NOVEL APPLICATIONS OF DISTRIBUTED FIBER-OPTIC SENSING IN GEOTECHNICAL ENGINEERING

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<td><strong>A</strong></td>
<td>[m²]</td>
<td>Cylindrical area of contact surface (shear area)</td>
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<tr>
<td><strong>A_c, A_f, A_p, A_s</strong></td>
<td>[m²]</td>
<td>Cross sections (cable, fiber, protection, steel)</td>
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<tr>
<td><strong>C_ε</strong></td>
<td>[MHz/%]</td>
<td>Strain coefficient</td>
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<td><strong>C_T</strong></td>
<td>[MHz/°C]</td>
<td>Temperature coefficient</td>
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<tr>
<td><strong>C_cd</strong></td>
<td>[-]</td>
<td>Coefficient of curvature</td>
</tr>
<tr>
<td><strong>C_ud</strong></td>
<td>[-]</td>
<td>Coefficient of uniformity</td>
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<td>[-]</td>
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<td><strong>K_0</strong></td>
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<td><strong>L_{sl}</strong></td>
<td>[m]</td>
<td>Interface slippage length</td>
</tr>
<tr>
<td><strong>L_j</strong></td>
<td>[-]</td>
<td>Point at left boundary of truncated section</td>
</tr>
<tr>
<td><strong>M_s, M_E</strong></td>
<td>[MPa]</td>
<td>Constrained modulus</td>
</tr>
<tr>
<td><strong>P</strong></td>
<td>[kN]</td>
<td>Force (pull, pullout)</td>
</tr>
<tr>
<td><strong>P_j</strong></td>
<td>[-]</td>
<td>Fixation point</td>
</tr>
<tr>
<td><strong>R</strong></td>
<td>[m]</td>
<td>Sand cylinder diameter</td>
</tr>
<tr>
<td><strong>R_a</strong></td>
<td>[kN]</td>
<td>External ultimate resistance</td>
</tr>
<tr>
<td><strong>R_i</strong></td>
<td>[kN]</td>
<td>Internal ultimate resistance</td>
</tr>
<tr>
<td><strong>R_j</strong></td>
<td>[-]</td>
<td>Point at right boundary of truncated section</td>
</tr>
<tr>
<td><strong>T</strong></td>
<td>[°C]</td>
<td>Temperature</td>
</tr>
<tr>
<td><strong>T_0</strong></td>
<td>[°C]</td>
<td>Reference temperature</td>
</tr>
</tbody>
</table>
**a** [m]  Cable diameter  
**c** [m/s]  Speed of light  
**c_k** [-]  Creep rate  
**c_{krit}** [mm]  Critical creep rate  
**c_k'** [kPa]  Characteristic value of effective cohesion  
**c_0** [m]  Initial fiber length along one coil  
**d_{10}, d_{30}, d_{60}** [mm]  Sand grain size parameters  
**d_g** [m]  Diameter of the grout body  
**dx** [m]  Incremental length  
**f_{eff}** [-]  Efficiency factor  
**f_p** [MPa]  Proportionality limit strength  
**f_u** [MPa]  Ultimate tensile strength  
**f_y** [MPa]  Yield strength  
**g**  Weighing function  
**h_0** [m]  Initial coil height  
**k** [-]  Load step  
**l_{fr}** [m]  Free anchor length  
**l_v** [m]  Fixed anchor length  
**l_{ve}** [m]  Apparent fixed anchor length  
**n** [-]  Index of refraction  
**p'** [kPa]  Mean effective stress  
**r_0** [m]  Initial radius of the tendon  
**s** [-]  Standard deviation  
**s_j** [m]  Gauge in point fixation  
**s_p** [-]  Standard deviation (in percent of applied strain)  
**t** [s]  Time  
**\tan(\delta)\)** [-]  Apparent friction coefficient  
**w** [m]  Spatial resolution  
**x** [m]  Along sensor (x-axis)
\( \alpha^2 \) [1/m\(^2\)] Parameter for analytical model
\( \beta \) [kN²/m\(^3\)] Parameter for analytical model
\( \gamma_c \) [kN/m\(^3\)] Unit weight of soil
\( \gamma \) [-] Shear strain of soil
\( \delta \) Delta function
\( \delta \) [m] Displacement, relative displacement
\( \overline{\delta} \) [°] Friction angle soil-grout
\( \varepsilon \) [% or \( \mu \varepsilon \)] Strain (reference, true, average, cable, optically measured)
\( \varepsilon_{SLimit} \) [% or \( \mu \varepsilon \)] Strain step limit
\( \varepsilon_j \) [% or \( \mu \varepsilon \)] Strain in the gauge
\( \varepsilon_{F,d} \) [% or \( \mu \varepsilon \)] Strain at fiber breaking
\( \varepsilon_{cm} \) [\( \mu \varepsilon \)] Maximum tensile strain allowing for optical measurements
\( \varepsilon_{cf} \) [\( \mu \varepsilon \)] Tensile strain at cable failure
\( \varepsilon_{cs} \) [\( \mu \varepsilon \)] Strain limit, above which slippage occurs
\( \lambda_0 \) [m] Wavelength of initial light
\( \mu \varepsilon \) [m/m] Microstrain (1 \( \mu \varepsilon = 0.0001\% \))
\( v \) [m/s] Group velocity
\( \nu \) [-] Poisson's ratio
\( v_A \) [m/s] Acoustic wave velocity
\( v_B \) [GHz] Brillouin frequency shift
\( v_{B0} \) [GHz] Reference Brillouin frequency shift
\( \rho_D \) [Mg/m\(^3\)] Dry density
\( \rho_{Dmax} \) [Mg/m\(^3\)] Maximum index density
\( \rho_{Dmin} \) [Mg/m\(^3\)] Minimum index density
\( \sigma \) [kPa] Stress (normal, vertical, axial, average)
\( \sigma_{xcf} \) [kPa] Tensile strength at cable failure
\( \tau \) [kPa] Shear stress (peak, average, residual)
\( \phi' \) [°] Peak angle of shear resistance
\( \phi'_{crit} \), \( \phi'_{r} \) [°] Residual angle of shear resistance
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi'_k$</td>
<td>$[^\circ]$</td>
<td>Characteristic angle of shear resistance</td>
</tr>
<tr>
<td>$\psi_{max}$</td>
<td>$[^\circ]$</td>
<td>Peak angle of dilation</td>
</tr>
<tr>
<td>$\omega$</td>
<td>[m]</td>
<td>Width of the weighing function / process zone</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>[-]</td>
<td>Increment</td>
</tr>
<tr>
<td>$\Delta c$</td>
<td>[m]</td>
<td>Length change of the coil</td>
</tr>
<tr>
<td>$\Delta \delta_{hopt}$, $\Delta P_{hpopl}$</td>
<td></td>
<td>Measurement error</td>
</tr>
<tr>
<td>$\Delta \varepsilon_{opt}$</td>
<td>[s]</td>
<td>Pulse width of the initial light</td>
</tr>
<tr>
<td>$\Delta x$</td>
<td>[m]</td>
<td>Sampling interval</td>
</tr>
<tr>
<td>$\Delta x_F$, $\Delta x_{j+1}$</td>
<td>[mm]</td>
<td>Cable fixation point displacement</td>
</tr>
<tr>
<td>$\Delta L_s$</td>
<td>[m]</td>
<td>Slippage progression</td>
</tr>
<tr>
<td>$\Theta$</td>
<td>$[^\circ]$</td>
<td>Direction of the light ray</td>
</tr>
<tr>
<td>$\Theta_c$</td>
<td>$[^\circ]$</td>
<td>Critical angle of light ray</td>
</tr>
</tbody>
</table>

**Abbreviations:**
- APC: Angled Physical Contact connector
- BDG-DS: Brillouin Dynamic Grating
- BEDS: Brillouin Echo Distributed Sensing
- BOCDA: Brillouin Optical Correlation Domain Analysis
- BOTDA: Brillouin Optical Time Domain Analysis
- BOTDR: Brillouin Optical Time Domain Reflectometry
- COST: European Cooperation in the Field of Scientific and Technical Research
- CP: Convolution product
- CTI: Swiss Innovation Promotion Agency
- CW: Continuous Wave
- DITEST: Distributed Temperature & Strain Testing Unit
- EMPA: Swiss Federal Laboratories for Materials Testing and Research
- EPFL: Swiss Federal Institute of Technology, Lausanne
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>ETHZ</td>
<td>Swiss Federal Institute of Technology, Zurich</td>
</tr>
<tr>
<td>FBG</td>
<td>Fiber Bragg Grating</td>
</tr>
<tr>
<td>FC</td>
<td>Ferrule type Connector</td>
</tr>
<tr>
<td>FE</td>
<td>Finite Element</td>
</tr>
<tr>
<td>GFO</td>
<td>Group of Fiber Optics (EPFL)</td>
</tr>
<tr>
<td>IGT</td>
<td>Institute for Geotechnical Engineering (ETHZ)</td>
</tr>
<tr>
<td>LAN</td>
<td>Local Area Network</td>
</tr>
<tr>
<td>OFDR</td>
<td>Optical Frequency Domain Reflectometry</td>
</tr>
<tr>
<td>OTDR</td>
<td>Optical Time Domain Reflectometer</td>
</tr>
<tr>
<td>PA</td>
<td>Polyamide</td>
</tr>
<tr>
<td>PC</td>
<td>Physical Contact connector</td>
</tr>
<tr>
<td>PPP-BOTDA</td>
<td>Pre-Pump-Pulse BOTDA</td>
</tr>
<tr>
<td>PU</td>
<td>Polyurethane</td>
</tr>
<tr>
<td>SOFO</td>
<td>Surveillance d’Ouvrages par Fibres Optiques</td>
</tr>
<tr>
<td>SP</td>
<td>Single measurement Point pick / Sand Poorly-graded</td>
</tr>
<tr>
<td>TAM</td>
<td>Truncated Average Method</td>
</tr>
<tr>
<td>USCS</td>
<td>Unified Soil Classification System</td>
</tr>
<tr>
<td>3-D</td>
<td>3-Dimensional</td>
</tr>
</tbody>
</table>
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Abstract

In the last two decades, Brillouin distributed fiber-optic sensing has became a widely accepted, mature technology. On the other hand, geotechnical monitoring applications of this technology are still rare, as the fragile fiber-optic and the harsh soil environment are a difficult combination. Additionally, due to high uncertainties in soil behavior, deeper understanding of geomechanical principles is necessary in order to achieve meaningful results when using these sensors.

In this study, novel applications of distributed fiber-optic sensing in geotechnical engineering were identified, developed, implemented and evaluated. Firstly, one-dimensional structures were considered:

- Strain distribution along a soil-embedded cable during pullout;
- Strain distribution along a monitoring ground anchor during pullout.

As a result, new insight into the progressive failure phenomenon was achieved by documenting the phenomenon of residual shear stress increase with increasing pullout load. This phenomenon is explained in a conceptual analytical model.

The successful implementation of the technology to one-dimensional structures inspired an attempt to apply the sensors in two- and three-dimensional problems:

- Road-embedded sensor for landslide boundary evaluation;
- Soil-embedded sensor for landslide boundary evaluation;
- Borehole-embedded sensor for landslide boundary evaluation.

For the ongoing landslide research and monitoring in St. Moritz, Switzerland, new understanding of the landslide mechanisms in the Brattas and Laret areas was achieved. The road-embedded sensor at the Brattas site detected an additional shear zone, which was later confirmed by a water pipe breakage that occurred at exactly the same location. The soil-embedded sensor at the
Laret site confirmed seasonal patterns of the surface displacement in a moving soil mass independently observed in inclinometer measurements.

To facilitate fiber-optic sensing for the above applications, significant advances in the technology, the sensors and the data interpretation were necessary:

- Spatial resolution of the Brillouin sensing technology had to be improved significantly. This was achieved by facilitating and testing the development of Brillouin Echo Distributed Sensing;

- Elaborate laboratory testing of the sensors and the sensing system led to the development and improvement of new commercial strain sensing cables. In addition, sensor integration techniques were developed and successfully applied;

- Options of improving the data interpretation had to be evaluated and applied.

The present study describes in detail the development and progress of these novel geotechnical monitoring applications at the IGT of ETH Zurich during the last 5 years.
Zusammenfassung


In der vorliegenden Studie wurden neue Anwendungen der kontinuierlichen Glasfasersensorik in der geotechnischen Überwachung erörtert, entwickelt, eingebaut und evaluiert. In einem ersten Schritt wurden eindimensionale Strukturen geprüft:

- Dehnungsverteilung entlang eines im Boden eingebauten Kabels während dem Herausziehen;
- Dehnungsverteilung entlang eines Bodenankers während dem Herausziehen.


Die erfolgreiche Anwendung der Technologie in eindimensionalen Strukturen motivierte dazu, die Sensoren auch für mehrdimensionale Fragestellungen einzusetzen:

- Instrumentierte Strasse zur Evaluierung einer Rutschhanggrenze;
- Direkt im Baugrund eingebautes „Mikro-Anker“ - Sensorsystem zum Messen von Bodenverschiebungen;
Instrumentiertes Bohrloch zur Evaluierung einer Rutschhang-Gleitfläche.


Um die oben erwähnten Anwendungen zu ermöglichen, waren Fortschritte in der Technologie, den Sensoren und der Dateninterpretation notwendig:

- Die längenverteilte Auflösung des Messsystems musste verbessert werden. Dazu wurde die an der EPFL entwickelte Brillouin Echo Distributed Sensing Technologie ein erstes Mal erfolgreich im geotechnischen Labor eingesetzt;

- Ausführliche Versuche mit Sensoren und Sensorsystemen ermöglichten die Entwicklung von neuen, verbesserten, kommerziellen Sensorkabeln. Dabei wurde auch die Sensorintegration weiter vorangetrieben;

- Optionen zur Dateninterpretation wurden evaluiert und angewandt.

Die vorliegende Arbeit beschreibt im Detail die Entwicklung und den Fortschritt mit neuen geotechnischen Anwendungen der kontinuierlichen Glasfasersensorik am IGT (ETH Zürich) über die letzten 5 Jahre.
Preface

Introduction

Distributed fiber-optic strain sensors are offering new possibilities in the field of geotechnical monitoring. By integrating a single fiber-optic cable into soil or a structure, an unprecedented amount of accurate, spatially resolved data can be obtained. Current commercially available technology allows for strain measurements in the microstrain range with a spatial resolution of 1 m along a 30 km long fiber. This means that thousands of “strain gauges” along a single cable connected to structures, embedded in soil or grouted into boreholes for example, can provide information about the current state of the object under supervision. The objects can include geological and civil structures, such as a construction site, a tunnel, a landslide prone area, or a pipeline. It is evident, that such a technology implies a benefit for placing fiber-optic cables anywhere possible on construction sites and in the green field. At first glance, monitoring with distributed fiber-optic sensors seems to require only the measurement unit and a relatively inexpensive cable. However, this is not the case, especially for geotechnical monitoring applications where the soil mechanics is a critical aspect for consideration.

In recent years, the use of fiber-optic sensors for structural health monitoring has rapidly accelerated with commercial services offered by several companies around the world. On the other hand, geotechnical applications are still rare, as the fragile fiber-optic and the harsh soil environment are factors difficult to reconcile. For example, the fiber-optic sensor integrated into a pile, in Figure 1, is subjected to various hazards during integration, such as mud, water, friction along rock outcrops, falling rocks etc. In addition, soil behavior is not always well understood and, thus, such sensors need to be carefully placed in order to achieve a meaningful result.
Objectives

The original objective of this work was to gain an improved understanding of the progressive failures in soil-structure interaction for ground anchors and cables. In the course of the research, it became evident that significant progress could only be achieved by applying a novel fiber-optic, strain-sensing technology capable of providing an unprecedented amount of strain data along the structure. Deploying this technology for the described application, however, required major adjustments which resulted in novel contributions to the technology itself, the development of new sensors, and improved data interpretation techniques.

Successful implementation of the technology to one-dimensional problems established new, challenging objectives: the application to two- and three-dimensional problems, such as the identification and monitoring of the landslide boundaries. The second set of objectives, rooted in the goal to better understand the landslide phenomenon, resulted in new potentials for geotechnical monitoring applications.

Main achievements and structure of the study

The study is divided into four parts, starting with an extensive introduction into fiber-optic strain sensor technology (Part I). Part II covers the application of the technology to one-dimensional structures. Part III is devoted to the
application of monitoring two- and three-dimensional problems. Part IV contains conclusions obtained from the study (Figure 2).

Figure 2: Four-part structure of the study.

In detail:

Part I describes the fiber-optic technology, leading from the very basic elements of fiber optics to the recent advances in fiber-optic strain sensing technology that have been made in this study. These advances involved interpretation techniques, cable development (in partnership with a cable manufacturer), sensor integration techniques, and the first application of a novel sensing technology (BEDS), which improves the spatial resolution by a factor of twenty.

Part II focuses on the soil-structure interaction phenomenon. Laboratory and field pullout tests of long, one-dimensional structures, such as sand-embedded cables and ground anchors, have been performed while monitoring strain along the structure during pullout via fiber-optic sensors. The resulting high resolution data showed an increase in residual shear stress, a phenomenon neglected in other studies. A conceptual analytical model is
proposed, which can explain this phenomenon and is validated against the experimental data.

Part III describes recent landslide monitoring applications of fiber-optic strain sensors. Presented are several large-scale, long-term field tests for locating landslide boundaries including a road-embedded sensor, a soil-embedded sensor system, and the reactivation of old inclinometer pipes.

Part IV summarizes the main findings of the study and provides ideas for future research on fiber-optic strain sensors and new applications in the field of geotechnical engineering.
PART I

ADVANCES OF FIBER-OPTIC STRAIN SENSING FOR GEOTECHNICAL APPLICATIONS
1 State of the Art in Fiber-Optic Monitoring

The focus in this study is the application of fiber-optic sensors in geotechnical monitoring and this study is the first one of its kind at the Institute of Geotechnical Engineering (IGT) of ETHZ. Therefore, an extensive introduction to fiber-optic strain sensors is given in this section. In-depth understanding of the fiber optics, the associated technologies, and the working principles for sensor applications is necessary to achieve meaningful results. In addition, this section is intended to provide a solid basis for further research at the IGT. The literature review summarizes the state of the art of fiber-optic sensors in geotechnical applications and identifies open questions that suggest the possibilities for further research.

1.1 Basics of Fiber Optics

1.1.1 Fundamental physical principles of light

The necessary fundamental physical principles are summarized below.

Light

Modern physics represents light as either electromagnetic waves or particle-like photons. Light travels in free space at approximately \( c \approx 3 \times 10^8 \text{m/s} \). The direction of travel is modified (reflected, refracted, etc.) at the surfaces, or at the interface, between two different mediums.

The electromagnetic-photon spectrum

The properties of the radiation (photons or waves travelling at the speed of light) can be measured by the frequency (wavelength) of the waves or by the photon energy of its incident photons. The electromagnetic-photon spectrum is the arrangement of the frequencies, wavelength, and energy (Figure 3): visible light is only a very small portion of the spectrum.
The index of refraction, \( n \), of a material is calculated as the ratio of the nominal value of the velocity of a beam (i.e. the speed of light in vacuum, \( c \)) to the apparent phase velocity, \( v \), in a homogeneous material:

\[
    n = \frac{c}{v} \tag{1.1}
\]

Given that refraction can only occur in materials and not in vacuum, the index is always greater than 1. For example, glass has a refractive index of around 1.5 and water about 1.33, while air is very nearly unity (Hecht, 2002). The refractive index also depends on the optical frequency.

**Reflection**

A light ray traveling through a transparent medium and striking the interface of another transparent medium of different density may be partially or totally
reflected at the interface (see incident light ray in Figure 4a). This phenomenon is called reflection.

**Refraction, Snell's Law and Fresnel reflection**

Instead of reflecting, the light ray above may also change its direction (θ) at the interface between the two mediums of refractive index \( n_x \) and \( n_y \) (see incident light ray in Figure 4b). This direction change can be calculated by the Law of Refraction, also known as Snell's Law:

\[
 n_x \cdot \cos(\Theta_i) = n_y \cdot \cos(\Theta_r) 
\]

(1.2)

Still, part of the ray is reflected at the interface. This is referred to as Fresnel reflection (generally less than 4% of the incident light).

![Figure 4: (a) Reflection and; (b) refraction of light for \( n_y > n_x \).](image)

**Critical angle**

When light travels from a medium with a relatively high refractive index to a medium with a lower refractive index, increasing the incident angle (\( \Theta_i \)) will result in an even greater increase in the angle of refraction (\( \Theta_r \)). At a certain incident angle, \( \Theta_i \), the angle of refraction \( \Theta_r \) equals 90° (light ray b in Figure 5). This is known as the “critical angle”, or \( \Theta_c \).
\[ \Theta_c = \arcsin \left( \frac{n_y}{n_x} \right) \] (1.3)

**Internal reflection**

For rays that have incident angles greater than the critical angle, the ray is said to be totally internally reflected.

![Diagram of internal reflection](image)

**Figure 5:** (a) Refraction; (b) critical angle and; (c) reflection of light in a medium setup similar to optical fibers \((n_y > n_x)\).

### 1.1.2 Scattering

Scattering occurs due to interaction of a light pulse with the medium particles and acoustic waves. In a dense and perfectly homogeneous medium, light scatters only in the forward direction (Niklès, 1997). Imperfections in the material structure, such as dust, flaws, and other impurities (Hecht, 2002) locally modify the optical properties of the medium. This modification results in a different configuration of the scattered waves with light also scattering laterally.


**Backscatter**

The scattered light that is transmitted opposite to the propagation direction is called “backscatter”. This is an important effect used in fiber-optic sensing.

**Rayleigh scattering**

Rayleigh scattering refers to scattering phenomena involving particles smaller than the incident wavelength; this normally means individual atoms and molecules (Wuilpart, 2009). Due to irregularities in the molecular structure of a material, Rayleigh scattering occurs in any direction. Rayleigh scattering is the major source of loss in optical fibers and this effect, is e.g. used for optical time-domain reflectometry (OTDR) measurements (see 1.1.6).

**Brillouin and Raman scattering**

Dynamic inhomogeneities in a material can give rise to still other types of scattering. For example, thermally influenced molecular vibrations cause Raman scattering and thermally excited acoustic waves cause Brillouin scattering (for details, see 1.2.1).

1.1.3 The optical fiber

The optical fiber guides signals (in the form of light) from one location to another. These signals are digital pulses or continuously modulated analog streams of light representing information. The advantage of optical fibers, over metallic wires or microwave frequencies in the air, is that they can transport more information over longer distances in less time (Awad, 2001). Additionally, the signal is not affected by the external interference from electrostatic and electromagnetic radiation.

**Fiber design**

A fiber consists of the core surrounded by cladding and a buffer coating. A cross section of a single mode fiber is shown in Figure 6. The trapping of the light inside the core is due to the core material having a higher refractive index, \( n \), than the cladding material. Therefore, the light travelling in the core
is reflected totally at the interface between the core and cladding, which are usually made of fused silica glass. The buffer coating adds strength to the fiber and protects the glass from environmental damage. It is a thin protective coating typically made of ultraviolet cured acrylate (a plastic material). Most coatings can be stripped mechanically and must be removed for splicing. All-plastic fibers for special purposes, such as large strains, are available. Fibers are differentiated as single or multi mode, depending on their light transport characteristics.

![Figure 6: Cross section of a single mode fiber with the corresponding diameter given.](image)

**Single mode fiber**

The single mode fiber (Figure 6) has a core diameter of 4 µm to 10 µm. Due to this small dimension, in comparison with the wavelength of the light travelling through the fiber, the light can only travel in a single path or mode. Single mode fibers are used for telecommunications and typically operate at 1300 nm and 1550 nm wavelength. However, the small core diameter can complicate coupling fibers together. All projects in this study have been carried out using single mode fibers.

**Multi mode fiber**

The multi mode fiber has a larger core diameter (25 µm to 150 µm) and, therefore, light can travel in many different paths or modes. The fibers are further differentiated into step-index multi mode fibers and graded-index multi mode fibers. These types of fibers are usually implemented in local area networks (LAN) and operate at 850 nm or 1300 nm wavelength.
1.1.4 Connecting fibers

To connect two fibers, joints are necessary. The connection must be mechanically strong enough to withstand pulling force and bending moments, while maximizing transmission of light. Accordingly, the fiber cores must fit together exactly. Connection types can either be permanent (splice) or temporary (connector).

**Stripping and cleaving**

The preparation for a permanent fiber joint includes stripping and cleaving. Stripping refers to removing all the layers above the core and cladding. A stripping tool of correct size (diameter) must be used to remove the 250 µm primary coating. Chemicals, such as methylene chloride, can also be used for the stripping process; however they may harm the cladding for long term applications. Subsequently, the fiber is broken in an exact 90° angle to the fiber axis by using a specialized tool called a cleaver.

**Splices**

Permanent joints are done by splicing two fibers together, either mechanically or by fusion. Mechanical splicing, however, does not meet the required splice accuracy for strain sensing equipment. Fusion splicing is achieved by an electric arc ionizing the space between the aligned fibers which eliminates the air and heats the fibers to a temperature of 1100°C. The softened fiber ends are then moved together with precise alignment to form the fusion joint. A perfect fusion splice results in a single fiber, with no indication that this is the result of two joined fibers. The tensile strength of a fusion splice is comparable to that of the original fiber. A protective device, such as a mechanical clip or a heat-shrink sleeve, is provided to replace the removed section of coating. For even better protection, a spliced cable is sometimes integrated into a splice case (Figure 7).
Figure 7: Different protections of spliced fibers: (a) Mechanical clip; (b) heat-shrink sleeve; (c) and (d) splice case.

Connectors

For temporary connections, a wide variety of connector types are available. The main differences are their dimensions and the methods of mechanical coupling. The connector is produced by gluing a fiber into a ferrule. Then, the end is cut and polished to be even with the face of the ferrule. A connector can be polished flat, with a slightly rounded dome for “physical contact” (PC) or an angled face “angled PC” (APC). The PC reduces the back reflection caused by air gaps between the fiber ends, while the angle (usually 8°) minimizes the back reflections at the point of connection. PC and APC connectors cannot be mated with each other, as this would crack the surface of the connectors. As a precaution, different colors distinguish connector types.

Generally, for a particular system or technology, the involved parties agree on using the same types of connectors. The connectors used for the equipment in this study are rigid ferrule connectors (FC) and the well protected E2000 connectors (Figure 8).
Figure 8: Fiber-optic connectors: (a) FC/APC; (b) E2000; (c) FC/PC and; (d) FC/PC.

**Pigtails**

As installation of a connector onto single mode fiber is a time-consuming process requiring special instrumentation and laboratory conditions, the most common method is to splice the fiber to another fiber on which a connector is already installed (a so-called pigtail). In this way, high quality can be assured, as connectors themselves are fabricated by a well-defined process under factory conditions.

**Connectors or splices?**

In general, splices offer a lower attenuation than connectors and are more economical. However, for certain tasks (e.g. within a strain sensing system), only temporary connections are desired and a connector offers this flexibility.

**1.1.5 Signal intensity loss and transmission interruption**

Not all optical networks, including fibers for monitoring, transmit the signal as smoothly as intended. Especially in rough conditions, such as the ones encountered in a geotechnical monitoring environment, this degradation can quickly translate into a serious problem. On the other hand, the advantage of controllable signal intensity loss is necessary for developing sensing instrumentation.
Fiber attenuation

The effect of loss in signal intensity along the fiber is called attenuation. In optical fibers, Rayleigh scattering is the major source of attenuation (Tur, 2009). As scattering is a function of wavelength, proportional to the inverse fourth power of the wavelength of the light (Wuilpart, 2009), a longer wavelength results in less scattering (Awad, 2001). Other causes of attenuation in a fiber include absorption, reflection, diffusion, dispersion, and bending and micro-bending. And of course, splice loss and connector loss add to the attenuation as well.

Attenuation in fiber optics is expressed in the logarithmic decibel scale (typically in decibels per kilometer dB/km). It describes the ratio of outgoing to incoming intensities. A typical attenuation rate for a single mode fiber is between 0.1 dB/km to 1 dB/km (Bailey & Wright, 2003).

Splice loss

Signal loss in a splice can result from lateral and angular misalignment of the fiber cores, variation in core diameters, inadvertent air spaces between the fibers, numerical aperture mismatch, and contamination caused by dirt or dust. Additionally, the unevenness in density or material composition in the fused section becomes large, increasing the loss due to Rayleigh scattering. Typical splice loss for single mode fibers is in the 0 dB to 0.15 dB range (Highhouse, 2001).

Connector loss

Signal loss from a connector is typically due to unclean connector faces, spacing between the connectors, and angular fiber misalignment from a connector mismatch. In addition, lateral misalignment, differences in core diameters, and numerical aperture mismatches can create loss. The typical single mode connector losses are from 0.1 dB to 1 dB (Highhouse, 2001).
**Interruption of signal transmission**

The worst case scenario for an optical fiber system is a complete signal transmission failure. Possible reasons include a fiber break, a splice problem, a connector problem, a localized high attenuation (often from bending or local stress concentrations), and a distributed high attenuation (microbending and distributed stresses). Localization of the fault is straightforward (see next section), however the appropriate steps to remedy this situation cannot always be taken due to limited access. For example, a strain sensing fiber glued inside a steel tendon of a ground anchor can simply not be accessed once in place. If this fiber has a fault, sensing will not be possible beyond the fault, or in some cases (as in a loop configuration), the entire sensor system will be lost.

**1.1.6 Fiber-optic instrumentation**

For the localization of the fault and the quantification of attenuation, several methods are available: the visual fault locator, the fiber-optic power meter, optical time domain reflectometry (OTDR), and the inspection microscope.

**Visual fault locator**

The visual fault locator provides the most basic fiber-optic testing by injecting a light from a visible source, such as a diode laser. Then, the fiber can be visually traced and the interruption located (Figure 9). This method is helpful as a quick, on-site troubleshooting technique.

*Figure 9: Light travelling from the source to the fiber end.*
**Optical power meter**

Measurement of cable attenuation is another common test of a fiber-optic system. This test is conducted by connecting a power source emitting a signal at a specific wavelength to one end of a fiber and a power meter is placed at the other end. The power meter measures the average optical power emanating from the fiber. With a calibrated source power, the attenuation can then be specified.

**Optical Time Domain Reflectometry (OTDR)**

OTDR, the most commonly used and best recognized method of analyzing a fiber-optic system, was invented by Barnoski & Jensen (1976). The principle of OTDR is to first send a short light pulse into the fiber and then subsequently measure the Rayleigh backscatter energy (due to impurities, cracks, breaks, connectors and splices). The pulse is attenuated on the outbound leg and the backscattered light is attenuated on the inbound leg, therefore, the returned signal is a function of twice the fiber loss. By time domain analysis, the distance to the reflection can be calculated.

For the tests performed in this work, the AQ7270 OTDR from Yokogawa has been used (Figure 10a). It allows for measurements at 1310 nm and 1550 nm wavelength with an optical allowance of 34 dB. The interval, at which data values are acquired (sampling resolution) can be as low as 5 cm and the event dead zone is 0.8 m (Yokogawa, 2007). A typical output of an OTDR measurement is shown in Figure 10b.

![Figure 10: (a) The Yokogawa OTDR used in this study and; (b) a typical OTDR output.](image-url)
Microscope

The inspection of cleaved fiber ends and polished connectors is performed with a fiber microscope (Figure 11). The microscope shows if there are any polishing defects, dirt, or scratches so that the exchange of the system parts, if necessary, can be made.

Figure 11: A fiber microscope.

1.2 Distributed Brillouin Sensing

Scattering effects (1.1.2) are the phenomena on which distributed fiber-optic sensing technologies are based upon. Special interest has been given lately to Brillouin scattered light, whose frequency depends on the longitudinal strain and the temperature in the fiber. Brillouin sensing forms the foundation from which the novel applications to geotechnical engineering are built on in this study.

1.2.1 Principles

Frequency shift due to Brillouin and Raman scattering

The main scattering effects of light are Rayleigh, Raman and Brillouin scattering. Raman scattering occurs due to thermally influenced molecular vibrations, while Brillouin scattering occurs due to thermally excited acoustic waves in the GHz range present in the silica fiber. A small fraction of the light is backscattered, containing light at the original wavelength (Rayleigh component) and light at shifted wavelengths (Raman and Brillouin
components). Figure 12 shows the main scattered wavelength components for a standard optical fiber, where $\lambda_0$ represents the wavelength of the incident light. The scattered components appear symmetrically on both sides at higher (Stokes components) and lower (anti-Stokes components) wavelengths.

The positions of the Raman peaks are fixed, but the intensity of the anti-Stokes component is temperature dependent while the intensity of the Stokes component is unaffected by temperature changes. Consequently, by comparing the two Raman peaks, the temperature information at the scattering location can be calculated.

The positions of the Brillouin peaks depend on the temperature and strain at the scattering location. Thus, the position is shifted if the temperature or strain changes. This is called the Brillouin frequency shift, $v_B$, which is a function of the refractive index of the fiber, $n$, the acoustic wave velocity, $v_a$, in the fiber ($\sim 5800$ m/s), and the wavelength of the initial light, $\lambda_0$, (Horiguchi et al., 1989).

$$v_B = \frac{2 \cdot n \cdot v_a}{\lambda_0}$$  \hspace{1cm} (1.4)

Figure 12: Spectrum of light scattered in an optical fiber (Thévenaz & Niklès, 2007).

The effect of temperature and strain dependency comes from the fact that the acoustic wave velocity depends on material density. And material density again depends on temperature (thermal expansion) and deformation (strain). Since these are all linear relations, strain and temperature changes cause
also the Brillouin frequency shift to change linearly according to the following relation (Horiguchi et al., 1995):

\[ v_B(T, \varepsilon) = C_\varepsilon \cdot (\varepsilon - \varepsilon_0) + C_T \cdot (T - T_0) + v_{B0}(T_0, \varepsilon_0) \tag{1.5} \]

where \( C_\varepsilon \) and \( C_T \) are the strain and temperature coefficients respectively, \( \varepsilon \) is the present strain, and \( T \) the current temperature. \( \varepsilon_0 \) and \( T_0 \) are the strain and temperature that correspond to the reference Brillouin frequency shift, \( v_{B0} \), respectively. Temperature and strain changes may also be expressed separately as:

\[ v_B(T_0, \varepsilon) = C_\varepsilon \cdot (\varepsilon - \varepsilon_0) + v_{B0}(T_0, \varepsilon_0) \tag{1.6} \]

\[ v_B(T, \varepsilon_0) = C_T \cdot (T - T_0) + v_{B0}(T_0, \varepsilon_0) \tag{1.7} \]

For standard telecommunication optical fibers, the strain coefficient, \( C_\varepsilon \), is about 500 MHz/% and the temperature coefficient, \( C_T \), is roughly 1 MHz/°C. The coefficients depend on the wavelength of the incident light. An example of the Brillouin frequency shift for strain and temperature change is displayed in Figure 13.

Due to the exponential decay of the phonons, the Brillouin scattered spectrum takes the form of a Lorentzian curve. The frequency corresponding to the peak of this curve is shifted by approximately 11 GHz from the incident light frequency at a wavelength of 1.55 µm (Shimizu et al., 1993) and about 11.5 GHz to 13 GHz at 1.3 µm (Niklès et al., 1994).

Figure 13. Brillouin frequency shift (after Thévenaz & Niklès, 2007).
Time-domain analysis

The distance, $x_i$, from the position where a pulsed light is launched to the position where the scattered light is generated can be determined using time-domain analysis:

$$x_i = \frac{c \cdot \Delta t}{2 \cdot n} \tag{1.8}$$

where $c$ is the velocity of light in vacuum, $n$ is the refractive index, and $\Delta t$ is the time interval between launching the pulse and receiving the backscatter.

