore, the loading tests were performed on samples of both pipes, denoted as pipe samples A and B respectively. The difference in properties may be due to different aging or due to different conditions in production. Pipe A was used in the experiment with only sand; pipe B was used in the experiment with grout (Figure 4-2).

The viscoelastic four-parameter model (Figure 4-5) is found to be appropriate to describe the time-dependent behavior of the pipe material (chapter 3). The parameters of the model (Table 4-1) were determined from creep tests performed in the transversal direction of the pipe and successfully validated against full-scale creep experiments conducted in the IDM box (chapter 3).

![Figure 4-5: Mechanical analog of the viscoelastic four-parameter model (Burgers model).](image)
Table 4-1: Parameters of the four-parameter model determined by transversal loading of the pipe.

<table>
<thead>
<tr>
<th>parameters</th>
<th>pipe A</th>
<th>pipe B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0$ [MPa]</td>
<td>2200</td>
<td>2490</td>
</tr>
<tr>
<td>$\eta_0$ [MPa d]</td>
<td>$3.0\times10^6$</td>
<td>$3.4\times10^6$</td>
</tr>
<tr>
<td>$E_1$ [MPa]</td>
<td>21120</td>
<td>22880</td>
</tr>
<tr>
<td>$\eta_1$ [MPa d]</td>
<td>$5.0\times10^4$</td>
<td>$5.5\times10^4$</td>
</tr>
</tbody>
</table>

In addition, loading tests at constant rate were performed in the transversal direction in order to check whether the model and its parameters still hold for the different loading conditions applied in the IDM box experiments. The test at the loading rate of 1 N/m/s is similar to the experiments performed in the IDM box in terms of loading condition and deformation. The test setup and the numerical model for interpretation are described in section 3.4. In Figure 4-6, the measured deformations are compared with the results of numerical calculations using finite elements. Because of the good agreement, the four-parameter model and its parameters are assumed to be appropriate for the analysis of the experiments performed in the IDM box. The Poisson’s ratio of the inclinometer pipe is specified as 0.34 by the manufacturer of the pipe.

![Figure 4-6: Comparison of measured and calculated deformations in loading tests in transversal direction at constant rate of 1 N/m/s: a) On pipe A, b) On pipe B.](image)

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4.4.2 Stiffness of the sand

Element tests were performed on the sand in order to determine its stiffness. The sample tested in triaxial compression was at a similar density to the sand in the IDM box (Table 4-2). The tested sample was slightly overconsolidated owing to its preparation.

Table 4-2: Relative density of the sand given by the density index $I_D$.

<table>
<thead>
<tr>
<th>test</th>
<th>$I_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>IDM box test without grout</td>
<td>69 %</td>
</tr>
<tr>
<td>IDM box test with grout surrounding the pipe</td>
<td>63 %</td>
</tr>
<tr>
<td>triaxial compression test: built in sample before saturation</td>
<td>63 %</td>
</tr>
<tr>
<td>triaxial compression test: slightly overconsolidated sample after saturation</td>
<td>88 %</td>
</tr>
</tbody>
</table>

The change in volume was measured via the excess of pore water while changing the mean effective stress $p'$ (Figure 4-7). The bulk stiffness is found to be dependent on pressure and stress path.

![Figure 4-7: a) Sample of sand built in a triaxial apparatus, b) Triaxial compression test on the sand in a triaxial shear apparatus.](image-url)
The tangent bulk modulus $K_t$ is assumed to depend on the mean effective pressure $p'$ according to the analytical function

$$K_t = k \left( \frac{p'}{p_r} \right)^m$$  \hspace{1cm} (4-1)

where $p_r = 1$ kPa is the reference pressure. The parameters $k = 245$ for loading, $k = 520$ for unloading and $m = 0.87$ are derived by a regression analysis, fitting the integrated function

$$\varepsilon_v = \frac{1}{k(1-m)} \left( \frac{p'}{p_r} \right)^{1-m}$$  \hspace{1cm} (4-2)

to the measured volumetric strains (Figure 4-7b). On the initial loading path, $k$ was fitted to higher pressures to make sure that the regression is not affected by the overconsolidation caused by the preparation of the sample.

The secant bulk modulus $K$ is derived for the range of pressure in the sand in the IDM box experiment ($K = 5.6$ MPa in initial loading; $K = 25$ MPa in reloading). Measurements of the shear modulus $G$ in triaxial shear tests provide a range for the Poisson’s ratio of the sand. The Poisson’s ratio of the sand was determined within this range as 0.23 in order to reproduce the measured $K_0$ in the IDM box experiments (see section 4.5.2). Assuming elasticity, $K$ can be translated into $E_s = 9$ MPa for the loading phase and 40 MPa for the unloading–reloading phase.

Triaxial shear tests were performed in order to investigate the frictional behavior of the sand. The samples were sheared in triaxial compression in drained conditions. The failure stress state at constant volume is shown in Figure 4-8. Assuming the Mohr–Coulomb failure envelope, its inclination in triaxial stress space $M$ is equal to 1.54, which corresponds to the angle of internal friction at constant volume $\varphi'_{CV}$ of 37.8°.
Chapter 4: Influence of the grout on earth pressure measurements taken by inclinometer

4.4.3 Stiffness of the grout

Unconfined compression tests were performed in order to assess the stiffness of the grout. The grout was mixed as in section 4.3.2 but using a smaller mixing device (Figure 4-9a). Two half pipes prescribing the diameter of the sample were filled with the liquid paste. After the first hardening phase (7 days), the sample was pre-consolidated within the half pipes by applying 20 kPa of vertical pressure by weight. Subsequently, the half pipes were removed after 28 days and the samples were tested in uniaxial compression at the strain rate of 0.08% / min (Figure 4-9b and Figure 4-9c). The force was measured corresponding to the applied displacement.

Figure 4-9: Unconfined compression test: a) Grout suspension during mixing, b) Failed sample (diameter = 56 mm, height = 75 mm, loading rate = 0.01 mm/s), c) Sample after large displacements.
For the grout mixture used in the IDM box experiment, the Young’s modulus was measured to be 71 MPa (Figure 4-10). The softer behavior at small strains is expected to be due to the limited contact between the press and the grout sample. The unconfined compressive strength was found to be 755 kPa. The Poisson’s ratio of the grout is assumed to be 0.2.

Several different grout mixtures were tested in uniaxial compression. The results obtained are reported in section 4.8.1.

![Figure 4-10: Uniaxial unconfined compression test on grout.](image)

4.5 **Numerical analysis of full-scale tests**

4.5.1 **Boundary value problem**

The pressure change can be back-calculated from the measured change in pipe diameters, provided the stiffness of the involved materials in this range of stress is known. A horizontal, two-dimensional cross-section of the IDM box is considered as the boundary value problem to be solved in inverse analysis using finite elements (Figure 4-2). The dimensions of the cross-section are 400 mm by 400 mm, corresponding to the inner dimensions of the IDM box (Figure 4-1c). The element behavior perpendicular to the cross-section is assumed to be plain strain for the sand and the grout, and plain stress for the pipe. Pressure boundary conditions are applied in the major principal direction (direction 1). The boundary conditions in the minor principal direction (direction 2) are constrained. The boundary value problem is solved
for the two different setups: the inclinometer pipe is surrounded by either (a) only sand or (b) a grout annulus surrounded by sand. The outer diameter of the grout annulus was measured to be 133 mm (see Figure 4-1c) after the experiment was completed.

4.5.2 Numerical model

IDM box tests were modeled with finite elements using the Abaqus code to back-calculate the applied pressures. The finite element meshes considered are shown in Figure 4-11. For the sand and the grout, quadratic plain strain elements (CPE4) with linear shape function and full integration scheme were chosen. Triangular plain stress elements (CPS6) with quadratic shape function are used for the pipe. Coulomb friction is assumed for the tangential behavior in the interface surrounding the pipe. The friction angle $\delta_p$ between the pipe and the sand is chosen according to the common assumption of $\delta_p = 2/3 \varphi_{CV}$, which corresponds to a friction coefficient of 0.47. The friction coefficient in the interface between the pipe and the grout is also considered to be 0.47. The sand and the grout are connected with a tie constraint; there is no interface taken into account.

![Figure 4-11: Finite element meshes: a) Model without grout, b) Model with grout.](attachment:image)

For the pipe, the viscoelastic four-parameter model is implemented according to chapter 3. The complex deformation behavior of the grout and the soil is simplified for back-calculation to a linear constitutive model defined by elastic parameters. The Young’s moduli of the sand and the grout are obtained in element tests; the Poisson’s ratio of the grout is assumed (section 4.4.2 and 4.4.3). The Poisson’s ratio of the sand is the main parameter controlling the contact pressure at the constrained boundaries. Therefore, the Poisson’s ratio $\nu_s$ is back-calculated using the finite element model in order to fit the measured $K_0$ ratio (section 4.3.3).
The parameters of the pipe, the grout and the sand are listed in Table 4-3, which is a summary of sections 4.4.1, 4.4.2 and 4.4.3.

Table 4-3: Material parameters: summary of sections 4.4.1–4.4.3.

<table>
<thead>
<tr>
<th>parameter description</th>
<th>parameter value</th>
<th>derivation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0$ parameter of the pipe A / B</td>
<td>2200 / 2490 MPa</td>
<td>transversal loading tests</td>
</tr>
<tr>
<td>$\eta_0$ parameter of the pipe A / B</td>
<td>3.0×10^6 / 3.4×10^6 MPa d</td>
<td>transversal loading tests</td>
</tr>
<tr>
<td>$E_1$ parameter of the pipe A / B</td>
<td>21120 / 22880 MPA</td>
<td>transversal loading tests</td>
</tr>
<tr>
<td>$\eta_1$ parameter of the pipe A / B</td>
<td>5.0×10^4 / 5.5×10^4 MPa d</td>
<td>transversal loading tests</td>
</tr>
<tr>
<td>$\nu_p$ Poisson’s ratio of the pipe</td>
<td>0.34</td>
<td>specified by the producer</td>
</tr>
<tr>
<td>$E_{s,l}$ modulus of the soil in loading</td>
<td>9 MPa</td>
<td>triaxial compression test</td>
</tr>
<tr>
<td>$E_{s,ul}$ modulus of the soil in unloading</td>
<td>40 MPa</td>
<td>triaxial compression test</td>
</tr>
<tr>
<td>$\nu_s$ Poisson’s ratio of the soil</td>
<td>0.23</td>
<td>$K_o$ measured in IDM box</td>
</tr>
<tr>
<td>$E_g$ modulus of the grout</td>
<td>71 MPa</td>
<td>unconf. compression test</td>
</tr>
<tr>
<td>$\nu_g$ Poisson’s ratio of the grout</td>
<td>0.2</td>
<td>assumed</td>
</tr>
</tbody>
</table>

4.6 Validation

The results of the full-scale laboratory experiments are compared with numerical calculations based on the parameters listed in Table 4-3.

The back-calculated pressures are very close to the applied pressures for the experiment with only sand surrounding the pipe (Figure 4-12a). The linear constitutive model and the derived material parameters are found to be appropriate. The difference between the initial loading path and the unloading–reloading path is explained by the difference in the bulk stiffness of the sand. Knowledge of the volumetric behavior of the sand allowed a precise and reliable back-calculation of pressures. Taking the nonlinearity of the volumetric behavior into account could even allow for more precise back-calculation. Nevertheless, for practical applica-
tion the linear constitutive model is shown to be appropriate; it also makes the back-calculation independent of initial stress conditions.

![Figure 4-12: The comparison between back-calculated and applied pressures: a) Experiment without grout, b) Experiment with grout surrounding the pipe.](image)

The pressures back-calculated from the experiment with grout surrounding the pipe are considerably larger than the applied pressures (Figure 4-12b). The pressure increments are over-estimated by around a factor of 2. The discrepancy is likely to be due to over-estimation of the stiffness of the grout ring. In order to check the condition of the grout, the IDM box was opened after the experiment was completed. The grout was found to be well-shaped but cracked. Two major cracks are forming on both sides in the direction of diameter 2 throughout the height of the grout (Figure 4-14a). Two minor cracks through the cylinder are observed in direction 1. The crack opening of the major cracks was less than 1 mm (Figure 4-13a); the minor cracks were very thin and could hardly be recognized. A regular pattern of cracks could be observed (Figure 4-13b).
A preliminary numerical analysis was performed to check whether the cracks could cause the softer behavior of the grout ring. Cracks were introduced into the finite element model where the large major cracks are observed in the experiment. For simplification, no normal and no shear forces were assumed to act within the cracks. The calculated deformations were found to be remarkably close to the measured pipe deformations (Figure 4-14b). Therefore, the softer behavior of the grout ring is probably due to the observed cracks.
4.7 Discussion

Reliable and precise back-calculation of pressures can be provided if the materials can be considered as a continuum. Unfortunately, cracks within the grout annulus can reduce the stiffness of the grout ring considerably. The equivalent stiffness of the continuum would be more than 20 times smaller owing to the cracks. Nevertheless, the IDM pressure measurements still provide a relatively good result because the back-calculated pressures are only affected by a factor of 2, owing to the low sensitivity to the grout stiffness.

The measured pipe deformations are similar in the experiment with only sand to the experiment with the cracked grout annulus surrounding the pipe. Therefore, the equivalent stiffness of the cracked grout body may be assumed to be similar to the stiffness of the soil. For the cracked grout, the analytical solution (presented in chapter 6) may be applied, which assumes the same stiffness for the grout and for the soil.

There are applications where this approach may not be appropriate. In the case of an intact grout ring surrounded by soft soil, the stiffness of the grout may be considerably larger than the stiffness of the soil. In this case, the grout stiffness may be taken into account by performing numerical simulations in order to back-calculate the pressures. This approach is presented in chapter 7 for the back-calculation of pressures in silty deposits next to an excavation.

The question remains as to whether the grout ring can be considered to be intact or cracked in field applications. The cracks in the laboratory tests may have occurred because of shrinkage of the grout. The dry sand surrounding the grout and the indoor climate may have supported the grout shrinking. Therefore, the grout ring is less likely to be damaged in ground water. Below the ground water table, the hypothesis of an intact grout ring may be made. If the grout is cracked above the ground water table, it may be recognized by a change in pipe deformations of around a factor of 2 at the level of the water table. For applications in very dry soils, where the grout is expected to shrink, the cement-bentonite grout may be replaced by sand to refill the borehole.

However, further research is required to understand the conditions under which the grout becomes cracked. In addition, the deformation behavior of the cracked ring needs to be further investigated.
4.8 Design of the grout composition

4.8.1 Influence of the grout composition on its properties

IDM pressure measurements are influenced by the properties of the grout which is used to refill the borehole. Therefore, the properties of the cement-bentonite grout are investigated for different compositions. This section focuses on the workability, the bleeding, the strength and the stiffness of the grout. All the grout mixtures were produced as described in section 4.3.2 and tested after 28 days.

The influence of the composition on the workability and stability towards sedimentation was reported by Jones (1963). Grout mixtures in the consistency regions a, b, c and d are pumpable (Figure 4-15). Grout compositions in the consistency region d are additionally indicated to be stable towards sedimentation. To refine the area of stable grouts, sedimentation tests were carried out on 40 different mixtures. The ratio between the volume of the segregated water at the top and the total volume was measured after 28 days of hardening. Several lines were obtained describing mixtures with the same amount of bleeding (Figure 4-15).

![Diagram showing workability and bleeding depending on grout composition](image)

Figure 4-15: Workability and bleeding depending on grout composition (portion of the compounds by weight): diagram and consistency regions a, b, c, d, e, f after Jones (1963).

The strength of different grout mixtures was determined in unconfined compression tests as described in section 4.4.3 (Figure 4-16). The mixture marked with the black dot was used in the full-scale experiment in the IDM box (Figure 4-16). The strength of the grout is mainly controlled by the water–cement ratio (Marsland, 1973). The lower the water–cement ratio the
higher is the unconfined compressive strength. The relation obtained between the strength and the water–cement ratio is similar to that of Contreras et al. (2008). Nevertheless, at high water–cement ratio, the results of Contreras et al. (2008) show higher strength than presented here.

![Diagram](image)

Figure 4-16: Unconfined compressive strength $f_c$ depending on the grout composition (portion of the compounds by weight).

The stiffness of different grout mixtures was obtained in uniaxial compression tests as in section 4.4.3. The measured Young’s modulus is shown in Figure 4-17 for different compositions.

The stiffness is found to be correlated to the unconfined compressive strength. Therefore it is not possible to match strength and stiffness with the same mix (Mikkelsen, 2002), especially if other requirements such as workability or stability towards sedimentation are considered as well.
4.8.2 Adjusting the grout mixture to the requirements of IDM

In order to be appropriate for IDM pressure measurements, the grout mixture is recommended to follow certain requirements:

1) The grout suspension should be pumpable.
2) To avoid the suspension settling in the borehole and hence risking a borehole with the topmost part filled with segregated water only, the grout mix should not show any bleeding,
3) The back-calculation of earth pressure changes becomes more difficult and less reliable if the grout has cracks or has entirely failed. Therefore, the grout should not fail.
4) The precision and sensitivity of the IDM pressure measurements are depending on the stiffness of the grout (Figure 4-18). The choice of the grout should allow for a high precision and sensitivity.

For a cement-bentonite grout, the following approach is expected to follow the requirements. The proposed approach is based on Figure 4-15, Figure 4-16 and Figure 4-17:
1) Workability: The grout suspension is pumpable as long as it belongs to consistency region d in Figure 4-15.

2) Stability towards sedimentation: The suspension is considered to be stable if the grout composition stays around the line of 1% of bleeding in Figure 4-15. This condition reduces the choice of the grout into a one-dimensional problem (Figure 4-15).

3) Strength: To avoid failure, a minimum strength is required. Therefore, grout compositions may not be admissible with water–cement ratio higher than the corresponding water–cement ratio (Figure 4-16).

4) Stiffness: Figure 4-17 and Figure 4-18 may be used to determine the composition of the grout in order to enhance precision and sensitivity. Figure 4-18 shows the precision as a function of the stiffness of the grout for an assumed set of parameters. The choice of grout composition may be based on the information provided by this figure. For many applications, the softest grout composition still admissible may be favorable in terms of precision and sensitivity. Therefore, the stable mixture providing the required minimum strength may be chosen.

Figure 4-18: Precision of the IDM pressure measurement as a function of the Young’s modulus of the grout for the following assumptions: constrained boundary conditions to the sides, plane stress boundary condition in vertical direction due to constant overburden, $E_p = 2000$ MPa, $\nu_p = 0.34$, $\nu_g = 0.2$, $\nu_s = 0.2$.

The optimum stiffness of the grout, where the highest precision can be achieved, is found to be quite low. The grout is recommended to be on the stiff side of the optimum stiffness, because the precision strongly deteriorates at stiffness smaller than the optimum (Figure 4-18). Therefore, Young’s modulus of the grout is recommended to be larger than 10 MPa.
4.9 Conclusions

Adjusting the mechanical properties of the grout to the soil is essential for earth pressure measurements taken by IDM. Based on laboratory tests on different grout compounds, a procedure is developed to design the optimal grout composition for the requirements of IDM measurements. The sensitivity of the IDM pressure measurements can be enhanced by using softer grout provided the Young’s modulus of the grout is no smaller than 10 MPa. In addition, there are other constraints for the design of the optimal grout compound: i.e. the workability, the stability towards sedimentation and the strength.

Full-scale laboratory tests were performed in a 2 m high calibration chamber incorporating the grout. The full-scale tests demonstrated that simple linear constitutive models can successfully be used as a first approximation for back-calculation. The back-calculation of pressure increments becomes reliable and independent of initial conditions. The smallest detectable pressure increment is shown to be as small as 0.2 kPa.

In general, the grout is expected to be designed to have similar deformation characteristics to the surrounding soil. Thus, the influence of the stiffness of the grout can be neglected for back-calculation of pressures, and an analytical solution may be applied, which assumes the same stiffness for the grout as for the soil. If the grout is stiffer than the soil but cracked, the influence of the stiffness of the grout remains small and may be neglected, so the analytical solution may again be applied. For an intact grout ring, which is considerably stiffer than the surrounding soil, the grout stiffness can be taken into account by performing numerical simulations in order to back-calculate the pressures. Fortunately, the back-calculation of stresses is not sensitive to the stiffness of the grout. Less than a twofold increase in the back-calculated stress increments is caused by a tenfold increase in the stiffness of the grout. Below the ground water table, the grout is likely to be intact. In dry soils, where the cement-bentonite grout is prone to shrink and crack, sand may be used as an alternative to refill the borehole.

