Spatial deformation estimation and long term structural health monitoring in underground engineering

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**SPATIAL DEFORMATION ESTIMATION
AND LONG TERM STRUCTURAL HEALTH
MONITORING IN UNDERGROUND
ENGINEERING**

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ENGINEERING**

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Spatial Deformation Estimation and Long Term Monitoring in
Underground Engineering

by

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ABSTRACT

The current understanding of geotechnical engineering and the development of underground structural health monitoring (SHM) methodology are still far behind the pace of the underground infrastructure development. The detection and estimation of time dependent displacement in tunnels and caverns have always been in the top priority to research in the underground engineering sectors. It is important to resolve the black box for both long term and short term time-dependent deformation in tunnels to provide estimates for support installation time, design consideration and possible collapse detection. This thesis aims to initiate solutions to estimate and monitor the time dependent deformation in underground infrastructures. It begins with a detail summary of the factors that influence the time dependent deformation and the traditional SHM techniques in underground engineering. This is followed by the development of a new excavation advancement rate incorporated longitudinal displacement profile (ARLDP) to address the short term time-dependent displacement in tunnel caused by the excavation activities. The complementary introduction of the superimposed longitudinal displacement profile (SLDP) mapping technique utilizes onsite convergence measurement to provide quick and reliable estimates of the tunnel displacement profile. The highlight of this thesis comes from the initiation development of the real-time long-term spatial SHM for underground engineering. The development starts with describing the SHM system layout and investigating its feasibility of using the Brillouin Optical Time Domain Reflectometry (BOTDR) fiber optic sensing technique in underground engineering. A geometrical analysis is performed aiming to transform the strain measurement acquired from the BOTDR sensors to displacement data. The major challenges in this research are found in the calibration of the BOTDR strain sensor and the investigation of the BOTDR fiber optic sensor (FOS) installation method. Through series of experimental studies, the author has successfully addressed the technical and physical deficiency in employing the BOTDR FOS to develop the underground spatial SHM system. An onsite demonstration is also presented in this thesis to prove the feasibility and the possibility to develop such real-time long-term spatial SHM system for underground engineering. It is hope that this research would make a significant contribution and valuable

reference for future development in underground engineering.

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LIST OF SYMBOLS

The following symbols are used in Chapter 2

σ_{rr} – Plastic zone and history of the radial stress

σ_{tt} – Tangential stress

σ_{yy} – Axial stress

σ_{ry} – Shear stress

a – Tunnel boundary = tunnel radius, r

$R = r$ – Tunnel radius

$X = x$ – Distance away from excavation face

$u = \bar{u}$ – Tunnel deformation

f_c – Uniaxial compressive strength of the ground

e – Unsupported span

k – Stiffness

p – Support pressure

d_i – Displacement corresponding to the convergence of a specific point in tunnel

t – Time in day(s)

ε – Strain

L – Length

V_a – Acoustic velocity

v_B – Brillouin frequency shift

λ_o – Wavelength of laser pump

n – Index of refraction

$g_B(v)$ – Gain resulted from exponential decay of the acoustic waves

g_o – Brillouin gain coefficient

Δv_B^{LF} – Change of Brillouin shift recorded in the lubricated cable

Δv_B^{SF} – Change of Brillouin shift recorded in the “strain sensor” fiber

c_ε^{SF} , c_T^{SF} – Coefficients of the “strain sensor”

c_ε^{LF} , c_T^{LF} – Coefficients of the lubricated cable

l_0 – Length of the elastic region

$\varepsilon_{m,\max}^p$ – Maximum plastic strain of the coating matrix

$\varepsilon_{g,\max}^p$ – Maximum strain in glass core

ε_m – Average compressive strain of the coating matrix

The following symbols are used in Chapter 3

$U_r(t)$ – Radial displacement of tunnel wall at time t

$F(x)$ – Internal pressure

P_o – Initial isotropic stress

a – Tunnel radius

G – Modulus of deformation of the rock

P_i – Internal pressure

g_{ve} – Compliance function

$F_c(t)$ – Creep function

$C_{(x,t)}$ – Final closure of the tunnel

$C_1(x)$ – Closure related to normalized distance

$C_2(t)$ – Closure related to time function

$C_{\infty x}$ – Instantaneous closure

X – Distance related to plastic radius

R_p – Plastic radius

T – Time related to deformation

V – Tunnel advancement rate (m/day)