3-D Brillouin scattering spectrum

In order to obtain a Brillouin frequency shift spectrum for any location along a fiber, a pulse of light is first launched into the fiber. Subsequently, the power of spontaneous Brillouin backscatter is measured by means of heterodyne detection and complemented with time-domain analysis. The scanning frequency of the incident light is then changed slightly and the same measurements are repeated for each scanned frequency. Consequently, a 3-D Brillouin spectrum is obtained. Figure 14 shows a frequency scan along a fiber which was subject to localized strain at 120 m. It can be seen that at 120 m, the maximum backscatter is at a different frequency than that along the rest of the fiber. By fitting a Lorentzian curve to the backscatter spectrum and calculating the peak of the Lorentzian curve, the peak power of the Brillouin spectrum is obtained (Smith et al., 1999).

![Figure 14. A 3-D Brillouin scattering spectrum (Niklès et al., 2004) for a fiber experiencing a localized strain.](image-url)
Spatial resolution

Strain data at any sampling point, $x_i$, along the fiber is calculated based on the frequency shift of the backscattered pulse. As this pulse is not infinitely short, but has a minimum width, the measured frequency shift corresponds to a fiber section in the vicinity of that point and not only to the point $x_i$ (Figure 15). The length of this section is called the spatial resolution, $w$, and is dictated by the pulse width of the initial light, $\Delta \tau$ (Horiguchi et al., 1995).

$$w = \frac{v \cdot \Delta \tau}{2}$$  \hspace{1cm} (1.9)

where $v$ is the light velocity (depending on $c$ and $n$ as stated in Eq. 1.1). Since the group velocity is $v \approx 2 \cdot 10^8 m/s$ in a silica fiber, the rule of thumb is that 10 ns corresponds to 1 m spatial resolution.

The spatial resolution determines the ability of Brillouin scattering based fiber-optic sensors to locate a strained segment. Higher spatial resolution can be obtained by narrowing the pulse width of the incident light. However, as the pulse width is further narrowed, the line width of the Brillouin frequency shift spectrum becomes wider, and, after it exceeds that of acoustic phonons, the accuracy of the strain measurement deteriorates abruptly. Therefore, minimum width of the light pulse is limited to about 10 ns, which limits spatial resolution to 1 m (e.g. Smith et al., 1999; Ravet et al., 2005; Zhang & Wu, 2008). This commonly accepted opinion is currently challenged, since it was questioned by Bao et al. (1999) when the authors observed an unexpected narrowing of the Brillouin gain spectrum with shorter than 10 ns pulses (for technologies with spatial resolution less than 1 m, see 1.2.2 BEDS).

![Figure 15: Spatial resolution and sampling interval.](image-url)
Sampling interval

Due to measurement device limitations in resolving the time of flight (time-domain), test points $x_i$ along the fiber cannot be continuously aligned behind each other. Accordingly, only discrete locations are measured. The distance between two subsequent test points $x_i$ and $x_{i+1}$, is the sampling interval, $\Delta x$ (Figure 15). Typical minimum sampling intervals are from 5 cm to 10 cm. In the literature, various terms are used to refer to the sampling interval, such as measurement interval, distance resolution, distance precision, and readout resolution.

1.2.2 Distributed Brillouin sensing technologies

The use of Brillouin scattering for distributed optical fiber strain and temperature measurements was first demonstrated in 1989 and has since evolved towards high performance instrumentation that can achieve 1 m spatial resolution over long fiber lengths up to 30 km with absolute strain measurements in the range of a few microstrains ($\mu\varepsilon$). At present, commercially available Brillouin sensing technology can be divided into two categories: spontaneous Brillouin scattering, also referred to Brillouin Optical Time Domain Reflectometry (BOTDR), and stimulated Brillouin scattering, referred to as Brillouin Optical Time Domain Analysis (BOTDA).

Under laboratory conditions, promising new technologies such as BOCDA, BEDS, and BDG-DS are currently tested. They offer strain and temperature measurements with spatial resolutions in the cm-range. However, no compact user-friendly measurement units are available yet.

- BOCDA: Brillouin Optical Correlation Domain Analysis (Hotate & Hasegawa, 2000; Hotate et Abe, 2005);
- BEDS: Brillouin Echoes Distributed Sensing (Thévenaz & Foaleng, 2008; Foaleng et al., 2009);
- BDG-DS: Brillouin dynamic grating (Song et al., 2009).
Brillouin Optical Time Domain Reflectometry (BOTDR)

The BOTDR technology is based on the detection of the spontaneous scattering created by the interference with an acoustic wave present in the fiber. A BOTDR instrument launches an input pulse from one end into a single mode fiber and observes the Brillouin backscattered light generated by the pulse at the same end of the fiber. Therefore, access to only one end of the fiber is required.

BOTDR measurement units are commercially available from Ando/Yokogawa and Sensornet UK (Sensornet, 2010). Their latest products are shown in Figure 16.

Brillouin Optical Time Domain Analysis (BOTDA)

In a more refined setup, scattering can be actively stimulated by two counter-propagating light waves. This technique has been proposed by Horiguchi & Tateda (1989a) and named as Brillouin optical time-domain analysis. In the basic configuration for BOTDA, in addition to the optical pulse (called the pump), a continuous wave (CW) optical signal (called the probe) is used to probe the Brillouin frequency profile of the fiber. The pulse is launched at \( x = 0 \) and propagates in the \(+x\) direction, while the CW probe is launched at the opposite end of the fiber, \( x = L_F \), and propagates in the \(-x\) direction (Horiguchi & Tateda, 1989b). Stimulation of Brillouin scattering occurs when the frequency difference of the pulse and the CW signal corresponds to the Brillouin frequency shift. The interaction leads to a larger scattering efficiency, resulting in an energy transfer from the pulse to the probe signal thereby amplifying the probe signal (Figure 17). The interaction of the probe with the pump is recorded in the time-domain. The main difficulty is to generate a pump and a probe with a fixed and stable frequency difference (Thévenaz, 2010). As for BOTDR, BOTDA is subject to the same limitations in spatial resolution and access to both fiber ends is required.

BOTDA measurement units are commercially available from Omnisens, Neubrex and OZ Optics. The Omnisens product (DITEST) and the OZ Optics product (Foresight DSTS) in a basic setup implement classical BOTDA.
technology. The Neubrexs sensing system, the Neubrescope, uses a so-called Pre-Pump-Pulse Brillouin Optical Time Domain Analysis (Kishida et al., 2005), abbreviated as PPP-BOTDA. The equipment mentioned is shown in Figure 16. For this study, an Omnisens DITEST STA-202 has been used.

Figure 16: Distributed fiber sensing units commercially available today: BOTDR - Yokogawa AQ8603 (left) and Sensornet DTSS (right); BOTDA - Omnisens DITEST STA-R (left), Neubrexs Neubrescope (middle) and OZ Optics Foresight DSTS (right).

Figure 17: BOTDA principle.

General specifications of measurement units

The typical performance of the selected Brillouin scattering measurement units are given in Table 1. The figures are according to the manufacturer's specification and one must be extremely careful when relying on this
information. E.g. several BOTDA instrument manufacturers claim that the instrument system can achieve spatial resolutions lower than 1 m regardless of the physical limits given by a 10 ns pulse. Of course, data processing techniques might play a role, but nowhere is that clearly stated.

In fact, despite the success of and the growing demand for such systems, there are neither guidelines nor standards available for the specification and characterization of fiber-optic sensing systems (Habel, 2007; Niklès, 2007). However, drafts of such standards are currently discussed as an outcome of a European project (COST Action 299: Optical Fibres for New Challenges Facing the Information Society) and a national project of the “German Association of Engineers” (VDI 2660: “Association for Experimental Stress Analysis”). Final versions of the drafts have been available since October 2009 (Habel et al, 2009). Additionally, a new European project (COST Action TD 1001) started in November 2010 encompasses the aim to formulate standards on sensor characterization and on-site validation (COST, 2010).

For BOTDR, the scattering intensity is between five and six orders of magnitude lower than the input signal. Thus, the detection of low intensity scattered light with a BOTDR measurement unit requires long averaging times (tens of minutes to hours) and long pulses which affect the spatial resolution. This leads to a poor signal-to-noise ratio that has an impact on both the measurement resolution and accuracy. In BOTDA, the optical stimulation leads to a much greater intensity of the scattering mechanism and, thus, an improved signal-to-noise ratio coincides. Compared to BOTDA, the measurement accuracy and time is the main disadvantage of BOTDR. However, BOTDR has the advantage that access to only one fiber end is necessary. This means that if the fiber is broken somewhere, it is still possible to take a measurement.
Table 1: Specifications of commercial Brillouin scattering measurement units.

<table>
<thead>
<tr>
<th>Instrument</th>
<th>AQ8603</th>
<th>Foresight DSTS</th>
<th>DITEST STA-R</th>
<th>Neubrescope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer</td>
<td>Yokogawa, Japan</td>
<td>OZ Optics, Canada</td>
<td>Omnisens, Switzerland</td>
<td>Neubrex, Japan</td>
</tr>
<tr>
<td>Technology</td>
<td>BOTDR</td>
<td>BOTDA</td>
<td>BOTDA</td>
<td>BOTDA</td>
</tr>
<tr>
<td>Sampling interval</td>
<td>0.05 m</td>
<td>0.05 m</td>
<td>0.1 m</td>
<td>0.05 m</td>
</tr>
<tr>
<td>Spatial resolution</td>
<td>1 m to 22 m</td>
<td>0.1 m to 50 m</td>
<td>0.5 m to 20 m</td>
<td>0.1 m to 1 m</td>
</tr>
<tr>
<td>Max. distance</td>
<td>80 km</td>
<td>100 km</td>
<td>50 km</td>
<td>25 km</td>
</tr>
<tr>
<td>Strain -Resolution</td>
<td>1 με</td>
<td>0.1 με</td>
<td>2 με</td>
<td>n/a</td>
</tr>
<tr>
<td>-Accuracy*</td>
<td>40 με</td>
<td>2 με</td>
<td>n/a</td>
<td>7.5 με</td>
</tr>
</tbody>
</table>

* [με = 10^{-4} %]; Resolution usually refers to the readout resolution of the unit (strain measurement resolution). Accuracy on the data sheets refers to the accuracy of the measured strain. The data sheets from the manufacturers usually provide no further details regarding how these two terms are defined.

1.3 Other Fiber-Optic Sensing Technologies

Besides the Brillouin sensing, there are other methods based on various phenomena such as scattering, optical loss and polarization. This results in sensors that can monitor anything from strain to pH values. Many technologies are still under development and are not available outside the scientific community, while others have reached a level of maturity, allowing routine use in a large number of applications (e.g. Fiber Bragg Gratings).
Although only Brillouin based strain sensors have been used in this study, it is the intention to provide the reader with a short overview of other methods as the comprehensive decision to use Brillouin technology required the understanding of the possible alternatives. The overview in this section covers strain and temperature sensing technologies that have already been successfully implemented in geotechnical laboratory and field applications. Not covered are the OTDR-based time-of-flight sensors, which were subjected to extensive research in the 1980’s and early 1990’s, but since then, publications and interest dropped massively (Lyöri, 2007).

### 1.3.1 Classification of strain sensors

Fiber-optic strain sensors can be divided into single and distributed measurement sensors.

- **A single measurement sensor** returns the strain value at a fiber section of gauge length \( w \). The gauge length may vary from a few millimeters to tens of meters depending on the technology applied. Multiplexing of several single measurement sensors leads to quasi-distributed sensors. Most single measurement fiber-optic sensors are based on Fiber Bragg Grating (FBG) and Interferometry (Fabry-Pérot & low coherence).

- **Distributed sensors**, on the other hand, measure strain at hundreds and even thousands of “gauges” of length \( w \). Those “gauges” are continuously aligned along a fiber of up to tens of kilometers of length. The majority of distributed strain sensing technologies are based on Brillouin scattering.

### 1.3.2 Single measurement sensors

**Fiber Bragg Grating (FBG)**

Bragg Gratings are periodic density alterations of the glass in the fiber core, produced by exposing the fiber to intense ultraviolet light. The principle of operation is that light with a wavelength corresponding to the grating period is reflected at the grating, while all other wavelengths pass the grating undisturbed. Since the grating period is strain and temperature dependent, the
spectrum of the reflected light changes with temperature and/or strain changes. As with other sensors, temperature compensation with an available reference grating dedicated to measuring only temperature is necessary. The gauge length of a FBG is about 10 mm, and therefore, it is often used to replace conventional strain gauges. The first in-fiber Bragg grating was demonstrated by Hill et al. (1978). Today, FBG are the most prominent type of fiber-optic strain sensor.

Multiplexing is possible by tuning the grating period to reflect at specific wavelengths. As the gratings have to share a spectrum of light, there is a trade-off between the number of gratings and the dynamic range (strain and temperature variations) of the measurements on each grating. Typically, combining of up to 16 gratings in a single fiber is possible.

A broad selection of FBG sensors are commercially available, ranging from bare fibers to textile-packed fibers and even steel-tube protected fibers. The idea is that protected FBG sensors can be glued or welded onto basically any structural component. Examples of differently packaged FBG sensors are shown in Figure 18a.

Measurement units are also widely available for FBG, offering up to 1 με resolution and 2 με accuracy (e.g. Fibersensing, 2010; Micronoptics, 2010). Acquisition rates in the range of 100 Hz are possible, which allows FBG to be used for dynamic monitoring. An example of a laptop-size measurement unit is shown in Figure 18b.

Figure 18: (a) Differently packaged FBG’s from Fibersensing and Micronoptics; (b) A Bragg Grating measurement unit from Fibersensing.
Interferometric sensors

Interferometric sensors make use of the principle of superposition by combining two separate waves of equal frequency in a way that the result can be analyzed to correlate to the state of the sensor. The two most common sensors, which have been used already for structural and geotechnical monitoring, are the Fabry-Pérot point sensor and the SOFO low-coherence long gauge sensor.

The Fabry-Pérot interferometer consists of a thin glass tube containing two cleaved fibers facing each other. An air cavity of a few microns is left between the two fibers, creating two reflecting boundaries (mirrors) at the air-glass interfaces that backscatter the light. By demodulating the interference of the two waves, the changes in the fiber spacing can be reconstructed (Ball, 2006; Betzler, 2002). Typically, the thin tube that two fibers are attached to is about 10 mm long. This means, Fabry-Pérot sensors are single, point measurement sensors.

The low-coherence system is based on a broadband source with a limited coherence length (Koch & Ulrich, 1991; Inaudi, 1997). This technology is implemented in the widely available commercial SOFO system.

The SOFO sensor consists of a pair of single mode fibers. One fiber (the measurement fiber), is in mechanical contact with a structure, while the other fiber (the reference fiber), is loose. Deformation of the measurement fiber results in a relative change in length between the two fibers, which can be analyzed by interferometry. The SOFO setup uses low-coherence interferometry (Glisic & Inaudi, 2007). SOFO sensors are long gauge sensors offering excellent long term stability, with typical gauge lengths of up to 20 m.

The technology has been developed at the Swiss Federal Institute of Technology in Lausanne (EPFL) and is now commercialized by Smartec (Rocnotest). For practical applications, both fibers are installed in the same pipe.

Interferometric sensors offer up to 2 με resolution and 0.2% strain accuracy (e.g. FISO, 2010; Smartec, 2010). Dynamic monitoring is possible.
Microbending sensors

Microbending sensors make use of the principle that strain induces bending of the fiber and, thus, affects the light power that propagates. This sensor was originally demonstrated by Fields & Cole (1980) and is currently a commercialized product (e.g. by the French company OSMOS).

1.3.3 Distributed measurement sensors

Distributed Raman scattering sensors

As introduced in section 1.2.1, the intensity of the lower Raman scattered peak is temperature dependent, while the intensity of the Raman scattered peak at higher wavelength is unaffected by temperature changes. By measuring the ratio of these intensities, local temperature can be obtained. This effect was first used in 1985 for temperature distribution assessment along a silica-based optical fiber by Dakin et al. (1985).

Raman systems work with multi mode fibers, which are prone to high attenuation and, thus, have a limited measurement range of a few kilometers. Specifications of commercially available Raman temperature sensors are: spatial resolution 1 m, sampling interval 0.5 m, temperature resolution 0.2°C and temperature accuracy around 1°C (e.g. Sensortran, 2010; Kher et al., 2002).

Other distributed sensing methods

It is beyond the scope of this study to provide a state of the art review of the other technologies available. A broad overview of other distributed fiber-optic sensing methods can be found e.g. in Alasaarela et al. (2002).
1.4 Geotechnical Monitoring Applications of Fiber-Optic Strain Sensors

1.4.1 Distributed Brillouin Sensors

The main focus in this study is on distributed Brillouin sensors in geotechnical monitoring. Therefore, a review of published geotechnical applications is given. Further details about selected applications can be found in the state of the art sections of the corresponding chapters (Chapter 5 and Chapter 9).

1.4.2 Distributed Brillouin sensors: small scale laboratory testing

With a spatial resolution of 1 m, distributed Brillouin sensors are not ideally suited for small scale laboratory testing. Thus, few publications have been found, e.g. a slope simulator and a flume.

**The slope simulator** is a 1 m high and 1.5 m square box at the Nanjing University in China, which can be filled with soil. Displacement on the soil is applied by a hydraulic jack. (Wang et al., 2009).

**The flume apparatus** is a 3.3 m long, about 1 m wide inclined box at the Second University of Naples, Italy, which can be filled with soil up to 0.5 m deep. It is designed to replicate conditions during a rainfall-induced slope failure in unsaturated soils (Olivares et al., 2009).

Several soil-embedded sensing systems ranging from bare cables to fiber-instrumented geotextiles have been tested in the laboratory (Table 2).
<table>
<thead>
<tr>
<th>Sensor system and technology</th>
<th>Description</th>
<th>Details, see e.g.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cable soil-embedded</strong></td>
<td><strong>BOTDR</strong></td>
<td><strong>Slope simulator:</strong> Strain in the cable during loading indicated the increase in applied strain from the jack. However, if the shear stress between the soil and the cable surface exceeded the shear strength, slippage of the cable inside the soil occurred. In addition, the spatial resolution of 1 m means that only average values of the strain can be read out in this 1.5 m box, which limits the identification of accurate soil strain.</td>
</tr>
<tr>
<td><strong>Cable glued to 1cm² geogrids soil-embedded</strong></td>
<td><strong>BOTDA</strong></td>
<td><strong>Flume:</strong> The very small geo-grids were glued to the cable in order to prevent relative movements. No indication about the interval size is given. Two cables were embedded parallel to the slope and subsequently, the slope was exposed to rainfall. Strain in the cable increased until failure.</td>
</tr>
<tr>
<td><strong>Geotextile-embedded fiber</strong></td>
<td><strong>BOTDR</strong></td>
<td><strong>Slope simulator:</strong> Fibers were fixed inside geotextiles and geogrids by Epoxy resin. Strain measured during loading indicated the increase in applied strain from the jack. However, due to the stiffer structure of the geogrid (and geotextile) in comparison with a cable only, measured strain was distributed over a larger distance. Also, the spatial resolution of 1 m sets some limits to the laboratory experiment.</td>
</tr>
</tbody>
</table>

**Critical assessment**

For all applications in the slope simulator and the flume, the spatial resolution of 1 m means that only average strain values were obtained. The same could have been obtained by single measurement sensors without utilization of distributed sensors. Thus, a qualitative analysis is possible, but the quantitative determination of strain at specific locations within the sample remains unknown and, thus, the evaluation of the proposed sensor-embedding is difficult.

Increased spatial resolution is therefore essential for testing of soil-embedded sensors at the laboratory scale.
1.4.3 Distributed Brillouin sensors: ground movement monitoring

Ground movement monitoring using distributed Brillouin sensors is proposed to investigate landslides, settlements, cavities, embankments, and dams. In addition, such monitoring is used to enhance safety of large, 1-dimensional infrastructure components such as pipelines, roads, tunnels, and railways. A wide range of such monitoring systems is summarized in Table 3.

Table 3: Distributed Brillouin sensing systems for ground movement monitoring.

<table>
<thead>
<tr>
<th>Sensor system and technology</th>
<th>Description</th>
<th>Details, see e.g.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable soil-embedded BOTDR</td>
<td><strong>Monitoring of cavities under an embankment</strong> by large scale testing in 8 m by 16 m embankments. Two 2.1 m diameter plates at 2 m depth simulated cavities. A cable was soil-embedded at 0.5 m and 1.25 m depth. Besides BOTDR, OFDR technology was used (optical frequency-domain reflectometry allowing for mm spatial resolution and, sensor length limited to 70 m). With OFDR, it was possible to detect a 5 mm movement of the simulated sinkholes (plates). BOTDR measurements were found to be less sensitive.</td>
<td>Lanticq et al., 2009</td>
</tr>
<tr>
<td>Cable soil-embedded BOTDA</td>
<td>Monitoring of soil settlements in a full scale 9 m long test box with simulated settlements over a zone of 0.5 m length. Settlements of 4 mm were detected and localized.</td>
<td>Belli et al., 2009</td>
</tr>
<tr>
<td>Cable-grid plate-fixed (soil-embedded) BOTDR</td>
<td>Monitoring of soil movements along a pipeline of 60 km length in Peru by a cable soil-embedded 0.15 m above the pipeline. No monitoring results have been published so far.</td>
<td>Ravet et al., 2008</td>
</tr>
<tr>
<td></td>
<td><strong>Monitoring of slopes</strong> by installing cables in a 0.5 m trench. The cables were fixed at undefined intervals to plates. A test in Korea showed that by applying external force on the buried cable, strain detection was possible. Johansson &amp; Farhadirousan (2005) claim that in complementary laboratory tests, which are not described in their paper, a deflection of a dam by just 5 mm is detectable with this system.</td>
<td>Kluth et al., 2006</td>
</tr>
</tbody>
</table>
### Distributed Brillouin sensing systems for ground movement monitoring (cont.)

<table>
<thead>
<tr>
<th>Sensor system and technology</th>
<th>Description</th>
<th>Details, see e.g.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cable-grid rod-fixed (soil-embedded)</strong></td>
<td>Monitoring of slopes by laying out sensing cables in a grid on, or slightly below, the surface. The cable was fixed to the soil or rock by steel rods. A case study has been carried out on a 30° inclined, 8 m high slope in Jiangsu, China. There, the cables were embedded 0.1 m deep in the soil and fixed at 5 m intervals to rods plunging to 0.5 m depth. Monitoring results detected sliding in an 8 m by 8 m section after a week of raining. This sliding could be confirmed by visual inspection.</td>
<td>Shi et al., 2007</td>
</tr>
<tr>
<td><strong>BOTDR</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Cable integrated into geotextile (soil-embedded)</strong></td>
<td>Monitoring of dam failure by fiber-equipped geotextiles provided by the company TenCate. 4 geotextiles stretching over the entire length of the dam were embedded in a 100 m by 27 m by 6 m high dam. This dam was then brought to collapse. A weak spot was detected at an early stage of the experiment, but the data of later stages was lost. Monitoring of soil settlements in a full scale 9 m long test box with simulated settlements over a zone of 0.5 m length. Settlements of 4 mm were detected and localized. In comparison with a bare soil-embedded cable (see above), no further improvement of sensitivity seems to be obtained by the integration into a geotextile. Monitoring of large deformations in geostuctures by polymer optical fibers woven into geotextiles. In this case, OTDR measurements are applied to measure the deformation.</td>
<td>Koelewijn, 2009</td>
</tr>
<tr>
<td><strong>BOTDA</strong></td>
<td></td>
<td>Belli et al., 2009</td>
</tr>
<tr>
<td><strong>OTDR</strong></td>
<td></td>
<td>Habel et al., 2007</td>
</tr>
<tr>
<td><strong>Cable fixed to aluminum strips (soil-embedded)</strong></td>
<td>Monitoring of dam behavior by cables attached to 0.5 mm thick and 55 mm wide aluminum strips. Large scale testing in 2.5 m by 20 m dam with 30° slope gradient. 3 strips were buried 0.45 m over the length of the dam. This dam was then watered and increasing strain in the strips could be monitored. In a field project, the strips were embedded in a river levee parallel to the expected soil movement direction.</td>
<td>Ohno et al., 2001</td>
</tr>
<tr>
<td><strong>BOTDR</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Critical assessment

As it can be seen, many methods have already contributed to the advancement of ground movement monitoring by fiber-optic sensors. However, several open questions remain. The most straightforward sensor integration strategy of simply soil-embedding a cable works only if the soil strain is accurately transferred into cable strain, rather than the soil just flowing around the cable. The drawback is that there is a maximum shear stress that can be transferred to the soil-cable interface after which interface slippage occurs (progressive failure) and the strain measurement ceases to correlate to the soil strain. Investigations of the progressive failure between soil and fiber-optic sensors are not discussed in the literature. However, the understanding of the progressive failure is necessary to conclude on the accuracy of the sensor system and the interpretation of the measured strain distribution.

Several authors have made an attempt to two-dimensionally fix the sensor to the soil (plate-fixed, rod-fixed and geotextile). With each of these systems, there is the risk that soil strain appears from the third dimension and, in this case, the sensitivity is severely diminished.

Additionally, the longitudinal stiffness of the sensors system plays an important role and studies of this influence are missing so far.

1.4.4 Distributed Brillouin sensors: monitoring of geotechnical structures

By attaching or integrating cables to geotechnical structures, a picture of their state, loading history, and safety can be obtained. The following examples of instrumented geotechnical structures represent the current state of the art in distributed Brillouin sensing for pile, pipeline, tunnel and mine applications (Table 4).
### Table 4: Distributed Brillouin sensing of geotechnical structures.

<table>
<thead>
<tr>
<th>Structure and technology</th>
<th>Description</th>
<th>Details, see e.g.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile</td>
<td><strong>Monitoring of secant piled wall during excavation</strong> by installing two cables on opposite sides of a 0.45 m diameter pile. Comparison with inclinometer readings indicated that the obtained deflection, gradient, and curvature are very much alike.</td>
<td>Mohamad et al., 2007</td>
</tr>
<tr>
<td>BOTDR</td>
<td><strong>Monitoring of mini-piles during pile load testing</strong> by epoxy-gluing cables to steel reinforcing bars. Comparison with vibrating wire strain gauges showed excellent agreement and provided data for further calculations of the load transfer from the pile to the soil.</td>
<td>Klar et al., 2006</td>
</tr>
<tr>
<td>Pipeline</td>
<td><strong>Monitoring of pipeline response to tunneling</strong> by fixing a cable to a pipeline. During a pipe jack tunneling operation at greater depth, the complete strain profile along a pipeline at a lesser depth was successfully monitored.</td>
<td>Vorster et al., 2006</td>
</tr>
<tr>
<td>BOTDR</td>
<td><strong>Monitoring of a pipeline crossing a landslide-prone area</strong> in Italy by epoxy-gluing 3 cables to the pipeline surface at 0°, 120° and -120° over 500 m. Strain changes after burying the pipeline were detected.</td>
<td>Inaudi &amp; Glisic, 2007b</td>
</tr>
<tr>
<td>Tunnel</td>
<td><strong>Monitoring of tunnel response to close-proximity tunneling</strong> by fixing cables to the circumference of the existing tunnel. During the excavation of the tunnel nearby, strain distribution was monitored and the practicality of such application demonstrated.</td>
<td>Mohamad et al., 2007</td>
</tr>
<tr>
<td>Mine</td>
<td><strong>Monitoring changes in the state of an underground mine</strong> by fixing cables to rockbolts in the ceiling of the tunnel at 3 m intervals. With this system, during a half year field trial, deformation due to ongoing extraction could be detected.</td>
<td>Naruse et al., 2007</td>
</tr>
</tbody>
</table>

**Critical assessment**

The literature covered and answered a lot of questions with regard to monitoring of geotechnical structures. But there are still several open questions.
With a relatively small pile (compared to the spatial resolution), the question remains if sufficient strain data can be obtained. Otherwise, this application is ready for commercial use.

With very large structures (pipeline, tunnel, and mine), it is not clear if the sensitivity is sufficient (especially if movements are perpendicular to the sensor, which is the most unfavorable direction).

The interpretation of the obtained Brillouin frequency shift is a key component for geotechnical structures monitoring. However, agreement on the applicable interpretation method is still lacking.

1.4.5 Fiber Bragg Grating

FBG sensors typically replace individual strain gauges. There is a tendency to apply them everywhere possible, which does not signify that the applications are always meaningful. This review is limited to a few examples of geotechnical monitoring using FBG sensors (Table 5).

In addition to geotechnical monitoring applications, Bragg Grating sensors have been used at ETH Zurich (Institute of Structural Engineering) for measuring strains in reinforced steel (Kenel & Marti, 2001; Kenel et al., 2005) and at EMPA for GFRP laminates (Frank, 2001).

Critical assessment

From the point of view of the present study, the main question is whether to use point sensors or distributed sensors for a given application. For micropiles, ground anchors, and inclinometers, distributed sensing has not been employed. However, point sensors are extremely sensitive to both the fixation of the sensor and local strain changes within the specimen. Distributed sensors are capable of smearing such local “imperfections” over the specimen length and may provide a better picture of the structure as a whole.
Table 5: Fiber Bragg Grating sensor applications for geotechnical monitoring.

<table>
<thead>
<tr>
<th>Application</th>
<th>Description</th>
<th>Details, see e.g.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Micro-pile and anchor</td>
<td>Strain distribution assessment during pullout tests based on eight FBG's aligned along the steel bar.</td>
<td>Habel et al., 2007</td>
</tr>
<tr>
<td></td>
<td>Strain distribution along two instrumented GFRP soil anchors of 3.6 m length in Hong Kong with FBG sensors and electrical strain gauges. The strain profile was obtained; however, the values of the different sensors did not match closely. At higher loads, some sensors failed.</td>
<td>Zhu et al., 2007</td>
</tr>
<tr>
<td></td>
<td>Strain distribution along a rock bolt (GFRP anchor) at four points with FBG at the testing cavern Hagerbach, Switzerland.</td>
<td>Frank, 2001</td>
</tr>
<tr>
<td>Inclinometer</td>
<td>The concept of FBG based inclinometer was proposed and studied by several authors. A 14 m deep inclinometer instrumented with two strings of eleven FBG’s each successfully monitored ground movements in Taiwan.</td>
<td>Schwarz, 2006; Ho et al., 2006</td>
</tr>
<tr>
<td>Soil shear profile</td>
<td>Laboratory tests of soil-embedded, highly flexible metallic strips instrumented with FBG’s. The soil shear profile during horizontal shearing could be successfully monitored.</td>
<td>Todd &amp; Overbey, 2007</td>
</tr>
<tr>
<td>Seismic monitoring</td>
<td>A seismic sensing system based on integration of three FBG dynamic sensors in a mechanical structure acting as an inverse pendulum.</td>
<td>Laudati et al., 2007</td>
</tr>
<tr>
<td>Asphalt monitoring</td>
<td>An instrumented asphalt road to detect strain affecting the serviceability of the road. The FBG’s were packaged in such a way that they were not damaged during the pavement lifetime.</td>
<td>Hu et al., 2010</td>
</tr>
<tr>
<td>Pressure monitoring</td>
<td>Transversely loaded FBG’s were used for downhole pressure monitoring.</td>
<td>Correia et al., 2007; Yamate et al., 2002</td>
</tr>
</tbody>
</table>

1.4.6 Interferometric sensors

Fabry-Pérot interferometry provides a method for point sensors that can replace piezometers, strain gauges, temperature sensors, pressure sensors and displacement sensors (Pinet, 2009). Low-coherence SOFO provides for
long gauge strain sensors. SOFO is widely commercialized and has been used to monitor more than 300 structures, including bridges, tunnels, piles, anchored walls, dams, historical monuments, nuclear power plants, and laboratory models (Inaudi & Glisic, 2007a). A review of a few interferometric sensor applications in geotechnical monitoring is provided in Table 6. Given the mature stage of these sensors, no further details are provided within the framework of the present study.

Table 6: Interferometric sensor applications for geotechnical monitoring.

<table>
<thead>
<tr>
<th>Application</th>
<th>Description</th>
<th>Details, see e.g.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile</td>
<td>Integrity assessment of large concrete foundation piles by a Fabry-Pérot sensor recording acoustic emission signals. Monitoring of average strains in several segments during axial compression, flexure, and pullout of cast-in-place piles with a low-coherence system.</td>
<td>Habel et al., 2007 Glisic et al., 2002</td>
</tr>
<tr>
<td>Anchor</td>
<td>Feasibility study of a low-coherence sensing system installation in a tendon. Assessment of rock movement with the installed anchor. However, no meaningful measurement was achieved.</td>
<td>Inaudi, 1997</td>
</tr>
<tr>
<td>Extensometer</td>
<td>Replacement of two mechanical extensometers with a low-coherence system to monitor rock deformations.</td>
<td>Inaudi, 1997</td>
</tr>
<tr>
<td>Tunnel</td>
<td>Deformation and crack monitoring of a cut-and-cover tunnel in Yverdon, Switzerland, with a low-coherence system. Monitoring the buttress at the entrance and the concrete lining inside the alptransit tunnel in Bodio, Switzerland.</td>
<td>Glisic et al., 2000 Inaudi, 2003</td>
</tr>
<tr>
<td>Pipeline</td>
<td>Study on feasibility of other interference systems (Sagnac and Mach-Zhender) to sense the acoustic signal caused by pipeline leakage.</td>
<td>Gao et al., 2007</td>
</tr>
</tbody>
</table>
1.4.7 General comment on applications

The state of the art review in this section (1.4) reveals that a wide range of applications have been suggested, tested, and implemented. At first sight, it seems that fiber-optic sensors have a solution for any geotechnical monitoring problem. But this interpretation has to be taken with some reservation, as often detailed information about the sensor system is not provided or additional assumptions have been used in order to obtain the data provided. This is common in research projects. For practical applications, however, additional verification in the rough construction environment is requisite.

1.5 Advantages and Limitations of the Technology

1.5.1 Advantages

During recent years, fiber-optic sensors have made a significant entrance into the monitoring world. After an initial euphoric phase, when fiber-optic sensors seemed a substitution for basically any other sensor, the focus has shifted towards cases, where fiber-optic technology offers superior performance compared with the more proven conventional sensors. The technology’s added value includes:

- Unprecedented amount of data (along a single cable of 10’s of km length);
- Automated and fast data acquisition (replacing manual readings and operator judgment, continuous monitoring, remote sensing);
- Insensitivity to external perturbations (e.g. chemicals, electricity and lightning). In addition, no electricity is used to measure (explosion-proof);
- Small size;
- Sometimes the only available solution;
- Lower sensor and lower measurement costs.
1.5.2 Challenges and limitations

Although there are many advantages to using distributed fiber-optic sensors, there are also challenges and limitations that must be understood in order to prevent failure of fiber-optic sensing systems:

- Sensor packaging (compromise between protecting the fiber and avoiding slippage between fiber and protection, handling and manipulation in the sensor);

- Sensor installation (fixing and embedding into the structure or soil);

- Optical loss (attenuation see 1.1.5): minimum bending radius must not go below 10 times the cable diameter (Rosenberg, 2001); maximum strain on the fiber must not exceed 1% to 4% (depending on the cable used); crush loads on the cable;

- Fiber break (rodents; ingress and egress points of the cables into the structure; construction and integration);

- Temperature range of the cable.
2 First Laboratory Application of Brillouin Echo Distributed Sensing

The objective of this study was to improve the understanding of the progressive failure in soil-structure interaction for ground anchors and cables. However, for the laboratory, the minimal spatial resolution of 1 m sets a serious limit on possible applications. Moreover, in several rather small scale geotechnical structures, a better resolution is desired. Thus, advancement to the fiber-optic sensor technology was required.