Further research is required to understand the conditions under which the grout becomes cracked. The deformation behavior of the cracked ring also needs to be further investigated.
Chapter 4: Influence of the grout on earth pressure measurements taken by inclinometer

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REFERENCES


NOTATION

$E_0$ elastic parameter of the Burgers model (four-parameter model)

$E_1$ elastic parameter of the Burgers model (four-parameter model)

$E_g$ Young’s modulus of the grout

$E_{s,l}$ Young’s modulus of the soil in loading

$E_{s,ul}$ Young’s modulus of the soil in unloading and reloading

$f_c$ unconfined compressive strength

$I_D$ density index of the sand

$K$ secant bulk modulus of the sand
$K_0$ ratio between the principal lateral stresses: $\sigma_2 / \sigma_1$

$K_t$ tangent bulk modulus of the sand

$k$ empirical parameter

$M$ inclination of the Mohr–Coulomb failure envelope in triaxial stress space

$m$ empirical parameter

$p$ mean effective stress

$q$ deviatoric stress

$\delta_p$ angle of friction in the interface between the pipe and the soil / grout

$\epsilon_v$ volumetric strain

$\eta_0$ viscous parameter of the Burgers model (four-parameter model)

$\eta_1$ viscous parameter of the Burgers model (four-parameter model)

$\nu_g$ Poisson’s ratio of the grout

$\nu_p$ Poisson’s ratio of the pipe

$\nu_s$ Poisson’s ratio of the soil

$\sigma_1$ major principal lateral stress; applied at boundary condition

$\sigma_2$ minor principal lateral stress; measured at boundary condition

$\varphi’_{CV}$ angle of internal friction at constant volume of the sand
Chapter 5: Effects of longitudinal bending on earth pressure measurements taken by inclinodeformeter

5 Effects of longitudinal bending on earth pressure measurements taken by inclinodeformeter

5.1 Abstract

A change in ovalization of an inclinometer pipe does not only occur because of changes in earth pressure. As a pipe is bent longitudinally its cross-section flattens into an oval shape. The inclinometer pipe is subjected to bending due to displacements of the surrounding soil, which may not be related to changes in pressure. Therefore, the IDM measurements have to be corrected. The amount of change in ovalization value due to longitudinal bending $\Delta \Omega \chi$ is subtracted from the measured change in ovalization value $\Delta \Omega$.

An analytical solution calibrated using numerical simulations describes the change in ovalization due to bending as a function of the curvature of the pipe. The curvature is obtained by numerical differentiation of the inclination of the pipe, which is measured by IDM in a profile with depth. The calculated change in ovalization value due to bending $\Delta \Omega \chi$ may not fully develop. If the length of the section of intense curvature is shorter than the required length of the transition zone, smaller pipe deformations will be observed.

However, the calibrated solution shows that the effects of longitudinal bending can be neglected for many applications. The cross-sectional deformations due to bending are usually smaller than the precision of the IDM device. The effects of bending can be ignored in particular within the sliding layer of a creeping landslide.
5.2 Introduction

As a thin tube is bent longitudinally, its cross-section flattens into an oval shape. The inclinometer tube is subjected to bending due to displacements of the surrounding soil, which may not be related to earth pressure changes. Therefore, the measured pipe deformations have to be corrected for the amount of ovalization due to bending.

One may think of examples in which ovalization changes due to bending with hardly any change in pressure. Figure 5-1 shows an inclinometer pipe in a creeping landslide. A thick shear zone at residual strength is assumed between the sliding layer and the stable ground. Within the shear zone, pressures do not change during shearing. The inclinometer pipe, which is embedded in the stable ground, is bent within the shear zone. The cross-section of the pipe is deformed by bending. A change in ovalization of the inclinometer pipe is observed by IDM measurements, although the lateral earth pressures do not change.

Another example is given by an inclinometer pipe embedded in soil in active failure behind a retaining wall (Figure 5-2). Although the wall continues rotating, the earth pressures behind the wall hardly change because of the failure. Nevertheless, the pipe is bent because of the displacements of the surrounding soil, causing ovalization of the pipe. Without correction of the effects due to bending, IDM measurements can be misleading. Therefore, the pipe deformations caused by bending should be calculated and subtracted from the measured deformations in order to correct the IDM pressure measurements.
Figure 5-2: Example of change in ovalization which is not related to pressure change: active failure behind a retaining wall.

5.3 Correction of the change in ovalization due to bending

The nonlinear bending response of initially straight tubes was studied by Brazier (1927). Assuming an oval shape, isotropic elastic material and no pressures, the ovalization is obtained by minimizing the total strain energy:

\[
\Delta D_\chi = -\frac{2\chi^2 (R - h/2)^3}{h^2} (1 - \nu_p^2) \cos(2\theta_\chi) \tag{5-1}
\]

\( \Delta D_\chi \) change in diameter due to bending  
\( \chi \) curvature  
\( \theta_\chi \) circumferential coordinate with respect to direction of the curvature  
\( R \) outer radius of the tube  
\( h \) thickness of the tube  
\( \nu_p \) Poisson’s ratio of the pipe

For this solution, higher order terms of the longitudinal stretching strain energy were neglected on the assumption of small ovalization. The curvature \( \chi_{max} \) of an inclinometer pipe was observed to be always smaller than 0.0001 mm\(^{-1}\) even in the case of a pipe which was almost sheared. Within this range the ovalization can be considered as small, and higher or-
nder terms can be neglected. Numerical analysis confirms that Brazier's solution provides an extremely close approximation for curvature smaller than $\chi_{\text{max}}$.

Although the analytical solution was developed for thin tubes, numerical analysis indicates that the solution can also be applied for thicker tubes such as inclinometer pipes subjected to curvatures smaller than $\chi_{\text{max}}$. In accordance with the work of Karamanos (2002) the cross-sectional radius $(R-h / 2)$ was used as the reference line in Equation (5-1).

![Figure 5-3: Finite element model of the inclinometer pipe subjected to constant curvature assuming linear elasticity: a) Undeformed pipe, b) Deformed pipe, c) Deformed cross-section due to curvature.](image)

Figure 5-3: Finite element model of the inclinometer pipe subjected to constant curvature assuming linear elasticity: a) Undeformed pipe, b) Deformed pipe, c) Deformed cross-section due to curvature.
Equation (5-1) cannot be applied to an inclinometer pipe because of its channels, which affect the deformed shape induced by bending. Nevertheless the analytical formula can be used as an approximation for the change of the diameters in the channels if the thickness $h$ of the tube is reduced. The reduced thickness $h_{red} = 5.35\,\text{mm}$ was calibrated using finite element simulations (Figure 5-3). The change in ovalization value due to bending is derived. The change in ovalization value is defined in the framework of IDM as the change in the difference of the two measured diameters normalized by the outer radius of the pipe. The amount of change in ovalization value due to bending must be subtracted:

$$
\Delta \Omega = \frac{\Delta D_A^x - \Delta D_A^z}{\Omega} = \frac{4 \sin^2 \left( \frac{R - h_{red}}{2} \right)^5}{R h_{red}^2} \left(1 - \frac{\rho^2}{\chi} \right) \cos(2 \delta_A^z) 
$$

(5-2)

The angle between the channel in direction A and the direction of the curvature is denoted as $\delta_A^x$. Figure 5-4 shows the comparison of the analytical approximation of Equation (5-2) to finite element calculations for different angles $\delta_A^x$.

Figure 5-4: Change in ovalization value due to bending of pipes at different rotation: comparison of the analytical function of Equation (5-2) with finite element calculations for different angles $\delta_A^x$. The angle $\delta_A^x$ denotes the rotation of the pipe with respect to the direction of the curvature.
IDM measurements are always taken with respect to an initial zero reading. If the pipe is considered to be initially curved, the initial curvature $\chi_0$ should be taken into account:

$$\Delta \Omega = \frac{\Delta D^y - \Delta D^z}{R} = \frac{4(\chi^2 - \chi_0^2)(R - \frac{h_{red}}{2})^3}{R h_{rad}^2} \left(1 - \nu_p^2\right) \cos(2\delta^z)$$ (5-3)

No correction is needed if the change in ovalization due to bending is smaller than the precision of the IDM measurement. The precision of the IDM measurement of $[\Delta D_B - \Delta D_A]_{min}$ of 2 micrometers yields the precision in terms of ovalization value $\Delta \Omega_{min}$ (see section 2.3.5). Thus, the effects of bending can be neglected if

$$|\Delta \Omega| \leq \Delta \Omega_{min} = 4.8 \times 10^{-5}$$ (5-4)

Hence, the condition for the curvature to be neglected is obtained from Equation (5-3) for the conservative assumption of $\delta^y = 0^\circ$ and $\nu_p = 0.34$:

$$|\chi^2 - \chi_0^2|_{\text{max}} \leq 1.7 \times 10^{-10} \text{ mm}^{-2}$$ (5-5)

For many applications, no correction of the measured ovalization is needed because the curvature is within the bounds provided by Equation (5-5).

### 5.4 Influence of pressure

The pressures acting on the installed inclinometer pipe have an influence on the deformations due to bending which has not been considered so far. An inclinometer pipe is usually filled with water during installation to prevent the pipe lifting up in the borehole. The water table remains inside the pipe. Lateral earth pressure and most often also groundwater pressure act on the outside of the pipe. The total pressure from outside is expected to be larger than the pressure inside the pipe.

Numerical calculations show that pipe deformations due to bending are considerably different in the two principal axes, when pressures are taken into account. Therefore Equation (5-1) is not appropriate when pressures are considered. Equations (5-2) and (5-3) still provide a remarkably good agreement to the numerical calculations, although they slightly overestimate the change in the difference of the diameter. Closed form solutions for thin pipes that
do take pressures into account, e.g. Reissner (1959), are not applicable owing to the relative thickness of the inclinometer pipe and owing to its channels.

Therefore, the error incurred by applying Equations (5-2) and (5-3) is assessed by numerical calculations under simplified conditions. Because the water table inside the pipe is often observed at a similar level to the ground water table, both are assumed to be at the level of the ground surface. A density $\gamma$ of 20 kN/m$^3$ and lateral earth pressure coefficient $K$ of 0.5 are assumed to derive horizontal earth pressures. The influence of pressures increasing with depth on the change in ovalization value is shown in Figure 5-5. The ovalization value influenced by pressures can be normalized for different curvature by the ovalization value in absence of pressures, which is obtained from the analytical function. The largest error occurs for very soft pipe material (e.g. $E_p = 1500$ MPa; $\nu_p = 0.34$) at a large depth. For this worst case scenario, the deformations due to bending are overestimated by Equations (5-2) and (5-3) by about 19% at a depth of 50 m (Figure 5-5).

Figure 5-5: Effects of pressure change with depth on the change in ovalization value due to curvature, for simplified conditions.
5.5 Transition between sections of different curvature

If the curvature is changing along the tube, the deformations due to bending form a kind of transition zone up to the point where the constant deformation described by the analytical solution is reached. According to Millard and Roche (1984), the length of this transition zone depends on geometrical parameters and on Poisson’s ratio. Therefore the propagation length can be considered to be constant, because the same type of inclinometer pipe is used. Finite element calculations (Figure 5-6) confirm that the length of the transition zone does not depend on the amount of change in curvature (Figure 5-7) nor on the length of sections with constant curvatures (Figure 5-8). The length of the transition zone was determined to be about 433 mm. If the section of constant curvature is long enough, the deformation will reach the analytical solution (Figure 5-7). The wavelike shape of the propagating ovalization is according to the analytical solution provided by Millard and Roche (1984).

![Finite element model of the inclinometer pipe with two sections subjected to constant different curvature (χ₁ and χ₂) assuming linear elasticity: a) Undeformed pipe, b) Deformed pipe.](image)
Chapter 5: Effects of longitudinal bending on earth pressure measurements taken by inclinometer

Figure 5-7: Propagation of the change in ovalization value for two sections of different constant curvature ($\chi_1$ and $\chi_2$).

Figure 5-8: Propagation of the change in ovalization value for two sections of constant curvature ($\chi_1 = 0.0001 \text{ mm}^{-1}; \chi_2 = 0 \text{ mm}^{-1}$) and different length.
5.6 Application

To quantify the influence of the deformations due to bending, the calibrated solution is applied to a specific IDM reading which can typically be observed in creeping landslides. Figure 5-9 shows the initial curvature \( \chi_0 \) and the curvature \( \chi \) of an inclinometer pipe located in the Brattas landslide in St Moritz (see chapter 6). The curvature is obtained by numerical differentiation of the inclination of the pipe, which is measured by IDM as a depth profile. The slip surface is located at a depth of 23 m, where large curvature is observed. However, the curvature is found to be small in the sliding layer above the slip surface.

![Curvature of an inclinometer pipe in a creeping landslide measured by IDM.](image)

The change in ovalization value due to bending \( \Delta \Omega^x \) can be calculated using Equation (5-3). Equation (5-5) provides a band within which no correction for bending is needed for the conservative assumption of the curvature occurring in the direction of the channels (\( \delta_{A}^x = 0^\circ \)).
The pipe deformations due to bending can be ignored within the sliding layer (see Figure 5-10), because they are smaller than the precision of the IDM diameter measurements. In the shear zone, the effects of longitudinal bending are considerably larger. Because pressures are not expected to change within the shear zone (see section 5.2), the ovalization of this pipe section is not of interest. Therefore, no correction for the influence of bending is needed for IDM measurements in creeping landslides.

In other applications where the curvature is larger, the effects of bending may no longer be negligible. The change in ovalization value can be corrected using Equation (5-3), although the pipe deformations caused by bending are overestimated for several reasons. The cross-sectional deformations are reduced by the effects of pressure (section 5.4). In addition, the ovalization caused by bending is reduced if the length of the section of intense curvature is limited compared with the length required to build up the full deformation (section 5.5).
5.7 Conclusions

An analytical solution calibrated using numerical simulations can describe the change in ovalization of an inclinometer pipe due to bending. The solution is able to capture the effects of the channels and the rotation of the pipe by considering a pipe section of constant curvature. The solution allows identification of many applications for IDM with relatively small curvature where the effects of longitudinal bending can be neglected. In particular, in the sliding layer of creeping landslides, the effects of longitudinal bending can be ignored. In the case of an application with intense curvature, longitudinal bending has to be considered. The solution can provide a preliminary estimate for correction of the measured deformation, although the correction is overestimated. The calculated deformations due to bending are overestimated owing to the surrounding pressures and to the limited length of the curved section. Additional studies are required in order to introduce these effects into an approach for the correction of the measured deformation.

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REFERENCES


**NOTATION**

- $E_p$: Young’s modulus of the pipe
- $h$: thickness of the pipe
- $h_{red}$: reduced thickness of the inclinometer pipe
- $K$: lateral earth pressure coefficient
- $R$: outer radius of the pipe
- $\gamma$: density of soil
- $\Delta D^X$: change in pipe diameter due to bending
- $\Delta D_A^X$: change in diameter in direction A due to bending
- $\Delta D_B^X$: change in diameter in direction B due to bending
- $\Delta \Omega_{\text{min}}$: precision of the IDM device in terms of ovalization value
- $\Delta \Omega^X$: change in ovalization value due to bending
- $\delta_A^X$: angle between the channel in direction A and the direction of the curvature
- $\theta^X$: circumferential coordinate with respect to direction of the curvature
- $\nu_p$: Poisson’s ratio of the pipe
- $\chi$: curvature
- $\chi_0$: initial curvature
- $\chi_{\text{max}}$: maximum curvature
6 Inclinodeformeter pressure measurements in creeping landslides: analytical solutions and field applications

6.1 Abstract

The chapter derives analytical solutions for the deformation of a viscoelastic pipe in elastic soil under far-field principal stress increments. These solutions are validated in laboratory tests and numerical analysis and provide the basis for the back-calculation of earth pressure increments from measured changes in shape of an inclinometer pipe cross-section using inclinodeformeter (IDM) technology. The procedure has been applied to back-calculating earth pressure changes and soil stiffness in three creeping landslides in Switzerland, which are similar in size but have a range of different boundary conditions and displacement rate fields. In these applications, pressure increments of less than 1 kPa could be reliably back-calculated allowing for identification of compression and extension zones, as well as yielding and failure of soil in the landslides under study, confirming the previously developed models of the landslide mechanisms and suggesting that IDM pressure measurements could serve as a reliable tool for analysis and monitoring of creeping landslides.
6.2 Introduction

Understanding the earth pressure changes in a sliding layer of a creeping landslide is essential for analysis and stabilization of such landslides. In particular, the rate of change of the lateral earth pressure has been found to be a critical parameter for the validation of models for constrained creeping landslides (Puzrin and Schmid, 2011, 2012; Puzrin et al., 2012).

The inclinodeformeter (IDM) is a device for back-calculation of the changes of earth pressure in the sliding layer of creeping landslides. IDM measures the change in dimensions of an inclinometer pipe and uses the measured deformations to back-calculate the change in pressure by solving the corresponding boundary value problem.

IDM makes use of the widespread inclinometer technology. In addition to the regular displacement profile, the IDM probe also provides a continuous profile of lateral earth pressures, using the same inclinometer pipe and requiring no further installation on the site.

In this chapter a simplified procedure is introduced for back-calculating the earth pressure changes. This procedure required derivation of analytical solutions for the deformation of viscoelastic pipes in elastic space under corresponding boundary conditions and calibration of these solutions against results of laboratory tests and numerical analysis. The procedure has then been applied to measuring earth pressure changes in three creeping landslides in Switzerland. These landslides are similar in size but have different boundary conditions and displacement rate fields, as listed below.

a) The St Moritz-Brattas landslide is slowing downhill towards a rock outcrop at the bottom.

b) The Braunwald landslide is accelerating downhill towards a vertical rock wall falling into a valley.

c) The Ganter landslide is moving uniformly downhill towards a river bed.

Pressure measurements taken by IDM allow the identification of compression and extension zones in the sliding body and provide unique quantitative data for understanding and mechanical analysis of these creeping landslides.
6.3 Measuring deformations of the inclinometer pipe

In order to measure inclinometer pipe deformations, the IDM probe is lowered down the pipe guided by three wheels along a pair of its channels. There are two tilt sensors on the probe to measure the inner diameter of the pipe: one sensor is located on the top of the probe; another tilt sensor is located on the lever of the middle wheel (see section 2.3.1). The inner diameter $D$ is a function of the relative inclination.

Diameter $D_A$ (Figure 6-1) between the channels in direction A, where the larger displacements are expected, is measured first; direction B is perpendicular to A (Figure 6-1). In addition, continuous measurements are taken of the inclination of the probe in both directions A and B, of the temperature and of the water pressure in the pipe, at a frequency of 40 readings per 1 cm of depth.

After completing measurements of diameter $D_A$, the probe is rotated by 90° and lowered down again, guided along the second pair of channels, so that the diameter $D_B$ can be measured along the pipe length. Because the tilt sensors are not independent of the inclination of the pipe out of the plane, both diameter readings are corrected for the influence of this inclination (see section 2.3.3). A correction function calibrated in laboratory tests is applied.

![Figure 6-1: Measured diameters in a horizontal cross-section of the installed inclinometer pipe.](image)
Next, the pipe ovalization value $\Omega$ is calculated, defined as the difference of the perpendicular diameters normalized by the nominal outer radius of the inclinometer pipe $R$.

$$\Omega = \frac{D_2 - D_1}{R}$$  \hspace{1cm} (6-1)

Unlike individual diameter measurements, the difference between the measured diameters does not appear to be sensitive to the temperature of the probe, long-term instability of the probe sensors, wear of the wheels, or to the influence of actual measurement conditions causing a uniform radial expansion or contraction of the pipe (such as temperature of the pipe, water pressures inside and outside of the pipe). Deformation changes of the pipe are always measured (and the corresponding pressure changes are back-calculated) relative to the first measurement in time, the so called zero-measurement. Further measurements at later points in time are taken after an expected change in lateral earth pressure has taken place. The deformation is described by the change in ovalization value $\Delta \Omega$ with respect to the zero measurement

$$\Delta \Omega_n = \Omega_n - \Omega_0 = \frac{(D_{B,0} - D_{A,0}) - (D_{B,n} - D_{A,n})}{R}$$  \hspace{1cm} (6-2)

Deformations of the pipe cross-section occur not only due to changes in earth pressures, but also as a result of the inclinometer pipe bending, causing its cross-section to flatten into an oval. The inclinometer pipe is subjected to bending due to displacements of the surrounding soil, which may not be related to earth pressure changes. The measured change in ovalization value can be corrected for bending, as described by an analytical formula calibrated against numerical simulations (chapter 5). However, if the pipe curvature is reasonably small, as is the case for many applications for IDM, no correction is necessary. The field measurements presented in this chapter have been obtained sufficiently far from the shear surface, so that the bending curvature did not exceed the threshold value defined in section 5.3. Therefore the cross-section deformation due to bending could be neglected for this study.

While the ovalization measurements are taken continuously over the thickness of the sliding layer, for back-calculation of the pressure changes a certain level of averaging and discretization is required to eliminate measurement noise. The inclinometer pipe is made out of 3 m long casing elements. Each casing element is treated as one sensor, with the measurements averaged within the element over the length of 12 times the circumference of the wheels of
the probe in order to reduce the influence of their imperfection. This leaves about 44 cm at the edges of the casing element which are neglected to eliminate the effects of the joints between the elements.

6.4 Analytical solutions for back-calculation of pressures

6.4.1 Boundary value problem: assumptions

The pressure increments causing deformations of a pipe cross-section in soil can be back-calculated from a solution of the corresponding boundary value problem, provided that the stiffness of the pipe, grout and soil are known in this range of pressures. Owing to the overburden, the vertical direction is assumed to be a principal stress direction with constant normal stress. The remaining non-zero components of the incremental stress tensor (i.e. the two principal stress increments $\Delta \sigma_1$ and $\Delta \sigma_2$ in the horizontal plane) are back-calculated by considering the boundary value problem given by a horizontal cross-section under generalized plane stress conditions (Figure 6-2). Within the plane, one principal stress direction of the increment is assumed to coincide with the direction of displacement, which is known from the conventional inclinometer measurements. The creeping landslides under consideration have wide moving fronts. Therefore, far away from the inclinometer pipe no horizontal displacements are assumed to occur in the direction perpendicular to the inclinometer measured displacement. Full slippage is assumed at the interface between the pipe and the soil. Radial deformations through the thickness of the pipe are assumed to be small.