V^* – Normalized advancement rate

R_T – Tunnel cross sectional radius

R^* – Ratio of plastic radius to tunnel radius

u – Displacement

$u_{\max}$ – Maximum displacement

The following symbols are used in Chapter 4

u_r – Displacement/Deformation
 σ_r – Radial load
 $u_{\max}$ – Maximum displacement
 X – Specified longitudinal position along the tunnel surface
 R_T – Tunnel cross sectional radius
 ν – Poisson's ratio
 R_p – Plastic radius
 R^* – Ratio of plastic radius to tunnel radius

The following symbols are used in Chapter 5

L – Support length
 ρ – Radius of curvature
 h – Beam depth
 ε – Strain
 R – Distance from center of circle, O to tunnel surface
 δ – Displacement
 α – Correction factor

The following symbols are used in Chapter 6 and 7

ε – Strain
 P – Compressive load
 E – Young's modulus
 A – Cross sectional area

CHAPTER 1 – INTRODUCTION

1.1 BACKGROUND

For the past decades, urban planners in mountainous and densely populated cities like Hong Kong, Singapore, Tokyo etc. have been drawing their blueprint and stepping forward to underground infrastructure development for the advantages of saving the scarcity available land space. Besides the underground transportation systems, tunnels and mining facilities, the underground infrastructure development has been further extended to storage utilities and others. From the Ministry of Defense (MINDEF), Singapore, an example of such is the MINDEF Underground Ammunition Facility (UAF) in the Bukit Timah granite, initiated by the Defence Science & Technology Agency (DSTA) in Singapore (MINDEF, 2001). As compared to the conventional aboveground storage facility, the underground storage facility has resulted in a 90% reduction of land use surrounding the ammunition facility which is needed to be sterilized for the development as safety buffers. In addition, having the facility underground, another 300 hectares of land space which is equivalent to 400 football fields is saved.

The Jurong Town Corporation (JTC), Singapore, has introduced the second underground storage facility in Singapore for commercial usage, the Jurong Rock Cavern (JRC) (JTC, 2007). The JRC which is still under construction will be used for oil storage at a capacity of 1.47 million cubic meters. The caverns are located beneath the Banyan Basin and approximately 130m below sea level on Jurong Island. With the completion of phase one, it would help to save up to 60 hectares of surface land space on Jurong Island which can be used for other higher value manufacturing operations.

However, frequent and prevalent geologic hazards have caused various catastrophic damage and fatality to such advanced underground infrastructure development. Some examples of these catastrophic cases reported by the news are the tunnel collapse along Gaza Strip and Egypt border (The Palestine Telegraph 2010), West Virginia Railroad Tunnel collapse (Justice News, 2009), Hangzhou

Tunnel collapse, China (Aljazeera News, 2008), the Subway Tunnel collapse in Cologne, Germany (ASCE Smart Brief, 2009) and Cramel Tunnel Collapse in Barcelona, Spain (Barcelona Field Studies Center, 2011). The above hazards shown the urgent need of an efficient structural monitoring technique to predict and prevent due to the uncertainties associated with underground engineering. The up-to-date knowledge in the rock engineering is apparently not sufficient to monitor both the construction process and during the usage lifespan of those underground structures to prevent such hazards. It is worth noting that the knowledge in monitoring technique in underground engineering is developing at a much slower pace compared to the demand in practices. One of apparent reasons is the infant development of an economic and robust spatial real-time monitoring system. It is always a challenge for researchers and engineers to have an accurate and efficient strategy or methodology to measure, monitor and predict the deformation of rock masses during and after excavation. However, regardless of the depth, conditions and design usage of the underground structure, the underground engineering monitoring techniques are usually limited by the insufficient measurement data collected either during construction or operation. Therefore, most of the geological hazards take place prior to any warning notice for escape, which would lead to major injuries, death and damage of surrounding facilities.