For this reason, collaboration with the Group of Fiber Optics at the Institute of Electrical Engineering of the EPFL was initiated with the aim of improving the spatial resolution of Brillouin Sensing for geotechnical monitoring applications. The 5 cm spatial resolution was achieved in 2008 (Thévenaz & Foaleng, 2008) with a Brillouin Echo Distributed Sensing (BEDS) setup. The initial result of this ongoing collaboration, was the first ever BEDS application outside of a fiber optics laboratory, and is presented in this study (Chapter 6 and Iten et al., 2009a).

2.1 Brillouin Echo Distributed Sensing

In BOTDA and BOTDR configurations, high spatial resolution and narrow Brillouin frequency profiles are incompatible due to the phonon lifetime. However, a significant breakthrough was realized when it was observed that the pre-excitation of the acoustic wave by a superimposed pulse on the continuous pump level can lead to a high spatial resolution while maintaining a sharp Brillouin gain profile (Bao et al., 1999).

In such a configuration, the pulse is reflected on the pre-existing acoustic wave that “sees” its amplitude mostly unchanged by the presence of the pulse (Thévenaz, 2010). Actually, the pulse is only probing locally for the existence of the continuous acoustic wave. This tiny back-reflected, out-of-phase light interferes destructively with the probe wave, resulting in an apparent loss proportional to the local amplitude of the pre-excited acoustic wave. With
extremely short pulse durations (500 ps) a 5 cm spatial resolution and a 10 με strain resolution can be achieved with a scanning frequency, of the probe wave, in 1 MHz steps. The sampling interval is equivalent to 0.25 cm (Thévenaz & Foaleng, 2008).

The data is processed to extract the local central frequency of the Brillouin gain spectrum, to eliminate the detrimental impact of the secondary echo on the measurements (Foaleng et al., 2009).

### 2.2 Laboratory Setup at IGT

BEDS is not yet commercially available, as the setup still requires the combination of several devices (Figure 19) and manual fine tuning. Such a setup is only possible in the laboratory and cannot be used for field measurements (in contrast to BOTDA interrogators). Data acquisition lasts between 10 to 30 minutes and accuracy is comparable to a BOTDA setup.

The collaboration with the Group of Fiber Optics (GFO) of EPFL allowed for testing BEDS in our geotechnical laboratory. The specifications of this setup (Foaleng et al., 2010) are given in Table 7. The measurements are presented in Chapter 4 (cable testing) and Chapter 6 (cable pullout). A second BEDS testing campaign was carried out in 2010 by Hauswirth (2010b).

![Figure 19: BEDS setup for laboratory fiber strain testing at the Institute for Geotechnical Engineering.](image)
Table 7: Specifications of the BEDS setup.

<table>
<thead>
<tr>
<th>Instrument</th>
<th>BEDS lab prototype</th>
<th>Spatial resolution</th>
<th>Max. distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer</td>
<td>GFO EPFL, Switzerland</td>
<td></td>
<td>50 mm</td>
</tr>
<tr>
<td>Technology</td>
<td>BEDS</td>
<td>Strain resolution &amp; accuracy</td>
<td>Comparable to BOTDA</td>
</tr>
<tr>
<td>Sampling interval</td>
<td>2.5 mm</td>
<td>Reference</td>
<td>Thevenaz, 2010</td>
</tr>
</tbody>
</table>
3 Development of Interpretation Techniques

The objective to improve data interpretation techniques provides a means for the efficient and successful application of distributed fiber-optic sensors. To improve the data interpretation, in-depth understanding of the technology and of the underlying factors affecting the measurements is required. Otherwise, measured data may not accurately represent the condition of the object under observation.

The literature notes the main factors affecting the measurements (e.g. fixation method, spatial resolution, and temperature). In addition, several data interpretation techniques can be found, such as the single measurement point, different kind of averages, and the convolution product. However, these suggestions on data interpretation are basic and the possibilities for improvement were realized. On the other hand, the sophisticated methods available are not applicable for the typical users of the technology due to their complexity.

Based on Iten & Puzrin (2009) and Iten et al. (2011), detailed investigation on the factors affecting BOTDA measurements is provided and the different techniques are organized allowing for consistent data interpretation. In addition, the current state of the interpretation, based on the convolution product, is challenged by the findings of this study.

3.1 Investigation on Factors Affecting BOTDA Measurements

In this section, the raw data obtained by BOTDA is analyzed. From a user point-of-view (in our case a geotechnical engineer), the focus must be on the data processing, interpretation, and performance evaluation of BOTDA data as provided by a commercially available interrogation unit (i.e. raw data).

3.1.1 Fiber fixation method

An optical fiber intended for strain measurements can be attached to a specimen in two ways: overall bonding and point fixation (Figure 20).
Figure 20: (a) A fiber fixed by overall bonding and; (b) point fixation to a specimen.

**Overall bonding**

With the overall bonding method, the fiber is bonded along the entire surface of the sample (Figure 20a). Since any strain in the sample is transferred directly to the fiber, strain measured can be assumed to be the real strain of the specimen at the fiber location. The strain distribution in an overall bonded fiber, $\varepsilon_{OB}(x)$, is equal to the true strain on the surface, $\varepsilon_T(x)$, to which the fiber is bonded to:

$$\varepsilon_{OB}(x) = \varepsilon_T(x) \quad (3.1)$$

In the case of perfect bonding, local disturbances (e.g. cracks) cause high strain in the fiber over short distances, while the other sections of the fiber remain under low strain or even unstrained. In such cases, the fiber may break. Thus, overall bonding can only be implemented in fiber attachment to materials where no cracks or excessively high strains are expected (e.g. iron reinforcing bars, construction steel and anchor tendons).

Cables directly embedded into concrete, grout or glacier boreholes are overall bonded after the cement hardens or water freezes. In these cases, there is a
high risk of failure due to cracks if the fiber cable is not able to redistribute some of the strain (slippage of the fiber inside the outer cable layers).

**Point fixation**

With the point fixation method, the fiber is bonded at distinct locations, $x_j$, along the specimen (Figure 20b). The gauge length, $L_{sj}$, between two fixation points, $(x_j, x_{j+1})$ is:

$$L_{sj} = x_{j+1} - x_j$$

(3.2)

In each gauge, $s_j$, a uniform average strain, $\varepsilon_j$, along the fiber develops. From the measured optical data, it remains unknown how the strain is distributed in the specimen (uniformly or locally high strains, cracks). Strain distribution, $\varepsilon_{PF}(x)$, in a point fixed fiber can be mathematically described using a unit step function, $H$, where

$$H(x) = \begin{cases} 0 : x \leq 0 \\ 1 : x > 0 \end{cases}$$

(3.3)

$$\varepsilon_{PF}(x) = \sum_{j=1}^{N_s} \varepsilon_j \left[ H(x-x_j) - H(x-x_{j+1}) \right]$$

(3.4)

A point fixed fiber is much less vulnerable to breakage than an overall bonded fiber. In fact, gauge lengths can be anticipated in such a way that the maximum strain a fiber can experience without breaking, $(\varepsilon_{F,d})$, is not reached within the extents of the monitoring range.

Point fixation is used for attaching fibers to various materials, i.e. concrete, masonry, and steel. Another way of point fixation is to put a fiber freely inside a tube and establishing a grip on the fiber only at discrete points along this tube. With such a system, point fixation of fibers inside asphalt, concrete, or soil is achieved.

**Combination of point fixation and overall bonding**

As illustrated in Part III, a combination of the two fixation methods is also possible (or, as a matter of fact, even inevitable). Consider for example a cable point fixed to soil with “micro-anchors”. Up to a certain level of shear
stress at the cable-soil interface, the shear force exposes the cable to overall bonding to the soil. Reaching the peak value of the interface shear stress, overall bonding fails and the force induced by the “micro-anchors” becomes the main fixation component.

3.1.2 BOTDA data processing around a fixation point

The fiber attachment to a structure often implies fixation at discrete points with sections of constant strains in between (Eq. 3.4). Nevertheless, interpretation of the measured data in the vicinity of the fixation point is not always trivial, due to the effects described below.

The output of a BOTDA interrogator, such as the DITEST, is the measured average strain, $\varepsilon_M(x)$, over the spatial resolution length, $w$, at discrete points, $x_i$, along the fiber (Figure 21). The distance between two measurement points, $x_i$, is the sampling interval, $\Delta x$, which can be chosen from a minimum of 0.1 m up to a maximum of 102 m.

![Figure 21: A point fixed fiber experiencing different strains at both sides of the fixation point.](image)

The measured strain is a function of the recorded central Brillouin frequency shift, $\nu_B$, multiplied by the calibration coefficients (e.g. tensile strain coefficient at the wave length of 1550 nm) of the fiber. The central Brillouin frequency shift is determined by fitting a Lorentzian curve over the Brillouin gain spectra (e.g. over the continuous bold line for $x_{i-5}$ in Figure 22a).
For a better understanding of the effect this curve fitting has on the BOTDA measured strain, a simple experiment has been performed. A long (>1m) section of a bare fiber between two fixation points was strained to different levels of the true strain, $\varepsilon_T$ (e.g. $\varepsilon_T = 9200 \mu \varepsilon$), with optical measurements taken at a sampling interval of $\Delta x = 0.1$ m with a spatial resolution of $w = 1$ m.

Assuming that the sampling point closest to the fixed section of the fiber has a coordinate, $x_i$, consider a measurement at sampling point $x_{i-5} = x_i - 5\Delta x = x_i - 0.5$ m (Figure 21), where the optical fiber experiences zero strain $\varepsilon_1 = 0 \mu \varepsilon$ over the entire spatial resolution length $w = 1$ m centered at the point $x_{i-5}$. For this measurement, the Brillouin gain spectra closely represents a Lorentzian curve (Figure 22a, $x_{i-5}$), with the peak at the frequency corresponding to $\varepsilon_1 = 0 \mu \varepsilon$. Another extreme case is the measurement at the sampling point $x_{i+6} = x_i + 6\Delta x = x_i + 0.6$ m within the fixed section (Figure 21), where the optical fiber experiences uniform true strain $\varepsilon_2 = 9200 \mu \varepsilon$ over the entire spatial resolution length $w = 1$ m centered at the point $x_{i+6}$. Again, the Brillouin gain spectra represents a Lorentzian curve, with a shifted peak (Figure 22a, $x_{i+6}$), corresponding to $\varepsilon_2 = 9200 \mu \varepsilon$. When a measurement is taken at the sampling points $x_i$ or $x_{i+1}$, where the spatial resolution length spans over two different fiber strains ($\varepsilon_1 = 0 \mu \varepsilon$ and $\varepsilon_2 = 9200 \mu \varepsilon$), two distinctive peaks can be observed (Figure 22a, $x_i$, $x_{i+1}$), corresponding to $\varepsilon_1 = 0 \mu \varepsilon$ and $\varepsilon_2 = 9200 \mu \varepsilon$, respectively. The higher peak corresponds to the true strain at the sampling point. Because in this case the Lorentzian curve is being fitted to the higher peak, the strain at the sampling point is measured correctly.

The picture, however, changes dramatically for smaller strain differences. For the true strain of $\varepsilon_T = 1250 \mu \varepsilon$, the Brillouin gain spectra for the sampling points $x_{i-5}$ and $x_{i+6}$ still have clear peaks with corresponding strain levels. A more complex picture can be observed when a measurement is taken at the sampling points $x_i$ or $x_{i+1}$. Here, the Brillouin gain spectra are a combination of the backscatter relating to the different section’s strains (Figure 22b, $x_i$, $x_{i+1}$). The frequency of the peak is shifted, reflecting some kind of superposition of the gain spectra, which have to be decomposed to allow for the true strains to be obtained. Fitting the Lorentzian curve to these superimposed spectra, however, does not always produce the correct strains, complicating data interpretation in the vicinity ($\pm w/2$) of the fixation point. This behavior can be
noticed for strain differences between the two sections below a certain strain step limit, \( \varepsilon_{SLimit} \). Figure 23 shows the obtained strain profile for BOTDA measurements at a small strain step (a) and a large strain step (b).

The strain limit, \( \varepsilon_{SLimit} \), specified for a BOTDR in the literature is about 2500 \( \mu \varepsilon \) (Zhang & Wu, 2008). According to the experiments in this study (performed with a different experimental setup), the limit is approximately 3000 \( \mu \varepsilon \).

Figure 22: Brillouin gain spectra for (a) a large strain step (9200 \( \mu \varepsilon \)) > \( \varepsilon_{SLimit} \) and; (b) a small strain step (1250 \( \mu \varepsilon \)) < \( \varepsilon_{SLimit} \).

Figure 23: Strain profile obtained by BOTDA measurements (dotted line) in comparison to the true strain in the fiber for (a) a small strain step and; (b) a large strain step.

3.2 Suggested BOTDA Data Interpretation for Point Fixation

As demonstrated, at small strain differences in optically measured data close to a fixation point can be influenced by the strain in the fiber on the opposing side of the fixation point. What can be done to correct data interpretation in this case?
There are three options described in the next sections:

- Single measurement point pick (SP);
- Truncated average method (TAM);
- Brillouin strain profile as a convolution product (CP).

Many other methods have been suggested in the literature, mainly with the goal to enhance spatial resolution by signal processing. For example, signal reconstruction by means of multidimensional minimization (Minardo et al., 2005) or signal processing including experimental parameters, such as pulsewidth, extinction ratio, pulse and pump powers, and sensing fiber characteristics (Ravet et al., 2007). In addition, methods to automatically process the data by finding strained sections through pattern recognition and converting the measurement to real strain by applying the trapezoidal law are suggested (Ding et al., 2010). The focus, however, in the present study is on simple data interpretation methods that can be carried out by geotechnical engineers without a strong background in optics.

### 3.2.1 Single measurement point pick (SP)

In the first interpretation method, called the single measurement point pick, only the strain, $\varepsilon_{SPj}$, at one point, $x_{SPj}$, within the strained section (gauge) is used for the strain determination of this section. All of the other points are simply discarded. The strain, $\varepsilon_j$, in the whole section is then assumed to be equal to the strain measured at this single point:

$$
\varepsilon_j = \varepsilon_{SPj} = \varepsilon_M(x_{SPj})
$$

Considering that the measurement point furthest from the two fixation points is least influenced by data processing effects around the fixation point (3.1.2), a reasonable pick can be the midsection measurement point $x_{j+0.5}$, as shown in Figure 24. Strain in the section is then:

$$
\varepsilon_j = \varepsilon_M(x_{j+0.5})
$$

Optical cables with protection layers above the fiber may be subject to slippage of the layers at higher strains (4.2.1) and to other non-linear effects...
influencing the strain within a section. Additionally, the measured strain at some data points may be completely random due to improper numerical fit of the Lorentzian curve. In such situations, it can be practical to select the single measurement point with the largest strain within the section (Figure 24).

\[
\varepsilon_j = \max \{\varepsilon_M(x_j \leq x \leq x_{j+1})\}
\]

(3.7)

Figure 24: Experimental data of optically measured strain along a fiber for the purpose of demonstrating different single measurement point picks.

### 3.2.2 Truncated average method (TAM)

In the truncated average method (TAM), all the data measured at the sampling points within the interval \( \pm w/2 \) around the fixation points, \( x_j \) and \( x_{j+1} \), is ignored. Only strains measured at sampling points of sufficient distance from the fixation points are considered. The strain in the section is then calculated as:

\[
\varepsilon_j = \varepsilon_{TAM} = \frac{\sum_{i=L_j}^{R_j} \varepsilon_M(x_i)}{(R_j - L_j + 1)}
\]

(3.8)

with, \( L_j = \text{int}\left(\frac{x_j + \frac{w}{2}}{\Delta x}\right) \) and \( R_j = \text{int}\left(\frac{x_{j+1} - \frac{w}{2}}{\Delta x}\right) \).
This model returns accurate data for strains above and below $\varepsilon_{\text{SLimit}}$ (Figure 23).

The advantage of the SP and TAM techniques are their simplicity. The disadvantage is that the amount of valid sampling points is significantly reduced, especially in the SP method and in the TAM, when the fixed strain section lengths are close to the spatial resolution, $w$.

### 3.2.3 Brillouin strain profile as a convolution product

In order to enhance the accuracy of optically measured strain, using all sampling points in the vicinity of the fixation, alternative interpretation models can be thought of, such as the deconvolution of the Brillouin strain profile taking into account the sensor parameters (Bernini et al., 2004; Ravet et al., 2007). Another approach is spectral decomposition (Ohsaki et al., 2002; Zhang et al., 2004), which can be seen as an “indirect” deconvolution.

**Brillouin strain profile as a convolution product**

According to Klar et al. (2006), a fiber-optic measurement strain profile, $\varepsilon_M(x)$, obtained by the BOTDR measurement unit from Yokogawa (Yokogawa, 2010), can be seen as a convolution product between the true strain profile, $\varepsilon_T(x)$, in an optical fiber with the light pulse and a weighting function $g(x)$ of the BOTDR. This convolution product can be written as:

$$
\varepsilon_M(x) = \int_{-\infty}^{+\infty} \varepsilon_T(\tau) g(x-\tau) \, d\tau \quad (3.9)
$$

The weighting function is not known a priori, but can be evaluated using the fundamental identity element property of the convolution function. This states that the convolute of a function, $g$, with the delta function, $\delta$ (Dirac delta), is equal to the function $g$:

$$
g * \delta = \delta * g = g \quad (3.10)
$$

$$
g(x) = \int_{-\infty}^{+\infty} \delta(\tau) g(x-\tau) \, d\tau \quad (3.11)
$$
The delta function can be reproduced in the fiber by a localized strain over very short distance (Figure 25a) and the resolution function follows from the measured data. Klar et al. (2006) suggested a strain profile with the shape of Gaussian normal distribution, specified by the distance from the center, $x$, and the width of the weighting function, $\omega$, (not to be confused with the spatial resolution, $w$):

$$
g(x) = \frac{1}{\omega \sqrt{2\pi}} \exp\left(-\frac{x^2}{2\omega^2}\right)$$

(3.12)

In fact, there is no particular physical reason for the weighting function to be of a Gaussian shape. For example, Bao and Chen (2008) have suggested it to be rectangular. In addition, the weighting function can also be found by analyzing the step response of the system (Bölcskei, 2008). The convolution of the unit step function, or Heaviside function, $H$, (Figure 25b) is given by:

$$
\varepsilon_M(x) = \int_{-\infty}^{+\infty} g(\tau) * H(x-\tau) d\tau = \int_{-\infty}^{x} g(\tau) d\tau
$$

(3.13)

so that:

$$
g(x) = \frac{d\varepsilon_M(x)}{dx}
$$

(3.14)

The width of the Gaussian weighting function suggested by Klar et al. (2006) for the Yokogawa system is $\omega = 0.287$ m, regardless of the strain step size.
Illustration of the convolution product

To illustrate the convolution product, as described above, consider a simple example of a short section, $L_{sj}$, fixed between the two points $x_j = 0$ and $x_{j+1} = L_{sj}$ where section strain of $\epsilon_j = 1$ has been applied. The true strain along the fiber can then be expressed by:

$$\epsilon_T(x) = \sum_{j=1}^{2} 1 \cdot \left[ H(x - 0) - H(x - L_{sj}) \right]$$

(3.15)

The convolution of $g$, specified by $\omega = 0.287$ m with $\epsilon_T$ (Eq. 3.9) will then produce the measurement strain profile, $\epsilon_M$, which is displayed in Figure 26 for various $L_{sj}$. As expected, the convolution product, $\epsilon_M$, takes a Gaussian shape.

For a step response, the same numerical experiment can be done using a long enough section of equal strain, while the rest of the cable remains unstrained. Such a step response is drawn in Figure 26.
Convolution product application on laboratory data

An attempt to specify the weighting function for the BOTDA measurement unit used in this study (DITEST) was undertaken by numerical convolution modeling of laboratory results. The true strain over the section $L_{sj} = 1\text{m}$ was convoluted with the Gaussian curve, $g(x)$ (Eq. 3.13), whose parameter, $\omega$, was adjusted to provide the best fit between the measured and calculated strain data within the strained section (Figure 27). As is seen from Figure 27, it was not possible to provide a good fit to the data measured for different strain step values with a single value of the parameter $\omega$. Indeed, this parameter shows a strong dependency on the strain step size up to the limit equal to $\varepsilon_{SLimit}$ introduced above (Figure 28). For comparison, Figure 27 also shows the convolution product, which uses the constant value of $\omega = 0.287\ \text{m}$ suggested by Klar et al. (2006). Clearly, a single weighting function is not suitable for all strain step values, but rather the function parameters have to be adjusted depending on the strain step used. This conclusion is also valid for other shapes of the weighting functions tested (rectangular, trapezoidal, and triangular).
3 Development of Interpretation Techniques

a) 472 \( \mu \varepsilon \); \( \omega = 0.21 \) m

b) 852 \( \mu \varepsilon \); \( \omega = 0.19 \) m

c) 1827 \( \mu \varepsilon \); \( \omega = 0.11 \) m

d) 3550 \( \mu \varepsilon \); \( \omega = 0.05 \) m

Figure 27: True strain in the fiber convoluted with a Gaussian weighting function of different width, compared with the optically measured strain for a true strain of (a) 472 \( \mu \varepsilon \); (b) 852 \( \mu \varepsilon \); (c) 1827 \( \mu \varepsilon \) and; (d) 3550 \( \mu \varepsilon \).

Figure 28: Dependence of \( \omega \) on the strain step size.
Deconvolution of the Brillouin strain profile

One approach to restore the undistorted information is to deconvolute the measured strain data (Bernini et al. 2004) and, in the process, calculate the true strain. More specifically, this means to find a match of the solution, \( c \), of the convolution equation with the measured strain profile.

\[
\varepsilon_T \ast g = c = \varepsilon_M
\]  

(3.16)

Due to measurement imprecision and the width of the weighting function’s strain step dependency, an iterative deconvolution algorithm is applied. Additionally, the fixation points, \( x_j \), location must be known. During this study, an algorithm was designed for finding the true strain, however the method of deconvolution was soon realized to be of very limited help in practical applications. Especially since the convolution product itself (delivered by the BOTDA data) is not as straightforward as it seemed from the theory developed by Klar et al. (2006).

Sub-spatial resolution strain data

Nevertheless, applying the convolution theory can provide for a first estimate of sub-spatial resolution BOTDA data. Because from Eq. (3.12), the maximum strain measured for a given \( L_{sj} \ll \omega \) is:

\[
\varepsilon_{M_{\text{max}}} = \frac{L_{sj}}{\omega \sqrt{2\pi}} \cdot \varepsilon_T
\]  

(3.17)

In addition, for \( 0 < L_{sj} < \omega \), Ohsaki et al. (2002) provide a useful relationship for the maximum strain measured:

\[
\varepsilon_{M_{\text{max}}} = \frac{L_{sj}}{W} \cdot \varepsilon_T
\]  

(3.18)

Both equations are only true for small strain steps below 1000 \( \mu \varepsilon \). In cases where the strain step is within the range of 1000 \( \mu \varepsilon \) and 2500 \( \mu \varepsilon \), the relationship given in the equations is not always valid. For strain steps above 2500 \( \mu \varepsilon \), any strain section shorter than \( W/2 \) will not be recognized at all by the BOTDA data processing.
3 Development of Interpretation Techniques

3.3 Suggested BOTDA Data Interpretation for Overall Bonding

For BOTDA data of overall bonding, there is no possibility to systematically process, or even delete, data that originates from error-prone sections (in contrast to point fixation, where data in the proximity of fixation points is either processed by deconvolution or simply deleted by the SP and TAM methods). Thus, data processing for the overall bonded fiber strain is much influenced by the engineering judgment and the understanding of the physical processes occurring in the compound material that the sensor is bonded to.

The data processing consists of two steps. The first one is almost always necessary; the second one depends on the project and the general quality of the data:

In the first step, the data is cleaned from evident errors originating from the BOTDA instrument data processing (e.g. if there is no obvious Brillouin gain peak within the scanned frequency, the data processing of the instrument assigns either the lowest or the highest scanned frequency to the sampling point. This results in very low or very high strains and can be usually recognized by simply looking at the strain profile along the fiber). To clean the data, a filter sorts out all of the unwanted data. The cleared points are either removed entirely or replaced with a calculated strain emerging from the surrounding strain data points.

In the second step, an appropriate fitting and smoothing of the data is performed. Often, simplified least square procedures (Savitzky and Golay, 1964) are sufficient. Curve fitting is also possible and countless functions, up to multi degree polynomials, are available (e.g. 10th degree polynomial adopted by Wu et al., 2006). The curve fitting, properly applied, can immensely improve the quality of the data. To this end, the mechanical properties of the medium, where the sensor is embedded, need to be understood. An example how this can be done with meaningful fitting is presented in Chapter 8. In addition, data from alternative sensors (e.g. conventional strain gauges, geodetical data etc.) provide for additional information to be included in the fitting function.
3.4 Temperature Compensation

As shown in Section 1.2.1 (Eq. 1.5), the Brillouin frequency shift depends on both strain and temperature. To distinguish a frequency shift due to a change in strain from a frequency shift due to a change in temperature an independent measurement of temperature is required. In a bare fiber, the frequency shift due to temperature variation is linear (Eq. 1.7). The temperature coefficient, \( C_T \), depends on the fiber type. For packaged fibers (cables), the frequency shift due to temperature variation is often not homogeneous along the cable.

In general, any strain measurement must be corrected by the component of the frequency shift due to temperature variation. The required measurement accuracy and the environment of the structure dictates whether temperature will be assessed in a distributed fashion along the whole sensor (E.g. placing of a loose fiber) or by alternative point sensing methods (E.g. conventional temperature sensors). For example, temperature variation of a cable embedded in soil is an order of magnitude lower than a cable at the surface of a steel structure exposed to the sun. In laboratory environment, the temperature is essentially constant, so its effect can generally be neglected when measuring strain (Bao & Chen, 2008). Further information on temperature compensation can be obtained from Cho & Lee (2004).

Temperature compensation should be the first data processing step on the raw BOTDA frequency data.
4 Development and Calibration of Strain Sensing Cables

In order to achieve the various novel monitoring applications, new fiber-optic sensors were required. Without accomplishing this objective, most of the applications in Part II and Part III would not have been possible.

Commercial telecom cables are designed to protect the optical fibers from mechanical stress, which may degrade data transmission performance and considerably reduce the optical fiber’s lifetime. Ideally, there would be no mechanical link between the fiber and the surrounding protection. On the other hand, in a strain sensing cable, stress must be efficiently transferred to the fiber core. The best conditions are obtained with a bare fiber because any protection layer above presents a risk of slippage between the protection layers. Nevertheless, protection of the strain sensing fibers from the rigors of the outside world during installation and operational life is crucial.

At the beginning of this study in 2005, commercial options for strain sensors were still scant and they appeared to have a series of handicaps varying from high signal loss (attenuation), tricky handling and inflexibility in project alterations to long production and delivery times and high prices. Therefore, it was chosen to custom build produce strain sensing cables in collaboration with the renowned cable manufacturer Brugg Cables AG. The collaboration with the cable manufacturer and the measurement unit producer (Omnisens SA) was further strengthened within the framework of a Swiss Innovation Promotion Agency (CTI) project. This resulted in a range of high quality strain sensing cables commercially available today.

All cables used in this study were prototypes. Therefore, each cable had to be tested thoroughly to obtain mechanical and optical sensor parameters, so that the laboratory and field data could be interpreted correctly. The development and calibration of these cables were reported on several occasions (e.g. Iten et al., 2009a, 2009b, 2009c). In addition, a new method to detect slippage of the fiber inside the protection was developed using BEDS measurements (Iten et al., 2011).
4.1 General Design of Fiber-Optic Cables

4.1.1 Loose tube and tight-buffer

Generally, fiber cables are distinguished between loose tube and tight-buffered types. A simple protected loose or tight tube cable typically has a diameter of 900 µm.

Loose tube

The fiber is placed loosely inside a hard tube (plastic, steel) and therefore, external strain on the cable is not directly transferred to the fiber inside. Such cables are used for outside telecom installations where fibers are likely to be subjected to external stresses.

Tight-buffered

A hard tube (plastic, steel) is connected to the fiber via a plastic buffer. Strain applied on a tight-buffered cable is, at least partially, transferred to the fiber core and such cables are thus, strain sensitive. Tight-buffered cables are used for most indoor installations.

4.1.2 Protection

Commercial communication cables have different types of protection depending on the application: aerial, underground, underwater, or indoor use. Heavy-duty cables have to withstand numerous hazards, such as tension, moisture, freezing water, rocky soils, construction activities, and rodents. For telecom applications, permanent strain transferred to the fibers must be limited to 0.33% strain. Short term strain (e.g. during installation) must not exceed 0.5% (Mitsunga et al., 1982; Swisscom, 2001). Figure 29 shows commercially available, well protected loose tube cables.
4 Development and Calibration of Strain Sensing Cables

Figure 29: Commercially available, well protected loose tube cables: (a) Metal protected cable with four fibers and; (b) duplex cable with four layers of plastic protection.

4.2 Requirements of Strain Cables

Fiber-optic strain cables for integration into different environments have to comply with several requirements, such as being strong enough to withstand harsh installation conditions, allowing unproblematic handling and offering flexible adjustment to project modifications. These requirements are in addition to being able to withstand hazards similar to those of telecom cables.

4.2.1 Minimal slippage

However, the largest issue to date is the transfer of strain applied on the jacket without loss to the fiber core. Slippage between different layers is common (Zhang & Wu, 2007; Ding et al., 2006) and thus, the issue of slippage needs to be addressed in any optical strain sensor development. The goal is to obtain minimal slippage without increasing attenuation due to the tight grip of the protection layer on the fiber. Then again, all layers of the cable jacket also have to be strippable (removable) in order to connect cables with each other and to repair broken fibers (splice). Strain transfer without slippage and a design allowing for flexible adjustments are in contradiction with one and other and provide an engineering challenge for cable manufacturers. In addition, the extrusion of an especially tight layer on the fiber can cause micro-bending in the fiber, which increases signal loss (attenuation) along the fiber dramatically.
4.3 Development of Custom Designed Strain Cables

As mentioned, specialty fiber-optic strain sensing cables can be found on the market. But only for two years have qualitatively good cables, that is, meeting the author’s quality requirement, been available. Other authors (e.g. Lanticq et al., 2008; Zhou et al., 2008) agree on this. Thus, for this study, mostly custom designed strain cables were used.

4.3.1 Commercially available fibers – The raw material

**BSM and TSM**

Fibers for BOTDA strain sensing are available in two designs; as a bare single mode fiber (BSM) of 0.25 mm diameter and as a tight buffered single mode fiber (TSM) of 0.9 mm diameter. The buffer, made of thermoplastic elastomer (TPE), offers a minimal strain sensitive protection. The BSM and TSM were used in the laboratory and for the monitoring ground anchor (PART II). Details are provided in Figure 30, with further optical and mechanical parameters later determined in Section 4.6.

**Specialty fibers**

Silica-based optical fibers are damaged or break at strains larger than 2 or 3 percent. Polymer optical fibers, allowing for strains up to 40%, (Habel, 2007) can be used for higher strains, but are not widely available.

4.3.2 Strain sensor cable 2006 (S06)

The first custom sensor (S06) was produced in 2006 at the Institute’s laboratory by integrating two tight TSM fibers into a heat shrink tube. After producing about 200 m of this 2 mm x 3 mm cable in a very labor intensive process, the attenuation of the cable was tested and showed no significant loss at any point along the cable. Thus, it was concluded that the heat applied during production had no negative impact. The S06 was used as a road-embedded sensor (Chapter 10) and allowed for monitoring during a 10 month period. Further details are provided in Figure 30.
4.3.3 Polyamide (P07) and metal (M07) strain sensor cable

The growing interest in fiber-optic monitoring enabled the convincing of a world leading cable manufacturer (Brugg Cables AG) for a joint project in 2007 aiming at developing better suited optical strain sensing cables. The first products of this collaboration were a polyamide (PA) coated strain sensor (P07) and a metal protected fiber cable (M07), both of them being strong enough to withstand stepping on and other minor impacts during integration.

P07

The 1.6 mm diameter P07 (Figure 30) was among the best strain cables available at this time. Besides good protection and reasonably good strain transfer, it offered very easy handling and pre-straining. The P07 cable was used in most applications presented in this study.

M07

The 0.9 mm diameter M07 (Figure 30) consists of a bare fiber embedded into a small steel tube. The steel tubing offers rodent protection, but limits pre-straining. Unfortunately, slippage between the tube and the fiber has been detected at early stages. Compared to the P07 cable, the slippage issue limits applications and, therefore, this sensor has only been used as a complementary sensor in selected projects. Further advancements of the M07 have been implemented in the core of the newest cables (4.4.2).

4.4 Development of Commercial Strain Cables

The cooperation in strain cable development significantly tightened, when the author together with Brugg Cables AG and Omnisens SA received a two year research grant from the Swiss Innovation Promotion Agency in 2008 (CTI, 2008). This allowed for active participation in the design of strain sensing cables for geotechnical monitoring applications. The results were 8 versions of cables for almost any kind of soil monitoring, and thus, a valuable contribution to this emerging technology industry.
4.4.1 Strain cable 2008 (S08)

The S08 is a compact, metal free 2.8 mm diameter cable with a protective polyurethane (PU) coating. It offers good protection and excellent strain transfer. Due to its relatively low longitudinal stiffness, it can be easily pre-strained during integration. The S08 was used as a soil-embedded sensor (Chapter 6 and Chapter 11) and was integrated into the monitoring ground anchor (Chapter 7). In addition, it was tested extensively in the laboratory.