Inclinometer pipes are fixed in the boreholes by grouting the annulus (Figure 6-1), with the grout around inclinometer pipes often chosen to mimic the deformation characteristics of the surrounding soil. The influence of the grout stiffness on the back-calculation of stresses is addressed in chapter 4. It appears that the back-calculation of stresses is not sensitive to the grout stiffness (Figure 4-18), with a ten-fold increase in the grout stiffness causing less than 30% maximum error in the back-calculated stress increments. Therefore, in the solution below it is assumed that the grout has the same stiffness as the sliding mass.
6.4.2 Boundary value problem: solution strategy

The solution for deformations of the viscoelastic pipe embedded in elastic soil under the far-field principal stress increments is derived through the following auxiliary steps.

a) First, deformations of the unsupported cylindrical cavity in elastic soil under far-field principal stress increments are calculated.

b) Second, a solution of the same problem, but with loaded cavity and zero far-field principal stress increments is derived.

c) Third, the general solution is derived for the deformations of the viscoelastic pipe subjected to variable radial pressures.

Finally, the pipe deformations caused by far-field changes in lateral earth pressure are obtained by combination of the three solutions derived above (i.e. deformation of the cylindrical cavity due to lateral earth pressure increments $u^{\Delta \sigma_1}$, deformation of the cavity due to pressure at the inclusion $u^q$ and deformation of the pipe due to external pressure $u^p$).

Indeed, compatibility of the soil and the pipe displacements in the radial direction requires

$$u^{\Delta \sigma_1} (r = R, \theta, t) + u^q (r = R, \theta, t) = u^p (r = R, \theta, t)$$

which, combined with the equilibrium of pressures at the boundary between the pipe and the cavity, allows for calculation of the deformation of the cross-section of a viscoelastic pipe embedded in elastic soil under the far-field principal stress increments.
6.4.3 Unsupported cylindrical cavity under far-field principal stress increments

In the first auxiliary step, deformations of the unsupported cylindrical cavity under far-field principal stresses are going to be derived (Figure 6-3a). The principal pressures at infinite distance from the cavity are assumed to be $\Delta \sigma_1$ and $\Delta \sigma_2 = K_0 \Delta \sigma_1$ owing to the far-field plane strain condition perpendicular to the landslide front, where $K_0 = \nu_s$. The static boundary conditions are decomposed into a hydrostatic component $\Delta \sigma_1 (1 + K_0)/2$ and a deviatoric component $\pm \Delta \sigma_1 (1 - K_0)/2$. The uniform contraction of the cavity caused by the hydrostatic component can be neglected because it has to be identical to the uniform contraction of the pipe, which is negligible compared to its ovalization. For the remaining deviatoric component, solution of the boundary value problem is the well-known Michell solution (e.g., Barber, 2002)

$$u^{\Delta \sigma_1} (r = R, \theta) = -\frac{2R \Delta \sigma_1 (1-K_0)\cos(2\theta)}{E_s} \quad (6-4)$$

where $E_s$ is the Young’s modulus and $\nu_s$ is the Poisson’s ratio of the soil; $R$ is the radius of the cavity (identical to the outer radius of the pipe) and $K_0$ is the ratio between the two principal stresses in the plane.

Figure 6-3: Schematic layout of the two auxiliary boundary value problems: a) Unsupported cylindrical cavity under far-field principal stress increments, b) Loaded cylindrical cavity with zero far-field principal stress increments.
6.4.4 Unsupported cylindrical cavity loaded by a variable radial pressure

The second auxiliary step requires solving the boundary value problem for the cylindrical cavity loaded by the pressure $-q(t)\cos(n\theta)$ at $r = R$, $n = 2, \ldots$ and zero far-field principal stresses (Figure 6-3b). Solution of this boundary value problem is another well-known Michell solution (e.g. Penzien and Wu, 1998; Barber, 2002):

$$u'(r = R, \theta) = \frac{Rq(t)\cos(n\theta)(5 - \nu_s)}{3E_s} \quad (6-5)$$

6.4.5 Viscoelastic solution for the pipe under plane stress conditions

In the third auxiliary step, viscoelastic deformations are derived for the pipe subjected to a variable radial pressure. Inclinometer pipes used in this study are made out of PVC. They show a time-dependent mechanical behavior, which can be described by the viscoelastic Burgers model (chapter 3). An analytical solution for the radial deformation of a pipe without channels is derived below for the case of constant pressure (creep) and constantly increasing pressure (constant rate), relevant for the long-term loading conditions in creeping landslides.

The external pressure $p(\theta, t)$ acting on the pipe is assumed to be the following function of the circumferential coordinate and time

$$p(\theta, t) = -p_c(t)\cos(n\theta) \quad n \geq 2 \quad (6-6)$$

where $n$ is an integer number. The hydrostatic case of $n = 0$ is neglected, because no ovalization is formed. The case of $n = 1$ is ignored owing to the lack of equilibrium. Static equilibrium of the thin-walled pipe in plane stress conditions is given by (Figure 6-4a)

$$Vd\theta + dN = 0 \quad (6-7)$$

$$dV - Nd\theta + p(\theta, t)R_md\theta = 0 \quad (6-8)$$

$$dM - VR_md\theta = 0 \quad (6-9)$$

where $R_m$ is the mean radius of the pipe, $N$ is the internal normal force, $V$ is the shear force and $M$ is the moment.
Combining equilibrium Equations (6-7)-(6-9), the following differential equations are obtained for the normal force and the moment

$$\frac{d^2N}{d\theta^2} + N = -p_n(t)R_n\cos(n\theta)$$  \hspace{1cm} (6-10)

$$\frac{d^2M}{d\theta^2} + M = p_n(t)R_n^2\cos(n\theta)$$  \hspace{1cm} (6-11)

Because the symmetry is required to be the same as for the applied pressure, the solutions of the Equations (6-10) and (6-11) are given by

$$N(\theta,t) = \frac{1}{n^2 - 1}p_n(t)R_n\cos(n\theta)$$  \hspace{1cm} (6-12)

$$M(\theta,t) = \frac{-1}{n^2 - 1}p_n(t)R_n^2\cos(n\theta)$$  \hspace{1cm} (6-13)

According to Bouma (1993), from the compatibility considerations, the strain profile through the thickness of the pipe (Figure 6-4b) can be described as

$$\varepsilon(z) = \frac{R_m}{R_n + z}(\varepsilon_0 + \chi z)$$  \hspace{1cm} (6-14)

where $\varepsilon_0 = \varepsilon(0)$ and $\chi = \varepsilon'(0) + \varepsilon(0)/R_m$, which are related to the radial displacement $u$ and the tangential displacement $v$ by the kinematic equations.
\[ e_0(\theta, t) = \frac{1}{R_m} \left( \frac{\partial v}{\partial \theta} + u \right) \]  

\[ \chi(\theta, t) = \frac{1}{R_m^2} \left( \frac{\partial v}{\partial \theta} - \frac{\partial^2 u}{\partial \theta^2} \right) \]  

(6-15)  

(6-16)

The constitutive equation of the pipe material in terms of circumferential stress and strain is given by the Burgers model (Figure 6-5)

\[ \sigma + p_1 \dot{\sigma} + p_2 \ddot{\sigma} = q_1 \dot{\varepsilon} + q_2 \ddot{\varepsilon} \]  

(6-17)

where \( q_1, q_2, p_1 \) and \( p_2 \) are parameters of the Burgers model

\[ p_1 = \frac{\eta_0}{E_0} + \frac{\eta_1}{E_1}; \quad p_2 = \frac{\eta_0 \eta_1}{E_0 E_1}; \quad q_1 = \eta_0; \quad q_2 = \frac{\eta_0 \eta_1}{E_1} \]  

(6-18)

\[ \begin{align*} 
N + p_1 \dot{N} + p_2 \ddot{N} &= q_1 \left( A \ddot{\varepsilon}_0 - \frac{I}{R_m} \dddot{\varepsilon} \right) + q_2 \left( A \ddot{\varepsilon}_0 - \frac{I}{R_m} \dddot{\varepsilon} \right) \\
M + p_1 \dot{M} + p_2 \dddot{M} &= q_1 \left( -\frac{I}{R_m} \dddot{\varepsilon}_0 + I \dddot{\varepsilon} \right) + q_2 \left( -\frac{I}{R_m} \dddot{\varepsilon}_0 - I \dddot{\varepsilon} \right)
\end{align*} \]  

(6-19)  

(6-20)

where \( A = 1 \times h \) is the area and \( I = 1 \times h^3/12 \) is the moment of inertia of the cross-section.
Equations (6-12) and (6-13), Equations (6-15) and (6-16), and Equations (6-19) and (6-20) are static, kinematic and constitutive equations, respectively, constituting together with the loading condition (Equation 6-6) the viscoelastic boundary value problem for a thin pipe under plane stress conditions. This problem will be solved using the elastic–viscoelastic correspondence principle (e.g. Findley et al., 1976).

As the first step, a simpler elastic problem will be solved. Assuming for the pipe material purely elastic constitutive behavior in terms of circumferential stress and strain $\sigma = E \varepsilon$, Equations (6-19) and (6-20) reduce to the following elastic constitutive equations in terms of forces and moments

$$N = E \left( A \varepsilon_0 - \frac{I}{R_m} \chi \right)$$  \hspace{1cm} (6-21)

$$M = E \left( -\frac{I}{R_m} \varepsilon_0 + I \chi \right)$$  \hspace{1cm} (6-22)

Combining Equations (6-12) and (6-13), Equations (6-15) and (6-16), and Equations (6-21) and (6-22) results in the following differential equation of the elastic boundary value problem

$$u + \frac{\partial^2 u}{\partial \theta^2} = \frac{1}{n^2 - 1} \frac{R_m^4}{EI} p_n \cos(n\theta)$$  \hspace{1cm} (6-23)

which has the solution

$$u(\theta) = -\frac{1}{\left(n^2 - 1\right)^2} \frac{R_m^4}{EI} p_n \cos(n\theta)$$  \hspace{1cm} (6-24)

In the second step, Equations (6-12) and (6-13), Equations (6-15) and (6-16), and Equations (6-19) and (6-20) of the viscoelastic boundary value problem are transformed into Laplace space

$$\hat{N}(\theta,s) = \frac{1}{n^2 - 1} \hat{p}_n(s) R_m \cos(n\theta)$$  \hspace{1cm} (6-25)

$$\hat{M}(\theta,s) = -\frac{1}{n^2 - 1} \hat{p}_n(s) R_m^2 \cos(n\theta)$$  \hspace{1cm} (6-26)

$$\hat{\varepsilon}_\phi(\theta,s) = \frac{1}{R_m} \left( \frac{\partial \hat{\varepsilon}_\phi}{\partial \theta} + \hat{u} \right)$$  \hspace{1cm} (6-27)
\[ \ddot{\chi}(\theta,s) = \frac{1}{R_m^4} \left( \frac{\partial^2 \ddot{\chi}}{\partial \theta^2} - \frac{\partial^2 \ddot{\chi}}{\partial \theta^2} \right) \] (6-28)

\[ \hat{N} = \frac{q_s + q_s^2}{1 + p_s + p_s^2} \left( \frac{1}{R_m} - \hat{\chi} \right) \] (6-29)

\[ \hat{M} = \frac{q_s + q_s^2}{1 + p_s + p_s^2} \left( -\frac{1}{R_m} \hat{\varepsilon}_0 + I \hat{\chi} \right) \] (6-30)

Equations (6-25)-(6-30) are equivalent to Equations (6-12) and (6-13), Equations (6-15) and (6-16), and Equations (6-21) and (6-22) of the elastic boundary value problem where the elastic modulus \( E \) is replaced by the transformed modulus

\[ \hat{E}(s) = \frac{q_s + q_s^2}{1 + p_s + p_s^2} \] (6-31)

Finally, according to the elastic–viscoelastic correspondence principle, the solution is equivalent to the elastic one in Equation (6-24) where true variables are replaced by the corresponding transformed ones

\[ \ddot{u}(\theta,s) = -\frac{1}{(n^2 - 1) \frac{R_m^4}{I}} \frac{1}{1 + p_s + p_s^2} \hat{\varepsilon}_0 \cos(n\theta) \] (6-32)

The inverse Laplace transformation is applied to Equation (6-32) first for the case of a load function of \( p_n = const \) corresponding to the transformed value of

\[ \hat{p}_n(s) = \frac{p_n}{s} \] (6-33)

and then for the case of a load function of \( \dot{p}_n = const \) corresponding to

\[ \hat{p}_n(s) = \frac{\dot{p}_n}{s^2} \] (6-34)

The resulting transformation to the time domain yields the viscoelastic solution

\[ u(\theta,t) = -\frac{1}{(n^2 - 1) \frac{R_m^4}{I}} p_n \cos(n\theta) \left( \frac{1}{E_0} + \frac{1}{E_1} \left( 1 - e^{-\frac{t}{\eta_0}} \right) \right) + \frac{1}{\eta_0} t \] (6-35)
for creep conditions with constant pressure $p_n$, and

$$u(\theta,t) = -\frac{1}{(n^2-1)^{\frac{3}{2}}} \frac{R^2}{I} \dot{p}_n \cos(n\theta) \left( \frac{\eta_1}{E_1} \left( e^{\frac{\eta_1}{\eta_0}} - 1 \right) + \frac{1}{2\eta_0} t^2 + \frac{1}{E_0} \right) t \right)$$  \quad (6-36)$$

for constant stress rate conditions with constant pressure rate $\dot{p}_n$.

### 6.4.6 Combined solution for the pipe–soil interaction

As is seen from Equation (6-4), the increment of the far-field principal stresses causes radial displacements of the cylindrical cavity proportional to $\cos(n\theta)$, where $n = 2$. These displacements will have to be compatible with the corresponding pipe displacements from Equations (6-35) and (6-36), that is, the deformation mode of the pipe will also be defined by $\cos(n\theta)$, with $n = 2$. According to Equation (6-6), this is only possible if the pressures acting on the pipe are proportional to $\cos(2\theta)$. According to the Third Law of Newton, the same pressures with the opposite sign will act on the boundary of the cavity. Therefore, in the second auxiliary step the cylindrical cavity will be loaded by the pressure $-q(t)\cos(2\theta)$ at $r = R$ and zero far-field principal stresses, which, following Equation (6-5), also produce the cavity deformation mode proportional to $\cos(2\theta)$, compatible with the corresponding pipe deformation mode.

Thus, the pipe deformations caused by changes in lateral earth pressure are obtained by combination of the derived solutions (i.e. deformation of the cylindrical cavity due to lateral earth pressure increments $u^{\Delta \sigma_1}$, deformation of the cavity due to pressure at the inclusion $u^q$ and deformation of the pipe due to external pressure $u^p$, both for the deformation mode of $n = 2$), provided the equilibrium of pressures at the boundary between the pipe and the cavity and the compatibility of the soil and the pipe displacements in the radial direction are satisfied

$$u^{\Delta \sigma_1} (r = R, \theta, t) + u^q (r = R, \theta, t) + u^p (r = R_m, \theta, t) \approx u^p (r = R_m, \theta, t)$$  \quad (6-37)$$

For formulating the equilibrium condition for the pressures at the boundary between the pipe and cylindrical cavity, the true pressure $q$ on the outside radius $R$ of the pipe has to be related to the pressure boundary condition for the thin-walled pipe solution: the pressure $p$ acting at the radius $R_m$. For this, the thin-walled solution and the thick-walled solution are required to produce the same displacements at $R_m$ (Appendix II). After neglecting higher
terms, this gives the equilibrium equation for the pressures needed for solving Equation (6-37)

\[ p(r = R_m,t) = \frac{R}{R_m} q(r = R,t) \]  

(6-38)

Equations (6-4), (6-5), (6-6), (6-38) and (6-35) for \( n = 2 \) are then substituted into Equation (6-37) and resolved with respect to the constant pressure \( p_n \), which, after being substituted back into pipe deformation Equation (6-35) for \( n = 2 \), produces for creep conditions (\( \sigma_1 = \text{const} \))

\[ u^\varphi(\theta,t) = -\frac{2R(1-K_n)\cos(2\theta)\sigma_1}{E_\varphi + aE_p} \]  

(6-39)

where

\[ \alpha = \frac{3(5 - \nu_1)}{R_m^3}; \quad E_p = \frac{1}{E_0} + \frac{1}{E_t} \left( 1 - e^{-\frac{\varepsilon_1}{\eta_0}} \right) + \frac{1}{\eta_0} \]  

(6-40)

Next, Equations (6-4), (6-5), (6-6), (6-38) and (6-36) for \( n = 2 \) are substituted into Equation (6-37) and resolved with respect to the constant pressure change \( \dot{p}_n \), which, after being substituted back into pipe deformation Equation (6-36) for \( n = 2 \), produces for constant rate conditions (\( \dot{\sigma}_1 = \text{const} \))

\[ u^\varphi(\theta,t) = -\frac{2R(1-K_n)\cos(2\theta)\dot{\sigma}_1 t}{E_\varphi + aE_p} \]  

(6-41)

where

\[ \alpha = \frac{3(5 - \nu_1)}{R_m^3}; \quad E_p = \frac{\eta_1}{E_1} \left( e^{-\frac{\varepsilon_1}{\eta_0}} - 1 \right) + \frac{t}{2\eta_0} \left( \frac{1}{E_0} + \frac{1}{E_1} \right) t \]  

(6-42)

Equations (6-39) and (6-41) can be used for interpretation of the IDM measurements by adopting the definition of the ovalization increment from Equation (6-2). Indeed, if the zero measurement is taken before the onset of loading, the major principal lateral pressure increment depends on the measured change in ovalization value by way of
\[ \Delta \sigma_1(t_n) = \frac{E_s + aE_p(t_n)}{8(1-K_0)\cos(2\delta)} \Delta \Omega(t_n) \quad (6-43) \]

where \( t_n \) is the time of the current measurement, with the time origin taken at the onset of loading; \( E_p \) and \( \alpha \) are given by Equations (6-40) and (6-42), for creep and constant rate conditions, respectively; \( \delta \) is the angle between the direction of the inclinometer pipe channels and the direction of principal stresses (Figure 6-2).

In case the onset of loading takes place before the zero measurement, Equation (6-43) evolves into the following two equations. For creep conditions, the pressure increment is obtained from the change in ovalization value measured over the time period \((t_0, t_n)\):

\[ \Delta \sigma_1(t_0, t_n) = \frac{\Delta \Omega(t_0, t_n)}{8(1-K_0)\cos(2\delta)} \left(\frac{1}{1 - \frac{t_n - t_0}{t_0}} \right) \quad (6-44) \]

where \( E_p \) and \( \alpha \) are given by Equation (6-40) with the time of the zero measurement \( t_0 \) and the time of the current measurement \( t_n \) measured from the onset of loading.

For constant rate conditions, the pressure increment is given by

\[ \Delta \sigma_1(t_0, t_n) = \frac{\Delta \Omega(t_0, t_n)}{8(1-K_0)\cos(2\delta)} \left(\frac{t_n-t_0}{E_s + aE_p(t_n) - E_s + aE_p(t_0)}\right) \quad (6-45) \]

where \( E_p \) and \( \alpha \) are defined from Equation (6-42).

The second principal lateral pressure increment is then given by

\[ \Delta \sigma_2 = K_0 \Delta \sigma_1 \quad (6-46) \]

Equations (6-43)-(6-46) represent analytical formulas allowing for back-calculation of the principal stress increments from the measured ovalization values of the inclinometer pipe cross-section.
6.5 Validation of the analytical solutions

6.5.1 Validation of the solution for the pipe

The analytical solution derived above for the pipe can be validated against laboratory tests on pipe samples performed in a simple test setup (Figure 6-6) by applying two equivalent opposing forces to the pipe in the transversal direction (see section 3.4).

![Figure 6-6: Laboratory tests performed on pipe samples: a) Test setup, b) Schematic layout of the test setup.](image)

Loading by two opposite forces \( P(t) \) acting at \( \theta = 0 \) and \( \theta = \pi \) can be presented by way of Fourier series

\[
p(\theta,t) = -\frac{P(t)}{\pi R_m} - \frac{2P(t)}{\pi R_m} \sum_{m=1}^{\infty} \cos(2n\theta)
\]

(6-47)

where the first term (hydrostatic pressure) is neglected due to its minor influence on the deformations. After substitution of Equation (6-47) into Equations (6-35) and (6-36), the solution is given by

\[
u(\theta,t) = -\frac{2PR_m^3}{\pi I} \left( \frac{1}{E_0} + \frac{1}{E_1} \left( 1 - e^{-\frac{E_0}{\eta_0}} \right) + \frac{1}{\eta_0} \right) \sum_{m=1}^{\infty} \frac{\cos(2n\theta)}{(2n)^2 - 1}
\]

(6-48)

for creep conditions with constant force \( P \), and
\[
    u(\theta,t) = -\frac{2\dot{P}R_m^3}{\pi l} \left( \frac{\eta}{E_0} - 1 \right) + \frac{1}{2\eta_0} t^2 + \left( \frac{1}{E_0} \frac{1}{E_1} \right) \sum_{n=1}^{\infty} \cos(2n\theta) \left( \frac{2n\theta}{(2n)^2 - 1} \right)^2
\]

(6-49)

for constant rate conditions with constant change in force \( \dot{P} \).