On the other hand, rock masses generated in the natural environment are complex with stochastic uncertainties in different extent of imperfection. These imperfections appear as bedding planes, joints, shear zones and faults, which make the rock mass a combined solid of continuum and discontinuum. The distinct feature of the rock masses creates tremendous difficulties in the analysis procedure. In the early stage, geotechnical engineering problems were usually analyzed through differential equations with specified boundary conditions. Crediting to past effort, there are few simple geometrical deformation models available in the closed-form theoretical expressions, e.g., the classic Ground Response Curves for rock tunnels analysis (Brown et al., 1983) and the most common practiced Convergence-Confinement method of tunnel design (Panet and Guenot, 1982). However, those solutions developed through two-dimensional plane strain analysis nominally over simplify the actual problem, resulting in significant discrepancy between the analysis and the real situation. To overcome those limitations from the theoretical

solutions, Park et al. (2008) commented that the ground response curve can be enhanced and improved through numerical methods investigation, and Carranza-Torres and Fairhurst (2000) had improved the applicability of the convergence-confinement method in tunnel design. The success of such researches motivated the rapid development of numerical analysis methods in the past a few years. The Poisson's ratio longitudinal displacement profile (LDP) (Unlu and Gercek, 2003) and the improved longitudinal displacement profile (ILDLP) (Vlachopoulos and Diederichs, 2009) analysis methods for tunnel deformation were then developed and have been widely applied in various engineering analysis. Many researchers and engineers applied these methods in their tunnel design, estimating the support installation time and setting the ultimate deformation limit during excavation. But in the rock engineering discipline, this method remains the shortcoming from the theoretical analysis on over simplifying the effect of time dependent deformation through the excavation activities. They inherit the small displacement/deformation assumption from the theoretical method with paying little or no attention to the deformation influence by the excavation advancement rate effect during construction and assuming that the geological parameters after excavation remains the same. The ignorance of the effect of these time dependent factors on the deformation reduces the efficiency and accuracy of the analysis.

Acknowledging to the above deficiency, some scientists have attempted to address the time dependent deformation in tunnel excavation through analytical approach. Various viscoelastic models such as the time-dependent displacement model (Sulem et al., 1987), the time-displacement interaction model between rock and shotcrete support (Pan et al., 1994) and the stress and displacement model in viscoelastic axisymmetric plane (Wang and Nie, 2010) have been proposed to explore and estimate the time dependent deformation in tunneling. However, in order to reduce the complexity in calculation and variation due to various assumptions found in the equations, Fortsakis and Kavvadas (2009) tried another approach to estimate the time dependent ground parameters and behavior through back analysis of tunnel convergence data. This approach requires the reliability of the actual tunnel deformation information used in the back analysis, thus leading to the challenge of collecting sufficient amount of reliable site measurements. Therefore, it has become challenging and critical for researchers to develop a

relatively low cost, reliable and robust spatial Structural Health Monitoring (SHM) system that can obtain accurate and sufficient amount of monitoring measurement data to rectify and improve their analysis.

On the other hand, good design, excellent construction methods and safety in development of the underground structures are the goals in the infrastructure construction. A long-term real time SHM system is the key towards achieving these goals and ensuring the sustainability of the underground infrastructural development. SHM is a process designed to provide accurate and in-time information relating to structural performance and condition. Tan et al. (2000) commended that new structural systems, construction technologies and analysis methodologies are increasingly being used in underground infrastructure development, thus it is necessary to increase knowledge about their on-site performance, to control the design, to verify performance, to create and calibrate the numerical models.