4.4.2 Strain cables developed during 2009 to 2011

As the present study was already concluding, when the new cables V2 to V8 came up, it was not possible to integrate them in the demonstrated field projects. Nevertheless, this study was a major contribution to the development of these mature cable products (Figure 31) that can be used in almost any kind of geotechnical monitoring environment. Applications using V2 to V8 can be found in Hauswirth (2010a), Hauswirth et al. (2010) and Hauswirth et al. (2011).
### Figure 30: Custom designed strain sensors (PA–polyamide; TPE-thermoplastic elastomer; EA–longitudinal stiffness).
<table>
<thead>
<tr>
<th>Cable Details</th>
<th>Sketch</th>
<th>Picture</th>
</tr>
</thead>
<tbody>
<tr>
<td>PU protected strain cable</td>
<td><img src="image1.png" alt="Sketch of a PU protected strain cable" /></td>
<td><img src="image2.png" alt="Picture of a PU protected strain cable" /></td>
</tr>
<tr>
<td>Diameter EA</td>
<td>6 mm x 3 mm</td>
<td>140 kN</td>
</tr>
<tr>
<td>Product type</td>
<td>Prototype from cable manufacturer</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cable Details</th>
<th>Diameter EA</th>
<th>Picture</th>
</tr>
</thead>
<tbody>
<tr>
<td>V2 Duplex (strain &amp; temperature) fibers in 0.15mm steel tube, PA outer protective sheet.</td>
<td>6 mm x 3 mm</td>
<td>140 kN</td>
</tr>
<tr>
<td>V3 Fiber in 0.15mm steel tube, steel armored (braiding), corrugated PA outer protective sheet.</td>
<td>7 mm</td>
<td>470 kN</td>
</tr>
<tr>
<td>V4 Fiber in 0.15mm steel tube, PA outer protective sheet.</td>
<td>3.2 mm</td>
<td>58 kN</td>
</tr>
<tr>
<td>V5 Fiber in 0.1mm steel tube, light steel armoring (braiding), PA outer protective sheet.</td>
<td>2.8 mm</td>
<td>44 kN</td>
</tr>
</tbody>
</table>

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<tr>
<th>Sketch</th>
<th>Picture</th>
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<tbody>
<tr>
<td><img src="image3.png" alt="Sketch of V2 cable" /></td>
<td><img src="image4.png" alt="Picture of V2 cable" /></td>
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</tbody>
</table>

<table>
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<tr>
<th>Cable Details</th>
<th>Diameter EA</th>
<th>Picture</th>
</tr>
</thead>
<tbody>
<tr>
<td>V6 Fiber in 0.1 mm steel tube, light steel armoring (braiding), corrugated PA outer protective sheet.</td>
<td>6.6 mm</td>
<td>Estimated 150 kN</td>
</tr>
<tr>
<td>V7 Fiber in 0.15mm steel tube, steel armored (braiding), corrugated PA outer protective sheet.</td>
<td>3.3 mm</td>
<td>44 kN</td>
</tr>
<tr>
<td>V8 Fiber in 0.15mm steel tube, PA outer protective sheet.</td>
<td>5 mm</td>
<td>Estimated 150 kN</td>
</tr>
</tbody>
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<tr>
<th>Sketch</th>
<th>Picture</th>
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</thead>
<tbody>
<tr>
<td><img src="image5.png" alt="Sketch of V6 cable" /></td>
<td><img src="image6.png" alt="Picture of V6 cable" /></td>
</tr>
</tbody>
</table>

Figure 31: Prototypes of commercial strain sensors developed within the framework of this study (PA-polyamide; PU-polyurethane; EA-longitudinal stiffness).
4.5 Laboratory Testing Setup and Sensor Parameters

For the implementation of fiber-optic strain sensors into geotechnical monitoring projects, several parameters of the proposed sensor must be known:

- Longitudinal stiffness;
- Cable maximum tensile strain;
- Strain and temperature coefficient of the Brillouin frequency shift;
- Approximate reference Brillouin frequency shift of the unstrained cable at room temperature;
- Qualitative and quantitative information about the fiber bonding to the protection layers (slippage).

To quantify these parameters, laboratory tension testing with and without simultaneous BOTDA/BEDS measurements were carried out.

4.5.1 Laboratory testing setup

For the laboratory testing, a displacement controlled setup called the fiber strain calibration device (FSC) has been designed. The FSC (Figure 32) allows for fixation and straining of fibers to predetermined strains. The principle of the FSC is that a fiber fixed at points \(x_j\) and \(x_{j+1}\) is strained over the section of length \(L_{sj}\) by applying displacement \(\Delta x_{j+1}\) through a step motor at \(x_{j+1}\). Pulling force, \(P\), at the step motor is measured by a load cell. At each load step the Brillouin frequency shift is obtained.

![Figure 32: Fiber strain calibration device (FSC).](image-url)
4.5.2 Mechanical sensor parameters

For all cables, stress-strain dependency has been recorded, whereas average stress, \( \sigma_{xc} \), in the cable was calculated from the load, \( P \), and the cable cross section of the unstrained cable, \( A_c \):

\[
\sigma_{xc} = \frac{P}{A_c} \tag{4.1}
\]

**Longitudinal stiffness**

Longitudinal stiffness, \( E_c A_c \), for elastic deformation is calculated from testing, using recorded pull force, \( P \), and displacement, \( \Delta x_{j+1} \). \( E_c \) is the mean apparent elastic modulus of the cable.

\[
E_c A_c = \frac{P}{\varepsilon_T} = \frac{P}{\Delta x_{j+1}/L_{ij}} \tag{4.2}
\]

**Relationship above elastic limit strain**

Testing of the cables usually went further than the elastic limit (the proportionality limit \( \varepsilon_y \)). For the range beyond, a polynomial fit to the test data expresses true strain, \( \varepsilon_T \), in the cable as a function of applied pull load, \( P \).

**Creep and aging**

It was out of the scope of this study to analyze creep and aging effects of the sensors. Further information on this topic can be found, e.g. in Lin et al. (2005), Ding et al. (2006) and Song et al. (2010).

**Slippage**

By analyzing the BOTDA/BEDS data, qualitative and quantitative information about the fiber bonding to the protection layers can be obtained.

When the strain in a cable changes rapidly from one section to another at a fixation point, the interfaces of the cable layers down to the fiber are subject to a large shear stress. Often, this shear stress surpasses its maximum and the interface strain redistributes over a larger area. This means, the fiber slips inside the protection. Although theoretical investigation into slippage has been
performed (e.g. LeBlanc, 2005), the optical measurement data allows for individual cable slippage characterization. Two values are of interest: strain limit, \( \varepsilon_{cs} \), above which slippage occurs and, \( \Delta L_s(\varepsilon_c) \), the slippage progression along the cable.

### 4.5.3 Optical sensor parameters

**Strain coefficient of Brillouin frequency shift**

As the experiments were carried out in a laboratory environment with almost constant temperature over the experimental period, the strain coefficient \( C_\varepsilon \) can be calculated from Eq. (1.6) as:

\[
C_\varepsilon = \frac{\nu_\beta(\varepsilon) - \nu_\beta(\varepsilon_0)}{(\varepsilon - \varepsilon_0)}
\]  
(4.3)

**Temperature coefficient of Brillouin frequency shift**

The temperature coefficient, \( C_T \), of the Brillouin frequency shift was not assessed as part of this study, but was obtained from sources mentioned in the corresponding sections.

**Reference Brillouin frequency shift of the unstrained cable at room temperature**

The reference Brillouin frequency shift of the unstrained cable at room temperature, \( \nu_{\beta 0}(\varepsilon_0, T_0) \), is easily obtained at the start of each measurement.

Note, that for advanced cable designs, this value might significantly change over the length of the cable. However, as strain measurements are always relative to the reference measurement, the impact of this irregularity is limited.

**Cable maximum tensile strain allowing for optical measurements**

The cables used in the tests were usually strained until the optical signal transmission failed (fiber break). In this case, the optical strain measured at the last step before failure was taken as the maximum tensile strain allowing for optical measurement, \( \varepsilon_{cm} \).
Note that this is a lower bound value, always below the tensile strain, $\varepsilon_{cf}$, and strength, $\sigma_{cf}$, at cable failure (cable break). No statistically valid study was carried out to find the maximum tensile strain, at which optical measurements were still possible.

### 4.6 Strain Testing

As a reminder: details of the strain sensing cables tested can be found in Figure 30 and Figure 31 (pp. 67).

#### 4.6.1 BSM

**Testing**

The laboratory testing of the BSM (Figure 30) has been performed on three different sections lengths, $L_s$ (0.1 m, 1 m and 2 m). Figure 33a shows typical displacement-load raw data for the BSM and Figure 33b shows the strain-stress dependency recorded during the three tests.

**Brillouin frequency shift measurements**

At each strain step, optical measurements were taken. In the case of the 1 m BSM, the Brillouin frequency shift was measured by BOTDA, whereas for the 0.1 m and the 2 m BSM, the applied method was BEDS. The Brillouin raw data was cleared of apparent errors (occurring due to the Lorentzian curve fit on the sometimes blurred Brillouin gain spectra). The measurements are shown in Figure 34.

**Obtained parameters**

The parameters obtained from the testing are summarized in Table 8 and Table 9.

As the BSM is at the core of all fiber cables, a short discourse follows here regarding the stress (and strain) that these fibers can be exposed to and its mean apparent elastic modulus.
Fiber failure

It has been reported that the intrinsic strength of silica fibers can reach, in short gauge lengths, maximum tensile strengths, $\sigma_{xcf}$, of 14 GPa (Tetelman & McEvily, 1967). However, usually defects (located either on the surface or internally) act as stress concentrators and initiate fractures at much lower levels of applied stress (Olshansky & Maurer, 1976). Additionally, the fixation may cause locally high stresses due to micro-bending or lateral pressure on the fiber. This was also the cause in the present BSM testing, as it always broke at the fixation point. Tensile strength, $\sigma_{xcf}$, of the BSM composite (glass and buffer coating) was encountered between 110 MPa and 310 MPa.

Mean apparent elastic modulus

From the BSM testing, the mean apparent elastic modulus was assessed as $E_c = 18.1$ GPa. For validation, $E_c$ can also be calculated from estimated values: $E_p$ (Figure 33b) of the protective coating ranges from 1.05 GPa to 2.65 GPa (Fabian et al., 2006; Antunes et al., 2008). $E_f$ (Figure 33b) for the core and cladding falls between 65 GPa and 72 GPa (Giallorenzi et al., 1982). With the respective cross sections of $A_c = 0.049 \text{ mm}^2$, $A_p = 0.037 \text{ mm}^2$, and $A_f = 0.012 \text{ mm}^2$, the calculated $E_c$ lies between 17 GPa and 20 GPa:

$$E_c = \frac{E_p \cdot A_p + E_f \cdot A_f}{A_c}$$  \hspace{1cm} (4.4)

4.6.2 TSM

Testing and measurements

Laboratory tests of the TSM had been performed on sections of length $L_{sj} = 3$ m. Figure 35 shows typical load-displacement raw data and the stress-strain dependency of the TSM. The Brillouin frequency shift was measured by BOTDA (Figure 36).
Figure 33: BSM laboratory testing: (a) Load-displacement and; (b) stress-strain from three tests of different length $L_{sj}$ and a fitted $E_c$.

Figure 34: BSM Brillouin frequency shift for a strained BSM section $L_{sj}$ (a) 2 m; (b) 1 m and; (c) 0.1 m. Optical measurements were taken using BOTDA (b) and BEDS (a, c).
Obtained parameters

The parameters obtained from the testing are summarized in Table 8 and Table 9.

When comparing the load-displacement curves of the TSM (Figure 35a) with the BSM (Figure 33a), a plastic behavior of the TSM becomes evident and surprisingly, failure load is smaller than reached by BSM. Given the structure of the TSM, which is basically a BSM with additional protection ($A_p = 0.62 \text{ mm}^2$), this fact can only be explained by taking the weak bonding between the protection and the fiber into consideration. Most of the load is absorbed by the protection and is not transferred to the fiber. Therefore, strain in the fiber is different than strain in the protection. At small strains up to 0.2%, TSM stress-strain relationship is elastic and similar to a BSM (Figure 35b). Protection contribution is negligible and bonding seems to be intact. Above 0.2%, the plastic behavior dominates.

Slippage can be clearly seen in Figure 36. The Brillouin frequency shift from the unstrained to the strained section increases over a transition zone in the meter range, where-as, compared to the BSM (Figure 34), this happens in one step. This is a problematic aspect of the TSM.

![Figure 35: TSM laboratory testing: (a) Raw data load-displacement and; (b) stress-strain from tests and a fitted curve of the stiffness.](image)
Figure 36: TSM Brillouin frequency shift obtained for 3m long strained sections.

Table 8: Mechanical cable parameters obtained from laboratory testing.

<table>
<thead>
<tr>
<th>Cable</th>
<th>Cable cross section</th>
<th>Longitudinal stiffness</th>
<th>Elastic limit strain</th>
<th>Load-strain relationship $\varepsilon(P)$ $\varepsilon \text{ [}\mu\text{e}]; P \text{ [}N]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSM</td>
<td>0.0491 [mm$^2$]</td>
<td>0.89 [kN]</td>
<td>-</td>
<td>$\varepsilon_T = \frac{P}{0.00089}$</td>
</tr>
<tr>
<td>TSM</td>
<td>0.636</td>
<td>0.79 [kN]</td>
<td>0.2</td>
<td>$\varepsilon_T = \frac{P}{0.00079}$; $\varepsilon_T = 89.7 \cdot P^3 - 616 \cdot P^2$ $+ 4804 \cdot P - 4440$</td>
</tr>
<tr>
<td>P07</td>
<td>2.01</td>
<td>3.3 [kN]</td>
<td>1.1</td>
<td>$\varepsilon_T = \frac{P}{0.0033}$; $\varepsilon_T = 2.88 \cdot P^2$ $+ 321 \cdot P - 5235$</td>
</tr>
<tr>
<td>S08</td>
<td>6.16</td>
<td>2.9 [kN]</td>
<td>0.9</td>
<td>$\varepsilon_T = \frac{P}{0.0029}$; $\varepsilon_T = 2.58 \cdot P^2$ $+ 241 \cdot P + 870$</td>
</tr>
</tbody>
</table>
Table 9: Cable parameters obtained from laboratory testing. (*Niklès, 2007; **Omnisens, 2008 and Hauswirth, 2010b)

<table>
<thead>
<tr>
<th>Cable</th>
<th>Brillouin frequency shift coefficients</th>
<th>Max. tensile strain reached</th>
<th>Slippage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$C_r$ [MHz/%] $C_T$ [MHz/°C] $\nu_{B0}$ [GHz] $\varepsilon_{cm}$ [%] $\varepsilon_{cs}$ [%]</td>
<td>$\Delta L_{s5}(\varepsilon_T=0.5%)$ $\Delta L_{s10}(\varepsilon_T=1%)$ [m]</td>
<td></td>
</tr>
<tr>
<td>BSM</td>
<td>507 0.95 * 10.86 1.2 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TSM</td>
<td>500 0.95 * 10.85 0.9 0.2</td>
<td>$\Delta L_{s5} = 2$ $m$ $\Delta L_{s10} = $ meters</td>
<td></td>
</tr>
<tr>
<td>P07</td>
<td>455 0.95 * 10.94 3.5 0.2</td>
<td>$\Delta L_{s5} = 0.2$ $m$ $\Delta L_{s10} = 0.4$ $m$</td>
<td></td>
</tr>
<tr>
<td>S08</td>
<td>500 4.8** 10.57 3.8 1.2</td>
<td>$\Delta L_{s5} = 0$ $m$ $\Delta L_{s10} = 0$ $m$</td>
<td></td>
</tr>
</tbody>
</table>

4.6.3 S06

This cable was a step towards the more refined P07 cable. It was only used in one field project and therefore, no laboratory testing was performed. As preliminary tests of pulling short TSM sections out of the S06 compound already indicated that the TSM breaks before debonding, the data obtained for the TSM (Table 9) was considered valid for the S06. Longitudinal stiffness is estimated at around 2 kN.

4.6.4 P07

Testing

A total of five laboratory strain tests with the P07 (Figure 30) were performed on 3 different sections of length $L_{sj}$ (0.1 m, 1 m and 2 m). Figure 37 shows the stress-strain dependency obtained.
Brillouin frequency shift measurements

The Brillouin frequency shift was measured by BEDS in two tests (e.g. Figure 38 a and b) and by BOTDA in the remaining 3 (e.g. Figure 38c).

Obtained parameters

The obtained parameters from the testing are summarized in Table 8 and Table 9.

From Figure 38, the slippage of the fiber inside the protection can be clearly seen (especially from the BEDS data, which offers a very detailed picture). This allows studying the slippage progression for increasing strain (Figure 39) and provides the limits for the application of the P07 cable. However, it is recognized that the P07 accurately monitors strain to very high levels and that slippage does not progress further than 1 m, which is in the spatial resolution range of BOTDA units. This cable was the best choice available at the time and contributed to the successful monitoring of many projects in this study.

For short sections, (e.g. $L_{sp} = 10$ cm), slippage quickly progressed into the strained section and, even at low strains, no exact measurements are possible. This excludes this cable for crack detection and similar applications.

![Figure 37: P07 stress-strain from tests and a fitted curve of the stiffness.](image)
Figure 38: P07 Brillouin frequency shift obtained by BEDS for $L_{sj}$ (a) 2 m; (b) 0.1 m and; (c) by BOTDA for $L_{sj}$ 1 m.

Figure 39: P07 slippage progression for increasing strain from several tests.
4.6.5 M07

This cable is a prototype, where slippage has been encountered at an early stage. Therefore, the cable was used in complementary arrangements with other sensor cables. Parameters were obtained by calculation ($E_cA_c = 70 \text{ kN}$) and back calculation of the laboratory anchor tests (the obtained $C_ε = 740 \text{ MHz/}%$ is much larger than with all other cables). The observations from the application of this cable were included by the cable manufacturer in the development of new, steel tube embedded cables. In general, steel tubing offers excellent temperature properties (uniform response) and allows for cyclic loading and rodent protection.

4.6.6 S08

Testing

Two laboratory strain tests with the S08 (Figure 30) were performed on sections of length $L_{sj} = 2 \text{ m}$. Figure 40 shows the stress-strain dependency obtained.

Brilouin frequency shift measurements

The Brilouin frequency shift was measured by BOTDA in one test (Figure 41a) and by BEDS in the other one (Figure 41b).

Obtained parameters

The obtained parameters from the testing are summarized in Table 8 and Table 9.

From Figure 41 it can be seen that slippage occurs only at high strains and its progression is very limited. In Figure 42, the slippage progressions from two tests are analyzed. Slippage in the S08 sensor occurs for strains larger than 1.2% and even then does not go beyond 0.5 m for very large strains up to 3%. The obtained parameters in terms of elasticity, strain readings up to very large strains and limited slippage, make this sensor applicable in many different monitoring projects, including crack detection and similar applications.
Figure 40: S08 stress-strain from tests and a fitted curve of the stiffness.

Figure 41: S08 Brillouin frequency shift obtained for \( L_{sj} = 2 \) m and strain up to almost 3% by (a) BOTDA and; (b) BEDS.
Figure 42: S08 slippage progression for increasing strain from several tests.

4.7 Conclusions on Sensor Performance

The sensor development and corresponding laboratory testing explicitly demonstrate the progress in sensor development over the last five years. It was shown that the most important issue, the slippage between the connection and the protection layers down to the bare fiber can be solved. A new method suggested in this study, the slippage progression monitoring by novel BEDS technology strongly supports this claim.

From the custom cables tested, the S08 is the most promising for geotechnical monitoring applications as it offers low longitudinal stiffness (pre-straining) and very limited slippage. Even if protection from rodents and the harsh construction environment is limited (due to its metal-free design), an ongoing project (Chapter 11) shows that this cable is able to survive the soil environment in the long term (currently > 30 months and counting).

A significant contribution to the industry could be achieved by implementing the findings of the laboratory and field testing in new cable designs. The parameters: longitudinal stiffness, protection, and slippage need to be deliberately arranged for different applications. The presented advanced cable versions, V2 to V8, offer applicability in many various environments and, thus, broaden the applicability of fiber-optic monitoring to geotechnical applications.
PART II

NOVEL FIBER-OPTIC APPLICATIONS IN THE FIELD OF SOIL-STRUCTURE INTERACTION
5 State of the Art

The second Part of the thesis focuses on the original objective: the improved understanding of progressive failure in soil-structure interaction. The fiber-optic strain sensing introduced in Part I will now be applied to monitoring strain along long, one-dimensional structures, particularly sand-embedded cables and ground anchors, subject to pullout testing. The experimental data reveals a phenomenon which had not been clearly stated in the geomechanical literature: namely that for certain boundary conditions, the residual shear stress increases with increasing pullout load. At the end of this Part, a simplified analytical model is derived in order to explain this phenomenon.

This Chapter contains an introduction to progressive failure and the literature review. To understand progressive failure, basic geomechanical concepts are first introduced. Then, a literature review summarizes the state of the art in the little-investigated lab scale pullout testing of cable-like structures and the more well-documented field scale pullout testing of ground anchors. Possibilities for further research and open questions are identified.

5.1 Introduction to Progressive Failure

The motivation for researching progressive failure is rooted in the fact that progressive failure leads to either significantly lower or higher bearing capacity of structures embedded in soil than predicted by conventional analysis.

In order to illustrate progressive failure, consider the following problem of a rod pulled out of sand (Figure 43a). The shear stress (τ) between rod and sand is drawn in Figure 43b and depends on the relative displacement (δ) between rod and sand. With increasing displacement, the shear stress is growing up to a peak value, τ_p, at δ_p and thereafter, for further displacements its value is decreasing to a residual value, τ_r.

For a perfectly rigid rod (inextensible), the problem simplifies, as relative displacement (δ) is uniform along the rod during pullout. Consequently, shear
stress reaches its maximum ($\tau_p$) uniformly along the rod simultaneously. Maximum pullout force can then be calculated by simply multiplying the shear area $A$ by $\tau_p$ (Figure 44a). After reaching this maximum, catastrophic failure will occur and pullout resistance drops to $\tau_r \times A$.

With a very low rigidity rod, relative displacement $\delta$ is not the same along its length and therefore, $\tau_p$ is not reached at the same time. So, if the loading is gradually increased, progressive failure occurs along the rod-sand interface. The maximum pullout force in this case is given as the residual shear stress ($\tau_r$) multiplied by the area (Figure 44a).

These two extreme cases produce upper and lower bound estimates for the pullout force. The peak pullout force in rods with finite rigidity, such as optical cables and ground anchors, actually fall somewhere in between (Figure 44a). The shear stress distribution along an infinitely rigid rod and one with finite rigidity is shown in Figure 44b.

The insufficient level of understanding of the progressive failure phenomenon at the individual structure level does not allow for the development of accurate, quantitative approaches for analysis and design of these structures. Improving the understanding may help to provide a safer design for structures like a ground anchor. In addition, efficient application of the observational method by monitoring progressive failure can be achieved.

![Figure 43: (a) A rod pulled out of sand and; (b) shear stress between rod and sand.](image)
Studies on progressive failure in soil-structure interaction are normally performed during pullout tests. For laboratory testing, objects are embedded in pullout boxes. For field testing, full scale ground anchors, soil nails and similar objects are pulled out.

In the most basic configuration, only the displacement and the pullout force are recorded, which leads to satisfying results in the two distinctive cases: very high and very low rigidity.

Testing conditions, where a material can be assumed as inextensible (very rigid), are obtained in small scale laboratory pullout tests (e.g. Hof, 2003; Chu & Yin, 2005; Benmokrane, 1995b). Catastrophic failure allows for peak shear stress calculation. Very low rigidity (e.g. planar reinforcement: Gurung et al., 1999; compact geotextiles: Zanziger, 2001; Meyer et al., 2004) allows for residual shear stress calculation.

The present study concentrates on a more refined setup, where strain in the structure is measured in addition to displacement and pullout force. This is necessary for structures with finite rigidity, e.g. ground anchors or cables embedded in soil. From quality distributed strain data, shear stress distribution can be readily calculated.

### Figure 44: (a) Total and residual pullout load of rods with different stiffness and; (b) shear stress distribution along a rod pulled out of sand.
5.2 Basic Geomechanical Concepts

5.2.1 Dilation in cohesionless soil

When soil is sheared along a slip plane, volume change can be noticed during shear to failure. Reynolds (1885) has shown, that for dense sands, the volume increases exhibiting a phenomenon now known as Reynold’s dilatancy, or simply dilatancy. Loose sand, in contrast, contracts in the same situation. Reynolds’s conclusions are based on particle movements during the process of shearing, which do not necessarily occur in the direction of applied shear stress.

To illustrate the above case, Figure 45 (after Taylor, 1948) shows data from direct shear tests on both loose and dense sands. It can be seen that, for dense sand, the volume increases during shear and, at considerably greater strains, a constant volume is reached, while the shear stress reaches a peak and later drops to the residual value. For loose sands, the volume first drops and later increases again. The maximum shear stress is only reached after considerable strain and without passing a previous peak.

Bolton (1986) summarized the works performed on strength and dilatancy of sands (among others: Rowe, 1962; Roscoe, 1970) and proposed a relative dilatancy index $I_R$, which is used to estimate the rate of dilatancy based on a correlation with the relative density $I_D$ (see also Eq. 6.1):

$$I_R = I_D (10 - \ln p') - 1$$  \hspace{1cm} (5.1)

where $p'$ is the mean effective stress at failure. Thus, the effective stress and soil density both affect the rate of dilatancy of soils and, thereby, the strength parameters. For a plane strain, Bolton found the following correlation:

$$\phi' - \phi'_{\text{crit}} = 0.8 \cdot \psi_{\text{max}} = 5 \cdot I_R$$  \hspace{1cm} (5.2)

where $\phi'$ is the peak angle of shear resistance and $\psi_{\text{max}}$ is the peak angle of dilation. The residual shear resistance ($\phi'_{\text{crit}}$) is the state at which the shear stress and density remain constant while the shear strain increases.
In drained conditions, the soil is free to dilate or contract during shear. Such conditions were present in the tests shown in the following Chapters.

Figure 45: Direct shear tests for loose and dense sands (after Taylor, 1948).

5.2.2 Progressive failure in soils

In the geotechnical literature, the term “progressive failure” is mainly associated with any growth of a slip surface in material with strain-softening behavior (e.g. Terzaghi & Peck, 1948; Bjerrum, 1967; Saurer, 2009). The slip surface develops as shear bands: areas of large shear deformation with thickness of about eight to ten times the mean grain diameter (Wong, 2000) between regions with relatively small shear deformation. Inside the shear bands, the large deformation causes shear resistance to drop from peak ($\tau_p$) to residual strength ($\tau_r$). In soils, the progressive failure phenomenon has
received a great deal of attention starting in the 1940’s (Terzaghi & Peck, 1948; Taylor, 1948).

5.2.3 Shear zone in soil-structure interaction

The development of a shear zone (or shear band) along a soil-structure interface depends on the roughness of the interface. When the interface is smooth (e.g. smooth steel or a cable surface), there is no formation of such a shear zone. The sand particles slide along the steel. Shear force and volume stay basically constant. If the interface is rough, particles close to the interface slip, roll and shift along the interface. Thus, a shear zone forms and, consequently, frictional resistance drops and volume change (dilation) occurs (Uesugi et al., 1988; Milligan & Tei, 1998). An extensive research summary on granular-continuum interface shear failure is provided by Frost et al. (2002).

5.3 Cable Pullout

The most convenient and popular means to obtaining maximum pullout resistance in soil-embedded objects are to first perform pullout tests. Such pullout tests have been performed on a large number of materials with different dimensions. However, the focus here is on the ones widely used in geotechnical constructions: steel nails and planar reinforcements (strips, geotextiles and geogrids). Nevertheless, some examples of pullout tests with cable-like structures exist, as well as the load transfer monitoring along these structures.

5.3.1 Laboratory pullout tests

In agreement with the small dimensions of cable-like structures, these tests are carried out in the laboratory in so-called pullout boxes, where stationary test procedures and boundary conditions can be provided. The research of Palmeira & Milligan (1989) is crucial with regards to the design of pullout boxes, as they have thoroughly investigated the influence of different boundary conditions on the pullout testing results.
The standard interpretation of pullout tests concludes with the apparent friction coefficient, $\tan \bar{\delta}$, (Abramento & Whittle, 1995a):

$$\tan \bar{\delta} = \frac{P_{\text{max}}}{A \cdot \sigma_n}$$ \hspace{1cm} (5.3)

where $P_{\text{max}}$ is the reported maximum pullout load, $A$ is the surface area and $\sigma_n$ the normal stress on the surface.

With regard to the objects pulled out, a few examples are given in Table 10. The review is separated into three sections: soil nails, planar reinforcements, and cable-like structures. As the literature does not provide for any pullout tests specific to sand-embedded cables, the term “cable-like structures” will refer to cylindrical and planar (width<<length) objects with comparably low stiffness. An additional and important property is the surface condition: smooth or rough.

### 5.3.2 Load transfer monitoring

The only report on laboratory load transfer monitoring along a cable-like soil-embedded structure is from Berilgen & Özaydin (1997). The aluminum pipe described in Table 10 was instrumented with three electrical strain gauges glued to the outer surface. Figure 46 shows the stress distribution obtained during pullout and compared with results from Finite Element Modeling (FEM). As is seen, in principle, it was possible to measure the normal stress at three points along the structure during pullout. However, the data seems to exhibit quite a large deviation and low accuracy.

Another laboratory test that can be considered cable pullout related is the monitoring of axial strain distribution along a fiber being pulled out of a fiber-reinforced composite. Such a case was reported by Peters et al. (2002), who monitored strain by FBG’s in fibers subjected to pullout forces. It was possible to back-calculate the fiber load from the strain measurements. However, it has to be mentioned that the bonding in the reinforced composite is mainly cohesive, not frictional, and thus, can barely be compared with soil-embedded cable pullout. Additional examples can be found in the studies of bond between concrete and reinforcement (e.g. Ullner, 2008).
Table 10: Examples of pullout resistance testing.

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Soil nail</strong></td>
<td><strong>Steel nail: Rough and smooth surface</strong></td>
</tr>
<tr>
<td></td>
<td>Examples of laboratory steel nail pullout can be found from various authors. E.g. Milligan &amp; Tei (1998) pulled out several 200 mm long nails with rough and smooth surface. Others: Junaideen et al. (2004); Chu &amp; Yin (2005).</td>
</tr>
<tr>
<td></td>
<td><strong>Aluminum pipe: Rough surface</strong></td>
</tr>
<tr>
<td></td>
<td>Berilgen &amp; Özaydin (1997) pulled out 200 mm long, 25 mm diameter sand-embedded aluminum pipes ($E_A c = 10^4$ kN). The rough surface was created by gluing sand grains to the pipe with epoxy. This pipe is about three orders of magnitude stiffer than the cables P07 and S08, but also has a ten times larger circumference.</td>
</tr>
<tr>
<td><strong>Planar reinforcement</strong></td>
<td><strong>Planar inclusions: Rough and smooth surface</strong></td>
</tr>
<tr>
<td></td>
<td>Abramento &amp; Whittle (1995b) performed a series of pullout test on about 0.4 m long and 130 mm wide steel (smooth) and nylon (rough) sheath inclusions.</td>
</tr>
<tr>
<td></td>
<td>Mevellec (1977) pulled 150 mm wide and 1000 mm long smooth bronze strip out of sand.</td>
</tr>
<tr>
<td></td>
<td>Ingold (1982) summarizes and reports on several pullout tests with various materials used for the development of reinforced earth.</td>
</tr>
<tr>
<td></td>
<td><strong>Geogrid/geotextile strip: Rough surface</strong></td>
</tr>
<tr>
<td></td>
<td>Palmeira et al. (1996) reported pullout tests of 200 mm long and 100 mm wide geotextile strips at relatively high confining pressure.</td>
</tr>
<tr>
<td></td>
<td>Alfaro &amp; Pathak (2005) pulled out 1200 mm long and 300 mm wide geogrids of setups which allowed for free and restrained dilation.</td>
</tr>
<tr>
<td></td>
<td>Farrag et al. (1993) obtained the pullout resistance of different geogrids (1000 mm by 300 mm).</td>
</tr>
<tr>
<td><strong>Cable-like structure</strong></td>
<td><strong>Rubber tube: Rough surface</strong></td>
</tr>
<tr>
<td></td>
<td>Milligan &amp; Tei (1998) pulled out 200 mm long, 3 mm diameter sand-embedded rubber tubes ($E_A c = 0.08$ kN). The cables of the present study (P07, S08), which are of equal dimensions, are 30 to 40 times stiffer than this rubber tube.</td>
</tr>
<tr>
<td></td>
<td><strong>Other</strong></td>
</tr>
<tr>
<td></td>
<td>For a first consultation, the works of Berilgen &amp; Özaydin (1997) and Abramento &amp; Whittle (1995b) on the aluminum pipe and the nylon sheath are useful.</td>
</tr>
</tbody>
</table>
5.3.3 Critical assessment

Even though a broad range of pullout tests have been performed, the specific case of pulling out a soil-embedded cable while continuously monitoring shear stress distribution along the cable has not been investigated. The main two reasons for this are:

1) Challenging instrumentation of small scale, extensible structures;

2) Insignificant economical interest (compared to ground anchors).

Nevertheless, from a scientific point of view, investigation of soil-structure interaction at such a small scale is of high interest. First of all, it helps to develop or verify models describing progressive failure in similar problems. In addition, the understanding of cable-soil interaction is necessary for the correct interpretation for the soil-embedded landslide monitoring system described in Chapter 11.

The challenge in the instrumentation of small scale, extensible structures lies in acquiring strain without interfering with the object. The attaching of numerous strain gauges might change the objects properties (longitudinal stiffness, dimensions). This is the point where fiber-optic sensors bring in
advantage of small size and a large number of strain gauges along a single fiber. The fiber-optic cable in these tests is both, the structure and the sensor.

From the literature review, it becomes evident that there is a lack of progressive failure monitoring tests along one-dimensional, flexible structures at the lab scale.

5.4 Ground Anchors

In contrast to the soil-embedded cable, ground anchors and rock bolts have been thoroughly investigated. They are, for safety as well as economic reasons, key components of many construction and mining projects. Rock bolts are generally shorter and lower capacity, and ground anchors are longer and with higher capacity (Barley & Windsor, 2001). In this study, the focus is put on ground anchors and, where applicable, a side glance at rock bolts is included.

5.4.1 Definitions and background on pre-stressed ground anchors

Temporary and permanent pre-stressed ground anchors are applicable to retaining structures, for slope stabilization and to prevent uplift. Their main structural components are: anchorage head, bearing plate, tendon, protective sheath, and grouted body as shown in Figure 47. The total anchor length is separated into two parts: the fixed anchor length, $l_{v}$, which is fully embedded in the grouted body and the free anchor length, $l_{f}$. The tendon can be a solid bar, a hollow bar, or a strand tendon.

In the case of bonding in ground which is not likely to creep, the limit load of a ground anchor corresponds to the smaller of the two following values:

- Internal ultimate resistance $R_i$ (failure of the tendon);
- External ultimate resistance $R_e$ (failure of the bond).

According to common practice (e.g. SIA 267, 2003), the external ultimate resistance shall be greater than the internal ultimate resistance. This allows for larger displacements at the anchor head before failure, as the tendon’s
potential strain is much larger than the displacement between the grout body and soil. The internal ultimate resistance is easily calculated using the material properties of the tendon. External ultimate resistance, empirically investigated or theoretically calculated, is not reliable. This is represented by the fact that most regulatory codes are reluctant to stipulate a specific procedure to evaluate the factor of safety for the calculated external ultimate resistance and, therefore, a minimum factor of safety of at least 2.5 to 3 is suggested unless full-scale field tests confirm using a lower value. (Benmokrane et al., 1995a; Xanthakos, 1991). In practice, external ultimate resistance is assessed by anchor tests.

Figure 47: Schematic representation for the terminology of a prestressed ground anchor (after SIA 267, 2003).

5.4.2 Conventional anchor testing and monitoring – Swiss Codes

In this section, a testing program complementing a construction project and the long-term monitoring of the anchors is reviewed. Please note: not covered are the stressing techniques and systems, anchor head and bearing plate
installations, and the corrosion protection issue that exists in the typically aggressive soil environment (humidity, water seepage and salt solutions).

Testing of a ground anchor is usually carried out after a minimum of 7 days following the installation (ASTRA, 2007), so that the grout has time to harden, before any stress is applied. These 7 days are a compromise between design criteria and the construction schedule (Xanthakos, 1991; Chu & Yin 2005).

**Anchor tests**

The Swiss Code (SIA 267, 2003) states that: “In general, for each zone of ground with similar geotechnical properties at least 3 anchor tests shall be carried out. Anchor tests are carried out in advance or at the beginning of the anchor installation works. Further tests have to be carried out during execution of the structure if unexpected ground conditions are encountered.”

The test anchor tendon has to be reinforced, so that loading is possible without causing internal failure $R_i$.