The inclinometer pipes installed in the landslide areas are assumed to have the same properties as the material of pipe A tested and described in chapter 3. Parameters of the Burgers model were obtained in independent laboratory tests with the pipe compressed in the longitudinal direction. In order to model the transversal behavior, the stiffness anisotropy in transversal direction has to be taken into account. This was achieved by reducing the elastic components of the Burgers model by a factor determined from back-analysis of transversal creep tests. This factor was found to be constant for different pipes. Resulting parameters for transversal modeling are presented in Table 6-1.

Table 6-1: Parameters of the four-parameter Burgers model for transversal loading of the pipe.

<table>
<thead>
<tr>
<th>parameter</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_0 )</td>
<td>2200 MPa</td>
</tr>
<tr>
<td>( \eta_0 )</td>
<td>( 3.0 \times 10^6 ) MPa( \times )day</td>
</tr>
<tr>
<td>( E_1 )</td>
<td>21120 MPa</td>
</tr>
<tr>
<td>( \eta_1 )</td>
<td>( 5.0 \times 10^4 ) MPa( \times )day</td>
</tr>
</tbody>
</table>

The analytical solutions, Equations (6-48) and (6-49), were validated against results of lab tests, where a sample of the pipe was loaded in the transversal direction by two opposite forces, in both the creep and constant rate modes (Figure 6-6b). In order to assess the effect of the inclinometer wheel channels, tests were performed on pipe samples at different axes rotation angles and finite element analysis was used for test interpretation (see section 3.4). These tests allowed for successful validation of the viscoelastic model for the true geometry of the channels. Because at the rotation of 45° the stiffness reduction caused by the channels is very small, the behavior of the pipe is well described by the analytical solution, allowing for back-calculation of the parameters of the viscoelastic model.
The measured pipe deformations are compared with the analytical solutions provided by Equations (6-48) and (6-49) using parameters in Table 6-1 (Figure 6-7). Figure 6-7a shows the comparison between measured and calculated deformations for a creep test lasting 10 days. Figure 6-7b shows deformations (normalized by the load rate) for four different tests at four different loading rates within four orders of magnitude. The longest test duration is 4 days. As can be seen, both analytical solutions provide remarkably good visual fit to the experimental data and their finite element simulations.

For other types of the inclinometer casing, parameters of the Burgers model can be found from the longitudinal and transversal compression, creep and constant rate lab tests similar to those presented in Figure 6-6 and Figure 6-7.

6.5.2 Validation of the combined solution

Validation of the combined solution was carried out against the results of numerical simulations, which in turn were validated against results of the full-scale laboratory tests performed in a 200 cm tall calibration chamber (IDM box) with a horizontal cross-section of 40 cm by 40 cm (see section 4.3). This two-step procedure allowed for eliminating of the effect of the boundaries present in the experimental setup.
In the first step, a numerical finite-element solution for a 40 cm by 40 cm model (see section 4.5) was calibrated against short-term (see section 4.6) and long-term (see section 3.5) laboratory tests, where the lateral pressure $\sigma_1$ was applied using pressure membranes with constrained kinematic boundary conditions in perpendicular direction (see Figure 4-2a). At each loading step the soil was assumed linear elastic with its parameters obtained in laboratory tests. The viscoelastic four-parameter Burgers model with parameters from Table 6-1 was used for the material of the pipe, which was modeled using true geometry with channels. As can be seen from Figure 4-12a and Figure 3-13a, both for the short term and long-term loading, the finite-element model managed to reproduce the experimental behavior reasonably well.

In the second step, the same numerical model, as validated in the first step, was applied to a different set of boundary conditions, which were closer to those of the analytical solution and more appropriate for the landslide applications. In particular, in order to eliminate effects of the boundaries, dimensions of the horizontal cross-section were increased to 2 m by 2 m. Plane stress conditions were introduced in the vertical direction. The major principal stress, increasing at a constant rate, was applied to one set of the boundaries. On the second perpendicular set of the boundaries, displacements were kinematically constrained. Comparison between numerical and analytical solutions is shown in Figure 6-8, where the obtained change in ovalization value is normalized by the stress rate in kilopascals per year. Although the channels lower the moment of inertia locally, the analytical solution, Equations (6-43)-(6-46), can provide a good approximation to the numerical one for a broad range of different Young’s moduli of soil $E_s$ and different angles $\delta$ between the principal stresses and pipe channels (Figure 6-8), provided the real thickness of the pipe is reduced to an equivalent value $h_{red}$ (see Table 6-2).
Figure 6-8: Change in ovalization value at constant stress rate: comparison of the analytical solution with finite-element calculations (pipe 84) for a 2 × 2 m model.

Table 6-2: Geometric parameters of the two most common inclinometer pipes used in Switzerland.

<table>
<thead>
<tr>
<th>symbol</th>
<th>parameter description</th>
<th>pipe 84</th>
<th>pipe 71</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R$</td>
<td>outer radius of the pipe</td>
<td>42 mm</td>
<td>35.35 mm</td>
</tr>
<tr>
<td>$R_m$</td>
<td>mean radius of the pipe</td>
<td>38.925 mm</td>
<td>32.275 mm</td>
</tr>
<tr>
<td>$h_{red}$</td>
<td>reduced thickness</td>
<td>5.5 mm</td>
<td>5.25 mm</td>
</tr>
</tbody>
</table>

6.6 Application of the analytical solutions to back-calculation of earth pressures in the creeping landslides

Application of the analytical solutions, Equations (6-43)-(6-46), to back-calculation of increments of earth pressures in creeping landslides requires the knowledge of the soil stiffness and of the principal stresses at the time of the inclinometer pipe installation. These values introduce significant uncertainty into the proposed method, and the present section describes
procedures allowing for these uncertainties to be significantly reduced in the back-calculation of the earth pressure increments.

6.6.1 Incorporating in-situ strain measurements

The incremental soil stiffness can be defined from laboratory or field tests, but considering its stress dependency, in particular when the soil exhibits yielding, this requires the knowledge of the in-situ stress state, which is another source of uncertainty. A practical way to reduce these uncertainties is by measuring soil displacements in the vicinity of the inclinometer pipe and calculating the average lateral strain increments \( \Delta \varepsilon \). The major principal stress increment \( \Delta \sigma_1 \) is then expected to occur in the direction of the observed major strain increment (assumed to coincide with the direction of the incremental displacement vector)

\[
\Delta \sigma_1(t_0, t_n) = \frac{E_s}{1 - \nu_s} \Delta \varepsilon(t_0, t_n) \tag{6-50}
\]

Each of the Equations (6-43)-(6-45) has two unknowns: the pressure increment and the soil stiffness. Equation (6-50) has the same two unknowns and, being solved together with one of the Equations (6-43)-(6-45), appropriate for a particular case under study, would allow for back-calculation of both the principal stress increments and the tangent stiffness of the soil by way of measured ovalization and average lateral strain.

6.6.2 Assumption of the isotropic initial stress state

Initial stress state after installation of the inclinometer pipe causes creep of the pipe and effects its deformations as described by Equations (6-39) and (6-40), even if no further changes in the earth pressures take place. When the major principal stress does experience changes (at the constant rate), pipe deformations are described by superposition of the solutions (6-39) and (6-41). Either way, the solution requires the knowledge of both principal stresses at the time of installation, which is a source of uncertainty.

This uncertainty can be significantly reduced by using the fact that the drilling of the bore-hole, installation and grouting of the pipe change the initial stress state around the pipe from anisotropic towards the isotropic state. The isotropic stress state does not contribute to the pipe ovalization, therefore the creep component of the pipe ovalization can be neglected, and the only solution to be used for calculating pipe deformations is given by Equations (6-41)
and (6-42), resulting in Equation (6-45), which allows for back-calculation of the major principal stress increment at any point in time, with the time origin taken at the moment of the completion of the pipe installation. The stiffness of PVC in short-term loading is found to be less than 2% higher at 8° compared to 20° (Domininghaus et al., 2008). Therefore, the influence of the temperature difference between the lab and the field on the viscoelastic model seems to be relatively small compared to other uncertainties of the presented approach.

The system of Equations (6-45) and (6-50) produces a quadratic equation for the soil stiffness, which has only one positive root. This solution is used in analysis of the three case studies presented in the following sections.

6.7 Case study: St Moritz-Brattas landslide, Switzerland

6.7.1 Description

The St Moritz slide is 1500 m long, 600–800 m wide and with average inclination of about 20° (Schlüchter, 1988). This landslide is constrained by the rock outcrop at the bottom of the sliding layer along the Via Maistra (Figure 6-9), which can be clearly identified from the contours of yearly horizontal displacements. The displacement rates increase uphill from the rock outcrop, while the field pressures in the sliding layer are expected to increase towards the toe of the slope.

A one-dimensional mechanical model considering the propagation of a zone of intense shearing along the slip surface was proposed for the St Moritz landslide by Puzrin and Schmid (2011). For validation of the model IDM measurements (inclinometer pipe 601) were taken close to the bottom boundary of the landslide (Figure 6-9). Additional IDM measurements (inclinometer pipe 702) were performed further away from the bottom boundary, but closer to the side boundary of the landslide.

Long-term stability of the landslide is of the greatest concern for the St Moritz community. Information about the earth pressure changes is essential, because the high pressures in the compression zone are suspected to be the cause of the landslide accelerations.
6.7.2 IDM pressure measurements

IDM measurements were taken in the borehole 601 from 2008 to 2012 (Figure 6-9). Inclinometer measurements could be performed in the same borehole from 2008 to 2011, after which the inclinometer probe could not pass through the shear zone any longer. The shear zone is observed at the depth of 16.5 m being less than 0.5 m thick. The average annual displacement at the ground surface is 22 mm. The average annual strain increment $\Delta \varepsilon$ is obtained between the borehole and the landslide boundary of zero displacement (Table 6-3).

The major principal stress increments are expected to occur in the direction of the observed compressive strain increments, that is, downslope. The change in ovalization of the pipe measured by IDM confirms the increase in pressures over almost the whole depth and measuring period (Figure 6-10). The inner diameter of the pipe in the direction of the slope became smaller with respect to the perpendicular diameter. Hence the ovalization value is increasing with time. The averaged change of the ovalization value $\Delta \Omega$ in time is calculated as 0.014 %/yr for the inclinometer pipe elements between 7.0 m and 12.1 m (the elements close to the ground surface or influenced by the shear zone were not considered).
The corresponding pressure increments and the tangent stiffness of soil can be back-calculated from Equations (6-45) and (6-50), after substituting Equation (6-42), and accounting for the angle $\delta$ between the pipe axes and the principal stress directions (Table 6-3).

The pressure increment $\Delta \sigma_1$ of 0.2 kPa/yr compares well with the range of 0.11–1.1 kPa/yr for the years 2008–2010 predicted by the analytical model (Puzrin and Schmid, 2011), which concluded that the earth pressure at the bottom reached the passive pressure around 1993. Comparing the very small back-calculated deformation modulus of the soil $E_x$ of 0.3 MPa with the elastic modulus of 17.9 MPa based on Marchetti dilatometer tests (Puzrin et al., 2008; Puzrin and Schmid, 2011) confirms that the soil exhibits secondary compression and post-failure hardening.

Figure 6-10: Pipe deformation of inclinometer 601 measured by IDM, St Moritz.

IDM measurements were also taken in borehole 702 (Figure 6-11) in the inclinometer pipe, which was sheared in 1997. The location of the slip surface at a depth of 21 m and the direc-
tion of the displacement vector are known from inclinometer measurements performed from 1995 to 1997. The current displacement rate of 33 mm/yr is obtained from nearby geodetic measurements. Although the strain rate is similar to that of inclinometer 601, the change in ovalization value $\Delta \Omega$ is considerably higher in the upper part of the borehole (Figure 6-11 and Table 6-3). Pipe elements from 1.8 m to 6.8 m were taken for averaging. In spite of the larger change in ovalization, the back-calculated deformation modulus $E_s$ of 0.6 MPa and the pressure increment of 0.4 kPa/yr are comparable to those in the borehole 601. This again confirms the soil failure, strain hardening and secondary compression in the compression zone, while the larger change in ovalization in the pipe of the borehole 702 can be explained by the fact that it was installed 16 years earlier than the pipe in the borehole 601, resulting in the lower viscoelastic stiffness of the pipe.

![Figure 6-11: Pipe deformation of inclinometer 702 measured by IDM, St Moritz.](image-url)
Table 6-3: Back-calculation of pressures in the St Moritz slide: parameters of each inclinometer pipe.

<table>
<thead>
<tr>
<th>parameter description</th>
<th>601</th>
<th>702</th>
<th>derivation</th>
</tr>
</thead>
<tbody>
<tr>
<td>year of completion</td>
<td>2006</td>
<td>1990</td>
<td></td>
</tr>
<tr>
<td>displacement rate</td>
<td>22 mm/yr</td>
<td>33 mm/yr</td>
<td>measured</td>
</tr>
<tr>
<td>distance to the boundary</td>
<td>40 m</td>
<td>56 m</td>
<td>measured</td>
</tr>
<tr>
<td>strain increment: $\Delta \epsilon$</td>
<td>0.056 %/yr</td>
<td>0.059 %/yr</td>
<td>measured</td>
</tr>
<tr>
<td>change in ovalization value: $\Delta \Omega$</td>
<td>0.014 %/yr</td>
<td>0.088 %/yr</td>
<td>measured</td>
</tr>
<tr>
<td>angle of the pipe: $\delta$</td>
<td>22.2°</td>
<td>19.3°</td>
<td>measured</td>
</tr>
<tr>
<td>deformation modulus of the soil: $E_s$</td>
<td>0.3 MPa</td>
<td>0.6 MPa</td>
<td>back-calculated</td>
</tr>
<tr>
<td>pressure increment: $\Delta \sigma_1$</td>
<td>0.2 kPa/yr</td>
<td>0.4 kPa/yr</td>
<td>back-calculated</td>
</tr>
</tbody>
</table>

6.8  Case study: Braunwald landslide, Switzerland

6.8.1  Description

The village of Braunwald (Figure 6-12) is built on a natural terrace with an area of 4 km$^2$, creeping towards the cliff facing the valley (Schindler, 1982; Schindler and Rageth, 1990), resulting at the bottom in an active pressure boundary condition. The slip surface is inclined by only 10–12° and mostly formed within a several-meters-thick clay layer of weathered schist above the moraine (Figure 6-13).

The upper part of the terrace is moving as one block, while the sliding mass in the lower part of the landslide is moving faster towards the cliffs (Figure 6-14). Locally, high spontaneous acceleration can be observed in the area close to the cliffs. Smaller local slides of several 10,000 m$^3$ are often triggered by heavy rainfall and fall over the cliffs (Bollinger et al., 2004). The authorities expect such larger events to happen every 15–20 years. Some parts of the village are located close to the cliffs.

The observed gradient in the displacement rate suggests that earth pressures in the sliding layer are expected to decrease towards the boundary. IDM measurements were performed in
two boreholes (inclinometers KB2 and KB6) in the lower part of the landslide, where most of the buildings and infrastructure are located.

Figure 6-12: The sliding terrace of Braunwald (after Braunwald-Klausenpass Tourismus).

Figure 6-13: The geology of the Braunwald slide (after Schindler, 1982).
6.8.2 IDM pressure measurements

Borehole KB2 is located next to the cable car station in an area of larger movements close to the cliff (Figure 6-14). The inclinometer pipe in the borehole has been sheared at the slip surface 10.5 m below the ground surface before 2009. The current ground movement of 34 mm/yr is known from a permanent GPS station installed at the station of the cable car. The strain rate of −0.034 %/yr has been derived between KB2 and a nearby inclinometer located in the same slope profile for the period of 2002–2006.

Borehole KB6 is located in a different area further up the slope, where the observed displacements are smaller (Figure 6-14). The inclinometer pipe has not yet been sheared. Nevertheless, IDM measurements have only been possible in the top 27 m. Inclinometer measurements show a surface displacement of 8 mm/yr. The strain rate of −0.011 %/yr for the period of 2009–2012 is obtained by considering another inclinometer further downhill in the same profile.
profile. Constant gradient in the displacement rate is assumed over the distance between the inclinometers.

IDM measurements were performed in both inclinometer pipes in the period between 2009 and 2012 (Figure 6-15 and Figure 6-16). The pipe elements in KB6 above the depth of 13.4 m were ignored, owing to possible local shallow slides in the area. Negative change in ovalization value $\Delta \Omega$ clearly indicates extension in the slope direction. The pipe diameter in the direction of the slope becomes larger with time with respect to the perpendicular diameter. The observed changes in ovalization value $\Delta \Omega$ are similar in both boreholes (Table 6-4), whereas the strain rates are different. The pressure increments and the deformation moduli of the soil are back-calculated solving the set of Equations (6-45) and (6-50), showing, as expected, a lower tangent stiffness and pressure drop at KB2, closer to the cliff where the soil has failed in the active mode (Table 6-4). Away from the cliff, at KB6, the soil experiences less yielding, but a very low back-calculated soil stiffness indicates its possible rate dependency.
Figure 6-16: Pipe deformation of inclinometer KB6 measured by IDM, Braunwald.

Table 6-4: Back-calculation of pressures in the Braunwald slide: parameters of each inclinometer pipe.

<table>
<thead>
<tr>
<th>parameter description</th>
<th>KB2</th>
<th>KB6</th>
<th>derivation</th>
</tr>
</thead>
<tbody>
<tr>
<td>year of completion</td>
<td>2002</td>
<td>2002</td>
<td></td>
</tr>
<tr>
<td>displacement rate</td>
<td>34 mm/yr</td>
<td>8 mm/yr</td>
<td>measured</td>
</tr>
<tr>
<td>distance to the cliff</td>
<td>180 m</td>
<td>360 m</td>
<td>measured</td>
</tr>
<tr>
<td>strain increment: $\Delta \varepsilon$</td>
<td>$-0.034 %$/yr</td>
<td>$-0.011 %$/yr</td>
<td>measured</td>
</tr>
<tr>
<td>change in ovalization value: $\Delta \Omega$</td>
<td>$-0.038 %$/yr</td>
<td>$-0.042 %$/yr</td>
<td>measured</td>
</tr>
<tr>
<td>angle of the pipe: $\delta$</td>
<td>2.4°</td>
<td>5.6°</td>
<td>measured</td>
</tr>
<tr>
<td>deformation modulus of the soil: $E_s$</td>
<td>0.6 MPa</td>
<td>4.3 MPa</td>
<td>back-calculated</td>
</tr>
<tr>
<td>pressure increment: $\Delta \sigma_1$</td>
<td>$-0.2$ kPa/yr</td>
<td>$-0.5$ kPa/yr</td>
<td>back-calculated</td>
</tr>
</tbody>
</table>
6.9 Case study: Ganter landslide, Switzerland

6.9.1 Description

The Simplon Pass connects Switzerland to Italy via the Alps. In 1980 a bridge was built crossing the valley 140 m over the Ganter river (Menn and Rigendinger, 1979). On the left bank of the river, the bridge is founded on a creeping landslide. The slope is inclined by 24° and covers an area of about 0.4 km². The ground surface next to the new bridge is moving at a rather uniform velocity of 4.2 mm/yr, as measured in the period from 1994 to 2004 (Puzrin and Schmid, 2012). Inclinometer measurements show displacements decreasing almost linearly with depth within the sliding body (Ritz, 1992). The soil consists of slope debris, moraine and stones of schist (Schaerer, 1975).

The landslide is creeping towards the Ganter river and is constrained by the rock outcrop at the opposite riverbank. Puzrin and Schmid (2012) have concluded that the soil close to the riverbed has reached passive failure. IDM measurements were performed in two boreholes (inclinometer 702 and 704) next to the river in the zone where the soil has possibly failed (Figure 6-17).

![Figure 6-17: The Ganter slide with displacement rates and location of the boreholes.](image)

6.9.2 IDM pressure measurements

Boreholes 702 and 704 are located at a distance of 17 m and 28 m from the opposite riverbank, respectively. IDM and inclinometer readings were taken in the period from 2010 to
2012. The displacement rate at the surface is observed to be 1.1 mm/yr in the borehole 702. The displacements are decreasing with depth; no clear shear surface could be identified.

![Figure 6-18: Pipe deformation of inclinometer 702 measured by IDM, Ganter.](image)

An averaged strain rate of 0.006 %/yr is obtained both between borehole 702 and the boundary and between 702 and the closest uphill geodetical measurement point (Table 6-5). For the borehole 704, the strain rate is assumed to be the same. In both boreholes, the change in ovalization value $\Delta \Omega$ is observed to be very small averaged over the depth of the boreholes (Figure 6-18 and Figure 6-19). The measured absolute deformations of the pipe of 0.5 micrometers for the period are within the precision of the IDM device of $\pm 2$ micrometers. Therefore in practice the change in ovalization value can be considered to be very close to zero, resulting from Equations (6-45) and (6-50) in zero values of both the pressure increment $\Delta \sigma_1$ and the deformation modulus of the soil $E_s$, confirming the passive failure of the soil close to the bottom boundary of the landslide (for details see Puzrin and Schmid, 2012).
Figure 6-19: Pipe deformation of inclinometer 704 measured by IDM, Ganter.