A conventional testing program consists of stressing the anchor stepwise and recording force-displacement. At each load step, $k$, creep behavior is analyzed in terms of creep rate, $c_k$:

$$c_k = \frac{\delta_{h2} - \delta_{h1}}{\log\left(\frac{t_2}{t_1}\right)}$$

Where, $\delta_{hi}$, is the anchor head displacement at time $t_i$. Once reaching the level at which the movement of the grouted body passes the critical value of creep rate, $c_{krit} = 2$ mm, testing is stopped. This is generally defined as the external ultimate resistance $R_a$ (SIA 267, 2003). Subsequent to each load step, the anchor is relaxed for elastic and plastic deformation evaluation.

**Control tests**

With the control test, the load carrying capacity of the installed anchors is assessed and satisfactory service performance is ensured. In addition, the test provides a basis for acceptance of the working anchors and establishes the actual factor of safety with which the design is implemented. The test is
based on the results of the previous anchor tests. This test is conducted on 10% of all anchors and a minimum of three anchors (SIA 267, 2003). For the test, the anchors are usually loaded in three steps up to 75% of $R_i$. At each load step, creep is recorded and then the load is released for evaluation of plastic deformation.

**Monitoring**

Anchorages have to be monitored during their whole working life (SIA 267, 2003; ASTRA, 2007). The number of the control anchors is governed by the importance of the project. The absolute minimum is 5% of all anchors and at least three anchors for each section of a structure. Important control parameters are: visual checks on the condition of the structure and load, displacement, deformation, and contact stress at the soil-structure interface.

Load monitoring is important for permanent structures where the anchoring is critical to long term performance. Combined load and deformation monitoring can be important for temporary or staged construction where interim loads or movement may influence the overall design. The load monitoring is usually carried out by jacks or load cells. Experience has shown that load cells are considered to be the most accurate, convenient and allowing for remote monitoring. However, they are also costly and can be unreliable in the long term (Wymer et al., 2003). Replacement of load cells by FBG sensors has been suggested (e.g. Moerman et al., 2005).

Typically, monitoring is carried out at 6 month intervals but can be extended up to 12 months (ASTRA, 2007) if results are favorable. Anchor load control is relatively expensive and, in the case of registered load decay, it usually is not easy to discern whether this is due to failure of the tendon or of the external resistance. Thus, the less common, but very promising, concept of load distribution monitoring along the fixed length can provide useful data to this problem.
5.4.3 External resistance of anchors in non-cohesive soil

The ultimate failure of ground anchors and rock bolts may take place in the following locations:

- tendon;
- grout;
- soil or rock;
- tendon-grout interface;
- grout-soil/rock interface;
- any combination of these failure modes.

For rock bolts, failure often takes place at the tendon-grout interface (e.g. Ren et al., 2009; Benmokrane et al., 1995b). For ground anchors, failure generally occurs at the grout-soil interface (e.g. Jalalifar, 2006; Xanthakos, 1991), as mechanical interlocking of the tendon ribs prevent failures between the tendon-grout interfaces. Given the grout surface roughness, the failure actually occurs along a shear zone (on the soil-grout interface) within the soil (Milligan & Tei, 1998).

**Linear dependence on fixed length**

In a first approach, the external resistance of a ground anchor in non-cohesive soil, \( R_a \), can be estimated as:

\[
R_a = \tau_{\text{avg}} \cdot A
\]

(5.5)

where \( \tau_{\text{avg}} \) is the average ultimate shear stress over the grouted body length (similar to the interpretation for cables Eq. 5.3) and \( A \) is the cylindrical area of contact surface, depending on the diameter of the grout body, \( d_g \), and the fixed anchor length, \( l_v \).

\[
A = \pi \cdot d_g \cdot l_v
\]

(5.6)

For practical anchor design, the assumption of uniform load distribution is adopted (Ivanovic & Neilson, 2009; Chu & Yin, 2005; Barley & Windsor, 2001). In this case, the design of the fixed anchor length is based on the
hypothesis of a linear relationship existing between the fixed anchor length and ultimate load capacity.

\[ R_a \propto l_v \]  

(5.7)

**Non-linear dependence on fixed length**

The above provides for a rough estimate only, and is at odds with field, experimental, and theoretical evidence, which indicate the distribution of shear stress to be highly non-uniform (Woods & Barkhordari, 1997; Ostermayer & Scheele, 1978). Attempts have been made to introduce non-linear factors into Eq. (5.5).

**Apparent fixed length** \( l_{ve} \) (Casanovas, 1989):

\[ R_a = \tau_{avg} \cdot \pi \cdot d_g \cdot l_{ve} \]  

(5.8)

\[ l_{ve} = l_v \cdot \log(0.1\tau_{avg}) \]  

(5.9)

**Efficiency factor** acknowledging progressive failure \( f_{eff} \) (Barley, 1995):

\[ R_a = f_{eff} \cdot \tau_{avg} \cdot \pi \cdot d_g \cdot l_v \]  

(5.10)

From back-analysis of failed test anchors in sand, the following dependence was suggested:

\[ f_{eff} = (0.91)^{\tan \varphi} \]  

(5.11)

**Dependence on several factors**

Not only does the fixed length play a role in the estimation of external ultimate load of anchors, but also several other factors do, the most important ones being:

a) Relative density and degree of uniformity of the soil;

b) Dilation;

c) Grout injection method and grout pressure used.

Various tests have been conducted to examine these parameters in non-cohesive soil (e.g. Hsu & Chang, 2007; Littlejohn & Weerasinghe, 1997;
Ostermayer & Scheele, 1978). As an example, the data from 30 tests of anchors brought to failure by Ostermayer & Scheele (1978) validates the influence on external ultimate load of anchors by soil type, density and fixed anchor length (Figure 48). The fixed anchor lengths have been varied from 2 m to 10 m, while the grouted body diameter has been kept almost constant. The graph shows clearly the non-linear dependence, as ultimate load increase tapers off with increasing length. Ostermayer & Scheele (1978) concluded that this must be due to the progressive failure mechanism.

Additionally, the average ultimate shear stress along the grout body, $\tau_{avg}$, can be calculated from these tests. As an example, $\tau_{avg}$ of a 4 m long, 0.1 m diameter grout anchor embedded 4 m to 5 m deep in sandy gravel reaches 1000 kPa, which is a value considerably larger than what would be predicted applying classical soil mechanics. Also, Shields et al. (1978) and Wernick (1978), have obtained unusually high values for skin friction. A possible cause for these high values is the increase of the normal stress on the shear interface. Many authors suggest that this increase may be due to laterally restricted dilation (Alfaro & Pathak, 2005; Luo et al., 2000; Abramento & Whittle, 1995a; Xanthakos, 1991; Shields et al., 1978). In the analytical model developed in Chapter 8, this normal stress increase is due to axial stress increase.

Increasing soil density amplifies the restriction of lateral expansion and thereby increases the ultimate load carrying capacity. This assumption can be verified in Figure 48 and Figure 49, as well as from Fujita et al. (1978), who performed a correlation assessment between $\tau_{avg}$ and $N$ values from standard penetration tests.

Pressure grouting may restress the soil medium and restore it to its initial in-situ value of relative density. However, it appears from back analysis that the extent of this effect is not always certain (Xanthakos, 1991).
Figure 48: Ultimate load carrying capacity of anchors in granular soils showing influence of soil type, density and bond-to-ground length (Ostermayer & Scheele, 1978).

Figure 49: Relationship between carrying capacity, bond length of anchors and dynamic penetration resistance in two types of non-cohesive soils (Ostermayer & Scheele, 1978).
5.4.4 Load transfer monitoring

The purpose of determining the load distribution along a ground anchor is to understand the anchor’s bearing behavior, as the anchor’s performance is ultimately limited by the efficiency of load transfer from the anchor tendon to the soil via the grout.

As shown, the relationship between bond length and external ultimate resistance is not linear. It is understood that this is the result of a general incompatibility between the elastic moduli of tendon, grout, and soil on one side, and the displacement dependent grout-soil interface shear stress on the other side (Moraci & Recalcati, 2006; Barley & Windsor, 2001; Woods & Barkhordari 1997; Fujita et al., 1978).

The shear stress, $\tau$, between the grout body and the surrounding soil depends on the differential displacement, $\delta$, between the two bodies. When load is applied to the anchor head, shear stress is first concentrated over part of the grouted section being the closest to the loaded anchor head. The far end of the grouted body stays unstressed (Figure 50). With increasing load, part of the shear stress reaches its residual value and the peak shear stress shifts towards the anchors rear. Prior to the failure of an anchor, the peak shear stress arrives at the far end of the anchor (Figure 50). Typically, an anchor tendon with a 6 m fixed length will, at proof load, need to extend some 15 to 20 mm at the close end of the fixed length before any load will be transferred by the tendon at the far end (Barley & Windsor, 2001).

![Figure 50: Progressive failure along ground anchor.](image-url)
Much of the work in the literature has been devoted to load transfer from anchors to rock/soil. For rock bolts (Hagan, 2004; Signer, 1990; Yu & Xian, 1983), it was generally found that most of the load transfer takes place near the bolt head, in contrast to ground anchors in softer soils, where there is a trend of a more uniform load transfer.

Typically, for load transfer assessment instrumented anchors measure strain, and the load is then calculated using the modulus of elasticity and the area of the tendon. Table 11 summarizes a literature review of load transfer monitoring studies conducted with conventional sensor technologies and shows selected results.

Recently, several researchers (Shi et al., 2007) have proposed and implemented fiber sensors into anchor tendons to monitor load transfer. Since bare fibers are very fragile for anchor monitoring, special packaging methods have been tested. Table 12 summarizes a literature review of authors using optical sensor technologies.

5.4.5 Critical assessment

Ground anchors are widely used in construction and many aspects of ground anchor resistance have been studied. However, the load transfer mechanism is still not entirely understood. The main reason is that the studies utilized a limited number of sensors, about ten to twelve, along the ground anchor. This does not provide for a complete picture of the load distribution nor does it allow for clear detection of the failure propagation.

For example, in Table 11 b, e, and f it appears that the residual shear stress ($\tau_r$) increases with increasing load, whereas in Table 11 d and g, residual shear stress ($\tau_r$) seems to be a constant. In Table 12, an estimation of the residual shear stress ($\tau_r$) is almost impossible. Also, it is difficult to quantify the progressive failure propagation in all Figures in Table 11 and Table 12.
Table 11: Current state of the art in load transfer assessment by conventional sensor technologies.

<table>
<thead>
<tr>
<th>Publication</th>
<th>Details / figure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a) Ostermayer &amp; Scheele (1978)</strong></td>
<td>Ground anchor tendons instrumented with six strain gauges over 3 m fixed length. A sample is shown on the right.</td>
</tr>
<tr>
<td><strong>b) Shields et al. (1978)</strong></td>
<td>Installed eight anchors of about 7 m fixed length with up to ten gauges. Because of harsh conditions, it was only possible to monitor two of them. In the other anchors, too many gauges failed. A sample is shown on the right.</td>
</tr>
<tr>
<td><strong>c) Benmokrane et al. (1995a)</strong></td>
<td>Performance of an anchor instrumented with six vibrating wire strain gauges over the fixed anchor length of 3.65 m. Additionally, laboratory pullout tests on instrumented ground anchors were performed.</td>
</tr>
</tbody>
</table>
Current state of the art in load transfer assessment by conventional sensor technologies (cont.)

<table>
<thead>
<tr>
<th>Publication</th>
<th>Details / figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>d) Weerasinghe &amp; Littlejohn (1997)</td>
<td>Ground anchor tendon instrumented with six strain gauges over 3 m length.</td>
</tr>
<tr>
<td></td>
<td><img src="image1.png" alt="Graph" /></td>
</tr>
<tr>
<td>e) Kim (2003)</td>
<td>Low-pressure grouted soil anchor instrumented with seven vibrating wire gauges in the tendon over the 4 m long bonded length (and additional gauges in the grout).</td>
</tr>
<tr>
<td></td>
<td><img src="image2.png" alt="Graph" /></td>
</tr>
<tr>
<td>f) Zhang et al. (2006)</td>
<td>FRP anchor grouted instrumented with ten strain gauges over 3 m fixed length.</td>
</tr>
<tr>
<td></td>
<td><img src="image3.png" alt="Graph" /></td>
</tr>
<tr>
<td>g) Signer (1990)</td>
<td>Load transfer in rock bolts: Signer instrumented rock bolts with up to twelve strain gauges placed pairwise at six locations along the bolt.</td>
</tr>
<tr>
<td></td>
<td><img src="image4.png" alt="Graph" /></td>
</tr>
</tbody>
</table>
### Table 12: Current state of the art in load transfer assessment using optical sensors.

<table>
<thead>
<tr>
<th>Publication</th>
<th>Details / figure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a)</strong> Zhu et al. (2007) and Yin et al. (2007)</td>
<td>Glass fiber reinforced polymer (GFRP) ground anchors instrumented with FBG sensors and electrical strain gauges over 3.6 m length (thereof 2.0 m fixed length). On each anchor, seven FBG were surface mounted and four FBG were enclosed into an aluminum tube (6 mm outer diameter, gauge length 0.1 m). A strain distribution was obtained, but the different sensor values do not match very closely. Above 140 kN (&lt; 75% of ultimate load capacity), the tube packaged sensors failed. The surface mounted sensors stayed in service until the ultimate load was reached.</td>
</tr>
<tr>
<td></td>
<td><img src="image1.png" alt="Image" /> <strong>FBG sensors: 140kN</strong></td>
</tr>
<tr>
<td></td>
<td><img src="image2.png" alt="Image" /> <strong>FBG sensors: 213kN</strong></td>
</tr>
<tr>
<td><strong>b)</strong> Habel (2008) and Dietz et al. (2008)</td>
<td>Instrumented large vertical steel anchors and piles with a FBG array of 9 sensors with 200 mm gauge length each over the 10 m fixed length and successfully performed pullout tests.</td>
</tr>
<tr>
<td></td>
<td><img src="image3.png" alt="Image" /> <strong>Kraft [kN]</strong></td>
</tr>
</tbody>
</table>


5.4.6 Conclusions

It is apparent that an increased number of strain sensing points is necessary to provide for a complete picture of shear stress distribution. The novel distributed fiber-optic strain sensors offer the possibility to increase the number of sensors without interfering with the ground anchor properties. This would allow for improved estimation of the ultimate bearing capacity and better predictions in the ground anchor applications.
6 Laboratory Pullout Tests using BEDS

The laboratory pullout tests in this Chapter were designed to achieve the following objectives: improve the understanding of progressive failure in soil-cable interaction and validate the new and improved BEDS technology.

When a cable is pulled out of sand, the soil fails at the point of contact with the cable. If the cable is sufficiently flexible, this failure does not occur simultaneously along the cable, but rather it initiates at the location of the maximum relative displacement (pullout location) and propagates along the cable length with increased load. This phenomenon has been introduced in the previous Chapter as a “progressive failure in soil-structure interaction” and the current state was shown in a literature review.

So far, it was not possible to study progressive failure properly at the laboratory scale because of the large amount of strain gages required per unit length/area of the structure, which would interfere with the mechanical properties of both the structure and the soil. Optical fiber sensors provide an alternative, but have so far been limited to point sensors (FBG) or distributed sensors with relatively large spatial resolution (BOTDA and others). The recently developed Brillouin Echo Distributed Sensor (BEDS) technology (Chapter 2) overcomes this dilemma by distributed readings at 5 cm spatial resolution. Overall, it provides an elegant solution: the cable is itself both the structure and the strain gage, allowing for at least 20 measuring points per meter, without affecting the stiffness of the cable or the soil.

For the first time it has become possible to observe the failure propagation at the lab scale with such a precision. Two laboratory testing programs have been successfully carried out at the ETH Zurich to verify applicability of BEDS for the study of progressive failure in soil-structure interaction. In this Chapter, the first ever geotechnical BEDS laboratory campaign from February 2009 is presented (Iten et al., 2009a). Details about the second laboratory campaign from April 2010 will be later published by Hauswirth (the second campaign is not a content of this Chapter). The data obtained sheds a new light on
understanding of the progressive failure, and in fact, a new model for the progressive failure is validated with the data in Chapter 8.

### 6.1 Testing Setup

#### 6.1.1 Pullout box

For the laboratory testing, a 2.0 m long, 0.1 m wide and 0.2 m deep pullout box was used (Figure 51). The concept is that a structural component embedded in soil (e.g. an anchor or a cable buried at 0.1 m depth in sand) can be pulled out of the soil filled box. The pulling force is applied by a step motor at the front of the box. This motor allows for controlled pullout by displacement, $\Delta x_F$. An independent displacement sensor monitors the displacement applied by the motor. The pullout force, $P$, is measured by a load cell. In addition, six pressure cells at the pullout box wall measure horizontal earth pressure variations during the test. Details of the pullout box sensors are shown in Table 13. The whole system is controlled by Labview software.

![Diagram of pullout box](image)

Figure 51: The pullout box.
Table 13: Overview of the pullout box sensors.

<table>
<thead>
<tr>
<th>Type</th>
<th>Displacement sensor</th>
<th>Load cell pullout force</th>
<th>Earth pressure cells</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range and accuracy</td>
<td>0 - 100 mm / 0.1%</td>
<td>0 - 2 kN / 0.1%</td>
<td>0 - 105 kPa / 0.25%</td>
</tr>
<tr>
<td>Picture</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.1.2 Tessin Sand

The filling material for the pullout box, named Tessin sand, was carefully selected to provide ideal testing conditions. The goal was to compose sand with properties similar to Ticino sand, for which extensive data is available in the literature (e.g. Baldi et al., 1989; Bellotti et al., 1996; Porcino et al., 2003).

Particle size distribution

The sand, obtained in Osogna (Ticino, Switzerland), was constituted in a predetermined particle size distribution with a maximum grain diameter of 2 mm (Figure 52). From the particle size distribution, several parameters (see Table 17) can be determined and the sand is classified in the USCS (unified soil classification system) as SP (sand poorly-graded). This classification is based on the Swiss Standard SN 670 004-2aNA (VSS, 2006) and the European Standard EN ISO 14688-2 (EN, 2004).
Figure 52: Tessin sand grain size distribution.

Table 14: Tessin sand grain size parameters (e.g. $d_{10} = 10\%$ (weight) of particles smaller than $d_{60}$).

<table>
<thead>
<tr>
<th>$d_{10}$</th>
<th>$d_{30}$</th>
<th>$d_{60}$</th>
<th>Coefficient of uniformity $C_{u/d}$</th>
<th>Coefficient of curvature $C_{c/d}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.29 mm</td>
<td>0.41 mm</td>
<td>0.66 mm</td>
<td>2.3</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Material properties

To further characterize the Tessin sand, several tests were carried out: maximum and minimum index density, triaxial compression test, Oedometer test and mineralogical composition. The obtained parameters and applied testing methods are summarized in Table 15.
Table 15: Tessin sand parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Obtained value</th>
<th>Method and references</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho_{D,\text{min}}$</td>
<td>1.383 Mg/m$^3$</td>
<td>Minimum index density represents the loosest condition of a cohesionless, free-draining soil that can be attained by a standard laboratory procedure. Standard test methods for minimum index density: ASTM D 4254 (ASTM, 2000a). A total of 6 tests (Method A) have been carried out (Hauswirth, 2008).</td>
</tr>
<tr>
<td>$\rho_{D,\text{max}}$</td>
<td>1.609 Mg/m$^3$</td>
<td>Maximum index density is determined by placing the soil in a mold, applying a surcharge to the surface and then vertically vibrating the mold. Standard test methods for maximum index density: ASTM D 4253 (ASTM, 2000b). A total of 6 tests (Method 1A) have been carried out (Hauswirth, 2008).</td>
</tr>
<tr>
<td>$\phi'_{\text{crit}}$</td>
<td>32.6° - 34.0°</td>
<td>The peak angle and the residual angle of shear resistance were obtained in triaxial tests on two sand samples; one at 25.6% relative density and the other one at 77.8%. The residual shear resistance angle, which is a material constant, was found between 32.6° and 34.0° (Hauswirth, 2008).</td>
</tr>
<tr>
<td>$\phi'_{\max}$</td>
<td>36.5° (26%), 39.1° (78%)</td>
<td></td>
</tr>
<tr>
<td>$G$</td>
<td>5 MPa (26%), 8 MPa (78%)</td>
<td>Additional parameters that were obtained from the triaxial tests above:</td>
</tr>
<tr>
<td>$K$</td>
<td>7 MPa (26%), 11MPa (78%)</td>
<td>- Shear modulus $G$</td>
</tr>
<tr>
<td>$M_E$</td>
<td>13MPa (26%), 22MPa (78%)</td>
<td>- Bulk modulus $K$ and $M_E$</td>
</tr>
<tr>
<td>$K_0$</td>
<td>0.24 (26%), 0.25 (78%)</td>
<td>- Earth pressure at rest $K_0$.</td>
</tr>
<tr>
<td>$M_E$</td>
<td>The dependence of $M_E$ on vertical pressure $\sigma_v$ was obtained by Odometer tests ASTM 2435 (ASTM, 2004) on dense and loose samples. Shown below are the results of 3 tests on dense ($I_D$ 85 to 100%) samples (Hauswirth, 2008).</td>
<td></td>
</tr>
</tbody>
</table>

![Graph](image)

Mineralogical composition | X-ray powder diffraction and Rietveld analyse provided for the mineralogical composition (in % of weight) of the Tessin sand (Plötze, 2007):
- Feldspar 42% (Plagioklas 32%, Kalifeldspar 10%)
- Quartz 40%
- Mica 15% (Muscovit 7%, Biotit 5%, Chlorit 3%)
- Carbonate 3%
Relative density

The maximum, $\rho_{D_{\text{max}}}$, and minimum, $\rho_{D_{\text{min}}}$, index density allow for the calculation of the relative density, $I_D^1$.

$$I_D = \frac{\rho_{D_{\text{max}}} (\rho_D - \rho_{D_{\text{min}}})}{\rho_{D_{\text{max}}} (\rho_{D_{\text{max}}} - \rho_{D_{\text{min}}})} \cdot 100$$  \hspace{1cm} (6.1)

Where, $\rho_D$, is the present dry density.

Compaction

As shown, the sand density influences the material properties. Thus, extensive literature research evaluated alternative preparation methods of reconstituted sand specimens for the pullout tests. The concepts of pluviation, moist tamping, undercompaction, vibration, and tamping have been considered (Ladd, 1978; Vaid & Negussey, 1988). Finally a tamping compaction procedure on dry sand was developed that allows specimen preparation in a controllable manner.

The method was to pour the sand into the box in 5 cm thick layers and compact each layer with a specific number of hammer drops. For the pouring, a specially designed hopper (Figure 53) was used. In this hopper, the sand grains flow through a slit at the bottom of the hopper into the pullout box. The slit width covers the entire width of the box and the pouring height is adjustable. The compaction was provided by a hammer (a known weight dropping a specified height to impact a wooden plate - Figure 53).

In order to assess the obtained sand density values from this procedure, tests were carried out for one to ten hammer drops. From the test results shown in Figure 54, it can be seen that at low numbers of drops (e.g. one drop), the

---

1 The literature provides for different denotations of relative index density. In the present study, the relative density is named according to European Standard EN ISO 14688-2 (EN, 2004). See also Eq. (6.1).
difference that an additional drop makes to the density is large. For higher
numbers of drops, the density changes less with additional drops. For the
tests of this study, it was chosen to compact each sand layer with five drops,
representing a compromise between a relatively large density and time of
specimen preparation.

<table>
<thead>
<tr>
<th>Weight of hammer</th>
<th>1636 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dropping height</td>
<td>300 mm</td>
</tr>
<tr>
<td>Plate dimensions</td>
<td>97x97 mm</td>
</tr>
<tr>
<td>Hopper width</td>
<td>99 mm</td>
</tr>
</tbody>
</table>

Figure 53: Details of the hopper and the hammer.

Figure 54: The impact of the hammer drops on the sand density.
6.1.3 Cable with periodic enlargements

Besides the standard cables, a cable with periodic enlargements was prepared which connects better to soil than a standard smooth surfaced cable. This allows analyzing the impact of the enlargements for further cable developments. The enlargements consist of small pellets of approximately 15 mm diameter fixed to the cable at each 0.1 m interval (Figure 55).

![The cable with periodic enlargements.](image)

Figure 55: The cable with periodic enlargements.

6.2 Soil-Embedded Cable Pullout Testing

6.2.1 Testing preparation

For the cable pullout testing, the pullout box was first filled with two 5 cm layers of compacted sand (five hammer drops each layer, about 77% relative density, $I_D$). At 0.1 m height, the cable was placed and another two sand layers covered the cable. Thus, the standard embedded depth of the cable is 0.1 m. Additional dead weight (lead) can be placed on top of the sand to simulate larger depth (e.g. 0.3 m). By weighing the sand that filled the pullout box an additional quality control measurement of the in-situ density was achieved.

The cable exiting the pullout box was then connected to the step motor and further on to the measurement unit. With the configuration as described above, pullout testing could begin.

6.2.2 Testing program

A total of ten pullout tests were carried out within the presented research. In four preparatory tests, only pullout force and displacement were measured. In
the following two tests, cable strain was obtained using BOTDA. In the four last and main tests, BEDS technology monitored continuous strain with high spatial resolution. Table 16 gives an overview of the tests performed.

Table 16: Cable pullout tests performed.

<table>
<thead>
<tr>
<th>Measurement</th>
<th>P &amp; $\Delta x_F$</th>
<th>BOTDA</th>
<th>BEDS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specifications</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 1: V3 (1 m long, 0.1 m depth, smooth)</td>
<td>Test 5: S08 (2 m long, 0.1 m depth, smooth)</td>
<td>Test 7: BSM (2 m long, 0.1 m depth, smooth)</td>
<td></td>
</tr>
<tr>
<td>Test 2: V3 (2 m long, 0.1 m depth, smooth)</td>
<td>Test 6: P07 (2 m long, 0.1 m depth, periodic enlargements)</td>
<td>Test 8: S08 (2 m long, 0.1 m depth, smooth)</td>
<td></td>
</tr>
<tr>
<td>Test 3: V3 (1 m long, 0.3 m depth, smooth)</td>
<td>Test 9: P07 (2 m long, 0.1 m depth, periodic enlargements)</td>
<td>Test 10: S08 (2 m long, 0.1 m depth, periodic enlargements)</td>
<td></td>
</tr>
<tr>
<td>Test 4: V3 (2 m long, 0.3 m depth, smooth)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.3 Soil-Embedded Cable Testing Results

6.3.1 Pullout force and displacement: Test 1 trough Test 4

The force displacement data of the four tests with the very stiff V3 cable are shown in Figure 56a. As expected, the 2 m long 0.3 m embedded cable provides the largest pullout resistance; whereas the 1 m long, 0.1 m embedded cable has the lowest pullout resistance. In Figure 56b, the average shear stress, $\tau_{avg}$, along the cable is plotted. Particularly for the cable embedded at 0.3 m depth, this value is very similar for the two tests.

6.3.2 BOTDA monitoring: Test 5 and Test 6

Test 5 and Test 6 provided the basis for the following tests with BEDS. The main intention was to verify optical signal quality at high pullout forces, thus it was not expected to achieve very accurate data. An example of the strain distribution along the P07 cable with periodic enlargements for specific load steps is shown in Figure 57. The interpretation of this data is rather difficult, as the size of the pullout box is in the same range as the spatial resolution of BOTDA.
Figure 56: (a) Force displacement for the pullout Test 1 through Test 4 and; (b) average shear stress along the cable.

Figure 57: BOTDA acquired strain distribution along a P07 cable with periodic enlargements during pullout.
6.3.3 BEDS monitoring smooth cables: Test 7 and Test 8

In Test 7 and Test 8, soil-embedded cable pullout was performed with a BSM and a S08 sensor.

The fragile BSM sensor allowed for 3 displacement steps controlled by the step motor before the fiber failed at the pulling fixation. The obtained strain data (relative to the zero measurement) is shown in Figure 58a. The fixation of the cable to the step motor is at the distance 3.40 m from the measuring unit. The better protected S08 sensor allowed for eight displacement steps up to more than 3.5% of strain (Figure 59a). At larger displacement steps, the strain profile propagates behind the fixation point (2.83 m) due to slippage of the glass fiber inside the protection.

The strain data obtained is then converted into stresses, by using the cable stiffness from the cable testing (4.5). Subsequently, a curve fit is performed to the stress data. For the BSM, a linear fit is used (Figure 58b). With the S08, two distinctive portions of the cable can be clearly identified for each load step: in the first the stress profile adheres to a linear fit, in the second – to a parabola (Figure 59b).

Figure 58: (a) Optically measured strain in the BSM for each load step $\Delta x_F$ and; (b) calculated stress for each load step $\Delta x_F$ and fitted curve.
PART II

Figure 59: (a) Optically measured strain in the S08 for each load step $\Delta x_F$ and; (b) calculated stress for each load step $\Delta x_F$ and fitted curve.

Validation of the optically measured strain data against independently measured strain is presented in Figure 60. First, numerical integration of the entire strain in the cable leads to the displacement at the fixation point, $\Delta x_F$, and this can be compared with the value applied by the step motor. Second, by using the optically measured maximum fiber strain at each load step, cable force at the fixation, $P$, can be calculated and compared to the applied load measured by the load cell.

For the BSM, the optically measured displacement is equal to the reference displacement only at 1 mm. Above 1 mm displacement, the cable slides through the far end of the box (confirmed by Figure 58) and thus, the integration of the strain measurement does not provide for an accurate value. For the correct displacement, the cable displacement at 5.4 m would have to be taken into consideration as well. The comparison of the force, $P$, provides values which are within the measurement accuracy of the load cell.

For both S08 values ($P$, $\Delta x_F$) the agreement between the optically and conventionally measured values is very good.
Figure 60: Applied vs. optically measured displacement and applied vs. optically measured pullout force for the (a) BSM and; (b) S08.

6.3.4 BEDS monitoring cables with periodic enlargements: Test 9 and Test 10

The distributed strain data shown in Figure 61 was obtained using a P07 and a S08 with periodic enlargements. The P07 was strained up to 1.2% and the S08 up to 3.5%. At these relatively high strains, slippage of the fiber inside the protection occurs around both the fixation point and the enlargements. Thus, the interpretation of the strain data becomes difficult to make. One phenomenon that can be easily recognized is the shorter anchoring length associated with the cables having periodic enlargements. E.g. at a strain of
2.5%, the smooth S08 transfers the force to the soil over roughly 1 m. Whereas the S08 with periodic enlargements requires only a half a meter to transfer the entire pullout force into the soil. The development of cables with periodic enlargements, although of smaller size, is still ongoing. Further testing on these cables can be found in Hauswirth (2011).

![Figure 61](image_url)

**Figure 61:** Optically measured strain along the cables with periodic enlargements for selected load steps: (a) P07 and; (b) S08.

### 6.4 Discussion and Conclusions on Laboratory Pullout Tests

For the successful study of progressive failure at a laboratory scale, several requirements need to be fulfilled: reproducible testing conditions, reliable external sensors to measure global parameters, and highly spatially resolved internal sensors that do not affect the structure’s properties.

In the present study, favorable testing conditions were obtained by the custom designed pullout box with well-concerted accuracy of the necessary sensors and the accompanying studies of the Tessin sand. The BEDS technology allowed using the cable as a combined internal sensor and structure.

The quality of the obtained data could be verified by independently acquired load and displacement data. Unfortunately, some testing data cannot be further interpreted due to slippage of the fiber inside the protection (mostly with the P07 cable). Nevertheless, sufficient data (e.g. Figure 62) was obtained to visualize the propagation of the progressive failure: the portion
where the interface slip has already taken place in Figure 62b is represented by the constant shear stress, and, as expected, its length grows with the applied displacement. What is less obvious is: (a) why this propagation slows when the displacement and pullout force increases (instead of accelerating and eventually leading to catastrophic failure), and (b) why the friction in this failure zone grows with the applied displacement and pullout force (instead of decreasing to the residual strength).

Answering these two questions would provide a better understanding of progressive failure. The literature review did not provide sufficient evidence to support this finding. The fact that the well documented laboratory testing of this Chapter clearly shows such an increase (Figure 62b) calls for a conceptual model, which can account for and predict this effect. Such a model is developed in Chapter 8.

In addition, this is the first ever geotechnical laboratory campaign using BEDS. The successful monitoring of strain distribution with high spatial resolution opens new possibilities for laboratory testing and field applications. Once this technology becomes commercially available, there are many field applications were this technology could be successfully applied to provide users with an unprecedented amount of high quality data.
Figure 62: Data obtained from the S08 sand-embedded pullout: (a) axial stress distribution and fitted stress distribution; (b) Load transfer (shear stress distribution).
7 Field Ground Anchor Monitoring using BOTDA

In this Chapter, a novel monitoring ground anchor using embedded optical fibers for continuous strain assessment along an anchor tendon is proposed. This allows for improved understanding of progressive failure in soil-structure interaction, which is one of the main objectives of this study. In addition, this allowed studying the applicability of the fiber-optic sensors in the harsh construction environment. Results from laboratory and field testing of such anchor tendons are presented in Iten & Puzrin, 2010.

For real scale ground anchor monitoring, three parameters are of interest:

1) Load at anchor head \((P)\);
2) Displacement at anchor head \((\Delta x_F)\);
3) Load distribution over the fixed length (allows for back-calculation of shear stress in fixed length).

Anchor load can be easily assessed by a load cell or, if no load cell is available, by applying a lift-off load. Anchor head displacement can be measured during pullout testing by micrometer or similar displacement sensor as well as geodetically during its whole lifetime. For the assessment of these two parameters, reliable standard methods are available and stipulated in the codes (e.g. SIA 267, 2003).

The determination of the load distribution along a ground anchor is less common. Nevertheless, it is essential for understanding the anchor’s load bearing behavior, as the performance of the anchor is limited by the efficiency of load transfer from the anchor tendon to the soil via the grout. Thus, significant interest has been put into measuring strain at distinctive points along anchor tendons by various types of sensors, such as conventional strain gauges and, more recently, FBG’s (see also 5.4). Other approaches are based on elongation measurements (using extensometers) in a very limited number of tendon sections. This can be seen as long-gauge strain sensors. Such monitoring anchor tendons, which offer strain readings in up to four sections, are commercially available and are regularly used in construction projects (e.g. Solexperts, 2011).
To begin, different techniques of optical fiber integration into tendons of 1.2 m to 3.3 m length have been carried out. The advantages and challenges of several sensor cables and sensor integration methods applied in this project are discussed. Then, the tendons where strained, stepwise in a tensile testing apparatus while optical strain measurements were taken at each displacement step. The optical strain was compared with independently acquired strain data. The evaluation of the laboratory testing led to the design and development of an 8 m long monitoring ground anchor for field application.

In August 2009, this anchor has been integrated into a wall supporting an excavation pit and subsequently, anchor pullout testing was performed. The anchor was loaded incrementally up to 470 kN, approaching the internal ultimate limit strength of the steel tendon. Optical strain measurements were successfully taken at each load step and the data could be used for the calculation of the applied pullout load in the free length and the anchor head displacement. The optically measured load closely matches the true applied load. This verified the quality of the obtained data, and therefore, further data interpretation steps (simplified analytical modeling) have been pursued (Chapter 8).

From a scientific point of view the motivation for this Chapter is similar to the one of the previous Chapter with a focus on the assessment of shear stress distribution along an embedded body in soil. Merely, the bodies, as well as the involved stresses, are of much larger scale. In addition, the motivation is stimulated by the practical application, as conventional monitoring ground anchors allow for measurements, albeit in a very limited amount of anchor tendon sections. However, for the understanding of the bearing behavior of this geotechnical structure, it is essential to assess the stress distribution along the entire tendon. Therefore, with a reliable new monitoring ground anchor offering distributed strain measurements, a significant contribution to practice can be made.
7.1 Development of Fiber-Optic Monitoring Anchor

As shown, strain distribution along the ground anchor tendon has been assessed so far by point sensors or long gauge strain sensors. However, the elastic-plastic mode of deformation (i.e. absence of localized fracture) of the steel anchor tendon positions this structural part as an ideal candidate for overall bonding a sensor to the tendon and thus, continuously monitoring strain. In materials where expected cracks could destroy the sensor cable at low strains, such an overall bonding would not be an appropriate solution.