Table 6-5: Back-calculation of pressures in the Ganter slide: parameters of each inclinometer pipe.

<table>
<thead>
<tr>
<th>parameter description</th>
<th>702</th>
<th>704</th>
<th>derivation</th>
</tr>
</thead>
<tbody>
<tr>
<td>year of completion</td>
<td>2007</td>
<td>2007</td>
<td></td>
</tr>
<tr>
<td>displacement rate</td>
<td>1.1 mm/yr</td>
<td>-</td>
<td>measured</td>
</tr>
<tr>
<td>distance to the boundary</td>
<td>17 m</td>
<td>28 m</td>
<td>measured</td>
</tr>
<tr>
<td>strain increment: $\Delta \varepsilon$</td>
<td>0.006 %/yr</td>
<td>0.006 %/yr</td>
<td>measured</td>
</tr>
<tr>
<td>change in ovalization value: $\Delta \Omega$</td>
<td>-0.0007 %/yr</td>
<td>-0.0007 %/yr</td>
<td>measured</td>
</tr>
<tr>
<td>angle of the pipe: $\delta$</td>
<td>13.8°</td>
<td>-</td>
<td>measured</td>
</tr>
<tr>
<td>deformation modulus of the soil: $E_s$</td>
<td>small</td>
<td>small</td>
<td>back-calculated</td>
</tr>
<tr>
<td>pressure increment: $\Delta \sigma_1$</td>
<td>small</td>
<td>small</td>
<td>back-calculated</td>
</tr>
</tbody>
</table>
6.10 Conclusions

The chapter derives analytical solutions for the deformation of a viscoelastic pipe in elastic soil under far-field principal stress increments. These solutions are validated in laboratory tests and numerical analysis and provide the basis for the back-calculation of earth pressure increments from measured changes in shape of an inclinometer pipe cross-section using IDM technology. The procedure has then been applied to back-calculating earth pressure changes in three creeping landslides in Switzerland, which are similar in size but have a range of different boundary conditions and displacement rate fields

a) the St Moritz-Brattas landslide, which is slowing downhill towards a rock outcrop at the bottom;
b) the Braunwald landslide, which is accelerating downhill towards a vertical rock wall falling into a valley;
c) the Ganter landslide, which is moving uniformly downhill towards a river bed.

In these applications, reasonably small pressure increments could be reliably back-calculated by applying the obtained analytical solution which takes the time-dependency of the pipe material properties into account. These pressure increments allowed for identification of compression and extension zones in the landslides under study, confirming the previously developed models of the landslide mechanisms.

Combining the IDM measurements with measurements of relative displacements, the tangent stiffness of the soil could also be clearly identified. The obtained in-situ deformation modulus for the incremental behavior indicates yielding, failure and possibly secondary compression of soil, confirming the previous landslide analysis and suggesting that IDM pressure measurements could serve as a reliable tool for analysis and monitoring of creeping landslides.

ACKNOWLEDGEMENTS

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tially supported by the ASTRA/VSS (grant no. VSS 2010/502) ‘Landslide-Road-Interaction: Applications’.

REFERENCES


Appendix I: viscoelastic constitutive equations in terms of pipe forces

Derivation of the constitutive relationships for a thin pipe in terms of its forces and moments uses compatibility considerations (Bouma, 1993) to obtain the strain distribution across the thickness of the pipe (Figure 6-4b)

\[ \varepsilon(z) = \frac{R_m}{R_m + z}(\varepsilon_0 + \chi z) \]  

(6-51)

The normal force and the moment are defined as

\[ N = \int_{-h/2}^{h/2} \sigma(z) dz \]  

(6-52)

\[ M = \int_{-h/2}^{h/2} \sigma(z) zdz \]  

(6-53)

where \( h \) is the thickness of the pipe.

The constitutive equation in terms of circumferential stresses and strains follows from the Burgers model

\[ \sigma + p_1 \dot{\sigma} + p_2 \ddot{\sigma} = q_1 \dot{\varepsilon} + q_2 \ddot{\varepsilon} \]  

(6-54)

Integrating both sides of Equation (6-54) and substituting Equations (6-52) and (6-53) into it gives

\[ N + p_1 \dot{N} + p_2 \ddot{N} = q_1 \int_{-h/2}^{h/2} \dot{\varepsilon}(z) dz + q_2 \int_{-h/2}^{h/2} \ddot{\varepsilon}(z) dz \]  

(6-55)

\[ M + p_1 \dot{M} + p_2 \ddot{M} = q_1 \int_{-h/2}^{h/2} \dot{\varepsilon}(z) zdz + q_2 \int_{-h/2}^{h/2} \ddot{\varepsilon}(z) zdz \]  

(6-56)

Next, expressing Equation (6-51) as Taylor series (Bouma, 1993), substituting it into Equations (6-55) and (6-56), integrating and neglecting terms of higher order than \( h^4 \) (thin pipe), results in the following constitutive relationships for a thin pipe in terms of the forces and moments
\[ N + p_1 \ddot{N} + p_2 \ddot{N} = q_1 \left( A \ddot{e}_0 - \frac{I}{R_m} \ddot{x} \right) + q_2 \left( A \ddot{e}_0 - \frac{I}{R_m} \ddot{x} \right) \] (6-57)

\[ M + p_1 \dddot{M} + p_2 \dddot{M} = q_1 \left( -\frac{I}{R_m} \dddot{e}_0 + I \dddot{x} \right) + q_2 \left( -\frac{I}{R_m} \dddot{e}_0 - I \dddot{x} \right) \] (6-58)

where \( A = 1 \times h \) is the area and \( I = 1 \times h^3 / 12 \) is the moment of inertia of the pipe cross-section.

**Appendix II: equivalent pressure acting on a thin-walled pipe**

A thin-walled pipe approximation requires a correspondence principle transforming the pressure \( q \) acting at the outer surface of the pipe at \( r = R = R_m + h/2 \) into the pressure \( p \) acting on the thin ring of the median radius \( r = R_m \) (Figure 6-20). This principle is based on the condition that pressures \( q \) and \( p \) should produce the same radial displacements at the same \( r = R_m \) for a thick-walled and a thin-walled pipe, respectively.

First, the thick-walled pipe with thickness \( h \) is considered to be under pressure \(-q \cos(2\theta)\) at the outer radius \( r = R = R_m + h/2 \).

The solution for the radial displacement at the center of the cross-section is given by the Michell solution (Barber, 2002) as
Chapter 6: Inclinometer pressure measurements in creeping landslides

\[ u_{\text{thick}}(r = R_m, \theta) = -\frac{4R_m^4q\cos(2\theta)}{3Eh^3}(1 + \frac{h}{2R_m^2} + \frac{3h^2}{4R_m^2} + \frac{3h^3}{8R_m^2} + \frac{h^4}{4R_m^2} + \frac{5h^5}{16R_m^2} - \frac{29h^4}{128R_m^4} + \frac{5h^5}{128R_m^4} - \frac{9h^6}{256R_m^4} + \frac{3h^7}{256R_m^4} + \frac{13h^8}{512R_m^4} + \frac{h^9}{512R_m^4} + \frac{5h^{10}}{512R_m^4} + \frac{h^{11}}{512R_m^4} + \frac{h^{12}}{256R_m^4} + \frac{9h^{13}}{2048R_m^4} + \frac{3h^{14}}{2048R_m^4} - \frac{h^{15}}{4096R_m^4} - \frac{h^{16}}{4096R_m^4} - \frac{h^{17}}{8192R_m^{10}} - \frac{h^{18}}{8192R_m^{10}}) \]  

\[ u_{\text{thin}}(r = R_m, \theta, p) = u_{\text{thick}}(r = R_m, \theta, q) \]  

(6-60)

The radial displacement of the thin pipe is given by Equation (6-24) for \( n = 2 \) and \( l = h^3/12 \) as

\[ u_{\text{thin}}(r = R_m, \theta) = -\frac{4R_m^4}{3Eh^3}p\cos(2\theta) \]  

(6-61)

Substituting Equations (6-59) and (6-61) into Equation (6-60) and neglecting the higher order terms gives the ratio \( p/q \)

\[ p(r = R_m)/q(r = R) \approx 1 + \frac{h}{2R_m} = \frac{R}{R_m} \]  

(6-62)

which provides the necessary equivalent pressure acting on a thin pipe

\[ p(r = R_m) = \frac{R}{R_m}q(r = R) \]  

(6-63)
NOTATION

$A$ area of the cross-section of the pipe

$D$ inner diameter of the inclinometer pipe

$D_A$ inner diameter of the pipe within the channels in direction A

$D_{A,0}$ zero measurement of the inner diameter of the pipe in direction A

$D_{A,n}$ further measurement of the inner diameter of the pipe in direction A

$D_B$ inner diameter of the pipe within the channels in direction B

$D_{B,0}$ zero measurement of the inner diameter of the pipe in direction B

$D_{B,n}$ further measurement of the inner diameter of the pipe in direction B

$E$ Young’s modulus of the pipe

$\hat{E}$ transformed modulus of the pipe

$E_p$ time-dependent pipe stiffness parameter

$E_s$ Young’s modulus of the soil

$E_0$ elastic parameter of the Burgers model

$E_1$ elastic parameter of the Burgers model

$h$ thickness of the pipe

$h_{red}$ reduced thickness of the inclinometer pipe

$I$ moment of inertia of the cross-section of the pipe

$K_0$ (incremental) earth pressure coefficient within the horizontal plane

$M$ bending moment in the pipe

$\hat{M}$ transformed moment

$N$ normal force in the pipe

$\hat{N}$ transformed normal force

$P$ force acting on the pipe

$p$ pressure acting on the pipe

$\hat{p}$ transformed pressure
\[ p_1 \quad \text{parameter of the viscoelastic Burgers model} \]
\[ p_2 \quad \text{parameter of the viscoelastic Burgers model} \]
\[ q \quad \text{pressure acting on the cavity} \]
\[ q_1 \quad \text{parameter of the viscoelastic Burgers model} \]
\[ q_2 \quad \text{parameter of the viscoelastic Burgers model} \]
\[ R \quad \text{nominal outer radius of the inclinometer pipe, radius of the cavity} \]
\[ R_m \quad \text{mean radius of the pipe} \]
\[ r \quad \text{radial coordinate} \]
\[ s \quad \text{Laplace variable} \]
\[ t \quad \text{time} \]
\[ t_0 \quad \text{time of the zero measurement} \]
\[ t_n \quad \text{time of the current measurement} \]
\[ u \quad \text{radial displacement} \]
\[ \hat{u} \quad \text{transformed radial displacement} \]
\[ u_{thick} \quad \text{radial displacement of the thick-walled pipe} \]
\[ u_{thin} \quad \text{radial displacement of the equivalent thin-walled pipe} \]
\[ u^p \quad \text{radial deformation of the pipe due to external pressure} \]
\[ u^q \quad \text{radial deformation of the soil due to pressure at the inclusion} \]
\[ u^{q_1} \quad \text{radial deformation of the soil due to lateral earth pressure} \]
\[ V \quad \text{shear force in the pipe} \]
\[ v \quad \text{tangential displacement} \]
\[ \hat{v} \quad \text{transformed tangential displacement} \]
\[ z \quad \text{coordinate within the cross-section of the pipe} \]
\[ \alpha \quad \text{dimensionless coefficient} \]
\[ \gamma_{r\theta} \quad \text{shear strain} \]
\[ \Delta \varepsilon \quad \text{lateral principal strain increment} \]
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta \sigma_1$</td>
<td>major principal stress increment in the horizontal plane</td>
</tr>
<tr>
<td>$\Delta \sigma_2$</td>
<td>minor principal stress increment in the horizontal plane</td>
</tr>
<tr>
<td>$\Delta \Omega$</td>
<td>change in ovalization value</td>
</tr>
<tr>
<td>$\delta$</td>
<td>angle between the pipe channels and the direction of principal stresses</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>strain</td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td>lateral principal strain</td>
</tr>
<tr>
<td>$\varepsilon_0$</td>
<td>axial strain in the pipe</td>
</tr>
<tr>
<td>$\dot{\varepsilon}_0$</td>
<td>transformed axial strain</td>
</tr>
<tr>
<td>$\varepsilon_r$</td>
<td>radial strain</td>
</tr>
<tr>
<td>$\varepsilon_\theta$</td>
<td>tangential strain</td>
</tr>
<tr>
<td>$\eta_0$</td>
<td>viscous parameter of the Burgers model</td>
</tr>
<tr>
<td>$\eta_1$</td>
<td>viscous parameter of the Burgers model</td>
</tr>
<tr>
<td>$\theta$</td>
<td>circumferential coordinate</td>
</tr>
<tr>
<td>$\nu_s$</td>
<td>Poisson’s ratio of the soil</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>stress</td>
</tr>
<tr>
<td>$\sigma_1$</td>
<td>principal stress within the horizontal plane</td>
</tr>
<tr>
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<td>principal stress within the horizontal plane</td>
</tr>
<tr>
<td>$\sigma_r$</td>
<td>normal stress in radial direction</td>
</tr>
<tr>
<td>$\sigma_\theta$</td>
<td>normal stress in tangential direction</td>
</tr>
<tr>
<td>$\tau_{r\theta}$</td>
<td>shear stress</td>
</tr>
<tr>
<td>$\chi$</td>
<td>curvature in the pipe wall</td>
</tr>
<tr>
<td>$\dot{\chi}$</td>
<td>transformed curvature in the pipe wall</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>ovalization value</td>
</tr>
<tr>
<td>$\Omega_0$</td>
<td>zero measurement of the ovalization value</td>
</tr>
<tr>
<td>$\Omega_n$</td>
<td>further measurement of the ovalization value</td>
</tr>
</tbody>
</table>
7  Inclinodeformeter pressure measurements in a vicinity of excavation

7.1  Abstract

Inclinodeformeter (IDM) pressure measurements were carried out behind a retaining wall during an excavation test. The strut loads measured during the excavation appeared to be considerably larger (up to 2.5 times) than those expected in the design. A possible scenario suspected for the development of such large forces was overpressure of the soil behind the slurry wall due to excessive jet-grouting pressures. IDM measurements were used for validation and quantification of this phenomenon. Using numerical analysis, the lateral earth pressure changes and the soil stiffness were back-calculated based on IDM and local displacement measurements. The back-calculated soil stiffness was validated for each of the six soil layers against the in-situ measurements obtained by CPT and Marchetti dilatometer. The excessive strut loads could be explained by the back-calculated pressure changes. For the excavation test, lateral earth pressure changes obtained by IDM measurements helped to improve understanding of the retaining wall reaction to installation processes, provide an early detection of potential problems, and help to prevent a possible excavation collapse.
7.2 Introduction

Understanding the lateral earth pressure changes behind a retaining wall of an excavation is essential for the design of the wall. Especially when using observational method, the information about the earth pressures becomes critical. Nevertheless, measuring the earth pressure changes behind a wall is still a challenging problem, especially for existing walls, where devices cannot be attached to the wall during construction.

Experimental research on earth pressures acting on retaining walls has been extensively reported in the literature (Terzaghi, 1943; Rowe and Peaker, 1965; Tsagareli, 1965; Narain et al., 1969; Rehnman and Broms, 1972; Matsuo et al., 1978; Fukuoka et al., 1981; Sherif et al., 1984; Fang and Ishibashi, 1986; Fang et al., 1997; Tweedie et al., 1998). The common method was for discrete measurements to be made directly on the wall using contact earth pressure cells. Another method is provided by self-boring dilatometer tests next to a retaining wall (Clarke and Wroth, 1984). The IDM device allows for a different approach, which provides a profile of lateral earth pressure changes without requiring any additional infrastructure on the construction site other than a standard inclinometer pipe behind the wall.

IDM pressure measurements are carried out behind a retaining wall of a strut supported excavation. The change in dimensions of an inclinometer pipe is measured using the IDM device (see chapter 2). For each soil layer, a separate boundary value problem is solved for a horizontal cross-section. Using numerical analysis, the lateral earth pressure changes and the soil stiffness are back-calculated based on IDM and local displacement measurements.

The back-calculated soil stiffness is successfully validated for each soil layer against the in-situ measurements obtained by CPT and Marchetti dilatometer. The strut loads, which were independently measured to be considerably higher than expected in the design, can be explained with the back-calculated pressure changes behind the retaining wall. The IDM device is shown to be a reliable tool for the back-calculation of lateral earth pressure changes and stiffness of soil.
7.3 Experimental setup

7.3.1 Site and geology

The Swiss Federal Railways SBB have performed feasibility studies for a new underground railway station below the existing station in the city of Lucerne, Switzerland. A large-scale excavation test was performed close to the existing station in order to obtain soil properties and to test construction processes.

The railway station is located next to lake Lucerne which affects the stratigraphy in the area. The bedrock, which is covered by moraine, can be found around 100 m below ground level. The lake was formed by the retreating glacier 15,500 years ago (Keller et al., 1995). The glacier remained for around 2,500 years, descending into today’s lake. The material brought by the retreating glacier into the lake settled to the bottom of the lake. In the process sediments were formed in layers consisting of either silt or fine sand.

7.3.2 Large-scale excavation test

A large-scale excavation test was performed to investigate construction options in this type of soil. The excavation will not be part of the future underground station. Slurry walls were built forming a box enclosing an area of 12 m by 4 m (Figure 7-1), to a depth of 24 m below ground level. At the bottom, the box was sealed by a 5 m thick jet-grouted slab from 18 m to 23 m below ground level (Figure 7-2). In the upper part of the box, jet-grouting was also performed locally in order to gain experience in jet-grouting in this kind of soil. Afterwards the soil inside the box was excavated in four stages to 18 m below ground level. Each of the first three excavation stages was supported by a level of struts made out of steel frames.
Figure 7-1: Plan view of the large-scale excavation test.

Strut forces were measured in the central strut of the steel frame during the excavation phase. The wall deformations were measured in the cross-section A-A (see Figure 7-1) on both sides of the central strut using a Trivec system (Koeppel et al., 1983). Inclinometer and IDM measurements were performed in the same cross-section 1.5 m behind the wall using an inclinometer pipe of 84 mm outer diameter installed to a depth of 35 m in the borehole M3 (Figure 7-1 and Figure 7-2). The borehole M3 was drilled with a diameter of 203 mm diameter in its upper part and 168 mm in the lower part (Figure 7-4). The borehole was refilled with a grout mixture made of water, cement and bentonite.

The large-scale excavation test was complemented by an extensive site investigation campaign (Rabaiotti et al., 2014). Among other tests, CPT profiling and Marchetti dilatometer tests were performed at several locations. Drilling core samples were used for soil classification, consolidation and triaxial shear tests (Springman et al., 2012).
Figure 7-2: Cross-section of the large-scale excavation test.
7.3.3 Soil profile

The location of the different soil layers could clearly be identified in the CPT profile. According to USCS classification, the soil profile consists of alternating layers of fine sand, silt and clay (Figure 7-3), with a few very thin layers containing peat at the surface, summarized into one stratigraphic layer, followed by a relatively thick layer of gravel. Soil properties obtained from field tests correlate well with the layers classification. The constrained modulus $M$, the undrained shear strength $s_u$ for the fine soils and the angle of internal friction $\varphi$ for the coarse soils are provided for each layer (Figure 7-3). These are obtained from the Marchetti dilatometer tests and the CPT tests. The coarse layers have considerably higher stiffness than the silt and clay layers.
IDM measurements were performed on the inclinometer pipe installed in borehole M3. The change in the cross-sectional shape of the inclinometer pipe was measured for each construction stage: the inner diameter of the pipe $D_A$ was measured in the channels in the direction perpendicular to the slurry wall, the inner diameter $D_B$ - in the direction parallel to the slurry wall.
wall (Figure 7-4 and chapter 2). The ovalization value $\Omega$ is obtained by normalizing the difference of the two diameters by the outer radius of the pipe ($R = 42 \text{ mm}$):

$$\Omega = \frac{D_b - D_t}{R}$$  \hspace{1cm} (7-1)

IDM measurements were taken before and after the jet-grouting phase and after each excavation stage. The deformations of the pipe are obtained relative to the so-called zero-measurement. Hence, the deformation is described by the change in ovalization value $\Delta \Omega$ with respect to the zero-measurement:

$$\Delta \Omega_n = \Omega_n - \Omega_0$$  \hspace{1cm} (7-2)

The measurement taken before the jet-grouting started is considered to be the zero-measurement for the jet-grouting phase. The measurement taken when the jet-grouting was completed is considered to be the zero-measurement for the excavation phase.

The obtained profile of the change in ovalization value is averaged in depth in order to smooth the data. The data is averaged over 35 cm intervals, where each interval is treated as one sensor, with length equivalent to 2 times the circumference of the wheels of the probe in order to reduce the influence of their imperfection. This results in 8 sensors per each 3 m long casing element.

Figure 7-4: Plan view of the inclinometer pipe installed behind the wall for the IDM measurements.
7.4 Measurement results

7.4.1 Jet-grouting phase

As a result of the jet-grouting inside the concrete box the walls were pushed against the soil with cracks opening in the slurry wall, mainly at the corners of the box (Rabaiotti et al., 2014). The maximum outward displacement of the wall of 130 mm was observed in the cross-section A-A (Figure 7-1) at a depth of 12 m. The inclinometer in borehole M3 located 1.5 m behind the wall showed a maximum outward displacement of 106 mm at the same depth (Figure 7-5a). The soil around the wall was compressed in the lateral direction owing to the expansion of the concrete box. The average horizontal strain increment between the wall and the borehole M3 is calculated by dividing the measured difference in corresponding horizontal displacements by the distance of 1.5 m (Figure 7-5b). The maximum average lateral strain is 2.2%. Considering that significant yielding occurred at a shear strain $\varepsilon_s$ of around 0.5% in lateral undrained triaxial tests performed on samples taken from the silt and clay layers at different depth (Springman et al., 2012), it can be concluded that the calculated average strains have a considerable plastic component.

Figure 7-5: Jet-grouting phase: a) Inclinometer readings in borehole M3, b) Average lateral strain increment $\Delta\varepsilon_{h,ap}$ in between the retaining wall and the inclinometer in borehole M3.
The measured ovalization value of the inclinometer pipe reacted to the jet-grouting (Figure 7-6). The diameter in the direction perpendicular to the wall $D_A$ became smaller compared with the diameter parallel to the wall $D_B$. The pipe deformations indicate increasing pressures in the soil outside the wall due to the jet-grouting inside of the concrete box. The profile of the pipe deformations (Figure 7-6) does not correlate well with the profile of the average horizontal strain increments (Figure 7-5b), indicating that the stiffness of soil changes with depth, as could be expected from the site investigation. Indeed, the soil profile obtained by CPT testing correlates with the measured change in ovalization value. Large change in ovalization value (Figure 7-6) is observed within the stiff and coarse layers 2, 4, 6 and 8 (Figure 7-3). The poor correlation of the average strain increment with the pipe deformations may also be considered as another indication for soil yielding around the wall owing to the excessive jet-grouting.

Figure 7-6: Pipe deformation during the jet-grouting phase measured by IDM.
7.4.2 Excavation phase

During the excavation the walls moved inwards the box. The maximum inward displacement of 22 mm was observed at a depth of 11 m after the excavation was finished. The inclinometer in borehole M3 located 1.5 m behind the wall showed a maximum inward displacement of 18 mm (Figure 7-7a). Below a depth of 20 m, at the level of the jet-grouted slab, no lateral soil displacements were observed. Due to the excavation the soil around the wall experienced incremental horizontal extension with the maximum average strain increment of −0.4% between the wall and the borehole M3 (Figure 7-7b). Yielding was observed to start at a shear strain $\varepsilon_s$ of around −0.5% in lateral extension tests performed in a triaxial apparatus under undrained conditions (Springman et al., 2012). Therefore, no significant plastic strains are expected to have developed during the excavation phase.

The ovalization increment changed its sign in the excavation phase compared with the jet-grouting phase (Figure 7-8). The inner diameter of the pipe in the direction of the excavation $D_A$ became larger with respect to the perpendicular inner diameter $D_B$. The pipe deformations react to decreasing lateral earth pressures behind the retaining wall caused by the excavation. No significant pipe deformations are measured at the depth of the jet-grouted slab. Large pipe deformations are observed within the stiff and coarse soil layers 2, 4 and 6 (Figure 7-3). Smaller pipe deformations are observed within the soft and fine soil layers 1, 3, 5 and 7.
The stiffer layers seem to have experienced larger lateral earth pressure increase during the jet-grouting phase and, respectively, larger lateral pressure decrease due to the unloading during the excavation phase.

![Figure 7-8: Pipe deformation during the excavation phase measured by IDM.](image)

### 7.5 Back-calculation of pressures and soil properties

#### 7.5.1 Approach

The deformations of the inclinometer pipe are assumed to follow changes in the surrounding stress field. The lateral stress increments close to the pipe can be back-calculated provided the incremental stiffness of all the materials involved is known. The stiffness of the pipe and of the grout, which is used to refill the borehole, is obtained in laboratory tests (see sections 7.5.4 and 7.5.5). The incremental stiffness of the soil may be assumed based on the site invest-
tigation. For relatively small stress increments within the elastic range, the stiffness measured with Marchetti dilatometer may provide reliable results. As soon as soil comes close to failure, this approach is no longer appropriate. Measurements of local displacements can as well provide additional information for back-calculating the stiffness of the soil. In the case presented here, the stiffness of soil is expected to be very difficult to predict for the jet-grouting phase, when soil has probably failed. Instead, the measured difference in lateral displacement between the retaining wall and the inclinometer pipe is introduced into the boundary value problem as an input quantity. It will allow back-calculation of both the pressure increments and the stiffness of the soil.

7.5.2 **Boundary value problem**

For each soil layer, the boundary value problem can be formulated in a horizontal cross-section (Figure 7-9). Constant stress conditions are assumed in the vertical direction owing to the constant overburden pressure. The inclinometer pipe is surrounded by the grout ring which is embedded in soil. Because the inclinometer pipe is located relatively close to the wall compared to the wall dimension of 13.6 m (see Figure 7-1), effect of the pipe on the stress-strain state of the soil diminishes along the wall. Based on a sensitivity analysis, introducing a constrained boundary condition at the distance of 2 m from the pipe axis is found to be sufficient to eliminate boundary effects. With respect to the rear boundary, a sensitivity analysis has shown that the boundary effects become negligible when the constrained boundary condition is introduced 1.5 m behind the pipe axis. The slurry wall displacements are applied uniformly at the front boundary in direction perpendicular to the wall. No friction is assumed between the slurry wall and the soil, resulting in the principal lateral stress increment being perpendicular to the wall.

For simplification, the interaction between the soil layers is neglected. The boundary value problem has been solved independently for each layer using the finite element code Abaqus. The mesh is built of three-dimensional elements (C3D8R) with linear shape function and reduced integration scheme. The cross-section has a thickness of one element for the soil and the grout. The overburden pressure is applied in the vertical direction on one side of the element. On the other side of the element, the vertical displacements are constrained.

Solving the boundary value problem separately for each horizontal cross-section does not account for the effect of unbalanced pressures acting on the pipe due to its bending with depth.
Therefore, the unbalanced pressure increment $\Delta p$ is estimated from the measured horizontal displacement of the pipe $w$ (Figure 7-5a and Figure 7-7a) using the following expression:

$$\Delta p(z) = \frac{EI}{D} \frac{d^4 w}{dz^4}$$

(7-3)

where $EI$ is the bending stiffness of the pipe, $D$ is the outer diameter of the pipe and $z$ is the vertical coordinate. In the case under study, the unbalanced pressure increments are found to be significantly lower than the back-calculated pressure increments in each horizontal cross-section and could, therefore, be neglected.

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**Figure 7-9:** Boundary value problem given in a horizontal cross-section.
The silt and clay layers (i.e. the layers 1, 3 and 5) are assumed to behave as fully undrained owing to their low permeability. For the coarse layers (i.e. 2, 4, and 6), a drained analysis was performed first. Later, an undrained analysis will also be performed for these layers, because calculations show that the fully drained behavior was unlikely. The constitutive law is chosen as linear elastic - perfect plastic. For the undrained analysis, the Tresca failure criterion is applied. Poisson’s ratio $\nu_s$ is equal to 0.5 owing to the constant volume. For the drained analysis, the Mohr–Coulomb failure criterion is assumed with zero cohesion and Poisson’s ratio $\nu_s$ of 0.25.

7.5.3 Back-calculation procedure

The back-calculation is performed separately for each layer. Within each layer, the lateral pressure increments $\Delta \sigma_h$ acting on the wall have to be determined. Usually, the pressure increments $\Delta \sigma_h$ could be obtained from the measured changes in ovalization value $\Delta \Omega$ provided the stiffness of all the materials is known. In the present case, the elasto-plastic soil stiffness is additionally back-calculated based on the additional information on the average lateral strain increments $\Delta \varepsilon_{h,av}$ between the wall and the inclinometer pipe. Therefore, there are two additional unknowns in the boundary value problem (i.e. the Young’s modulus of the soil $E_s$ and either the undrained shear strength $s_u$ for undrained analysis or the angle of internal friction $\varphi'$ for drained analysis). The back-calculation procedure is applied to obtain the two stress increments ($\Delta \sigma_h^l$ in loading and $\Delta \sigma_h^u$ in unloading) and the two unknown soil parameters $E_s$ and $s_u$ (or $\varphi'$) from the two measured changes in ovalization value $\Delta \Omega^l$ and $\Delta \Omega^u$ (in loading and unloading) and the two corresponding measured lateral strain increments $\Delta \varepsilon_{h,av}^l$ and $\Delta \varepsilon_{h,av}^u$ (Figure 7-10), using the following steps:

1) The soil is assumed to be elastic in unloading during the excavation phase.
2) The Young’s modulus of the soil $E_s$ and the pressure increment $\Delta \sigma_h^u$ in unloading are obtained from the inverse analysis of the elastic boundary value problem for $\Delta \Omega^u$ and $\Delta \varepsilon_{h,av}^u$ measured during the excavation phase.
3) The pressure increment $\Delta \sigma_h^l$ in loading is obtained from the inverse analysis of the elastic boundary value problem for $\Delta \Omega^l$ measured during the jet-grouting phase and the modulus $E_s$ back-calculated above for the excavation phase.
4) The lateral strain increment $\Delta \varepsilon_{h,av}^l$ is calculated from the pressure increment $\Delta \sigma_h^l$ in loading and the elastic modulus $E_s$, back-calculated above for the excavation phase.
5) If this calculated lateral strain increment $\Delta \varepsilon_{h,av}^l$ is smaller than the corresponding measured strain increment in loading, the soil is assumed to have failed. The lateral pressure at failure $\sigma_f$ can be calculated by adding the back-calculated pressure increment $\Delta \sigma_h^l$ in loading to the assumed initial lateral earth pressure $\sigma_0$: $\sigma_f = \sigma_0 + \Delta \sigma_h^l$.

6) The strength parameter of the soil layer (i.e. $s_u$ for undrained analysis, or $\varphi'$ for drained analysis) is calculated from the stress state at failure obtained above using the appropriate failure criterion.

7) The assumption of elastic behavior in unloading is verified based on the back-calculated pressure increments $\Delta \sigma_h^u$ and the back-calculated soil parameters ($E_s; s_u$ or $\varphi'$).

The initial stress conditions on the wall are difficult to define. The earth pressure coefficient at rest, $K_0$, was found by Marchetti dilatometer tests to lie within the range of 0.72 to 0.9. The initial lateral earth pressure coefficient on the wall, $K_0^{wall}$, is assumed to be equal to 1 owing to the overpressure of the liquid concrete acting during the construction of the slurry wall.

**7.5.4 Stiffness of the inclinometer pipe**

The stiffness of the inclinometer pipe is determined in order to be able to back-calculate the change in lateral earth pressure. The inclinometer pipe used here is made out of PVC. It shows a time-dependent mechanical behavior. Samples of the built-in pipe were tested in the laboratory under creep and constant stress rate conditions in the transversal direction (see
section 3.4). The applied constant stress rate was varied within two orders of magnitude for different tests. The linear viscoelastic four-parameter model was calibrated against the laboratory test results according to chapter 3 to obtain its parameters (Table 7-1). The viscoelastic four-parameter model can be transferred into a generalized Maxwell model in order to be implemented into a common finite element code using Prony series. The Poisson’s ratio of the pipe is given by the manufacturer as 0.34.

Table 7-1: Parameters of the four-parameter model determined by transversal loading of the pipe.

<table>
<thead>
<tr>
<th>parameter</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0$</td>
<td>2070 MPa</td>
</tr>
<tr>
<td>$\eta_0$</td>
<td>$2.0 \times 10^6$ MPa$\times$day</td>
</tr>
<tr>
<td>$E_1$</td>
<td>14000 MPa</td>
</tr>
<tr>
<td>$\eta_1$</td>
<td>$4.5 \times 10^4$ MPa$\times$day</td>
</tr>
</tbody>
</table>

### 7.5.5 Stiffness of the grout

A cement-bentonite grout mixture was used to refill the annulus around the inclinometer pipe in the borehole. Following Mikkelsen (2002), cement and water were mixed first for 5 minutes in a paddle mixer. Bentonite was then added and mixed for another 5 minutes to insure homogenous grout properties. The grout composition is given by the water : cement : bentonite ratio of 2.6 : 1 : 0.4. Laboratory tests were performed on this mixture to establish its stability towards sedimentation, strength and stiffness (see section 4.8). Within the stress range of the problem, the behavior of the grout is assumed to be linear elastic. The Young’s modulus $E_g$ is found to be 71 MPa in uniaxial compression after 28 days of hydration (see section 4.4.3). The Poisson’s ratio is assumed to be 0.5 owing to the full saturation of the grout and the undrained conditions.

### 7.5.6 Results

Numerical analysis is performed for the top six soil layers which are excavated within the box. Within each of the six layers, the representative horizontal cross-section is assumed in
the middle of the layer. The input parameters measured in-situ are averaged within the thickness of each layer (Table 7-2).

Table 7-2: Input parameters for each soil layer measured in-situ: average later strain increment between the wall and the pipe $\Delta \varepsilon_{h,av}$ and change in ovalization value $\Delta \Omega$.

<table>
<thead>
<tr>
<th>layers</th>
<th>jet-grouting phase</th>
<th>excavation phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>no. USCS</td>
<td>$\Delta \varepsilon_{h,av}^I$ [%]</td>
<td>$\Delta \Omega^I$ [%]</td>
</tr>
<tr>
<td>1</td>
<td>-0.32</td>
<td>-0.09</td>
</tr>
<tr>
<td>2 GP-GM</td>
<td>-0.82</td>
<td>-0.39</td>
</tr>
<tr>
<td>3 MH</td>
<td>-1.50</td>
<td>-0.18</td>
</tr>
<tr>
<td>4 SP-SM</td>
<td>-1.75</td>
<td>-1.10</td>
</tr>
<tr>
<td>5 CM</td>
<td>-2.07</td>
<td>-0.30</td>
</tr>
<tr>
<td>6 SP-SM</td>
<td>-1.85</td>
<td>-0.32</td>
</tr>
</tbody>
</table>

For each layer, the pressure increment in unloading $\Delta \sigma_h^U$ and the Young’s modulus $E_s$ are back-calculated from the input parameters measured in unloading during the excavation phase (Table 7-3).

Table 7-3: Back-calculated stress change and stiffness during excavation (unloading).

<table>
<thead>
<tr>
<th>layer no.</th>
<th>USCS</th>
<th>$\Delta \sigma_h^U$ [kPa]</th>
<th>$E_s$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>-6</td>
<td>4.8</td>
</tr>
<tr>
<td>2 GP-GM</td>
<td></td>
<td>-27</td>
<td>35.2</td>
</tr>
<tr>
<td>3 MH</td>
<td></td>
<td>-11</td>
<td>3.4</td>
</tr>
<tr>
<td>4 SP-SM</td>
<td></td>
<td>-71</td>
<td>20.4</td>
</tr>
<tr>
<td>5 CM</td>
<td></td>
<td>-23</td>
<td>4.5</td>
</tr>
<tr>
<td>6 SP-SM</td>
<td></td>
<td>-23</td>
<td>16.5</td>
</tr>
</tbody>
</table>
The pressure increment in loading $\Delta \sigma_h^l$ is back-calculated from the change in ovalization value $\Delta \Omega^l$ during the jet-grouting phase using the Young’s modulus $E_s$ of Table 7-3 which was derived previously. The lateral earth pressure at failure $\sigma_f = \sigma_0 + \Delta \sigma_h^l$ is given by the assumed initial lateral pressure $\sigma_0$ and the back-calculated pressure increment $\Delta \sigma_h^l$. The strength parameter of the soil layer (i.e. the undrained shear strength $s_u$ for the fine layers and the angle of internal friction $\varphi'$ for the coarse layers) is calculated from the stress state at failure, assuming the Tresca model for the undrained analysis of the fine layers and the Mohr Coulomb model for the drained analysis of the coarse layers (Table 7-4).

<table>
<thead>
<tr>
<th>layer no.</th>
<th>USCS</th>
<th>$\Delta \sigma_h^l$ [kPa]</th>
<th>$s_u$ [kPa]</th>
<th>$\varphi'$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>13</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>GP-GM</td>
<td>53</td>
<td>19</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>MH</td>
<td>24</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>SP-SM</td>
<td>131</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>CM</td>
<td>40</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>SP-SM</td>
<td>37</td>
<td>7</td>
<td></td>
</tr>
</tbody>
</table>

According to the numerical model, all the layers failed during the jet-grouting phase and unloaded elastically during the excavation phase. Therefore, the parameters obtained from unloading (i.e. the modulus $E_s$ and the change in lateral pressure $\Delta \sigma_h^u$ during excavation; Table 7-3) are assumed to be reliable. The reliability of the parameters back-calculated from the post-failure phase during loading (i.e. the strength and the change in lateral pressure $\Delta \sigma_h^l$ during jet-grouting; Table 7-4) is expected to be considerably lower.

The back-calculated angles of internal friction $\varphi'$ between 7° and 23° (Table 7-4) are rather small compared with the results of the CPT testing, where $\varphi'$ is within the range of 34° to 41° (Figure 7-3). The reduced strength indicates that the coarse layers behave as only partially drained during the jet-grouting phase due to the presence of fines and the fast impact of the excessive pressure surge caused by the jet-grouting. Pressure jumps measured by piezome-
ters in coarse layers during jet-grouting have confirmed this finding. The very low back-calculated shear strength of the coarse layer no. 6 may be explained by hydraulic shocks which were induced by the jet-grouting and may have propagated through the cracked wall. Based on these considerations, pressure increments and soil parameters are back-calculated again assuming undrained conditions for all the layers (Table 7-5).

Table 7-5: Back-calculated stress change, stiffness and shear strength for undrained conditions for all the layers.

<table>
<thead>
<tr>
<th>layers no.</th>
<th>USCS</th>
<th>jet-grouting</th>
<th>excavation</th>
<th>soil parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\Delta \sigma_h^J$ [kPa]</td>
<td>$\Delta \sigma_h^E$ [kPa]</td>
<td>$E_s$ [MPa]</td>
</tr>
<tr>
<td>1</td>
<td>13</td>
<td>-6</td>
<td>4.8</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>GP-GM</td>
<td>76</td>
<td>-38</td>
<td>40.7</td>
</tr>
<tr>
<td>3</td>
<td>MH</td>
<td>24</td>
<td>-11</td>
<td>3.4</td>
</tr>
<tr>
<td>4</td>
<td>SP-SM</td>
<td>184</td>
<td>-99</td>
<td>22.9</td>
</tr>
<tr>
<td>5</td>
<td>CM</td>
<td>40</td>
<td>-23</td>
<td>4.5</td>
</tr>
<tr>
<td>6</td>
<td>SP-SM</td>
<td>51</td>
<td>-33</td>
<td>18.4</td>
</tr>
</tbody>
</table>

The positive lateral pressure increments due to the jet-grouting are observed to be larger than the negative increments caused by the excavation process (Table 7-5). Hence, the final lateral earth pressure acting on the wall is expected to be larger than the initial earth pressure.

7.6 Validation

7.6.1 Validation of the back-calculation by comparing the soil parameters

The back-calculation procedure is validated against independent in-situ measurements of soil properties. The back-calculated Young’s moduli $E_s$ (Table 7-5) are compared with the constrained moduli $M$ obtained from Marchetti dilatometer tests (Figure 7-3). For this purpose, the constrained modulus $M$ is converted to the Young’s modulus for undrained conditions $E_u$ using the following expression:
Chapter 7: Inclinodeformeter pressure measurements in a vicinity of excavation

\[ E_u = \frac{3(1-2\nu_d)}{2(1-\nu_d)} M \]  

(7-4)

where \( \nu_d \) is the drained Poisson’s ratio of the soil.

For the Poisson’s ratio \( \nu_d \) equal to 0.25, the soil stiffness back-calculated from the IDM measurements in unloading shows a very good agreement (within 10% - 20%) with the stiffness measured by the Marchetti dilatometer (Table 7-6).

The back-calculated undrained shear strength (Table 7-6) is compared with the shear strength obtained from CPT profiling (Figure 7-3). In the silt and the clay layers, the back-calculated undrained shear strength is observed to be slightly smaller than the in-situ measured strength. The difference may be due to the assumption of the linear elastic, perfect plastic soil behavior, because the model does not allow for plastic strains before reaching failure. The comparison of the values obtained for the top layer is of minor significance because of the inhomogeneous structure of the layer containing peat.

Table 7-6: Comparison of the measured and back-calculated soil parameters (i.e. Young’s modulus \( E_u \) in undrained conditions and the undrained shear strength \( s_u \)).