7.1.1 Tendon

In this study, a hollow steel tendon with a threaded outer surface is used as shown in Figure 63a. The tendon’s outer diameter is 35 mm and the inner diameter is 15 mm. The ultimate tensile load and ultimate limit strength of the tendon are known (Belloli, 2004). From three uniaxial strain tests performed in 2008, yield strength, proportionality limit stress, and stiffness of the tendon (Figure 63b) have also been determined (see Table 17 below). These parameters are in agreement with those of comparable products (e.g. Stahlton, 1998).

Table 17: Parameters of the steel anchor tendon.

<table>
<thead>
<tr>
<th>$A_s$</th>
<th>$R_t$</th>
<th>$f_u$</th>
<th>$f_y$</th>
<th>$f_p$</th>
<th>$E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel cross section</td>
<td>Ultimate tensile load</td>
<td>Ultimate tensile strength</td>
<td>Yield strength (0.2% offset)</td>
<td>Proportionality limit strength</td>
<td>Stiffness (elastic modulus)</td>
</tr>
<tr>
<td>785 mm$^2$</td>
<td>533 kN</td>
<td>679 MPa</td>
<td>584 MPa</td>
<td>370 MPa</td>
<td>207 GPa</td>
</tr>
</tbody>
</table>

The tendon’s stress-strain relationship in the strain range of interest for optical measurements (up to 1.5%) can be approximated by the following formulas (found from the three uniaxial tension tests performed):

Elastic behavior: For stress up to the proportionality limit strength ($f_p = 370$ MPa; $\varepsilon_x \leq 1.787 \mu\varepsilon$):
\[ \sigma_x = \frac{207}{[GPa]} \cdot \varepsilon_x \quad (7.1) \]

**Plastic behavior:** For stresses above the proportionality limit strength \((f_p = 370 \text{ MPa}; \varepsilon_x > 1'787 \mu \varepsilon)\) and below 1.5% strain (651 MPa):

\[ \sigma_x = 604.9 \cdot e^{\varepsilon - 4.935} - 745.2 \cdot e^{-\varepsilon - 630.9} \quad (7.2) \]

---

Figure 63: (a) Anchor tendon and; (b) tensile testing of the tendon.

### 7.1.2 Fiber sensor integration

In a first step, different techniques for optical fiber integration into tendons were tested. Special focus was on the best possible protection of the sensor, as the anticipated application environment is very rough. Three methods (Figure 64) proved applicable for the laboratory tests: integration in an external longitudinal trench, internal integration, and helix integration, respectively. For the field testing, only two methods were pursued. Table 18 gives an overview of the evaluated methods.
7.2 Laboratory Strain Testing

7.2.1 Testing Program

In order to evaluate the performance of the different sensor integration methods, a laboratory uniaxial strain testing program was designed. The testing program consisted of two phases. In Phase 1, short tendons 1.3 m long \( (L_T) \) were strained over 1 m length \( (L_e) \). Based on the evaluation of Phase 1, an extensive experimental program has been developed for selected sensor integration methods on long tendons \( (L_T = 3.3 \text{ m}) \) in Phase 2. Table 19 shows the type and amount of strain sensors tested during the testing program and the maximum strain applied on the tendons.
Table 18: Evaluated methods for sensor integration.

<table>
<thead>
<tr>
<th>Method</th>
<th>Trench</th>
<th>Internal</th>
<th>Helix</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Description</strong></td>
<td>The sensor is aligned in a trench (1 mm wide, 2 mm deep) that is cut along the entire length of the tendon. [Figure 64a]</td>
<td>The sensor is placed inside the hollow tendon, which is then filled with a low viscosity injection resin. [Figure 64b]</td>
<td>The sensor is curled around the outside of the tendon following the thread geometry (about 8 coils per 100 mm). [Figure 64c]</td>
</tr>
<tr>
<td><strong>Fixation</strong></td>
<td>Sensor is continuously fixed with a two component epoxy inside the trench.</td>
<td>Sensor is continuously fixed by the low viscosity injection resin on epoxy base (alternatively, the tendon is filled with a quartz sand epoxy mixture).</td>
<td>Sensor is continuously fixed with a two component epoxy inside a trench cut into the thread spline (alternatively, cable is attached to the surface of the thread groove).</td>
</tr>
<tr>
<td><strong>Possible FOS</strong></td>
<td>BSM, TSM, P07, M07</td>
<td>BSM, TSM, P07, M07, S08, and all newer cable versions</td>
<td>BSM, TSM, P07, M07</td>
</tr>
<tr>
<td><strong>Advantages</strong></td>
<td>• Sensor is well protected. • Sensor is directly fixed to tendon.</td>
<td>• Sensor is well protected. • Steel cross section is not affected. • Simple preparation and integration.</td>
<td>• Sensor is well protected. • Steel cross section is not affected. • Compression measurements possible.</td>
</tr>
<tr>
<td><strong>Challenges</strong></td>
<td>• Steel cross section is reduced. • Preparation of trench is laborious. • Pretension of sensor.</td>
<td>• Sensor is not directly fixed to tendon. • Pretension of sensor. • Expensive resin.</td>
<td>• Preparation of trench and integration is very laborious. • Scaling of measured strain (lateral extension, which might not be homogenous, plays a role).</td>
</tr>
</tbody>
</table>
Table 19: Type and amount of strain sensors tested and maximum strain applied.

<table>
<thead>
<tr>
<th>Phase 1</th>
<th>Trench</th>
<th>Internal</th>
<th>Helix</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tendon 1</strong></td>
<td>1 TSM – surface mounted (total sensor length ≈ 8m)</td>
<td>1 BSM – trench (total sensor length ≈ 8m)</td>
<td>1'800 to + 6'000 µs</td>
</tr>
<tr>
<td>(LT = 1.3 m, Lε = 1 m)</td>
<td>1 BSM – trench (total sensor length ≈ 8m)</td>
<td>+ 2'500 µs</td>
<td></td>
</tr>
<tr>
<td><strong>Tendon 2</strong></td>
<td>1 M07 – trench (total sensor length ≈ 8m)</td>
<td>4 BSM aligned (total sensor length ≈ 5m)</td>
<td>+ 1’500 µs</td>
</tr>
<tr>
<td>(LT = 1.3 m, Lε = 1 m)</td>
<td>4 BSM aligned (total sensor length ≈ 5m)</td>
<td>+ 1’500 µs</td>
<td></td>
</tr>
<tr>
<td><strong>Tendon 3</strong></td>
<td>1 TSM / 1 P07 / 1 M07 (each ≈ 3 m long)</td>
<td>2 P07 (each ≈ 3 m long)</td>
<td>+ 15’000 µs</td>
</tr>
<tr>
<td>(LT = 3.3 m, Lε = 3 m)</td>
<td>2 P07 (each ≈ 3 m long)</td>
<td>+ 15’000 µs</td>
<td></td>
</tr>
</tbody>
</table>

**7.2.2 Testing Setup**

To test the sensor equipped tendons, hydraulic uniaxial tension machines were chosen. The short tendons were tested on a machine capable of delivering up to 480 kN axial force (Schenk 480 – Figure 65). The long tendons were tested on a massive machine allowing for 1600 kN pull force (Schenk 1600). In these machines, the tendon is inserted into the test frame and gripped at both ends. Uniaxial force (extension and compression) or displacement can then be applied. During loading, it is possible to monitor
both the load (with an accuracy of 0.02% of the total applied load) and the axial displacement. In addition, strain on the surface of the tendon can be measured by an extensometer. The steel tendons were strained in a displacement controlled manner. At each displacement step, optical measurements were taken. As an example, Figure 66 shows a typical straining of the tendon vs. the test time.

Figure 65: Hydraulic uniaxial tension machine (Schenk 480) used for the 1.3 m long tendon.

Figure 66: Typical straining of the steel tendon in the time domain.
7.2.3 Phase 1: testing on short tendons, results and interpretation

The first phase of the testing program was designed with the intention to obtain information about the serviceability of the proposed sensor integration methods. As the strained section \( L_\varepsilon \) was only 1 m long, and thus, on the order of magnitude as the spatial resolution of the DITEST, the testing in Phase 1 provides for a qualitative assessment only and no quantitatively accurate results were expected.

**Helix integration**

The helix integration method was tested on two tendons (Tendon 1 and Tendon 2), where the sensor was coiled 76 times around each tendon, resulting in a strain sensor length of about 8.5 m. Several sensor types were integrated (in total three pre-strained sensors, BSM, TSM, M07) and compression and elongation strain tests were performed between -1'800 µε and +6'000 µε.

An accurate interpretation of applied strain in terms of optically measured strain required the effect of lateral extension be accounted for. This is because elongation of the tendon causes compression of the coiled sensor. This effect can be described by the following, simple model:

The initial fiber length along one coil \( c_0 \) of the helix depends on initial coil height \( h_0 \) and initial tendon radius \( r_0 \):

\[
c_0 = \sqrt{(2 \cdot \pi \cdot r_0)^2 + h_0^2}
\]  
(7.3)

Length change of the fiber coil \( \Delta c \) due to change in longitudinal strain in the tendon \( \varepsilon_x \), can then be expressed as:

\[
\Delta c = c - c_0 = \sqrt{(2 \cdot \pi \cdot r_0)^2 + h^2} - c_0 = \sqrt{(2 \cdot \pi \cdot r_0 \cdot (1 - \nu \cdot \varepsilon_x))^2 + [h_0(1 + \varepsilon_x)]^2} - c_0
\]  
(7.4)

For the elastic strain, \( \nu \) is the Poisson’s ratio of steel and for plastic strain \( \nu \) is 0.5. The strain in the coil \( \varepsilon_f \) can thus be expressed as a function of the longitudinal strain in the tendon \( \varepsilon_x \):
\[ \varepsilon_f = \frac{\Delta c}{c_0} = \frac{\sqrt{a \cdot \varepsilon^2 + b \cdot \varepsilon + c^2}}{c_0} - 1 \]  

(7.5)

where

\[ a = h_0^2 + 4 \cdot \pi^2 \cdot \nu^2 \]  

(7.6)

\[ b = 2 \cdot h_0^2 - 8 \cdot \pi^2 \cdot \nu^2 \]  

(7.7)

From an analytical model in the elastic range using Hooke's Law and from elasto-plastic FE calculations (Abaqus), the parameters \( a \) and \( b \) have been derived and are plotted in Figure 67.

![Figure 67: Parameters a and b for the helix model.](image)

Both tendons were strained to different levels while the applied strain is compared with optically measured strain as in Figure 68a. For elongation (which means compressing the sensor), it can be seen that with increasing tendon strain, the optically acquired tendon strain does not adequately agree with the applied strain. In addition, if a loading-unloading cycle is driven (as shown here with the BSM), the matching is even worse. The worst results are obtained with the M07 sensor. For compression (which means elongation of the sensor), the optically acquired tendon strain follows a practically straight line, but does not agree with the values of the applied strain. This fact does not disqualify these results, rather it suggests a reconsideration of the boundary conditions used in the helix model (Eq. (7.3) to Eq. (7.7)) and as a consequence, a modified model to improve the correlation. Additionally, a scaling factor may also prove helpful at this point.
Nevertheless, further interpretation problems are encountered with the helix integration method, particularly when looking at the measured strain along the sensor (Figure 68b). Clearly, the optically measured strain distribution over the 8.5 m sensor length is unrealistic. These results, in conjunction with the difficult and time consuming sensor integration, led to the decision at the end of Phase 1 to discard the helix integration for the monitoring ground anchor.

a) Applied vs. optically acquired strain

b) Strain along the sensor optically measured for selected strain steps

Figure 68: Helix integration testing in Phase 1.
**Trench and internal integration**

In Tendon 3 (trench) and Tendon 4 (internal), the sensor (BSM) was routed four times along the entire tendon length (Figure 69), resulting in a (continuous) strain sensor length of about 5 m. Both tendons were strained stepwise to 1'500 µε and the applied strain is compared with optically measured strain in Figure 70a. It can be seen that the two curves agree very well with the 45° line. A closer look at the strain along the sensor for selected strain steps (Figure 70b) also shows a reasonable distribution, especially considering the short sensor sections used. In light of these promising results, it was decided to carry the trench and internal integration methods along to Phase 2. Difficulties encountered with the handling of the BSM in the internal integration configuration led to the decision not to consider the BSM for the internal integration method.

![Figure 69: Sensor alignment for the BSM in Tendon 3 and Tendon 4.](image)

**7.2.4 Phase 2: testing on long tendons**

For the second phase of the testing program, two 3.3 m long tendons (Tendon 5 and Tendon 6) were sensor-equipped by applying the two selected methods (trench and internal). As sensors, the BSM, TSM, and M07 from Phase 1 were chosen, as well as the newly available P07.

In Tendon 5, a TSM, M07, and P07 sensor were integrated internally. The tendon was then strained to 4'000 µε (Test 1). 2 months later, the same tendon was used again and strained to 9'000 µε (Test 2). No negative effects on the bond between the injection grout and the fiber-optic sensor could be detected as a result of this elapsed time.
In Tendon 6, a BSM and a TSM sensor were trench integrated and, additionally, two P07 sensors (Sensor 1 and Sensor 2) were internally integrated to allow for further testing of this sensor as a consequence of the good results with Tendon 5. The tendon was then strained up to almost 15’000 µε.

a) Applied vs. optically measured strain

![Graph showing applied vs. optically measured strain for BSM and 45° in Trench and Internal.]

b) Strain along the sensor optically measured for selected strain steps

![Graph showing strain along the sensor optically measured for selected strain steps in Trench and Internal.]

Figure 70: Trench and internal integration testing in Phase 1.
7.2.5 Phase 2: internal integration testing results and interpretation

**TSM and M07**

The correlation between the applied strain and the optically measured strain for the two internally integrated sensors is shown in Figure 71. It can be seen that strain is closely monitored optically by the TSM. But, as recognized for Test 1, slippage of the fiber inside the protection may occur, resulting in a strain drop in the fiber and a redistribution of strain along the sensor beyond the tendon boundary. This redistribution of strain may even result in a completely inhomogeneous distribution of strain along the sensor (Figure 72 - Test 1). During Test 2, the sensor section length over which the maximum strain is detected decreased with increasing load. Generally, this is a clear sign indicating slippage of the fiber inside the protection. In agreement with preceding strain sensor testing results, the TSM sensor can only be applied for accurate strain sensing in a tendon, with expected axial strains not exceeding 0.4% during its lifetime.

For the M07, the correlation between the applied and the measured strain is poor. In addition, repeatability between Test 1 and Test 2 is not possible. Once again, Figure 73 points out slippage as the likely problem. In addition, inhomogeneous interlocking of some sensor sections may have occurred during Test 1. Overall, the M07 sensor serviceability is deemed insufficient.

**P07**

The correlation between the applied strain and the optically measured strain for the internally integrated P07 sensors is shown in Figure 74. It can be seen that for three tests including three independent P07 sensors, applied strain is closely monitored optically up to 15'000 µε. Slippage of the fiber inside the protection may occur, as in Figure 75 (Tendon 5, Test 1), but to a smaller extent than with the other sensors described above. And, under certain circumstances (that can be due to inhomogeneous protection fiber adhesion), slippage does not occur at all (Figure 75 – Tendon 6, Sensor 2). Nevertheless, for short strained sensor sections or for strain points close to a strain step, this has to be taken into account.
Figure 71: Comparison between optically measured and applied strain for the internal integration method.

Figure 72: Strain along the TSM internally integrated for selected strain steps.

Figure 73: Strain along the M07 internally integrated for selected strain steps.
Figure 74: Comparison between optically measured and applied strain for the internal integration method using P07 sensors.

Figure 75: Strain along the P07 internally integrated for selected strain steps.

### 7.2.6 Phase 2: trench integration testing results and interpretation

**BSM**

The excellent correlation between the applied strain to the optically measured strain for the BSM trench integrated for strain levels up to 1.5% is shown in Figure 76. In addition, Figure 77a confirms the quality of the sensor data, as optically measured strain is shown to be evenly distributed over the tendon length.
TSM

For the TSM, slippage of the fiber inside the protection is already evident below 0.5% applied strain, resulting in a strain drop in the fiber. This effect can also be clearly seen in Figure 77b and is in agreement with the internal integration testing results of this sensor in Phase 2.

Figure 76: Comparison between optically measured and applied strain for the trench integration method.

Figure 77: Strain along the BSM and TSM trench integrated sensors for selected strain steps.
7.2.7 Comparison and conclusions for laboratory testing

Comparison

In addition to the discussion of the obtained results in the preceding sections, a comparison of the Phase 2 results was also performed. For this comparison, two values were calculated:

Individual measurement error ($\Delta \epsilon_{\text{opt}}$) at each load step ($k$):

$$\Delta \epsilon_{\text{opt}}(k) = \left| \epsilon_{\text{opt}}(k) - \epsilon_{\text{ref}}(k) \right| \quad (7.8)$$

where, $\epsilon_{\text{opt}}$ is the optically measured strain and $\epsilon_{\text{ref}}$ is the applied strain. This value is an indication of the sensors accuracy.

Standard deviation ($s$) along the sensor in percent of the applied strain ($s_{p}$) for each load step ($k$):

$$s_{p}(k) = \frac{s(k)}{\epsilon_{\text{ref}}(k)} \cdot 100 = \frac{1}{\epsilon_{\text{ref}}(k)} \cdot 100 \cdot \sqrt{\frac{1}{N-1} \sum_{i=1}^{N} (x_i - \bar{x})^2} \quad (7.9)$$

This value is an indication of the homogeneity of the strain distribution along the sensor.

In Table 20, the measurement errors for the sensors integrated into Tendon 5 and Tendon 6 are summarized. It can be seen, that with the TSM and M07 sensors, large measurement errors were obtained, independent of the chosen integration method (for TSM). The measurement error of the P07 lies within acceptable limits (also for strains above 1%). The measurement error of the BSM sensor is very low (less than 0.004%) for strains up to 0.5% and still stays within a small range for higher strains.

In Table 21 the standard deviation along the sensor in percent of the applied strain is shown. This indication of the homogeneity of the strain distribution along the sensor indicates either localized slippage between the fiber and the protection or insufficient sensor fixation. It can be seen, that the P07 sensor produces reasonably accurate results up to high strains. The values from the BSM sensor are lowest, and therefore, most accurate.
Table 20: Summary of measurement error ($\Delta \varepsilon_{\text{opt}}$) for all tested sensors in Phase 2. The values of the error are expressed in $\mu\varepsilon$ for each range of applied strain (in %).

<table>
<thead>
<tr>
<th>Method</th>
<th>Strain range</th>
<th>0 to 0.2%</th>
<th>0.2 to 0.5%</th>
<th>0.5 to 1%</th>
<th>1 to 1.5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>P07</td>
<td>150</td>
<td>180</td>
<td>280</td>
<td>370</td>
<td></td>
</tr>
<tr>
<td>M07</td>
<td>1000</td>
<td>3000</td>
<td>7000</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>TSM</td>
<td>-</td>
<td>1700</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>BSM</td>
<td>40</td>
<td>40</td>
<td>150</td>
<td>210</td>
<td></td>
</tr>
<tr>
<td>TSM</td>
<td>120</td>
<td>3000</td>
<td>7000</td>
<td>10’000</td>
<td></td>
</tr>
</tbody>
</table>

Table 21: Summary of standard deviation in percent of the applied strain ($s_p$) for all tested sensors in Phase 2 for each range of applied strain.

<table>
<thead>
<tr>
<th>Method</th>
<th>Strain range</th>
<th>0 to 0.2%</th>
<th>0.2 to 0.5%</th>
<th>0.5 to 1%</th>
<th>1 to 1.5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>P07</td>
<td>34%</td>
<td>6%</td>
<td>10%</td>
<td>10%</td>
<td></td>
</tr>
<tr>
<td>M07</td>
<td>60%</td>
<td>22%</td>
<td>27%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>TSM</td>
<td>-</td>
<td>30%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>BSM</td>
<td>6.5%</td>
<td>2.5%</td>
<td>1.3%</td>
<td>0.8%</td>
<td></td>
</tr>
<tr>
<td>TSM</td>
<td>5%</td>
<td>25%</td>
<td>28%</td>
<td>28%</td>
<td></td>
</tr>
</tbody>
</table>

Conclusions

From the laboratory testing, it can be concluded that obtaining good quality strain data in an optical monitoring anchor strongly depends on the fiber-optic sensor selection and the sensor integration method. In the extensive
laboratory strain testing program, several combinations of sensors and integration methods were tested. The main problems encountered were:

1) Insufficient protection of the fiber;
2) Inhomogeneous optically measured strain distribution along the sensor;
3) Slippage of the fiber inside the protection layer(s).

Due to these problems, the following sensors and integration methods were excluded for the field testing:

1) BSM for internal integration method;
2) Helix integration method;
3) TSM and M07 sensors.

For the field monitoring anchor, the trench and internal integration method with BSM and P07 cable can be considered an optimal choice. The monitoring results of strains between 0.5% and 1.5% with the P07 sensor can be expected to fall within 20% of the applied strain with a 95% confidence interval and for the BSM trench integration within about 2.5% of the applied strain. For lower strains between 0 and 0.5%, the sensors attached to the tendon react less accurately. However, with controlled pre-straining of the sensor during integration, an improvement can be expected, as the sensor reacts immediately, once straining starts.

### 7.3 Field Ground Anchor Monitoring

With the encouraging results and experience from the laboratory testing, a full size 8 m long monitoring ground anchor for field pullout testing was designed, installed into a wall supporting an excavation pit, and a successful distributed anchor load monitoring of a pull test was performed.

### 7.3.1 Design of monitoring anchor

Commercially available, hollow steel tendons that are currently used by monitoring companies for conventional monitoring anchors come in 4 m long
Field Ground Anchor Monitoring using BOTDA

pieces. If longer anchor lengths are desired, coupling or welding is necessary. In the current project, an 8 m long tendon was required. Due to the extreme heat developed during welding, only coupling of two 4 m tendons came into consideration. For the coupling, a standard ferrule was modified so that no differential, circular displacements induced by torsion between the two tendons were possible. Such torsion would immediately break the fibers leading from one tendon to the other. The ferrule was, for this reason, equipped with locking bolts. ²

In addition to the sensors selected through the laboratory testing process, the newly available, better protected, and less slippage prone S08 was also used in the field anchor. It was endeavored to try as many sensor and integration combinations as possible while ensuring, that at least some of the sensors would produce suitable results. Therefore, one BSM sensor and the S08 cable were run only through the bottom part, so that high plastic strains in the anchor head vicinity did not affect the serviceability of these sensors. Internal fixation was also carried out only in the bottom part. Figure 78 shows the sensor naming scheme and setup along the anchor and in Figure 79, the details of the monitoring anchor tendon are pictured.

7.3.2 Installation

In August 2009, the 8 m ground anchor was installed at the PLATFORM construction site in Zürich Hardbrücke (part of the Prime Tower construction, currently Zürich’s highest building).

Placement

The test anchor could be integrated in a row of strand tendon anchors supporting a sheet piling wall (Figure 80). The tendon leads from the anchor head, which is at 2.8 m depth with a 22° inclination with respect to the bottom

² The cutting of the trenches required a workshop with extra space, which was found at SOLEXPERTS AG (Switzerland) and kindly carried out by them.
of the anchor, which is embedded about 5.5 m deep in the soil. The fixed anchor length is 5.75 m and the grout body diameter, $d_g$, is estimated to be between 0.13 m and 0.19 m (the borehole diameter is 0.13 m; the injected grout volume leads to a cylinder diameter of about 0.19 m). A detailed sketch can be found in Figure 81.

Figure 78: Field monitoring anchor tendon. Schematic sketch of the placement of the different sensors used.

Figure 79: Field monitoring anchor tendon; pictures of selected details.
Geotechnical profile

The soil in the vicinity of the anchor has been identified by the responsible geotechnical engineer as a “silty gravel” GM (VSS, 2006), lying above the groundwater level. The corresponding parameters from the geotechnical profile for the soil are listed in Table 22.

![Geotechnical profile](image)

**Figure 80:** Test anchor integrated at the construction site besides a row of strand tendon anchors.

![Sketch of the installed test anchor](image)

**Figure 81:** Sketch of the installed test anchor.
Table 22: Geotechnical parameters for the silty gravel at PLATFORM.

<table>
<thead>
<tr>
<th>$\phi_k'$</th>
<th>$c_k'$</th>
<th>$\gamma_e$</th>
<th>$M_E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic value of effective angle of internal friction</td>
<td>Characteristic value of effective cohesion</td>
<td>Unit weight</td>
<td>Bulk modulus</td>
</tr>
<tr>
<td>40 [°]</td>
<td>0-5 [kPa]</td>
<td>23 [kN/m$^3$]</td>
<td>40-60 [MPa]</td>
</tr>
</tbody>
</table>

**Rough conventional estimate of ultimate bearing capacity**

A first estimate of the anchors capacity can be done using classical soil mechanics.

Average vertical stress on the grout body:

$$\sigma_{v_{avg}} = \gamma_e \cdot \frac{(3.3m + 5.45m)}{2} = 101kPa \quad (7.10)$$

Average shear stress along the grout body:

$$\tau_{avg} = \sigma_{v_{avg}} \cdot \frac{(1 + K)}{2} \cdot \tan(\delta) \quad (7.11)$$

$K$ is the earth pressure coefficient, ranging somewhere between active, at rest, and passive modes. $\delta$ is the soil-grout friction angle and can reach at its maximum the effective angle of internal friction of the soil. Cohesion is neglected.

The ultimate bearing capacity, $R_u$, (Eq. 5.5) can then be calculated by estimating $K$, $d_g$, and $\delta$ (Table 23). For an expert, the estimated capacity seems very low and it must be assumed that even the highest value (290 kN) is still below the anchors capacity in reality. Restrained dilatancy has not been considered. Pullout testing in the next section will show if classical soil mechanics can provide a reasonable solution for the anchor.
Table 23: A parameter study for the ultimate bearing capacity of the anchor.

<table>
<thead>
<tr>
<th>$K$</th>
<th>$\overline{\delta}$</th>
<th>$d_g$</th>
<th>$R_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_a = \tan^2\left(45^\circ - \frac{\phi_k'}{2}\right)$</td>
<td>$\overline{\delta} = 2 \cdot \frac{\phi_k'}{3}$</td>
<td>0.13 m</td>
<td>72 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.19 m</td>
<td>106 kN</td>
</tr>
<tr>
<td>$K = 1$</td>
<td>$\overline{\delta} = \phi_k'$</td>
<td>0.13 m</td>
<td>121 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.19 m</td>
<td>176 kN</td>
</tr>
</tbody>
</table>

7.3.3 Pullout testing

Testing setup

For pullout testing, a hydraulic jack which allowed for loading the anchor at specified load steps was used (Figure 82). In addition to the jack load monitoring, displacement of the anchor tendon head and displacement of the bearing plate was recorded. The optical cables were run through the entire length of the anchor (including the hydraulic jack section) and were led to the DITEST in the vicinity of the test site.

Testing procedure

Pullout testing was performed 12 days after the anchor installation, which allowed sufficient time for grout hardening. Eighteen load steps between 0 kN and 500 kN were carried out and, at each load step, optical data was acquired. The time each test load was held before continuing to the next load step was set to 15 min, so that there was enough time for optical strain monitoring while still allowing the maximum number of load steps desired. Figure 83 shows the anchor head displacement for each load step. Between 400 kN and 440 kN, a problem with the testing equipment was encountered, as can be seen on the load-displacement graph. The problem could be solved and further testing was possible.
7.3.4 Monitoring results

*Strain distribution raw data*

To begin, the raw data of the frequency change along the sensors was transferred into strain by the frequency coefficients for each of the sensors (Table 9). The strain data was also filtered in half of the spatial resolution vicinity of the tendon joint \((x = 4 \, \text{m})\) and at the bottom of the tendon \((x = 0 \, \text{m})\). As an example, Figure 84 shows the distributed strain data for the BSM and S08 sensors for all load steps.
Load distribution

Next, the load in the tendon was calculated from the optically measured strain using elasto-plastic deformation modulus of steel (Eq. 7.1 and Eq. 7.2). A first assessment of the serviceability of the different sensors and methods can be performed by comparing the optically measured load distribution in each individual sensor at a specific load step. In Figure 85, this is shown for a selected set of load steps from 0 kN up to 400 kN. It is readily visible that all sensors produced very similar data in the Bottom Tendon even at higher loads. In the Head Tendon, the picture is different, as the P07 sensor shows an irregular distribution. This irregularity occurs most pronounced in the vicinity of the tendon joint.

Fitted load distribution

For further interpretation, the tendon length is divided into five sections and a curve is fitted to the load data in each section. The curve fit's properties are guided by a general understanding of the interface properties in the corresponding sections. For example, in Section 1, a homogeneous load distribution is expected, as this section is not embedded in soil. On the other hand, in Section 4, practically linear load dissipation is predicted, as only

Figure 84: Raw data of strain distribution along the tendon for the (a) BSM and; (b) S08 sensors for all load steps.
residual shear stress is acting along the soil-grout interface in this section. In addition, the fit has to assure that shear stress is continuous along the grout. The boundaries between the sections are not fixed in place, but shift further into the soil (away from the anchor head) with increasing pullout loads (except for Section 1 and Section 2 boundaries). For better understanding, the sections and their corresponding boundaries are sketched in Figure 86 and details are explained in Table 24.

For each individual sensor, such a load distribution fitting was carried out. Figure 87 (BSM), Figure 88 (P07) and Figure 89 (S08) show the curve fitting for selected load steps.

**Pullout load and anchor displacement**

To verify the overall quality of the optical strain data, a comparison with the independently measured load and displacement data can be performed.

In the case of load, the optically measured tendon force at the anchor head \( P_{hopt} \) at \( x = 0.5 \text{ m} \) is compared with the applied pullout load \( P_{href} \) (Figure 90a). In addition, the measurement error \( \Delta P_{hopt} \) at each load step is plotted in Figure 90b.

In order to obtain the optically measured displacement of the anchor head \( (\delta_{hopt}) \) the integral of the strain \( (\varepsilon_{opt}) \) from the anchor bottom to the anchor head has to be taken:

\[
\delta_{hopt}(k) = \int_{x=0}^{x=8m} \varepsilon_{opt}(k) \cdot dx
\]  

(7.12)

The optically measured head displacement is then compared with an independently acquired head displacement for each sensor (BSM Head, BSM All, and P07) in Figure 91a and the measurement error \( \Delta \delta_{hopt} \) at each load step is plotted in Figure 91b. Additionally, the optically measured tendon joint displacement was calculated and is plotted against the applied load in Figure 92.
Figure 85: Optically measured load distribution in each individual sensor at specific load steps.
Figure 86: Separation of the tendon length into sections of similar soil-grout interface properties (as explained in Table 24 below).

Table 24: Separation of the soil-embedded tendon into 5 sections.

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Fit</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Outside free length</td>
<td>$P_{opt_fit} = \sum_{i=1}^{n} P_{opt}(x_i) / n$</td>
<td>$P$ is equal to the applied pullout force.</td>
</tr>
<tr>
<td>2</td>
<td>Embedded free length</td>
<td>$P_{opt_fit} = a \cdot x_i + b$</td>
<td>Some load is dissipated along the free length due to undesired friction.</td>
</tr>
<tr>
<td>3</td>
<td>Growing free length</td>
<td>Spline interpolation</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Constant shear stress</td>
<td>$P_{opt_fit} = a \cdot x_i + b$</td>
<td>$a = \tau_{res}$</td>
</tr>
<tr>
<td>5</td>
<td>Fully bonded</td>
<td>$P_{opt_fit} = a - b \cdot (e^{-a}x_i - 1)$</td>
<td></td>
</tr>
</tbody>
</table>
Figure 87: BSM sensor measured and fitted load distribution for selected load steps.

Figure 88: P07 sensor measured and fitted load distribution for selected load steps.
Figure 89: S08 sensor measured and fitted load distribution for selected load steps.

Figure 90: (a) Optically measured load vs. independently measured load and; (b) measurement error.
7.3.5 Comparison and conclusions for the field testing

The quality of the obtained field data could be verified by independently acquired load and displacement data. The agreement of the data is good and the prediction from the laboratory testing, which states that accuracy of the optically measured force lies within 20% of the actual value, was confirmed.

In addition, when comparing the data from the different sensors with each other (e.g. Figure 85), it becomes evident that all sensors provided reasonable
results. Nevertheless, the P07 showed some irregularities in the Head Tendon at higher strains. It is therefore suggested, that this cable is not further used for anchor load monitoring. Yet, even with this sensor, satisfying results for limited geotechnical engineering applications could be obtained.

In general, the challenges and problems pointed out in the laboratory testing could be overcome by a careful sensor selection, pre-straining of the sensor, and cautious handling of the cables. This is also represented by the fact that all the sensors integrated into the anchor functioned properly up to high loads. No failures due to transport or anchor integration were registered.

For future sensor integrations into anchor tendons, either the trench integration method with a BSM sensor or the inside integration method with the S08 sensor is suggested. The decision on which method is appropriate will have to be made taking into consideration the dimensions of the tendon(s), the cost of trench cut and resin, and the cost of the sensor itself.

The initial calculated anchor capacity (7.3.2) differs significantly from the maximum pull force observed, which reached 500 kN. In agreement with the literature (5.4), it can be stated that applying classical soil mechanics does not deliver reliable results in ground anchor projects. Thus, anchor tests and monitoring are absolutely necessary for ground anchor load bearing capacity and safety determinations.

### 7.4 Discussion and Conclusions on the Monitoring Anchor

Successful shear stress distribution monitoring along the ground anchor (Figure 93) provides for an improved understanding of progressive failure in soil-structure interaction. As with the soil-embedded cable pullout (Chapter 6), the phenomenon of increasing residual shear stress with increasing pullout load (Figure 93b) was observed. The fact that the ground anchor pullout was well documented and the quality of the data could be verified independently strongly underlines this finding and calls for further study of failure propagation in soil-structure interaction. Thus, a conceptual model, which adequately incorporates this effect, is developed in the next Chapter.
Moreover, it was shown that embedding optical fibers into a ground anchor tendon for continuous strain assessment is a promising application in geotechnical engineering. The process of developing such a monitoring anchor consisted of the laboratory testing of several sensor integration methods and of a full scale field test. To a geotechnical engineer, this proposed monitoring anchor provides a powerful tool for the acquisition of the pullout load, anchor head displacement, and load distribution in the anchor.
tendon. Other authors have acquired load distribution by means of point sensors or long gauge strain sensors, but there have not been any publications addressing distributed strain assessment along such anchors. Even though the elastic-plastic steel tendon mode of deformation makes the anchor ideal for continuous fixing of a sensor to the tendon and, thus, continuous strain monitoring. The proposed field monitoring anchor offers the possibility of monitoring such structures in a comprehensive way and can provide for continuous estimation of the residual factor of safety. This saves money for the constructor, while enhancing structural safety and maximizing the life of the structure.
8 Modeling Shear Stress Distribution

This Chapter presents a conceptual model capable of explaining the phenomenon of increasing residual shear stress, $\tau_{\text{res}}$, with increasing pullout load. This phenomenon, to the knowledge of the authors, has not been clearly identified and explained in the literature. However, it has been documented during the pullout tests of the cable and the ground anchor in the preceding Chapters. The distributed sensing technology provided for a large amount of data, which was independently verified with conventional load and displacement sensors. The fact that, in both testing conditions (cable and ground anchor pullout), the phenomenon of residual shear stress increase was well documented, leads to the conclusion that this is a real phenomenon.