<table>
<thead>
<tr>
<th>layer no.</th>
<th>USCS</th>
<th>( E_s,\text{DMT} ) [MPa]</th>
<th>( E_s,\text{B} ) [MPa]</th>
<th>( s_u,\text{CPT} ) [kPa]</th>
<th>( s_u,\text{B} ) [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>4.8</td>
<td>17</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>GP-GM</td>
<td>no test results</td>
<td>40.7</td>
<td>n. a.</td>
<td>38</td>
</tr>
<tr>
<td>3</td>
<td>MH</td>
<td>3</td>
<td>3.4</td>
<td>17</td>
<td>12</td>
</tr>
<tr>
<td>4</td>
<td>SP-SM</td>
<td>24</td>
<td>22.9</td>
<td>n. a.</td>
<td>92</td>
</tr>
<tr>
<td>5</td>
<td>CM</td>
<td>4</td>
<td>4.5</td>
<td>22</td>
<td>20</td>
</tr>
<tr>
<td>6</td>
<td>SP-SM</td>
<td>22</td>
<td>18.4</td>
<td>n. a.</td>
<td>26</td>
</tr>
</tbody>
</table>

7.6.2 Validation of the back-calculation by comparing the strut loads

The strains in the three central steel struts (Figure 7-1 and Figure 7-2) were measured during the excavation phase. The measured strut loads (Table 7-7) are up to 2.5 times larger (Ra-
baiotti et al., 2014) than those predicted using conventional earth pressure analysis. Therefore, the struts had to be reinforced during construction, and additional struts had to be installed next to the existing ones.

The measured strut loads allow additional validation of the back-calculation of earth pressures. The back-calculated pressure changes due to jet-grouting and due to excavating (Table 7-5) are added to the initial lateral earth pressures which are assumed based on the averaged weight $\gamma = 19$ kN/m$^3$ (Springman et al., 2012) and based on the initial lateral earth pressure coefficient at the wall $K_0^{wall} = 1$. The vertical pressures due to the weight of the 1.2 m high embankment (Figure 7-2) are assessed using the Boussinesq solution for the elastic half-space. In the vertical section, the cracked slurry wall is supported by three strut levels and the jet-grouted slab (Figure 7-2). The central strut of each strut level is assumed to support the back-calculated pressures acting on the wall within a rectangular area of influence ($h \times b$). The struts on the levels 1 and 2 are assumed to support the wall within half the vertical distance to the next strut level, which gives the height of the areas of influence $h_1 = 6.7$ m and $h_2 = 4.0$ m. The assumed remaining dimensions of the areas of influence, i.e. the height for the third strut level $h_3 = 3.2$ m and the horizontal width for all three central struts $b = 3.8$ m, have been validated by three-dimensional finite-element analysis of the excavation by Rabaiotti et al. (2014).

In spite of the simplified nature of the pressure distribution assumptions, the strut loads back-calculated from the IDM measurements appear to be very close to measured loads (Table 7-7).

<table>
<thead>
<tr>
<th>strut loads</th>
<th>measured</th>
<th>back-calculated</th>
<th>$\delta%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>strut level 1</td>
<td>1830 kN</td>
<td>1770 kN</td>
<td>- 3</td>
</tr>
<tr>
<td>strut level 2</td>
<td>2610 kN</td>
<td>2570 kN</td>
<td>- 1</td>
</tr>
<tr>
<td>strut level 3</td>
<td>3130 kN</td>
<td>3150 kN</td>
<td>+ 1</td>
</tr>
</tbody>
</table>
7.7 Discussion

The strut loads measured during the excavation appeared to be considerably larger than those expected in the design. A possible scenario suspected for the development of these large earth pressures was overpressure of the soil behind the slurry wall due to excessive jet-grouting pressures. IDM measurements allow for validation and quantification of this phenomenon. The large strut loads have been confirmed to result from the expansion of the concrete box during the jet-grouting phase. Lateral earth pressures in the surrounding soil increased because of the expansion. Following the excavation the pressures decreased, but a significant residual portion of excess lateral earth pressures remained, affecting the developing strut loads. Geotechnical monitoring of the strut loads complemented by the IDM measurements of earth pressures was critical for saving the excavation from collapse. Advantage of the IDM measurements is that they could detect large earth pressure increments resulting from the jet-grouting long before the excavation and strut forces measurements commenced, allowing for an early prediction of a potentially dangerous development.

7.8 Conclusions

Despite the highly non-homogeneous soil profile and the challenging loading history, the combination of IDM and displacement measurements allowed for back-calculation of soil stiffness and lateral earth pressure changes. The back-calculated quantities have been validated with both the stiffness and earth pressure increment values confirmed by independent in-situ measurements in six different layers of soil and by strut forces measured during the excavation.

IDM measurements did not require any additional infrastructure on the construction site, as they used the same inclinometer pipe installed for measuring displacements. When these displacement measurements are complemented by measuring displacements of the wall, the IDM measurements can provide not only the earth pressure increments, but also the stiffness of soil under in-situ pressures for each layer of soil.

Note, that without monitoring the wall displacements, the lateral earth pressure changes could still be back-calculated provided the stiffness of the soil is known. In the case under study, Marchetti dilatometer tests provided reliable soil stiffness within the elastic range. For large strains, it is still recommended to validate the soil stiffness by monitoring displacements.
For the excavation test under study, the lateral earth pressure changes obtained by IDM measurements helped to improve understanding the retaining wall reaction to installation processes, to provide an early detection of potential problems, and ultimately to prevent a possible excavation collapse. Real-time monitoring of the lateral earth pressures using IDM allowed for successful application of the observational method to minimize hazards due to underestimated earth pressures.

ACKNOWLEDGEMENTS

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REFERENCES


**NOTATION**

\( D \) outer diameter of the inclinometer pipe

\( D_A \) inner diameter of the pipe within the channels in direction A
\( D_B \)  inner diameter of the pipe within the channels in direction B
\( E_0 \)  elastic parameter of the Burgers model (four-parameter model)
\( E_1 \)  elastic parameter of the Burgers model (four-parameter model)
\( E_g \)  Young's modulus of the grout
\( E_I \)  bending stiffness of the inclinometer pipe
\( E_s \)  Young's modulus of the soil
\( E_{s,B} \)  Young's modulus of the soil back-calculated from IDM measurements
\( E_{s,DMT} \)  Young's modulus of the soil obtained from Marchetti dilatometer tests
\( E_{u} \)  Young's modulus of the soil for undrained conditions
\( K \)  earth pressure coefficient
\( K_0 \)  earth pressure coefficient at rest
\( K_{0,Wall} \)  initial lateral earth pressure coefficient behind the retaining wall
\( L \)  distance between the inclinometer in borehole M3 and the retaining wall
\( M \)  constrained modulus of the soil
\( R \)  nominal outer radius of the inclinometer pipe
\( s_u \)  undrained shear strength of the soil
\( s_{u,B} \)  undrained shear strength back-calculated from IDM measurements
\( s_{u,CPT} \)  undrained shear strength obtained from cone penetration tests
\( u_x \)  lateral displacement of the wall in direction \( x \)
\( u_z \)  lateral displacement of the inclinometer pipe in direction \( x \)
\( u_x \)  lateral displacement in direction \( x \)
\( u_y \)  lateral displacement in direction \( y \)
\( w \)  horizontal displacement of the pipe
\( z \)  vertical coordinate
\( \gamma \)  unit weight of the soil
\( \Delta p \)  unbalanced pressure increment acting on the pipe due to bending
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta \epsilon_{h, av}$</td>
<td>average lateral strain increment</td>
</tr>
<tr>
<td>$\Delta \epsilon_{h, av}^l$</td>
<td>average lateral strain increment in loading</td>
</tr>
<tr>
<td>$\Delta \epsilon_{h, av}^u$</td>
<td>average lateral strain increment in unloading</td>
</tr>
<tr>
<td>$\Delta \sigma_h$</td>
<td>lateral earth pressure increment</td>
</tr>
<tr>
<td>$\Delta \sigma_h^l$</td>
<td>lateral earth pressure increment in loading</td>
</tr>
<tr>
<td>$\Delta \sigma_h^u$</td>
<td>lateral earth pressure increment in unloading</td>
</tr>
<tr>
<td>$\Delta \Omega$</td>
<td>change in ovalization value</td>
</tr>
<tr>
<td>$\Delta \Omega^l$</td>
<td>change in ovalization value in loading</td>
</tr>
<tr>
<td>$\Delta \Omega^u$</td>
<td>change in ovalization value in unloading</td>
</tr>
<tr>
<td>$\epsilon_{h, av}$</td>
<td>average lateral strain</td>
</tr>
<tr>
<td>$\epsilon_s$</td>
<td>shear strain</td>
</tr>
<tr>
<td>$\eta_0$</td>
<td>viscous parameter of the Burgers model (four-parameter model)</td>
</tr>
<tr>
<td>$\eta_1$</td>
<td>viscous parameter of the Burgers model (four-parameter model)</td>
</tr>
<tr>
<td>$\nu_s$</td>
<td>Poisson’s ratio of the soil</td>
</tr>
<tr>
<td>$\nu_d$</td>
<td>drained Poisson’s ratio of the soil</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>initial lateral earth pressure</td>
</tr>
<tr>
<td>$\sigma_f$</td>
<td>lateral earth pressure at failure</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>lateral earth pressure</td>
</tr>
<tr>
<td>$\varphi'$</td>
<td>angle of internal friction of the soil</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>ovalization value</td>
</tr>
<tr>
<td>$\Omega_0$</td>
<td>zero measurement of the ovalization value</td>
</tr>
<tr>
<td>$\Omega_n$</td>
<td>further measurement of the ovalization value</td>
</tr>
</tbody>
</table>
8 Conclusions and outlook

8.1 Main results

Lateral earth pressures are a key factor for many practical problems in geotechnical engineering. Measuring these earth pressures, however, remains one of the most challenging problems in geotechnical monitoring. A novel device and procedure have been developed to determine the change in lateral earth pressure. Inverse analysis of the measured change in shape of the cross-section of an inclinometer pipe allows back-calculation of lateral earth pressure changes in the surrounding soil.

An extensive program of full-scale laboratory tests and field measurements has validated the applicability of the monitoring device developed here to geotechnical problems. The novel device provides a step towards more reliable and precise measurements of changes in lateral earth pressure. Changes in dimensions of an inclinometer pipe are measured with a precision of +/-2 micrometers. Lateral earth pressure changes are back-calculated with a precision within the range of 0.1 kPa to 0.7 kPa depending on the stiffness of the soil and the grout surrounding the pipe. The softer the soil and the grout, the higher is the precision.

The major aspects affecting the back-calculation of pressures have been studied: these are (i) the influence of the time-dependent behavior of the pipe, (ii) the influence of the mechanical behavior of the grout and (iii) the effects of longitudinal bending of the inclinometer pipe.

The back-calculation procedure has been successfully validated in full-scale laboratory tests.

i) Time-dependent deformations of an inclinometer pipe can be described by the viscoelastic four-parameter model. Its parameters can be obtained from laboratory tests performed in the transversal direction on the pipe sample. The viscoelastic model provides reliable back-calculation of pressures both for instantaneous and continuously applied loads.

ii) Simple linear constitutive models have been found to be appropriate, which makes back-calculation of pressures reliable and independent of initial conditions. In general, the grout is expected to be designed to have similar deformation characteristics to the surrounding soil. Thus, the influence of the stiffness of the grout can be neglected for back-calculation of pressures, and an analytical solu-
tion may be applied, which assumes the same stiffness for the grout as for the soil. If the grout is stiffer than the soil but cracked, the influence of the stiffness of the grout remains small and may be neglected, so the analytical solution may again be applied. For an intact grout ring, which is considerably stiffer than the surrounding soil, the grout stiffness can be taken into account by performing numerical simulations in order to back-calculate the pressures. Fortunately, the backcalculation of stresses is not sensitive to the stiffness of the grout. Less than a two-fold increase in the back-calculated stress increments is caused by a tenfold increase in the stiffness of the grout. Below the ground water table, the grout is likely to be intact.

The stiffness of the grout can be determined in unconfined compression tests. The sensitivity of the IDM pressure measurements can be enhanced by using softer grout provided the Young’s modulus of the grout is no smaller than 10 MPa. Further constraints for the design of the optimal grout compound are the workability, the stability towards sedimentation and the strength.

iii) An analytical solution calibrated using numerical simulations allows correction of the ovalization of an inclinometer pipe due to bending. The solution provided allows the identification of many applications for IDM with relatively small curvature, where the effects of longitudinal bending can be neglected. In particular, in the sliding layer of creeping landslides, the effects of longitudinal bending can be ignored.

The IDM device allowed quantification of the earth pressure changes in a depth profile in two different applications: (iv) in creeping landslides and (v) behind a retaining wall during jet-grouting and excavation.

iv) IDM pressure measurements provide a reliable tool for analysis and monitoring of creeping landslides. Measuring the change in shape of an inclinometer pipe cross-section allows identification of compression and extension zones. Reasonably small pressure increments can be reliably back-calculated by applying an analytical solution which takes the time-dependency of the pipe material into account. Additionally, the tangent stiffness of the soil can be obtained by combining the IDM measurements with measurements of relative displacements. Thus, the back-calculation procedure becomes independent of the measurement of stiffness.
of soil. The lateral modulus obtained in-situ for the incremental behavior distinctly indicates yielding and failure within the sliding layer.

v) The lateral earth pressure changes obtained by IDM measurements can improve understanding and analysis of supported excavations. Real-time monitoring of the lateral earth pressures with IDM allows the successful application of the observational method. IDM pressure measurements for monitoring can minimize hazards due to underestimated earth pressures. The combination of IDM and displacement measurements allows reliable back-calculation of the profile with depth of the lateral stiffness of the soil. For layered soils, soil parameters and pressure increments are obtained for each layer.

The device that has been developed looks promising owing to its simplicity and accuracy of measurements. A standard inclinometer pipe is the only installation required, and this is regularly installed anyway for many applications. The IDM device can continue to make use of an inclinometer pipe even long after it may have been sheared and become unsuitable for inclinometer measurements. The proposed device and method are expected to contribute to a better understanding of lateral earth pressures in geotechnical engineering problems.

8.2 Future research

A novel approach to measuring lateral earth pressure changes is presented. The device and procedure developed here provide a basis for its application in geotechnical practice. Additional research could take this approach to the next level and allow it to approach its full potential.

Although the mechanical interaction between the soil and the monitoring tools has been investigated through numerous laboratory tests, full-scale laboratory tests and numerical analysis, further research should be carried out on (i) the influence of the mechanical behavior of the grout and (ii) the influence of longitudinal bending of the pipe.

i) Further research is required to understand the conditions under which the cement-bentonite grout can become cracked. The deformation behavior of the cracked grout ring also needs to be investigated further. Alternatively, sand could be used to refill the borehole in dry soils, where the cement-bentonite grout is prone to shrink and crack.
Additional studies are required to improve the correction of the influence of longitudinal bending of the pipe. For a particular application with intense curvature, the effects due to the surrounding pressures and due to the limited length of the curved section may have to be considered and therefore introduced into the approach for correction.

The strategic direction of research should be to focus (iii) on the improvement of the pipe and (iv) on further applications of the presented approach.

The results presented here were obtained with standard inclinometer pipes which are commonly installed in Switzerland. Although it is favourable to be able to make use of a large number of installed pipes, a purpose-designed pipe for IDM could be developed since there might be significant potential for improvement. The developed pipe should show larger and time-independent deformations to enhance the sensitivity and the reliability of the method. If the initial shape of the purpose-designed pipe is well-known, the zero measurement may provide information about the initial stress state in the soil around the borehole.

The device and procedure have been applied to creeping landslides and to an excavation. The results obtained may encourage investigation of further applications in geotechnical engineering. Understanding and analysis of geotechnical problems may benefit highly from the new insight gained through the method developed here. Further research in this area would be of significant interest.
9 Appendix: device and method description

9.1 Abstract

The inclinodeformeter (IDM) is a technology for back-calculating the changes in the lateral earth pressures based on the measured changes in dimensions of an inclinometer pipe installed in a vertical borehole in soil. A continuous profile of changes of lateral earth pressure is obtained. The IDM technology comprises a device to measure dimensions of an inclinometer pipe and a method to interpret the measurements in terms of lateral earth pressures.

The device measures the inner diameter of the cross-section of an inclinometer pipe at a predetermined longitudinal position. The device consists of a probe detecting the inner diameter and of a positioning system detecting the longitudinal position. The appendix provides a description of the design of the probe and of the design of the positioning system.

The appendix also provides a description of the method of interpreting the measurements in terms of lateral earth pressures. The method comprises 11 steps: A borehole is drilled (I) wherein an inclinometer pipe (II) is installed. Inner diameter readings are taken (III) and corrected (IV) in order to derive the ovalization of the cross-section of the pipe (V). Several measurements (VI) are required over time to obtain the change in ovalization (VII) which is corrected for effects of longitudinal bending (VIII) and averaged with depth (IV). The stiffness of the pipe and the soil (X) is assessed in order to back-calculate the change in lateral earth pressure (XI).

The appendix is based on a filed patent application (Puzrin et al., 2013).
9.2 Introduction

The inclinodeformeter is introduced for measurement of lateral earth pressure changes in geotechnical monitoring practice. The appendix provides a description of the device and of the method of interpreting the measurements in terms of lateral earth pressures.

The description of the device comprises the mechanical design of the probe, the mechanical design of the positioning system and electronic design of the assembly. The method for interpreting the measurements in terms of lateral earth pressures comprises 11 steps. It provides a guide-line for geotechnical monitoring practice. Therefore, the method includes the most common assumptions made in the framework of IDM. For particular application, the method may be modified based on the findings discussed in chapters 2–7. However, the procedure presented here can be straightforwardly applied for many applications in geotechnical engineering.

The appendix is based on a filed patent application (Puzrin et al., 2013).

9.3 Inclinodeformeter: concept

The inclinodeformeter (IDM) is a technology for back-calculating the changes in the lateral earth pressures based on the measured changes in dimensions of an inclinometer pipe installed in a vertical borehole in soil. A continuous profile of changes of lateral earth pressure is obtained (Figure 9-1).

Inclinometer pipes are widely used for measuring soil displacements with depth using a standard inclinometer probe (Figure 9-1).

The curve $\Delta \sigma(z)$ in Figure 9-1 corresponds to the principal change in stress which may be assumed co-axial with the displacement vector $u(z)$. From this change in normal stress the stress increments in other directions can be derived.

The IDM technology comprises a device to measure dimensions of an inclinometer pipe and a method to interpret the measurements in terms of lateral earth pressures. The device is only applicable to inclinometer pipes owing to its specific design, whereas the method can be
applied to any kind of pipe. The method is formulated in a general sense and is not dependent on the measurement technique used to obtain the pipe dimensions.

![Diagram of IDM pressure measurements](image)

- $u(z)$: horizontal displacement of the pipe measured by conventional inclinometers
- $D(z)$: inner diameters of the pipe measured by IDM
- $\Delta \sigma(z)$: change in lateral earth pressure back-calculated by IDM

Figure 9-1: Conceptual diagram illustrating the principle of IDM pressure measurements.
9.4 Description of the device

The device is measuring the inner diameter of the cross-section of an inclinometer pipe at a predetermined longitudinal position. The device consists of a probe detecting the inner diameter and of a positioning system detecting the longitudinal position.

9.4.1 Design of the probe

The IDM probe is lowered down the pipe on three wheels guided along the channels of the pipe (Figure 9-2). The upper and the lower wheels roll in the same channel. These wheels are fixed to the probe. The middle wheel is connected via a lever with two springs, so that it is pressed against the opposite channel. A change in the diameter leads to change of the position of the middle wheel in respect to the probe. There are two tilt sensors to measure the inner diameter of the pipe: One sensor is located on the top of the probe and detects the inclination of the probe $\alpha_P$; another tilt sensor is located on the lever of the middle single wheel and detects the inclination of the lever $\alpha_L$. The inner diameter $D$ is a function of the relative inclination $(\alpha_L - \alpha_P)$. The base of the lever $Y$ is given by the distance from the center of rotation of the lever to the center of rotation of the middle wheel. The center of rotation of the lever is located at a distance $X$ from the line connecting the centers of rotations of the upper and the lower wheel.

The inner diameter $D$ is calculated from the geometry $(d, X, Y)$ and the two measured inclinations $(\alpha_L, \alpha_P)$:

$$D = d + X + Y \sin(\alpha_L - \alpha_P) \quad (9-1)$$

- $D$ inner diameter of the inclinometer pipe
- $d$ diameter of the wheels of the probe
- $X$ distance describing the location of the center of rotation of the lever
- $Y$ base length of the lever
- $\alpha_L$ inclination of the lever
- $\alpha_P$ inclination of the probe
In addition to the two sensors in the plane of the measured diameter, there is another tilt sensor measuring the inclination $\beta$ of the probe in a perpendicular direction out of the plane (Figure 9-2). This sensor is used for correction of the measurements due to the out-of-plane inclination of the probe.

Figure 9-2: The IDM probe.
Above the top wheel there is a pressure cell to measure the water pressure in the inclinometer pipe (Figure 9-3). The temperature of the probe is also measured continuously in order to give an opportunity to correct for the influence of the temperature.