Yet, this is in disagreement with the progressive failure concept introduced in 5.1, 5.4.4, and other analytical models (8.2), where residual shear stress is usually assumed to be constant. In order to understand the mechanism behind this increase and to provide conceptual support for this phenomenon, a simplified analytical model for the shear stress distribution along the soil-embedded cable pullout is developed.

The model is based on the hypothesis that additional compression in the sand around the cable, due to the cable pullout, leads to increased residual shear stress. Applying the model to the sand-embedded cable pullout tests provided for a good prediction and convincing data consistency. With the ground anchor pullout, an improved analytical model, or numerical modeling, is necessary.

The model developed in this Chapter is content of a publication recently submitted by Puzrin et al. (2011).
8.1 Experimental Shear Stress Distribution

8.1.1 Curve fitting

The pullout testing of the sand-embedded cable resulted in an axial stress distribution data as presented in Figure 59b. For the analysis, this data has been processed so that the tip of the pullout box (where the pulling is applied) is at $x = 0$ (Figure 62bis). In addition, a curve, $\sigma_{\text{fit}}(x)$, was fitted to the data, taking into consideration, that two distinctive portions of the cable could be clearly recognized. Thus, the fit is a piece-wise function consisting of a linear function in the first part and an exponential function (as for the elastic interface sections described under 8.2) in the second part.

$$\sigma_{\text{fit}}(x) = \sigma_{\text{fit}1} - b \cdot x \quad \text{for} \quad 0 \leq x \leq L_{\text{sl} \_\text{fit}}$$  \hspace{1cm} (8.1)

$$\sigma_{\text{fit}}(x) = \left(\sigma_{\text{fit}1} - b \cdot L_{\text{sl} \_\text{fit}}\right) \cdot e^{-c(x-L_{\text{sl} \_\text{fit}})} \quad \text{for} \quad L_{\text{sl} \_\text{fit}} \leq x \leq \infty$$  \hspace{1cm} (8.2)

All parameters $\sigma_{\text{fit}1}, b, c, L_{\text{sl} \_\text{fit}}$ were arbitrarily chosen to provide the best fit. Figure 62abis shows the curve fit axial cable stress distribution. Moreover, the differentiation of the stress distribution provides us with the shear stress distribution along the cable (Figure 62bbis).

8.1.2 Evidence of the progressive failure

The portion where the interface slip has occurred in Figure 62bbis is represented by the constant shear stress, and, as expected, its length grows with the applied displacement. The friction in this failure zone also grows with the applied displacement and pullout force.

8.2 Analytical Models for Similar Problems in the Literature

First, a short review of similar problems described in the literature is provided.
8.2.1 Elastic rod pullout of stiff medium

From structural analysis, models for the pullout of an elastic rod out of a stiff medium are available (Figure 94a). The normal force change, \( dN \), in the rod (of radius \( a \)) depends on the shear stress \( \tau \):

\[
dN = \tau \cdot a \cdot \pi \cdot dx
\]  

(8.3)

Figure 62bis: Data obtained from the S08 sand-embedded pullout (6.3.3): (a) axial stress distribution and fitted stress distribution; (b) Load transfer (shear stress distribution).
The derivative of the rod displacement, \( \delta \), is

\[
d\delta = \frac{N}{E \cdot a^2 \cdot \pi}
\]  

(8.4)

where \( E \) is the elastic modulus of rod. And thus,

\[
\frac{d^2 \delta}{dx^2} \frac{\tau \cdot 2}{E \cdot a} = 0
\]  

(8.5)

If the shear stress is known (depending on the rod displacement \( \delta \)), a solution for Eq. (8.5) can be found. For example, Marti (2006) derives, for different relationships of \( \tau \) to \( \delta \), an analytical solution. One possible relationship is plotted in Figure 94b.

![Figure 94: (a) Elastic rod pullout out of a stiff medium; (b) Linear relationship between interface displacement and shear stress and; (c) tri-linear bond slip model.](image)

8.2.2 Analytical models for rock bolts

For rock bolts, several analytical studies have been conducted. In these studies, the medium where the elastic bolt is embedded is also deformable. As soon as interface slippage between the bolt and the rock appears, the shear stress drops. To model this, a large amount of the studies assume a so-
called tri-linear bond slip model (Figure 94c) describing the relationship of $\tau$ to $\delta$ for three different interface sections (elastic, softening, and debonding). The earlier works (Farmer, 1975) for this model focused on the elastic part, Li & Stillborg (1999), as well as Ren et al. (2010), widened the study to the softening and debonding part. In all these studies, the axial rod stress over the elastic section decays exponentially.

### 8.2.3 Shear-lag approximations

The load-transfer behavior for pullout tests with extensible, planar reinforcements may also be described using shear-lag approximations (commonly used in the mechanics of composites as shear-lag analysis - Cox, 1952; Abramento & Whittle, 1995a). In shear-lag approximations, the development and distribution of tensile stresses in the reinforcement (and thus the shear stress) is based on the constituent material properties and the test geometry. The shear-lag modeled axial strain has been evaluated in comparison with pullout tests performed on instrumented inclusions (Abramento & Whittle, 1995b).

### 8.2.4 Restrained dilatancy

Some models are also based on restrained dilatancy, which results in an increase in normal stresses, or mobilization of dilatant stresses at the soil-reinforcement interface, during pullout. E.g. Liang & Feng (2002) or Alfaro & Pathak (2005).

### 8.3 Formulation of the Problem and Assumptions

Both the problem and the analytical model are formulated for the specific cases of the sand-embedded S08 pullout test (6.3.3). In 8.6, the model is also applied to the ground anchor pullout.
8.3.1 Formulation

In Figure 95 a 2 m long cable is sand-embedded in the pullout box. Subsequently, pullout force, $P$, is applied at the cable fixation, generating a cable tip displacement, $\Delta x_F$. The force ($P$) is transferred along the cable-sand interface into the sand. The cable is separated into two sections (similar to Abramento & Whittle, 1995a):

1. **No interface slippage**: The interface shear stress between sand and cable is below or equal to the peak value and thus, the displacement of the cable, $\delta_c$, is equal to the displacement of the sand, $\delta_s$.

2. **Interface slippage**: The interface shear stress between sand and cable is past peak value and thus, the displacement of the cable, $\delta_c$, and the sand, $\delta_s$, are not equal.

The interface slippage length, $L_{sl}$, increases with increasing tip displacement (and increasing pullout load). The differential displacement between cable, $\delta_c$, and the sand, $\delta_s$, is, $\delta_d$.

![Figure 95: Analytical model of the sand-embedded cable pullout test.]

8.3.2 Assumptions

For simplification, several assumptions have been made:
• The friction angle, $\varphi$, on the cable-sand interface depends on the differential displacement, $\delta_d$. The assumed relationship is shown in Figure 96a. Apart from the small zone, $\omega$ (the process zone), where softening occurs, the friction angle over the disconnected length, $L_{sl}$, is equal to the residual value, $\varphi_r$.

• The process zone, $\omega << L_{sl}$, is neglected in the calculations. This effectively means that for $\delta_d > 0$, residual friction is applicable;

• The cable of diameter $a$, is embedded in a sand cylinder of diameter $R$ (Figure 96b). The sand outside $R$ is assumed to be rigid and $R$ must be smaller than the pullout box cross section;

• The cable stress-strain behavior is elastic according to Hooke’s Law $\sigma_c = \varepsilon_c \cdot E_c$ ($E_c$ is the elastic modulus of the cable);

• The sand is linear elastic with constrained modulus, $M_s$, and shear modulus, $G_s$;

• The radial stress from the sand on the cable, $\sigma_{rs}$, is taken as an averaged value over the horizontal and vertical components.

Figure 96: (a) Relationship between the friction angle and the relative displacement along the cable-sand interface; (b) The cable embedded in the sand cylinder.

8.4 The Model

The model is developed for the two sections individually: the no interface slippage length and the interface slippage length.
8.4.1 No interface slippage

Stresses

For the analysis of the stresses where no interface slippage occurs, a slice of the cable-soil composite, with incremental length $dx$, is considered. This slice is restrained at its external boundary, $R$, and constant stress $\sigma_s$ is applied to the soil. The cable in the center is subjected to force $P_c$, which causes a displacement, $\delta$ (Figure 97a). The slice deforms as sketched in Figure 97b.

The applied cable force, $P_c$, is transferred to the soil as shear stress, $\tau$:

$$P_c = \tau \cdot 2 \cdot \pi \cdot a \cdot dx$$  \hspace{1cm} (8.6)

The equilibrium of forces acting in x-direction on the soil element can be written as

$$\tau \cdot 2 \cdot \pi \cdot r \cdot dx = \left( \tau + d\tau \right) \cdot 2 \cdot \pi \cdot (r + dr) \cdot dx$$  \hspace{1cm} (8.7)

which after omitting the term $d\tau \cdot dr$, can be reduced to

$$\frac{\tau}{r} + \frac{d\tau}{dr} = 0$$  \hspace{1cm} (8.8)

The solution of this first-order linear ordinary differential equation has been found to be of the form:

$$\tau(r) = \frac{c}{r}$$  \hspace{1cm} (8.9)

Using the boundary condition, $\tau(a) = \tau_c$, the shear stress is

$$\tau(r) = \tau_c \cdot \frac{a}{r}$$  \hspace{1cm} (8.10)
Strains and displacements

The shear strain, $\gamma$, (Figure 97c) is expressed as

$$\gamma = \frac{d\delta}{dr}$$

(8.11)

and is linked to the shear stress through the shear modulus of the sand, $G_s$:

$$\gamma(r) = \frac{\tau}{G_s} = \frac{\tau_c \cdot a}{G_s \cdot r}$$

(8.12)

By integrating $\gamma$ over the whole slice, from $a$ to $R$, the displacement, $\delta$ becomes

$$\delta = \int_a^R \gamma \cdot dr = \frac{\tau_c \cdot a}{G_s} \int_a^R \frac{dr}{r} = \frac{\tau_c \cdot a}{G_s} \ln \left( \frac{R}{a} \right)$$

(8.13)

Displacement along the cable

The axial and shear stresses acting on a cable element are displayed in Figure 98 in a local coordinate system starting with $x = 0$ at the boundary.
between interface slippage and no interface slippage. The equilibrium of the forces in this element can be expressed as

\[ \sigma_c \cdot \pi \cdot a^2 + \tau_c \cdot 2\pi \cdot a \cdot dx = \left( \sigma_c + \frac{d\sigma_c}{dx} \right) \cdot \pi \cdot a^2 \]  

(8.14)

which leads to

\[ \frac{d\sigma_c}{dx} = \frac{2 \cdot \tau_c}{a} \]  

(8.15)

It is expected that cable strain does not exceed the elastic limit and thus,

\[ \frac{d\sigma_c}{dx} = E_c \cdot \frac{d\varepsilon}{dx} = E_c \cdot \frac{d^2\delta}{dx^2} \]  

(8.16)

And, by Eq. (8.13) resolved for \( \tau_c \) and substituted into Eq. (8.15), the following differential equation is found:

\[ \frac{\partial \sigma_c}{\partial x} = 2 \cdot \frac{\tau_c}{a} = 2 \cdot G_s \cdot \frac{\delta}{a^2 \cdot \ln \left( \frac{R}{a} \right)} = E_c \cdot \frac{d^2\delta}{dx^2} \]  

(8.17)

The model parameters are lumped into the parameter, \( \alpha^2 \), and the differential equation is written in the form:

\[ \frac{d^2\delta}{dx^2} = \alpha^2 \cdot \delta \]  

(8.18)

\[ \alpha^2 = \frac{G_s}{E_c} \cdot \frac{2}{a^2 \cdot \ln \left( \frac{R}{a} \right)} \]  

(8.19)

The general solution of this second-order linear ordinary differential equation is in the form of:

\[ \delta = A \cdot e^{-\alpha x} + B \cdot e^{\alpha x} \]  

(8.20)

Using the boundary condition \( \delta(\infty) = 0 \), the second part of the equation is cancelled, leaving

\[ \delta = A \cdot e^{-\alpha x} \]  

(8.21)
The other boundary condition is; \( \delta_B = \delta(0) = A \). According to Eq. (8.13)

\[
\delta_B = \delta(0) = \frac{\tau_B \cdot a}{G_s} \ln \left( \frac{R}{a} \right) = A
\] (8.22)

where \( \tau_B \) is the shear stress at the boundary \( x = 0 \), in the local coordinate system. The cable and soil displacement and strain at the cable surface \( (r = a) \) are then

\[
\delta(x) = \frac{\tau_B \cdot a}{G_s} \ln \left( \frac{R}{a} \right) \cdot e^{-\alpha x}
\] (8.23)

\[
\frac{d\delta}{dx} = \varepsilon(x) = -\alpha \cdot \frac{\tau_B \cdot a}{G_s} \ln \left( \frac{R}{a} \right) \cdot e^{-\alpha x}
\] (8.24)

and \( \varepsilon_B \) is

\[
\varepsilon_B = -\alpha \cdot \delta_B
\] (8.25)

By squaring this equation and using Eq. (8.19) and Eq. (8.22), \( \delta_B \) can also be expressed as

\[
\delta_B = \varepsilon_B^2 \cdot \frac{E}{2} \cdot \frac{1}{\tau_B}
\] (8.26)

Figure 98: (a) Stresses acting on a cable element along the no interface slippage section and; (b) shear stress distribution along the cable.
8.4.2 Interface slippage

Shear stresses

Now, the shear stresses between sand and cable when interface slippage occurs are considered. The residual shear stress, $\tau_{res}$ (acting over the interface slippage length, $L_{sl}$), depends on the radial stress on the cable, $\sigma_{rc}$, and the friction angle, $\varphi'$. (Figure 96a). Starting from the initial stress conditions within the sand, $\sigma_{rc0}$, the shear stress may increase with additional radial stress, $\Delta \sigma_{rc}$:

$$\tau_{res} = (\sigma_{rc0} + \Delta \sigma_{rc}) \cdot \tan(\varphi') = \tau_{res0} + \Delta \sigma_{rc} \cdot \tan(\varphi')$$  \hspace{1cm} (8.27)

The only source of additional stress in the soil comes from the compression term, $\Delta \sigma_{xs}$, due to the displacement, $\delta_B$. This displacement compresses the soil over $L_{sl}$, as the boundary at the cable tip side of the pullout box remains rigid. Additional stress in the radial direction, $\Delta \sigma_{rc}$, is obtained using constrained modulus, $M_s$, and earth pressure at rest, $K_0$:

$$\Delta \sigma_{rc} = K_0 \cdot \Delta \sigma_{xs} = K_0 \cdot M_s \cdot \Delta \varepsilon_{xs} = K_0 \cdot M_s \cdot \frac{\delta_B}{L_{sl}}$$  \hspace{1cm} (8.28)

Which leads to the following equation:

$$(\tau_{res} - \tau_{res0}) = K_0 \cdot M_s \cdot \frac{\delta_B}{L_{sl}} \cdot \tan(\varphi')$$  \hspace{1cm} (8.29)

Introducing the constant, $\beta$, applying the assumption $\tau_{res} = \tau_B$, neglecting the influence of $\varphi'_p$ (this means also the influence of the small triangle at $\tau_B$ in Figure 98b), and using Eq. (8.26), this can be expressed as:

$$\frac{(\tau_{res} - \tau_{res0}) \cdot \tau_{res}}{L_{sl}^2} \cdot \beta = \frac{E_s}{2} \cdot \frac{a}{L_{sl}} \cdot K_0 \cdot M_s \cdot \tan(\varphi')$$  \hspace{1cm} (8.30)

$$\beta = \frac{E_s \cdot a}{2} \cdot K_0 \cdot M_s \cdot \tan(\varphi')$$  \hspace{1cm} (8.31)
Preliminary model validation against experimental data

For the testing data shown in 8.1 (sand-embedded cable pullout test), a preliminary validation of this model can be carried out (Figure 99). The chosen model parameters are shown in Table 25 and an approximate value of \( \beta \) is also calculated from the test parameters. The convincing data consistency (Figure 99) and the agreement of the estimated and fitted \( \beta \) further justifies the development of the model.

Table 25: Parameters for Eq. (8.30) from the testing data.

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>Data Fit ( \tau_{v0} ) [kPa]</th>
<th>Data Fit ( \beta ) [kPa^2 m]</th>
<th>Estimated ( \beta ) (Eq. 8.31)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_c \approx 470 ) MPa ; ( M_s \approx 2.5 ) MPa ; ( a \approx 0.0014 ) m ; ( K \approx 1 ) ; ( \tan(\phi_v) \approx \tan(34^\circ) )</td>
<td>4.2</td>
<td>1.5\times10^6</td>
<td>1.2\times10^6</td>
</tr>
</tbody>
</table>

Figure 99: A preliminary validation of the pullout testing results by Eq. (8.30) for the laboratory pullout test (Chapter 6).
8.4.3 The entire (combined) cable length

Axial stress distribution along the cable

Summarizing the above sections, the axial stress in the cable in a coordinate system with \( x = 0 \) at the pulling location (cable exit of pullout box at step motor) is expressed as:

\[
\sigma_c(x) = \sigma_p - \frac{2 \cdot \tau_{res} \cdot x}{a} \quad \text{for} \quad 0 \leq x \leq L_{sl}
\]

\[
\sigma_c(x) = \sigma_B \cdot e^{-a(x-L_{sl})} \quad \text{for} \quad L_{sl} \leq x \leq \infty
\]

Cable stress at boundary and boundary location

With the condition that the derivative \( \left( \frac{d \sigma_c}{dx} \right) \) of the two equations above must be the same at \( L_{sl} \):

\[
\sigma_B = \frac{2 \cdot \tau_{res}}{a}
\]

The above equation can as well be included to express the boundary location \( L_{sl} \):

\[
L_{sl} = \frac{\sigma_p - \sigma_B}{2 \cdot \tau_{res} \cdot a} = \frac{\sigma_p}{2 \cdot \tau_{res} \cdot a} - \frac{1}{a}
\]

Model parameter \( \alpha \) as a function of pullout stress, residual shear stress and tip displacement

The displacement over the interface slippage length is

\[
\delta_{L_{sl}} = \int_{0}^{L_{sl}} \left( \frac{\sigma_c}{E_c} \right) dx = \frac{1}{E_c} \cdot \int_{0}^{L_{sl}} \left( \sigma_p - \frac{2 \cdot \tau_{res} \cdot x}{a} \right) dx
\]

And by using Eq. (8.35), we obtain
The displacement over the no slippage section is from Eq. (8.33) and Eq. (8.34):

\[ \delta_B = \int_{L_{oi}}^{L_{oi}+\alpha L_{oi}} \frac{\sigma_c}{E_c} dx = \frac{1}{E_c} \int_{L_{oi}}^{L_{oi}+\alpha (L_{oi} - L_{oi})} \frac{\sigma_B}{E_c} e^{-\alpha (x-L_{oi})} dx = \frac{2 \tau_{res}}{E_c \alpha^2 a} \]  

(8.38)

The tip displacement, \( \Delta x_F \), is the sum of the displacements in the two sections:

\[ \Delta x_F = \delta_{L_{oi}} + \delta_B = \frac{\sigma_p}{4 \cdot E_c} \frac{a}{\tau_{res}} + \frac{\tau_{res}}{E_c a^2} \]  

(8.39)

This can then be resolved for \( \alpha^2 \) as:

\[ \alpha^2 = \frac{\tau_{res}}{E_c a \left( \frac{\Delta x_F}{\tau_{res}} \right)} \]  

(8.40)

**Boundary location as a function of pullout stress, residual shear stress and tip displacement**

Substituting Eq. (8.40) into Eq. (8.35), the interface slippage length can be expressed as a function of the relatively easily measurable values of pullout stress, residual shear stress, and tip displacement:

\[ L_{sl} = \frac{\sigma_p a}{2 \tau_{res}} \sqrt{\frac{E_c a \Delta x_F}{\tau_{res}} \frac{\sigma_p^2 a^2}{4 \tau_{res}}} \]  

(8.41)
8.5 Validation of the Model

8.5.1 Shear stress and boundary location

In Figure 62b, the shear stress distribution given from the fit of the experimental data is shown. The relationship between interface slippage length, \( L_{sl} \), and residual shear stress, \( \tau_{res} \), can be read out and then plotted in Figure 100. Using Eq. (8.41), the model can also predict this relationship from measured pullout stress, residual shear stress, and tip displacement data (Table 26). As it can be seen in Figure 100, the two values are in good agreement with each other and thus, validate this conceptual model.

Table 26: Estimation of \( \alpha^2 \) from the test parameters.

<table>
<thead>
<tr>
<th>( \Delta x_F ) [m]</th>
<th>4.8( \times )10(^{-4} )</th>
<th>9.3( \times )10(^{-4} )</th>
<th>1.3( \times )10(^{-3} )</th>
<th>2.2( \times )10(^{-3} )</th>
<th>6.7( \times )10(^{-3} )</th>
<th>1.1( \times )10(^{-2} )</th>
<th>1.5( \times )10(^{-2} )</th>
<th>1.8( \times )10(^{-2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_p ) [kPa]</td>
<td>1.6( \times )10(^3 )</td>
<td>2.3( \times )10(^3 )</td>
<td>3.0( \times )10(^3 )</td>
<td>4.1( \times )10(^3 )</td>
<td>8.2( \times )10(^3 )</td>
<td>1.1( \times )10(^4 )</td>
<td>1.4( \times )10(^4 )</td>
<td>1.7( \times )10(^4 )</td>
</tr>
<tr>
<td>( \tau_{res} ) [kPa]</td>
<td>4.5</td>
<td>4.7</td>
<td>5.4</td>
<td>5.9</td>
<td>8.7</td>
<td>9.6</td>
<td>11.3</td>
<td>12.4</td>
</tr>
</tbody>
</table>

Figure 100: Shear stress and boundary location from the testing data and obtained by the model.
8.5.2 Progression of interface slippage length

In Figure 101, the growth of the interface slippage length, $L_{sl}$, with increasing applied cable stress, $\sigma_p$, is plotted from the fit on the measured stress distribution (Figure 62) and the model prediction using measured pullout stress, residual shear stress, and tip displacement data (Table 26) inserted into Eq. (8.41). The retarding of the length propagation with increasing stress can be clearly observed. The two values are in good agreement with each other, further validating this conceptual model.

![Figure 101: Growth of the interface slippage length as a function of the applied stress for the cable pullout testing.](image)

8.5.3 Lateral stresses

Finally, the model can also be validated by its ability to simulate the evolution of the earth pressures acting on the pullout box wall. The stress change (radially from the cable axis) was measured at the pullout box side wall (Figure 51) by pressure cells ($\Delta\sigma_{cell}$). In 8.4.2 it was explained that the soil displacement, $\delta_s$, (Figure 102a, taken from Figure 95) causes a strain change, $\Delta\varepsilon_{ss}$, in the soil (Figure 102b). This strain change is directly related to
stress change in the sand body. The stress change is deduced below for the two sections individually.

\[ \delta_B - \delta_s = \Delta \varepsilon_x \]

\[ \delta_s \]

\[ \delta \]

\[ \Delta \varepsilon \]

\[ \Delta \varepsilon_{ss} \]

\[ \Delta \varepsilon_{xs} \]

\[ \Delta \varepsilon_x \]

Figure 102: (a) Soil displacement along the cable and; (b) strain change in the soil due to cable pullout.

**No interface slippage**

When the interface slippage length, \( L_{sl} \), propagates towards the pressure cell located in a no interface slippage length at distance \( x \) from the tip of the cable, radial stress change, \( \Delta \sigma_{wall} \) (the wall pressure), is proportional to (Eq. 8.23):

\[ \Delta \sigma_{wall} \sim \sigma_B \cdot e^{-\alpha(x-L_x)} \]  \hspace{1cm} (8.42)

where

\[ \sigma_B = \frac{2 \cdot \tau_{res}}{\alpha \cdot a} \]  \hspace{1cm} (8.34bis)

\[ \alpha^2 = \frac{\tau_{res}}{E_c \cdot a \cdot \left( \Delta x_p - \frac{\sigma_p^2 \cdot a}{4 \cdot E_c \cdot \tau_{res}} \right)} \]  \hspace{1cm} (8.40bis)
Interface slippage

The stress change in radial direction on the cable ($\Delta \sigma_{rc}$) over the interface slippage length was calculated in Eq. (8.28). The stress change on the pullout box side wall ($\Delta \sigma_{wall}$) is then proportional to $\Delta \sigma_{rc}$:

\[
\Delta \sigma_{wall} \sim \Delta \sigma_{rc}
\]  
\[ (8.43) \]

Using Eq. (8.26), this can be written as:

\[
\Delta \sigma_{wall} \sim \frac{E_b^2 \cdot E_c \cdot \alpha}{\tau_{res} \cdot L_{sl}} \cdot \frac{1}{L_{sl}} = \frac{\sigma_b^2 \cdot \alpha}{2 \cdot E_c \cdot \tau_{res} \cdot L_{sl}}
\]
\[ (8.44) \]

Stress change calculation

The pressure cells considered for the model validation are the ones at 0.4 m distance from the pullout box front wall (Figure 51). Thus, $x$ is equal to 0.4 m in Eq. (8.44). For these pressure cells, model predictions of different lengths $L_{sl}$ are fitted to the measured pressures (Figure 103) to define one proportionality coefficient for each of the equations (Eq. 8.42 and Eq. 8.44). The remaining parameters are taken from the measurements (Table 26).

As it can be seen, the model is capable of capturing both decrease of the wall pressure when the cell is located in the no interface slippage length, and subsequent increase, when the cell finds itself in the interface slippage length.

![Figure 103: Lateral stress increments at the pullout box wall measured and modelled.](image)
8.5.4 Limitations of the model

There are two assumptions, which are not satisfied for larger pullout stresses:

- The diameter, \( R \), of the sand cylinder is smaller than the pullout box cross section;

- The cable stress-strain behavior is linear elastic.

**Sand cylinder diameter**

The sand cylinder diameter, \( R \), can be calculated by solving Eq. (8.19) for \( R \), calculating \( \alpha^2 \) from the testing data (Table 26) using Eq. (8.40), and inserting the testing parameters from Table 27:

\[
R = a \cdot e^{E_c \cdot a^2 / a}
\]

(8.45)

According to this calculation, the sand cylinder \( R \) reaches the location of the pullout box wall at pullout stress \( \sigma_p \) above about 5000 kPa. Thereafter, the assumption \( R < 0.05 \) m (pullout box wall) is not satisfied anymore.

Table 27: Test parameters for the sand and the cable.

<table>
<thead>
<tr>
<th>Test</th>
<th>( G_s ) [MPa]</th>
<th>( E_c ) [MPa]</th>
<th>( a ) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEDS Test 8</td>
<td>~ 0.15</td>
<td>~ 470</td>
<td>~ 0.0014</td>
</tr>
<tr>
<td>(S08 cable)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Elastic cable range**

From Table 8, the elastic range of the S08 cable is limited to 0.9% strain (4.2 MPa). Above, part of the cable is subjected to plastic strains and the assumption of Hooke’s law is not valid anymore.

**Effect of the limitations on the model**

Curiously, in spite of the fact that the two assumptions mentioned above were partially violated, the conceptual model still produced a good fit to the
experimental data (Figure 100 and Figure 101). This implies that the model is not very sensitive to these simplifications and is capable of accurately predicting the experimental results. Thus, for the present study, no further model modification was undertaken.

8.6 Application of the Model to the Ground Anchor

The ground anchor is of much larger scale and the boundary conditions in the field are never as well defined as in the pullout box. Nevertheless, the same tendency of shear stress increase was evident from the experimental data and therefore, the conceptual validation of the model for the ground anchor pullout is anticipated in this section.

8.6.1 Differences to the soil-embedded cable pullout

The full scale ground anchor has been employed in a real construction project under field conditions. Therefore, several differences to the precise laboratory conditions are recognized:

- Grout (unknown and inconstant diameter, inconstant $E$-modulus of grout and steel cross sections);
- Tendons (jointed together with possible stress concentration at this location);
- Soil (not compacted, not homogeneous properties);
- Depth (varying embedded depth);
- Front boundary (front wall is not a rigid boundary).

These differences influence the quality of the monitoring results and the deviation of the input parameters and geometry for the model.
8.6.2 Experimental shear stress distribution

In the modeling, the optically measured force along the steel tendon is first converted into stress in an idealized cylinder (which consists of the grout and the tendon, as the ground anchor fails at the grout-soil interface). The radius of the cylinder is assumed to be 10 cm (approximately the injected grout volume). The 0-point of the x-axis is chosen to coincidence with the tip of the grouted length (start of section 4 in Table 24).

The stress in the cylinder, obtained from curve fitting optically measured strain data (using the procedure described in 8.1.1), are shown in Figure 93abis. Please note that the fit is not as good as with the cable, due to the differences described above. The differentiation of the stress distribution provides the shear stress distribution along the anchor (Figure 93bbis). As with the cable pullout, a clear tendency of growing residual shear stress is observed.

8.6.3 Validation of the model for the ground anchor

Preliminary model validation against experimental data

A preliminary validation is performed by inserting the testing data above into Eq. (8.30). The chosen model parameters are shown in Table 28 and an approximate value of $\beta$ is also calculated from the test parameters. Figure 104 shows a satisfying data consistency. However, the estimated value of $\beta$ is about one order of magnitude different than obtained from the fitting. This may be an indication, that the model in the way it was developed for the cable is not applicable for the anchor.

Table 28: Parameters for Eq. (8.30) from the testing data.

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>Data Fit $\tau_{\text{res}}$ [kPa]</th>
<th>Data Fit $\beta$ [kPa²*m]</th>
<th>Estimated $\beta$ (Eq. 8.31)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0 \approx 6$ GPa; $M_s \approx 60$ MPa; $a \approx 0.1$ m; $K \approx 1$; $\tan(\phi') \approx \tan(40^\circ)$</td>
<td>19</td>
<td>$3.4 \times 10^{11}$</td>
<td>$2 \times 10^{10}$</td>
</tr>
</tbody>
</table>
Figure 93bis: Data obtained from the ground anchor pullout: (a) axial stress distribution and fitted stress distribution; (b) load transfer (shear stress distribution).
PART II

Figure 104: A preliminary validation of the pullout testing results by Eq. (8.30) for the ground anchor pullout (Chapter 7).

Further validation

From the shear stress distribution (curve fit on the measured stresses) in Figure 93bis, the relationship between interface slippage length, $L_{sl}$, and residual shear stress, $\tau_{res}$, can be extracted. These two values are plotted in Figure 105. Using Eq. (8.41), the model can also predict this relationship from measured pullout stress, residual shear stress, and tip displacement data (Table 29).

The plot of the experimental data is, in shape comparable with the soil-embedded cable pullout (Figure 100). The modeled data can qualitatively show the same trend, but does not fit quantitatively. The same is true for the growth of the interface slippage length, $L_{sl}$, with increasing applied cable stress, $\sigma_p$, plotted in Figure 106.

Unfortunately, further model validation with the data from the field ground anchor pullout test was not possible. A first problem indication was already
obtained above with $\beta$ (calculated and modelled) differing by an order of magnitude. Additionally, the uncertainties listed under 8.6.1 do influence the model predictions strongly. The effects of the missing rigid front boundary and the inconstant grout diameter are believed to play a role. Thus, the limit of the model applicability was reached, as it was not possible to obtain reasonable agreement with the ground anchor pullout testing data.

Table 29: Estimation of $\alpha^2$ from the ground anchor pullout test parameters.

<table>
<thead>
<tr>
<th>$\Delta x_F$ [m]</th>
<th>3.6 $\times 10^{-4}$</th>
<th>5.7 $\times 10^{-4}$</th>
<th>2.0 $\times 10^{-3}$</th>
<th>3.6 $\times 10^{-3}$</th>
<th>4.7 $\times 10^{-3}$</th>
<th>5.7 $\times 10^{-3}$</th>
<th>6.7 $\times 10^{-3}$</th>
<th>7.7 $\times 10^{-3}$</th>
<th>9.4 $\times 10^{-3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_p$ [kPa]</td>
<td>1.5 $\times 10^{3}$</td>
<td>2.0 $\times 10^{3}$</td>
<td>4.6 $\times 10^{3}$</td>
<td>6.7 $\times 10^{3}$</td>
<td>8.7 $\times 10^{3}$</td>
<td>1.0 $\times 10^{4}$</td>
<td>1.1 $\times 10^{4}$</td>
<td>1.2 $\times 10^{4}$</td>
<td>1.3 $\times 10^{4}$</td>
</tr>
<tr>
<td>$\tau_{res}$ [kPa]</td>
<td>45</td>
<td>50</td>
<td>80</td>
<td>105</td>
<td>132</td>
<td>150</td>
<td>155</td>
<td>165</td>
<td>175</td>
</tr>
</tbody>
</table>

Figure 105: Shear stress and boundary location from the ground anchor pullout data and obtained by the model.
8.7 Discussion and Conclusions on the Simplified Analytical Model

The observation of residual shear stress increase with increasing pullout load, loosely recognized in the literature, but strongly documented by laboratory and field tests of this study, called for an explanation of this phenomenon. Therefore, a simplified analytical model for progressive failure in soil-structure interaction was developed. In contrast to other models, this model allows the residual shear stress over the interface slippage length to grow due to additional confining stress in the sand cylinder around the cable.

In the laboratory, with well-defined boundary conditions, the model predictions match accurately the optically measured data obtained with the high resolution BEDS technology. Thus, the concept’s application is convincing and is able to explain the residual shear stress growth recognized in the tests. For further model validation, a parametric study (sand density, cable stiffness and diameter, surface roughness, etc) is suggested.

It was then attempted to apply the model to the ground anchor pullout field test, even though different boundary conditions apply. However, the model predictions were not satisfying. The reasons are believed to be in the rigid
front wall, which is missing in the field, and in the many parameters, which are
not known with good accuracy for the materials present in the field (grout
diameter, embedded depth, soil conditions, pullout force and displacement at
tip of fixed length, and general anchor installation parameters). An additional
field test with better defined boundary conditions could provide further insight
into the applicability of the model to the field conditions. For this test, the
anchor should be pulled directly out of a concrete retaining wall, with load and
displacement measurements at the tip of the fixed length.

To conclude, it can be said that the model is capable of explaining the
phenomenon of the presented pullout tests. Moreover, it emphasizes the
importance of direct measurements of interface slippage length and residual
shear stress for accurate predictions of progressive failure and ultimate
bearing capacity. To quantify these phenomena for a broad range of boundary
conditions, more sophisticated models have to be developed, which in turn
require more experimental data. Currently, measurements allowing for
sufficient accuracy are only possible using distributed fiber-optics technology.
It is suggested to use BEDS for structures of one to ten meters and BOTDA
for structures of from ten to one hundred meters.
PART III

NOVEL FIBER-OPTIC APPLICATIONS FOR LANDSLIDE MONITORING

A summary of this Part III has been published in the December 2010 issue of Geotechnical News (Puzrin et al., 2010).
9 Introduction and State of the Art

Part III focuses on the application of novel fiber-optic sensors to landslide monitoring. Following the successful implementation of the technology described in Part II to one-dimensional problems, new, challenging objectives were defined: implementation of the fiber-optic sensor into two- and three-dimensional problems.