The same probe can be used for different kind of inclinometer pipes with different inner diameters. Therefore the bearings of the lever can be fixed in two different positions (Figure 9-3, Figure 9-4). The lever rotates in its bearings which are built within hangers. These hangers are attached to the rest of the probe with screws. The screws connecting the hangers to the probe can be fixed in two different positions in order to move the center of rotation of the lever. This allows the same probe to be used for different type of inclinometer pipes.
Figure 9-3: Mechanical design of the IDM probe.
Figure 9-4: Mechanical design of the IDM probe (drawing without the front panel).
9.4.2 Design of the positioning system

At the top of the borehole, the cable on which the probe is hanging goes around a wheel. An incremental rotation sensor measures the wheel rotation, which determines the longitudinal position of the probe in the pipe. As the probe is lowered down, all the sensor measurements are saved for the corresponding longitudinal position.

A connecting piece of pipe is fixed on top of the inclinometer pipe in order to elongate the pipe above the ground surface (Figure 9-5). The winch is fixed on top of the connecting piece. In order to be able to compare measurements it is essential to take reliable and precise measurements of the position of the probe.

Therefore corresponding markers on the cable and on the winch exactly define the starting position of the probe. The position of the probe is recorded relative to the starting position.

The change in position is measured by the rotation of the winding wheel. In order to reach high precision, slippage of the cable has to be avoided by squeezing it between the winding wheel and another wheel (Figure 9-6). The center of rotation of the winding wheel is not fixed vertically (Figure 9-7). Therefore the load of the cable on the winding wheel is used to squeeze the cable. The contact pressure where the cable is squeezed increases with the length of the winded cable. This setup automatically increases friction as necessary.
Figure 9-5: The positioning system.
Appendix: Device and method description

Figure 9-6: Features of the positioning system.
Figure 9-7: Mechanical design of the positioning system.
9.4.3 Electrical design of the device

The analog signal of the five sensors on the probe (i.e. three tilt sensors, a temperature sensor and a pressure transmitter) is transferred uphole on the cable on which the probe is hanging (Figure 9-8). There are slip rings in order to allow the cable to pass over the reel. The analog signal is converted into a digital signal triggered by the rotation sensor at a frequency of 40 readings/cm of depth. The data corresponding to the longitudinal position is saved on the computer. The computer also provides the power supply for the whole device.

Figure 9-8: Electrical scheme of the device.
9.5 Description of the method

The method for interpreting the IDM measurements in terms of lateral earth pressures comprises 11 steps (Figure 9-9):

- Drill borehole
- Install pipe
- Measure diameters in transversal direction
- Correct single diameter readings
- Derive ovalization
- Measurements are taken before and after the pressure change to be observed?
- YES
  - Derive change in ovalization
  - Correct change in ovalization
  - Average change in ovalization
  - Assess stiffness
  - Derive change in pressure
- NO
  - Derive change in ovalization
  - Correct change in ovalization
  - Average change in ovalization
  - Assess stiffness
  - Derive change in pressure

Figure 9-9: The method flow diagram.
I. Drilling borehole

A vertical borehole is drilled into the ground.

II. Installing pipe

An inclinometer pipe with several pairs of longitudinal opposite channels inside is installed along the borehole. It may be a standard inclinometer pipe with two pairs of opposite channels oriented perpendicular to each other. The inclinometer pipe consists of casing elements connected to each other. The pipe is closed at the bottom of the borehole. At the top of the borehole the pipe can be opened by removing the closure head. The borehole is refilled with grout, filling the empty space around the pipe.

The proposed method is not limited to inclinometer pipes. It can be applied for any kind of pipe.

III. Measuring diameters in transversal direction

At least two profiles of inner diameter $D$ of the pipe are measured in different transversal directions.

More diameter measurements may be taken in different transversal directions in order to describe better the shape of the pipe. More measurements of inner diameters enhance the accuracy of the proposed method.

In the case of a standard inclinometer pipe, the two perpendicular diameters in the two pairs of channels are measured (Figure 9-10).

The diameter measurements are taken at different longitudinal positions of the pipe in order to obtain a profile along the pipe or to accumulate more data for a larger base for data reduction.
IV. Correcting single diameter readings

Every single diameter measurement is corrected for the influence of two errors due to the design of the probe. One error (a) is due to the tilt sensors not being independent of the inclination out of the plane. The other error (b) occurs because the diameter measured by three wheels at a certain distance is not equal to the true diameter at one single cross-section of the pipe.

a) The tilt sensors are not independent of the inclination out of the plane. It is sufficient to correct the difference \((\alpha_L - \alpha_P)\) between the measured inclination of the lever \(\alpha_L\) and the measured inclination of the probe \(\alpha_P\) in order to correct the diameter. Therefore a correction term \(\Delta\) is subtracted from the difference \((\alpha_L - \alpha_P)\). The correction term is dependent on the inclination of the probe out of the plane \(\beta\) (Fig. 2), the inclination of the lever \(\alpha_L\), the inclination of the probe in the plane \(\alpha_P\), and the correction coefficients \(A_1, A_2, A_3, C_1, C_2\) and \(C_3\):

\[
\Delta = (A_1 \alpha_L + A_2 \alpha_P + A_3) \beta^2 + (C_1 \alpha_L + C_2 \alpha_P + C_3) \beta
\]  

(9-2)
The correction coefficients $A_1$, $A_2$, $A_3$, $C_1$, $C_2$ and $C_3$ are determined by fitting laboratory calibration measurements.

The sensors in the plane are assumed to show true values if the probe is standing vertically. The sensor measuring the inclination out of the plane cannot be built into the probe perfectly straight. Therefore the measured inclination out of the plane $\beta_0$ of the vertical probe is introduced. The correction term $\Delta_0$ for the probe within the plane is obtained:

$$\Delta_0 = (A_1 \alpha_L + A_2 \alpha_P + A_3) \beta_0^2 + (C_1 \alpha_L + C_2 \alpha_P + C_3) \beta_0$$ (9-3)

The corrected inner diameter of the pipe is calculated:

$$D = d + X + Y \sin(\alpha_L - \alpha_P - (\Delta - \Delta_0))$$ (9-4)

b) The inner diameter is detected by measuring the distance from the middle wheel to the probe (Figure 9-2). The probe is guided by the upper and the lower wheel in the opposite channel. The probe describes a straight line between the upper and the lower wheel. For a straight pipe, the straight probe represents the opposite channel very well. The mechanical design of the probe implies a linear interpolation of the longitudinal shape of the pipe between the upper and the lower wheel.

For a curved pipe, the error due to the geometry of the probe needs to be corrected. Instead of the linear interpolation, the true curved longitudinal shape of the pipe is taken into account between the upper and the lower wheel (Figure 9-11). The difference between the linear interpolation and the curved shape of the pipe at the longitudinal position of the middle wheel is added to the measured diameter.

The curved longitudinal shape of the pipe can be measured using a standard inclinometer probe. Alternatively, the IDM measurements can be processed in the same way, also providing the longitudinal shape of the pipe.
true inner diameter $D$

error in the diameter measurement due to the geometry of the probe: to be corrected

longitudinal shape of the pipe: extremely exaggerated in order to highlight the error

Figure 9-11: Illustrated error in the diameter measurement due to the geometry of the probe.

V. Deriving ovalization value

The shape of the pipe is characterized as an oval, with a major axis length $MA$ and a minor axis length $MI$, then the ovalization value $\Omega$ can be determined by subtracting the minor axis length from the major axis length and normalizing the difference by the outer radius of the pipe $R$:

$$\Omega = \frac{MA - MI}{R}$$ (9-5)

The outer radius of the pipe $R$ is defined as the nominal outer radius given by the manufacturer of the pipe. An equivalent oval is found by fitting the plurality of inner diameter meas-
measurements in order to obtain the major and minor axis length to derive the ovalization value (Figure 9-12). Owing to theoretical considerations the oval is described by an average radius $r_{av}$, an amplitude $A$ and a rotation of the oval $\rho$. The equivalent oval is given in the polar coordinates $r$ and $\varphi$:

$$r(\varphi) = r_{av} + A \cos(2(\varphi + \rho))$$

(9-6)

The parameters $r_{av}$, $A$ and $\rho$ of the equivalent oval are determined by fitting the measured plurality of inner diameters $D$. In the case of two inner diameter measurements the rotation of the oval $\rho$ is assumed to be zero.

![equivalent oval](image)

**Figure: 9-12: The equivalent oval.**

The major axis length $MA$ and the minor axis length $MI$ can be determined from the parameters of the equivalent oval:

$$MA = 2(r_{av} + A)$$

(9-7)

$$MI = 2(r_{av} - A)$$

(9-8)
Hence, the ovalization value $\Omega$ can be derived from the parameters of the equivalent oval:

$$\Omega = \frac{4A}{R} \quad (9-9)$$

In the case of two measured perpendicular inner diameters, consequently the ovalization value $\Omega$ can be calculated by subtracting the smaller inner diameter $D$ from the larger inner diameter $D$ and normalizing the difference by the outer radius $R$. This is the case for a standard inclinometer pipe, where two perpendicular diameters are measured within the two pairs of channels.

The ovalization value is derived for all predetermined longitudinal positions where measurements of inner diameters have been taken.

The definition of the ovalization value as a normalized difference between the major axis length and the minor axis length allows for reducing the influence of errors affecting all the diameter measurements (e.g., influence of the long-term stability of the probe; influence of the actual field conditions: temperature, humidity, water pressures inside and outside of the pipe).

**VI. Required sets of measurements**

The pressure in the ground is expected to change with time. In order to observe the change in pressure the ovalization value has to be determined before and after the period of interest. The reference set of inner diameter measurements is taken before the period of interest, and the reference ovalization value $\Omega_0$ can be derived. At least one set of measurements is taken after the period of interest, and a subsequent ovalization value $\Omega_1$ can be derived.

As long as the observation period is not enclosed by measurement sets, further measurements should be taken.
VII. Deriving change in ovalization value

The change of shape of the pipe with time is characterized by the change in the ovalization value. The change in the ovalization value $\Delta \Omega$ over the observation period is derived by subtracting the reference value from the subsequent ovalization value:

$$\Delta \Omega = \Omega_i - \Omega_o$$  \hspace{1cm} (9-10)

The change in ovalization value is derived for all predetermined longitudinal positions where measurements of inner diameters have been taken.

VIII. Correcting change in ovalization value

Change in ovalization does not only occur because of changes in pressure in the ground. As a pipe is bent longitudinally its cross-section flattens into an oval shape. The pipe is subject to bending due to displacements of the surrounding ground, which may not be related to change in pressure. Therefore the change in ovalization can be corrected: The amount of change in ovalization value due to longitudinal bending $\Delta \Omega^X$ can be subtracted from the measured change in ovalization value $\Delta \Omega$.

The maximum possible change in ovalization value due to bending $\Delta \Omega^X$ can be derived from the curvature at the reference measurement $\chi_0$, the curvature at the subsequent measurement $\chi_1$, the outer radius of the pipe $R$, the thickness $h$ of the pipe, the Poisson’s ratio of the pipe material $\nu_p$ and the angle $\delta^X$ between the direction of the curvature and the direction of the minor axis of the equivalent oval, as follows:

$$\Delta \Omega^X = 4 \left( \frac{\chi_1 - \chi_0}{\chi_0} \right) \left( R - \frac{h}{2} \right)^3 \left( 1 - \nu_p^2 \right) \cos(2 \delta^X)$$  \hspace{1cm} (9-11)

The curvature of the pipe at the reference measurement $\chi_0$ and the curvature at the subsequent measurement $\chi_1$ can be found by numerical differentiation of the longitudinal shape of the pipe. The curvature may be obtained by adding the components of curvature obtained in two directions. The direction of the curvature may be obtained by combining the orientation parameters.
of the two components. The longitudinal shape of the pipe can be measured using a standard inclinometer probe. Alternatively the IDM measurements can be processed in the same way, also providing the longitudinal shape of the pipe.

In the case of an inclinometer pipe, the thickness $h$ of the pipe is reduced to take the influence of the channels into account. The reduced thickness of the pipe $h_{\text{red}}^X$ is calibrated against numerical simulations that take the real cross-section, including channels, as an input. The solution for the change in ovalization value due to bending $\Delta \Omega^X$ can still be applied as an approximation calibrated against numerical simulations.

The calculated change in ovalization value due to bending $\Delta \Omega^X$ may not fully develop. If the length of the section of intense curvature is shorter than the required length of the transition zone, smaller pipe deformations will be observed. Nevertheless, the calculated change in ovalization value $\Delta \Omega^X$ is still appropriate as an upper bound for the deformations due to bending.

IX. Averaging change in ovalization value

The change in ovalization value $\Delta \Omega$ is obtained at many predetermined longitudinal positions in the pipe, forming a profile along the pipe. The amount of measured data provides opportunity for data reduction. The data is averaged over a certain section along the pipe axes in order to smoothen the profile.

The length of the section for averaging is taken to be several times the circumference of the wheels of the probe. Thus the influence of noise at the frequency of the wheels is reduced. The length of the section for averaging depends on the required spatial resolution. In order to be able to observe very small deformations, the spatial resolution is lowered.

In the case of a pipe consisting of several casing elements, the section of the pipe where the probe is influenced by the joints is neglected (Figure 9-13). Hence a section of at least the length of the probe is ignored at each joint.

The change in ovalization value is averaged within each casing element, considering the proposed restrictions. In order to observe very small deformations each casing element is
represented by just one value. The spatial resolution can be increased by reducing the length of the section for averaging. Several values are obtained within each casing element.

Figure 9-13: Averaging change in ovalization value.

X. Assessing stiffness

The pressure increment to be observed is related to the change in ovalization via the incremental stiffness of the involved materials (i.e. the pipe material, the grout and the soil). The incremental stiffness of the involved materials can be expressed via constants of the elasticity theory (e.g. Young’s modulus and Poisson’s ratio).
The grout is assumed to have the same stiffness as the soil. The stiffness of the soil is obtained from standard site investigation. It is measured in field or laboratory tests or assessed by other means.

The stiffness of the pipe material is assessed based on laboratory tests. The pipe material shows time-dependent behavior, which is taken into account. The stiffness of the pipe for the time step in between the measurement sets is obtained by considering a viscoelastic model (Burgers model). The analytical solution for the occurring strains with time is derived assuming the stress conditions with time (creep conditions or constant stress rate conditions). Dividing the expression for the stress by the expression for the strain at the end of the time step is providing the equivalent stress-independent average stiffness over the time step.

The Young’s modulus of the pipe $E_p$ depends on the time step $t$ and on the model parameters, which are the Young’s modulus of the spring in series $E_0$, the Young’s modulus of the spring in parallel $E_1$, the viscosity of the dashpot in series $\eta_0$ and the viscosity of the dashpot in parallel $\eta_1$ of the mechanical analog assuming the Burgers model (Figure 9-14). The Young’s modulus of the pipe $E_p$ is calculated assuming constant stress rate conditions:

$$E_p = \frac{1}{E_0} \left( \frac{1}{E_t} - \frac{\eta_1}{\eta_0} \right) + \frac{1}{t} \left( 1 - e^{\frac{-E_t}{\eta_1} \cdot t} \right)$$

or assuming creep conditions:

$$E_p = \frac{1}{E_0} \left( \frac{1}{E_t} + \frac{\eta_1}{\eta_0} \right) + \frac{1}{\eta_0} \cdot t$$

The parameters of the Burgers model $E_0, E_1, \eta_0$ and $\eta_1$ are determined by laboratory tests performed on the pipe material.
XI. Deriving change in pressure

The pressure changes with depth $\Delta \sigma_1$ are back-calculated by solving the corresponding boundary value problem given in a horizontal cross-section at each predetermined longitudinal position. The change in pressure at each position is obtained from the change in the static boundary condition, which has to be applied in order to reproduce the measured pipe deformations.

The change in lateral pressure in the surrounding ground can be derived from the change in ovalization value by applying an analytical solution based on elasticity theory. The boundary value problem is solved for a horizontal cross-section under generalized plane-stress conditions due to the constant overburden pressure. The change in lateral pressure $\Delta \sigma_1$ is defined as the major principal stress increment in a horizontal plane at infinity. The change in pressure $\Delta \sigma_1$ is proportionally dependent on the change in ovalization value $\Delta \Omega$. The change in pressure $\Delta \sigma_1(z)$ also depends on the stiffness of the pipe and the soil (i.e. Young’s moduli of the pipe $E_p$ and of the soil $E_s$, Poisson’s ratio of the soil $\nu_s$), the geometry (i.e. the mean radius of the pipe $R_m$, the circumferential cross-section moment of inertia of the pipe $I$), the angle $\delta$ between the direction of the major principal stress increment and the direction of the minor axis of the equivalent oval and the stress ratio $K_0$ between the principal stress increments in the plane at infinity:

![Diagram of the Burgers model](image-url)
\[
\Delta \sigma_1 = \frac{(E_r + \alpha E_p)}{8(1 - K_0)\cos(2\delta)} \Delta \Omega
\]  
(9-14)

with

\[
\alpha = \frac{3(5 - \nu_s)I}{R_m^3}
\]  
(9-15)

The stress ratio \( K_0 \) is defined by dividing the minor principal stress increment by the major principal stress increment:

\[
K_0 = \frac{\Delta \sigma_2}{\Delta \sigma_1}
\]  
(9-16)

Constrained boundary conditions can be assumed at infinity in direction of the minor principal stress increment for many applications. Thus, the ratio \( K_0 \) is equal to the Poisson’s ratio of the ground \( \nu_s \).

The angle \( \delta \) depends on the direction of the major principal stress increment in the plane. If this direction is not known, it may be obtained based on the following assumptions:

The major principal stress increment may be assumed to occur in direction of the horizontal displacement of the ground \( u(z) \). The horizontal displacement of the ground \( u(z) \) can be measured using a standard inclinometer probe (Figure 9-1). Alternatively, the IDM measurements can be processed in the same way to provide the horizontal displacement of the ground. If more than two inner diameters are measured, the direction of the major principal stress increment may also be assumed according to the deformed shape of the pipe. The major stress direction may coincide with the minor principal axis of the equivalent oval.

In the case of an inclinometer pipe, the circumferential cross-section moment of inertia of the pipe \( I \) is reduced to take the influence of the channels into account. The overall moment of inertia of the pipe \( I_{\text{red}} \) is calibrated against numerical simulations that consider the real cross-section with channels. The solution for the change in pressure \( \Delta \sigma_1 \) can still be applied as an approximation calibrated against numerical simulations.
REFERENCES


NOTATION

$A$ amplitude of the equivalent oval
$A_1$ coefficient of the error function
$A_2$ coefficient of the error function
$A_3$ coefficient of the error function
$C_1$ coefficient of the error function
$C_2$ coefficient of the error function
$C_3$ coefficient of the error function
$D$ inner diameter of the pipe
$d$ diameter of the wheels of the probe
$E_0$ elastic parameter of the Burgers model
$E_1$ elastic parameter of the Burgers model
$E_p$ Young’s modulus of the pipe
$E_s$ Young’s modulus of the soil
$d$ thickness of the pipe
$h_{red}^\chi$ reduced thickness due the channels to correct for the influence of bending
$I$ circumferential cross-section moment of inertia of the pipe
$I_{red}$ reduced moment of inertia due to the channels
$K_0$ ratio between the principal stress increments in the plane
$MA$ major axis length of the equivalent oval
\( M_I \)  minor axis length of the equivalent oval  
\( R \)  nominal outer radius of the pipe  
\( R_m \)  mean radius of the pipe  
\( r \)  radius of the equivalent oval  
\( r_{av} \)  average radius of the equivalent oval  
\( t \)  time step  
\( u \)  horizontal displacement of the pipe  
\( X \)  distance describing the location of the center of rotation of the lever  
\( Y \)  base length of the lever  
\( z \)  coordinate along the pipe (approximately vertical coordinate)  
\( \alpha_L \)  measured inclination of the lever on the probe  
\( \alpha_P \)  measured inclination of the probe  
\( \beta \)  measured inclination of the probe out of the plane  
\( \beta_0 \)  measured inclination out of the plane of a vertical probe  
\( \Delta \)  term to correct for the influence of the inclination out of the plane  
\( \Delta_0 \)  correction term for the probe within the plane  
\( \Delta \sigma \)  change in lateral earth pressure  
\( \Delta \sigma_1 \)  major principal stress increment in the horizontal plane  
\( \Delta \sigma_2 \)  minor principal stress increment in the horizontal plane  
\( \Delta \Omega \)  change in ovalization value  
\( \Delta \Omega^X \)  change in ovalization value due to bending of the pipe  
\( \delta \)  angle between the direction of the major principal stress increment and the direction of the minor axis of the equivalent oval  
\( \delta^X \)  angle between the direction of the curvature and the direction of the minor axis of the equivalent oval  
\( \eta_0 \)  viscous parameter of the Burgers model
Appendix: Device and method description

\( \eta_1 \) viscous parameter of the Burgers model

\( \nu_p \) Poisson’s ratio of the pipe

\( \nu_s \) Poisson’s ratio of the soil

\( \rho \) rotation of the equivalent oval

\( \sigma \) lateral earth pressure

\( \varphi \) angular coordinate

\( \chi_0 \) curvature of the pipe at the reference measurement

\( \chi_1 \) curvature of the pipe at the subsequent measurement

\( \Omega \) ovalization value

\( \Omega_0 \) reference ovalization value

\( \Omega_1 \) subsequent ovalization value