The large-scale objects under surveillance in Part III are 3-dimensional creeping landslides. Defining and monitoring their boundaries is essential for the landslide understanding, analysis, and stabilization. Current conventional technologies, such as geodetic measurements and inclinometers are based on manual point measurements. Newer methods include radar and laser scanning. A complete picture of a landslide, however, involves the use of several complementary methods. The accuracy, as well as the time and cost, of measurements (particularly manual measurements) often form the limitations on the applicability of the method. Distributed fiber-optic sensors offer new possibilities in the field: by applying fiber sensors properly, the accuracy can be improved, while the costs and measurement time are lowered.

Presented in Part III are several long-term, field monitoring systems that have been designed, installed, and tested during this study. These include: a road-embedded, a soil-embedded, and a borehole-embedded sensor system. All landslide monitoring was carried out in the Swiss mountain resort of St. Moritz. The location was chosen since the IGT has a long history of monitoring there and thus, a large amount of data and experience was internally available. A summary of the systems is available in Iten et al. (2009b) and Puzrin et al. (2010).
9.1 Introduction

9.1.1 Landslide boundary

Differential soil displacements initiated by creeping landslides can cause immense problems by damaging infrastructure and buildings in the sliding area (Figure 107). Moreover, special construction and reinforcement requirements, or even total halt of construction within a landslide area may be demanded by local construction laws. Needless to say, the land owners, contractors, and authorities are each interested in defining the exact area affected by a landslide.

Figure 107: Damages caused by differential soil displacements.

Current mapping and monitoring is based on traditional techniques, such as geodetic measurements of the surface. This requires a very dense mesh of the markers in the vicinity of the landslide boundary and making this extremely time-consuming for larger areas.

In addition, for a comprehensive analysis, the boundary needs to be identified and monitored at the subsurface as well. Inclinometers serve for the detection of the boundary underneath the surface, called the sliding surface. However, once an inclinometer pipe is sheared, a conventional inclinometer probe can not be inserted anymore and the inclinometer will no longer produce results.

9.1.2 Detection of transition zone by fiber-optic sensor

The present study resulted in the development of several systems to detect and monitor the transition zone between the stable ground and the moving area (Figure 6). The basic idea is to meaningfully integrate the new distributed
strain sensors (BOTDA), which offer an unprecedented amount of quality data at reasonable costs, and thus, provide for the possibility to monitor large areas. This is not only useful in urban areas, but also for the monitoring of infrastructure (such as roads, railways and pipelines) crossing unstable terrain.

![Diagram](attachment://diagram.png)

**Figure 108: Simple model for movement interpretation from strain data.**

The following fiber-optic sensor projects have been developed with the aim to determine this boundary between the landslide and the stable ground.

The first system, an asphalt road-embedded sensor cable (Chapter 10), serves for the boundary evaluation in an urban region. A road, which intersects this boundary, can be seen as a large-scale strain gauge. Commencing in 2006, up to date, three such road-embedded sensor systems have been integrated and tested in the field.

For boundary identification in an area where no road or other infrastructure exists to attach a fiber cable to, a soil-embedded “micro-anchor”-cable system (Chapter 11) has been developed. The principle of this second system is that a cable, fixed to “micro-anchors” buried in soil, experiences the same movement as the soil around it. Laboratory testing of system parts started in 2007 and the first field integration of the novel, custom developed cables and anchors took place in July 2008.

The third monitoring system takes advantage of old, out of service inclinometer pipes (Chapter 12). In order to continue using such pipes, a fiber cable is placed inside and the pipe is filled with grout. The current sliding surface can then be identified and the displacements at this surface back calculated. Such a fiber-optic equipped inclinometer has been installed on site in July 2008.
9.2 State of the Art

9.2.1 Landslide monitoring

Conventional

There are many conventional means of landslide monitoring. It is not the intention to provide here a detailed overview of the methods and their performance. The monitoring can be anything from simple visual inspection to very specific instrumentation. The instrumentation includes all types of physical measurements (both periodic and continuous), which are carried out manually, automatically with a data logger, or fully automatic and remote. Table 30 lists the most common conventional landslide monitoring methods. Further reading about these methods and their precision can be found in the literature (e.g. Mikkelsen, 1996; Gili, 2000).

Novel Methods

Newer methods include radar interferometry, laser scanning and novel geotechnical sensors. Table 30 provides details of the novel methods (Löw et al., 2010; Schwager et al., 2010; Schmid et al., 2009).

9.2.2 The Brattas landslide in St. Moritz

The field applications of the fiber-optic sensors developed in Part III were all carried out in the vicinity of the renowned Swiss mountain resort St. Moritz. A major geotechnical hazard in this area is the creeping Brattas landslide, were the IGT of ETH Zurich has been actively involved over the last 30 years in all geotechnical aspects of the problem (e.g. Puzrin & Sterba, 2006). The landslide is of special interest, because it only stops in the middle of the town (Figure 109a). This means, there are urban areas (with remarkably high real estate prices) affected by the movement. An extensive displacement monitoring program is in progress and the town has adopted special construction laws for the affected areas. The Leaning Tower of St. Moritz most prominently shows how part of the urban area is on the moving zone, while
the other part (the church tower in the back) is on stable ground (Figure 109b).

Table 30: Conventional and novel monitoring methods for landslides.

<table>
<thead>
<tr>
<th>Conventional method</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual inspection</td>
<td>Taking notes, damaged infrastructure, plants.</td>
</tr>
<tr>
<td>Photogrammetry</td>
<td>Terrestrial, airborne or satellite pictures.</td>
</tr>
<tr>
<td>Geodetic</td>
<td>GPS, total stations, geometrical leveling.</td>
</tr>
<tr>
<td>Geotechnical sensors</td>
<td>Extensometers, inclinometers, piezometers, strain gauges, pressure cells, dilatometers, tilt sensors, crack meters, seismometers.</td>
</tr>
<tr>
<td>Meteorological sensors</td>
<td>Temperature, humidity, pressure, wind, radiation, precipitation.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Novel method</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radar interferometry</td>
<td>SAR (synthetic aperture radar). The most accurate form is the differential interferometry (InSAR). Terrestrial and satellite.</td>
</tr>
<tr>
<td>Laser scanning</td>
<td>LiDAR (Light detection and ranging). Terrestrial and airborne.</td>
</tr>
<tr>
<td>Geotechnical sensors</td>
<td>Various fiber-optic sensors (as discussed already in 1.4.3), IDM (inclinodeformometer).</td>
</tr>
</tbody>
</table>

The Brattas landslide is the lower section of the unstable northern slope above St. Moritz (Figure 110). It extends over a horizontal distance of 800 m, a height difference of 300 m, and is 600 m wide. The main sliding surface revealed in one boring reaches a depth of about 50 m (Alonso et al., 2010). The existing yearly displacement data (Figure 111) reveals the approximate location of the landslide boundary. At the southern end of the landslide, the Via Maistra is getting narrower by about 0.5 cm every year. Uphill, the displacement rate increases. An overview of all the sites described in the next Chapters is given in Figure 112.
Figure 109: (a) A view of the lake and the town of St. Moritz from the South with the Brattas landslide and; (b) the Leaning Tower.

Figure 110: The geology of the northern slope (after Müller and Messina, 1992).
Figure 111: Yearly horizontal displacements (after Schlüchter, 1988). The red circle indicates the location of the road-embedded sensor and the red dot the location of the borehole-embedded sensor.

Figure 112: The St. Moritz area from the South with the sites of the sensors outlined in the next Chapters: 1 road-embedded, 2 soil-embedded, and 3 borehole-embedded.
10 Road-Embedded Sensor for Landslide Boundary Evaluation

The objective of applying fiber-optic sensors to monitor the landslide was first approached with a road-embedded sensor. Such sensor embedded into an asphalt road seems to be a convenient solution for the boundary evaluation in an urban area. At first, the sensor cable was manually coated and integrated into a road in 2006 (Iten et al., 2008; Schmid et al., 2009). However, it was realized that the sensor coating required improvement, which led to the objective of developing new sensors. This was achieved in cooperation with Brugg Cables AG and resulted in new, better protected sensor cables that were subsequently integrated into the same location as the 2006 cables (Iten & Puzrin, 2009). Both sensors were able to localize the landslide boundary.

10.1 The Idea

The boundary of the creeping Brattas landslide (10.2.2) crosses the Via Tinus road (see circle in Figure 111 and site 1 in Figure 112) in the south-western region of the landslide. The road has an inclination of approximately 4.5% and is surrounded by buildings on both sides. No cracks have been observed in the asphalt, which indicates that the boundary shear zone is sufficiently wide for the asphalt to absorb deformations without cracking between scheduled repairs. To locate the boundary shear zone, it was decided to instrument the road with a fiber-optic sensor cable.

10.2 Field Instrumentation

2006 cables

The installation of the first set of sensors took place in October 2006. Along the hillside road boundary, an 89 m long, 7 cm deep, and 1 cm wide trench was cut into the asphalt (Figure 113). At the upper end of the trench (Figure 114), a shaft for the connectors was placed. At the lower end of the trench,
the two fibers of the S06 were spliced together in a loop (Figure 115). The S06 was then attached to the bottom of the trench by clips at every 2 m along. This procedure allowed pre-straining of the cable over the entire length. Next, the functionality of the cable in the trench was checked before applying a two-component epoxy which rigidly fixed the cable every meter to the asphalt at the trench bottom. On top of the strain cable, a temperature cable with a loop at its end was loosely positioned and the trench was filled with an elastic cold sealing compound (Figure 113). The cables functioned for 7 months and allowed for the strain data to be collected.

![Figure 113: Cross section of the trench instrumented with strain sensing fibers.](image)

**2007 cables**

After the 2006 cables failed in mid-2007, a second set of sensors was installed in August 2007. As it was not possible to retrieve the 2006 cables and clean the trench, another 9 cm deep trench was cut parallel about 10 cm away from the old trench. In addition, a second shaft was cut at the lower end of the trench. This setup, with two shafts, allows for better troubleshooting and a more flexible system (e.g. if one of the fiber in the loop is broken, the sensor can still be used as the remaining cable can be spliced to another fiber at the distant shaft). A pair of P07 and a pair of M07 cables were then pre-strained
and attached to the trench in a similar procedure as in 2006 (Figure 115). On top of the strain cables, a temperature sensor was loosely positioned and the trench was filled with the same elastic cold sealing compound as before.

*Figure 114: Via Tinus with part of the trench and upper shaft position at the far end of the trench.*

*Figure 115: Sketch of the trench along Via Tinus and the setup for the 2006 and 2007 strain sensors including the shafts.*
10.3 Results and Conclusions


An extensive set of strain measurements on the S06 were successfully carried out in fall 2006 and again in spring 2007. In order to filter out the noise, the data was processed by taking the moving average over a 3 m section. This procedure resulted in a sharp picture of the strain change. Figure 116 shows the change in strain in the fiber along the road in this 7 months period. It is immediately recognizable that, in a 15 m long section (between 43 and 58 m from the upper shaft), the strain increased by about 1000 με, while in the other parts of the fiber, the strain change stays within a 400 με band around the zero line. This strained section of the cable is sketched in Figure 117.

Temperature measurements where successfully performed in October 2006, however, they could not be repeated in May 2007, as the signal loss was already exceeding the required level for measuring with BOTDA. Therefore, temperature compensation is not incorporated into the strain measurement results. Air temperature at noon during the measurements in October was around 3°C and in May around 18°C. The air temperature difference of about 15°C can influence measured cable strain by about 270 με at the average, with locally higher values if the road is exposed directly to sun radiation. Considering that the cable is 7 cm deep in the road and insulated by the sealing compound, and taking into account the fact that the measurements were taken in the morning, it can be concluded that temperature influence is likely not exceeding 200 to 300 με. Furthermore, the strain shift due to temperature change would be similarly distributed along the 89 m of cable.
10.3.2 Monitoring 2007 cables: October 2007 – October 2008

The 2007 sensors at the Brattas landslide site accumulated an extensive set of data from August 2007 to October 2008. Since the two sensing fibers show a similar trend, the strain change in the 12 month period was calculated as an average of the two sensors’ measurements (Figure 117). The monitoring data showed that, in a 24 m long section (between 35 m and 59 m from the upper shaft), strain increased at about 300 με. While in the other parts of the fiber, the strain change stays within a 200 με band centered about the zero line (except on a short section close to the lower shaft (trench meter 16 to 21), where a strain increase of about 350 με was encountered). As a downside, temperature measurements could only be successfully carried out twice in between, but not for the whole period. For this reason, temperature compensation had to be calculated considering the air temperature at the time of the measurements. Despite this shortcoming, in comparison to the 2006 cables, an improved temperature compensation was still possible.
10.3.3 Monitoring results

The main results are (Figure 118a):

- The position of the transition zone;
- The length of the transition zone (increases from 15 m to 24 m);
- The average strain in the transition zone (decreases from about 1000 to 300 με).

The only parameter missing is the angle at which the landslide boundary crosses the cable. From geodetical data and Figure 111, the yearly displacement of the landslide is about 1 to 2 cm in this area. Thus, an angle of the displacement can be found as a function of the transition zone length and average strain in the transition zone, which corresponds to displacements on the order of 1 cm to 2 cm (Figure 118b). This angle of displacement was found to be about 45° to the cable axis.
10.3.4 Validation of the results

Visual observations

Visual observations of neighboring buildings were performed in August 2007 and showed that a number of them have serious cracks in their structures (Figure 107). Some cracks appeared to be fresh, i.e. not observed in 2006. The sites of cracked structures, and the area where BOTDA measurements showed strain transition, have been mapped in Figure 119. The line that can be drawn connecting these sites would also cross the fiber-optic equipped trench in the road at approximately a 45° angle.

Damaged water pipeline

Additional validation of the results came up by chance. When the optical strain measurements were being scheduled to continue after the winter 2008/2009, a fiber breakage occurred. During visual observation of the trench, it was found that the asphalt was patched in a section (Figure 120). Neighbors explained that a water pipeline broke just a few weeks ago and, of course, during the emergency opening of the ground, the cable was destroyed. The sensor was lost, but it was realized, that this happened exactly at the location, where in 2008 a short section of sharp strain increase of about 350 με was encountered (Figure 117). This strain increase indicated an additional shear...
zone inside the sliding mass, which eventually broke the pipe. This provided another, unexpected validation of the monitoring results.

Figure 119: Visual observations of cracks in neighboring buildings and structures (small points) and strained area in cable (large point).

Figure 120: The water pipeline breakage in early 2009 crossing the 2007 cables.

10.3.5 Conclusions

The road-embedded sensor for landslide boundary evaluation was the first field project of this study. The largest problem was that there were not
appropriate strain cables available to begin. This problem was first solved by a self-made cable (S06) and one year later by custom produced cables (P07, M07). The S06 produced results for 7 months. In terms of cable development, this project initiated the cooperation with the industry, as it convinced the cable manufacturer to realize the demand for well protected strain sensing cables. The 2007 custom cables were already a dramatic improvement and they provided measurements until they were destroyed in an emergency.

In general, it has to be mentioned that the road is a difficult place for sensor placement. Although sensor integration into the road can protect the sensor well from the heavy loads and harsh weather conditions, this integration allows no access to the cable at later stages. Thus, if the sensor is broken somewhere, the entire sensor is lost. Of course, this can be improved upon in further projects. However, in addition, a road is a place where regular and unscheduled construction activities occur often (pipes are buried, shafts need to be opened and closed, asphalt needs to be repaired, etc). Therefore, it will be very difficult to keep all stakeholders informed and aware to preserve the embedded sensor cable. Further improvements on sensor integration and protection, as well as access to repair broken cables, is required.

The present project demonstrated the applicability of road-embedded cables for landslide boundary monitoring. With this system, reasonable results are obtained. In combination with the new sensing cables developed during this study, the system can be more sensitive than conventional methods and allows for monitoring over larger areas. This makes road-embedded cables a powerful monitoring tool in urban regions.

For the ongoing monitoring and consulting of the IGT in St. Moritz, new knowledge was obtained with respect to the Brattas landslide. First of all, it was possible to locate the landslide boundary and specify the width of the shear zone between stable and moving ground. And second, the existence of an additional shear zone within the moving ground was detected (where the pipe broke).
11 Soil-Embedded Sensor for Landslide Boundary Evaluation

The objective of identifying and monitoring landslide boundaries is, as with the road-embedded sensor, the driving force behind the soil-embedded sensor development. However, when no road or other infrastructure exists to which the cable could be attached to, the sensor must be connected to the soil directly. The development of the “micro-anchors” to facilitate connection to the soil, the understanding of the behavior of this connection under load, and the further development of well protected sensors were part of this thesis (Iten et al., 2009a; Iten et al., 2009c) and is the subject of further studies in conjunction with another thesis at the IGT (Hauswirth, 2010a; Hauswirth et al., 2011).

The soil-embedded sensor serves not only for the boundary evaluation in the Greenfield, but can also be applied to pipeline monitoring by embedding the sensor system in the trench of the pipeline.

11.1 The Idea

In order to measure the soil strain accurately in the cable’s axial direction, no slippage between the soil and the embedded cable is allowed to occur. As seen in Chapter 8, this is only true up to specific cable stress (cable load), which is, in turn, transferred to the sand by surface friction. Additionally, in the direction perpendicular to the cable, it must be ensured that the cable makes the same movement as the soil, rather than the soil flowing around the cable. Theoretically, fixation of the cable to the soil can be controlled by cable stiffness and the friction between the soil and the cable. For field projects with natural soil and commercially available cables, this is not possible.

11.1.1 The soil-embedded “micro-anchor”-cable system

For this reason, a CTI project together with the cable manufacturer Brugg Cables AG and the measurement unit producer Omnisens SA, was initiated with the title “Novel fiber-optic sensing systems for soil displacement
monitoring” (CTI, 2008). The soil-embedded sensor system consists of two parts: “micro-anchors” and optical cable (Figure 121a). The purpose of the “micro-anchors” is to connect the cable to the soil at the specified fixation points, so that the cable experiences the same movement as the surrounding soil. The “micro-anchor” consists of three perpendicular planes, which provide bearing resistance in all directions, acting as three dimensional “dead” anchors (Figure 121b). Furthermore, the “micro-anchors” allow pre-stressing the cable during integration and, therefore, to measure not only tension but also compression.

11.1.2 Laboratory testing of the system

For the dimensioning of the system parts (“micro-anchors” and cables), two tests were developed: anchor failure test (longitudinal pullout) in the pullout box (Chapter 6) and full scale simulation of a shear zone in a large shear box. The largest part of the laboratory testing was carried out by D. Hauswirth (Hauswirth et al., 2010). Therefore, only a short overview of the testing is provided here, with results published elsewhere in the near future.

Anchor failure test

The mechanical pullout properties of the anchors and the tension elements in general were investigated in several studies (e.g. Neely et al., 1973;
Abramento & Whittle, 1995). However, for the “micro-anchors” used in this study, additional testing was necessary. This is because the design of the system depends on the exact specification of the “micro-anchor” pullout behavior and bearing capacity. This allows for determining separately the pullout load transferred from the cable to the soil and from the anchor to the soil. With the knowledge obtained, it is possible to adjust the anchor size and its embedded depth to the desired cable stiffness. The task was approached by pulling sand-embedded cables and cables with an anchor attached to it out of the pullout box (Figure 122 - the sand-embedded cable pullout is analyzed in depth in Part II).

Figure 122: Setup of the laboratory tests with the pullout box.

Shear zone simulation

The anchor failure test only provides the longitudinal sensor behavior. Close to the landslide boundary, the sensor also experiences movements transverse to the cable axis. For an accurate simulation of the real sensor behavior in a shear zone, a full scale shear zone test was designed.

The test setup consists of two 4 m long, 0.6 m wide, and 0.4 m deep boxes filled with sand and an initial 1.16 m long shear zone in between (Figure 123). Cables can then be embedded at covering depths from 0.1 m to 0.3 m in compacted moist sand. A mechanism in the shear zone allows moving one
box (while the other stays in place) along, in principle, any displacement path in x- and y-directions. A similar full scale test for a fiber-optic sensor equipped geotextile exposed to local settlements was recently performed by Belli et al. (2009).

As an example of the shear zone simulation tests, Figure 124 shows the measured strain for several sensors (with and without anchors) compared to the calculated strain from the applied displacements between the two boxes. The measurements clearly demonstrate the effectiveness of the “micro-anchor”-cable system.

Selection of appropriate sensor parameters

The intention of the laboratory testing was to understand and quantify the influence of cable longitudinal stiffness, anchor size, sand density, and embedded depth on the anchor bearing capacity, the system’s sensitivity, and the measurement accuracy. Such understanding allows an application to dictate the appropriate cable and “micro-anchor” and define integration procedures (depth, compaction and pre-straining).

Figure 123: Plan view of the shear zone simulation (after Hauswirth et al., 2010).
Figure 124: Strain measured in a shear box by a cable only and by the “micro-anchor”-cable system in comparison to the applied strain (after Puzrin et al., 2010).

11.2 Field Instrumentation

The “micro-anchor”-cable system was integrated in 2008 into a hiking path to monitor a supposed creeping landslide about 500 m southwest from the road-embedded sensor (site 2 in Figure 112) in the vicinity of St. Moritz, Switzerland. Downhill from the path, a building was severely damaged in 2006 due to differential displacements. At that point, concerns began to rise that a creeping landslide may exist at this location. It is assumed that the landslide is almost stationary under normal conditions, but may be triggered by a nearby excavation, such as was undertaken in 2006 for the construction of a new building.

For the installation of the sensor system, an approximately 0.4 m deep trench was cut along 80 m of the path (Figure 125a). It was anticipated that part of the trench would lie on stable ground while the rest is in the creeping zone. This would allow for the sensor to intersect the possible landslide boundary. The bottom of the trench was then filled with a 10 cm layer of compacted sand. On that sand, the sensor cable was placed in the trench and the “micro-
anchors” attached to the S08 cable at every 2 m (Figure 125b). The sensor was slightly pre-strained and covered with about 15 cm of compacted sand (Figure 125c). At the top, the original soil was poured and the path restored by vibration compaction. In addition to the “micro-anchor”-cable system, an S08 cable without anchors was put in the trench for the sake of comparison and validation. For temperature compensation means, a loose tube cable was placed as well.

![Figure 125: (a) The trench cut in the hiking path; (b) attachment of the “micro-anchors” to the cable and; (c) compacting of the soil above the cable.](image)

### 11.3 Results and Conclusions

Reference readings of the strain were taken the day following the installation (July 2008). Monitoring was performed 3, 12, 14 and 17 months after installation. The strain change (temperature compensated) in the “micro-anchor”-cable system and in the soil-embedded cable for this 17 month monitoring period is shown in Figure 126a. From the strain readings, the following can be recognized:

- At first, the data looks like a random distribution. But one has to keep in mind that the two cables (with and without anchors) measure independently the same strain peaks at the same locations. Thus, the measured strain accurately represents the strain present in the soil;

- The “micro-anchor”-cable system is more sensitive to local strain changes than the cable only. Strain peaks are detected by the system,
whereas the cable redistributes this over longer distance (e.g. trench meter 4, trench meter 23 to 24, and trench meter 68);

- The pre-straining was successful, as large negative strain can be monitored;

- Strain changes during the first 3 months are negligible, so as are the strain changes between month 12 and 17 are. Both correspond to the late summer and fall period of the year.

- As expected, due to seasonal changes in the landslide movements (largest activity during the spring), the biggest increase in strain happens between month 3 and 12. This can also be read out of Figure 126b, where the strain during the late summer and full period (first 3 months) is compared with a strain for an average 3 months period between late fall and early summer (i.e. the strain increase between month 3 and month 12 is taken and divided by factor 3). This observation of seasonal changes in the landslide movement is consistent with inclinometer readings taken nearby;

- The landslide boundary is not detected. The irregular strain distribution leads to the conclusion, that the sensor must be embedded in a moving mass. In this mass, there are no two points, which move 100% parallel to each other, resulting in high tensile and compressive strains in between (and thus, accounts for the irregular strain distribution along the sensor).

Event though the goal of detecting a landslide boundary was not achieved (unfortunately, at project planning, sufficient geodetical data was not available and thus, the boundary was only a rough estimation), a large amount of new knowledge was obtained:

- Adequate protection of sensors during integration (harsh construction environment) and lifetime application scales are possible, which makes this system feasible for large scale sensing projects on real landslides;
Figure 126: Strain change along the S08 sensors trench-embedded below the hiking path for (a) the whole monitoring period and; (b) for 3 month periods in different seasons.
• The high sensitivity of the soil-embedded sensor leads to a possible application for site investigation purposes. The detection of strain changes in the moving mass already in the first 3 months means that this tool may identify potential hazardous zones quicker than conventional methods. In addition, the greater sensitivity of the “micro-anchor”-cable system in comparison to the soil-embedded cable only was also proven in the field;

• Data acquisition in any season was possible (the monitoring was successfully performed even during the winter months at minus temperatures). From this, it can be concluded that quasi year-round availability of field monitoring can be achieved, even though the equipment and the staff were pushed to their limits (Figure 127);

• Long term performance (the sensors work up to today (last measurement taken in November 2010), with no problems or breaks encountered. This can serve as a positive example for the long term serviceability of the little protected S08 sensor. From January 2010 on, the monitoring duty was passed to D. Hauswirth, who will use the future results for his dissertation).

Figure 127: Difficult access to the monitoring site in St. Moritz during winter 2009/2010.
12 Borehole-Embedded Sensor for Landslide Boundary Evaluation

The last Chapter of Part III focuses on the evaluation and monitoring of the landslide boundaries in the vertical direction. In combination with the road-embedded or soil-embedded sensor, the objective of monitoring the landslide in all three dimensions is achieved. A first borehole-embedded sensor cable for the sliding surface detection (the boundary in the vertical direction) was tested on the Brattas landslide in St. Moritz (Iten et al., 2009b). The system was able to detect the boundary quickly and precisely. In addition, the horizontal displacements along the borehole were back-calculated and compared with inclinometer measurements at the same location.

12.1 The Idea

Close to the compression zone and lower boundary of the Brattas landslide (red dot where Via Tinus and Via Maistra meet in Figure 111 and site 3 in Figure 112), an inclinometer pipe was installed in 1982 for the localization of the sliding surface and monitoring of displacements. The inclinometer pipe has a standard 71 mm diameter, and goes 14.75 m deep into the ground. Since 1987, the inclinometer cannot be monitored anymore because the conventional probe does not fit all the way through the pipe any longer. However, a thin object, such as a weight on a string still makes its way down to the bottom. For that reason, it was decided to reactivate the inclinometer by installing a fiber-optic sensor.

12.2 Field Instrumentation

The sensor consists of the P07 cable, which was inserted into the inclinometer to 14 m depth and slightly pre-strained (by attaching a weight at the sensor bottom). The whole pipe was then filled with a cement-bentonite grout backfill (Figure 128). The grout is intended for the overall bonding of the fiber along the pipe. No temperature compensation sensor was integrated, as it is
assumed that the temperature change in the ground is not significant compared to the expected strain.

Figure 128: The fiber-optic instrumentation of the old, out-of-service inclinometer.

12.3 Results and Conclusions

In July 2008, after the grout hardened, zero readings of the strain in the fiber were taken. Monitoring was performed 3, 12 and 14 months after the installation. The change in strain along the cable for the 14 month period is displayed in Figure 130a. It can be seen, that between 5.8 m and 6.8 m depth, the strain in the fiber increased as much as 5000 $\mu\varepsilon$, while in the other parts, the strain increase is small. Even after only 3 months, the strain increase in this zone was already visible (and reported by Iten et al., 2009b), underlining the overwhelming sensitivity of this application. As expected, temperature influence was insignificant compared to the high strain values obtained.

Using the measured strain, the horizontal displacements along the borehole can be approximately calculated. For this, the angle $\beta$, at which the sliding surface crosses the inclinometer pipe, has to be estimated (Figure 130b). In the case of the present borehole, which is situated at the lower boundary of the landslide, the angle $\beta$ was estimated to be $20^\circ$. The approximate horizontal displacements are shown in Figure 130a. The horizontal displacements at the surface of the borehole are comparable with the displacements expected in that area (Figure 111) and geodetically measured displacements at the nearby leaning tower. A comparison with inclinometer
data from the 1980’s (Figure 130b) strengthens the conclusion that this simple application of fiber sensors produces reasonable results. The sliding surface is still at the exact same location, and, as the monitoring results suggest, still highly active.

**Figure 129:** (a) Average strain change in the borehole-embedded fiber sensor and; (b) sketch of the borehole with the sliding surface crossing at angle $\beta$.

Of course, this application does not substitute for inclinometers, as no indication about the direction of the displacement is obtained in this measurement. Fiber-optic inclinometers have been studied (see 1.4.5), but doubts about the practicality of the application is being raised (especially the instrumentation, as conventional inclinometer pipes are connected together onsite, whereas the fiber cable cannot be simply connected every few meters on the construction site).
However, the exact direction of the sliding is not always the most important issue and can also be obtained through alternative measurements (geodetical measurement of borehole location). Thus, this application is a cost-efficient and straightforward way to reactivate old inclinometer pipes. In addition, any borehole created during geotechnical prospecting and no longer used could be, at a later stage, instrumented with a fiber cable grouted inside, offering the geotechnical engineer a large amount of significant data.

![Figure 130: (a) Calculated horizontal displacements in the borehole-embedded sensor and; (b) inclinometer readings in the same borehole 25 years ago.](image)
PART IV

CONCLUSIONS AND RECOMMENDATIONS
13 Conclusions

Summary of applications

Over the last years, Brillouin distributed sensing has gained wide acceptance in the structural health monitoring field. The geotechnical monitoring field, with its harsh soil environment and often not so easily determinable parameters is still lagging behind. In this study, novel applications of distributed fiber-optic sensing to geotechnical engineering were identified, developed, implemented, and evaluated. First, one-dimensional structures were considered:

- Strain distribution monitoring along a soil-embedded cable during pullout;
- Strain distribution along a monitoring ground anchor during pullout.

The successful implementation of the technology to one-dimensional structures led to the application of the sensors in two- and three-dimensional problems:

- Road-embedded sensor for landslide boundary evaluation;
- Soil-embedded sensor for landslide boundary evaluation;
- Borehole-embedded sensor for landslide boundary evaluation.

However, in order to achieve meaningful results, significant contributions to the technology itself, the sensors, and the data interpretation were required:

Contributions to distributed fiber-optic sensing

New technologies: New distributed fiber-optic sensing technologies, especially the ones focusing on smaller spatial resolution, higher acquisition rate, and larger strain range widen the range of possible applications in geotechnical monitoring. Providing an appropriate and potent first application for such a new technology offers the possibility to promote a powerful picture of this new technology. By creating an impact on the involved scientific
community, further development is pushed. With the sand-embedded cable pullout and the successive data interpretation, the potential of the novel BEDS technology was demonstrated (Chapters 2, 6 and 8).

**Sensor development and sensor integration techniques:** Only the in-depth understanding of the soil mechanics behind a specific problem allows the fiber-optic sensor to be integrated meaningfully, and thus, obtain reasonable geotechnical monitoring results. In addition, the mechanics within the sensor (e.g. the “micro-anchor”-cable system or the cable protection layers around the fiber) need to be tested and controlled. In the present study, sensor integration techniques were developed and successfully applied, ranging from simply embedding in sand, to point fixation with “micro-anchors”, and to fully bonded sensors in epoxy and grout. Elaborate laboratory testing of the sensor and the sensing system pushed the development and improvement of new commercial strain sensing cables (Chapters 4, 7, 10, 11 and 12).

**Data interpretation:** The interpretation of the resulting monitoring data can, in the best case, accurately sketch the problem and lead to correct decisions on the necessary measures to take. In the worst case, incompetent data interpretation leads to uncomfortable situations endangering lives and valuable infrastructure. The present study showed options of improving the data interpretation and strengthening the conclusions by including physics and mechanics:

- The Brillouin frequency shift as a convolution product (Chapter 3);
- Simplified analytical model of shear stress distribution (Chapter 8).

As a result of the novel distributed fiber-optic sensor applications, new geotechnical knowledge in progressive failure and landslide monitoring was obtained.
New geotechnical knowledge

Progressive failure: Detection of the phenomenon of residual shear stress increase with increasing pullout load. Documentation of the failure progression in high resolution laboratory tests and in a field test. Drawing a possible explanation of this phenomenon with a conceptual, analytical model (Chapters 6, 7 and 8).

Landslide monitoring: For the ongoing monitoring and consulting of the IGT in St. Moritz, new knowledge was obtained with respect to the Brattas and Laret areas. The road-embedded sensor at the Brattas site detected primary and secondary shear zones. Both zone locations could be confirmed independently from the damage to buildings and infrastructure. The soil-embedded sensor at the Laret site shows the extent of inequalities of the surface displacement in a moving soil mass (Chapters 10 and 11).

Commercial potential

Last, but not least, the potential of distributed fiber-optic sensing applications in geotechnical monitoring was not only demonstrated in the scientific area, but also to industry. During the last year, the author’s work successfully convinced several industry stakeholders and earned two Swiss-wide business plan contests (Venture 2010 and Venture Kick) with the idea of starting an innovative geotechnical monitoring company based on fiber-optic sensors (even in the face stout competition such as new vaccinations and cancer medications). Distributed Brillouin sensing for geotechnical monitoring is a technology with rapidly emerging market.
14 Further Research

As this study is the first of its kind at the IGT of ETH Zurich, several recommendations for further research can be formulated.

Sensor integration techniques

Today’s knowledge regarding fiber-optic sensor integration techniques is quite advanced, the same being true for sensors and fixations currently available. For future projects, quality testing of the commercially available sensors is necessary. In addition, long-term behavior and the parameters influencing fixations and anchorages (glue, grout, mechanical clip, creep of the cable jacket, temperature variations) need to be better understood.

Data interpretation

With respect to data interpretation, two areas offer the possibility for further studies: data interpretation standards and parametric study for the analytical model.

As mentioned, fiber-optic strain sensor data interpretation standards are not available as of yet. Though drafts exist and standards are due to be a main focus in the new European COST TD100 project. However, especially for geotechnical fiber-optic monitoring, there is still much to do. More research and publications on data interpretation standards and sensor accuracy are very welcome.

The simplified, analytical model is able to conceptually prove the growth of residual shear stress. However, the model can be improved and strengthened by further parametric studies with laboratory tests (changing sand density, cable stiffness and diameter, surface roughness, etc.). In addition, the model needs to be modified for field conditions. Quantitative data from a ground
anchor pullout test with boundary conditions similar to the sand-embedded cable pullout is required.

**New technologies**

For field applications, the author is looking forward to a compact BEDS measurement unit that can be taken into the field. Once such a unit is available, many new applications can be thought of, as one single fiber-optic cable basically substitutes for hundreds of strain gauges. This is a concept that is a revolutionary idea for structural, as well as for geotechnical engineers.

**Commercial projects**

Large amounts of data are soon to be obtained in commercial projects (e.g. in monitoring of pipelines crossing unstable mountainous terrain). Mostly, the industrial partners use the data only as a check, if certain threshold values are exceeded, then an alarm is triggered. It would be advantageous if the research community gains access to the immense data resource. A careful analysis of the data would certainly produce exhaustive conclusions about the serviceability, the long-term behavior, the accuracy, and the advantages of fiber-optic sensors in geotechnical monitoring.
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Curriculum Vitae

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08/1996 – 06/1997 Bonners Ferry High School, USA
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Language Skills
Native: German
Good: English, Portuguese
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Internships
2003 NEAT tunnel construction in Bodio, Switzerland, with Zschokke Locher AG (today Implenia AG)
2002 CGE contractor, Londrina, Brazil

Awards
2011 Venture Leaders (Swiss National Startup Team)
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2009 Young researchers award from the Japanese geotechnical society
2005 Culmann-Award (ETH Zurich) for excellent master thesis
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