Through years of development, a variety of point sensing instrumentation and measurement techniques have been developed in the past to measure the deformation of tunnel/cavern during or post construction. To incorporate with the convergence-confinement method for deformation detection, linear variable differential transformer (LVDT), tilt-meter, plate jacking, dilatometers, pressure-meters, strain gauges, borehole extensometers, convergence tape and the total station are some of the most common instrumentations used in measurement acquisition. Inaudi and Glisic (2003) illustrated their instrumentation setup with an onsite example of convergence monitoring in tunnels, during and after excavation at the Luzzzone dam inspection gallery. Two-point extensometers, multipoint extensometers and surface sensors were used in the convergence monitoring system. The convergence measured after excavation of a particular zone showed +3mm to -3mm displacement of the profile in the cavern section. Thevenaz et al. (2005) performed geotechnical monitoring of the Macquarie Park Station (Australia) cavern excavation. Their SHM system included inclinometers, extensometers, surface settlement points, endoscopes and convergence points. Bertuzzi and Anderson (2005) highlighted that monitoring of displacement and stress around a tunnel under construction should be part of the tunnel design process, though the type and detail of ‘appropriate’ monitoring are often debated. They presented their

experience with monitoring instruments in two road tunnels excavated in Hawkesbury Sandstone in Sydney, Australia and compared it with the published experience. Nevertheless, the above instrumentations faced the same challenges in low degree of automation, limited life-span and only able to achieve point measurement, which limited the amount of data collected for performance analysis of tunnels. Since the cost per sensing point is relatively high to develop a spatial monitoring system using those point sensing instrumentations, it is highly desirable to have a monitoring system which is capable of providing global and distributed measurements of an underground structure at reasonable cost.

In order to overcome the limitations from the traditional instrumentations, continuous distributed fiber optic sensors (FOSs) have made a gradual but momentous entrance into the sensor panorama. A single optical fiber can be used as a linear uninterrupted sensor, providing numerous measurement points spatially and continuously distributed over the whole fiber length. This technology is attractive and successful in cases such as the oil and gas industry, where FOSs are sometimes the only solution for reliable and long-term SHM. However, factors like quality of fiber core, polymer of fiber outer cover and method of installation can significantly affect the accuracy of measurements and the applicability of this SHM system.

Nevertheless, the successful introduction of these FOSs inspires research and development to deploy FOSs in underground civil infrastructure where the SHM technique is limited and still heavily relies on the traditional instrumentation and methodology. A number of surface-mountable FOS systems have been developed in the past years, among which the Brillouin Optical Time Domain Reflectometry (BOTDR) system has received much research attention in terms of its technology development and application. Different from the traditional monitoring instruments, the BOTDR system provides distributed, long distance, real-time, interference free and reliable measurement data (Haruyoshi et al., 1997). Liu and Zheng (1999) suggested that the BOTDR technology will serve as a promising solution to overcome the limitations of traditional instrumentations and be able to acquire substantial amount of field measurement to improve on geotechnical analysis and underground structure design methods. However, there is still a gap between the laboratory experiments and practical field applications.

Limited research has been conducted on how to maximize its possible applications in underground SHM.

Detailed literature review, examination of possible solution in the technical deficiency of installation methods, investigation of the BOTDR FOSs with different protection packages and calibration of the sensors are the keys to bridge this high-potential technology in designing a real time SHM system for underground applications. Though various efforts have been contributed to improving the displacement analysis in tunneling and advancement in instrumentation methodologies, the SHM methodologies, measurement acquisition through BOTDR FOS and the longitudinal displacement profile (LDP) analysis is still far from maturely developed. The drive for researchers and engineers to seek for robust real-time spatial monitoring systems is urgent and critical. Also, the improvement in methodologies on tunnel deformation monitoring and prediction for analyzing and solving geotechnical problems become necessary to prevent and reduce the disastrous damage, and increase the consciousness regarding construction safety and the economic feasibility in the civil and specially the underground engineering field.

1.2 PROBLEM STATEMENT

The current understanding of geotechnical engineering and the development of SHM methodology are still far behind the pace of the underground infrastructure development. As the excavation of underground tunnels is a dynamic process, the time-dependent deformation is an important parameter in support of estimate of the installation time, design consideration and possible collapse detection. However, the current longitudinal displacement profile (LDP) analysis method developed through plane strain analysis does not include the effect of tunneling excavation advancement rate in the analysis. This may cause large discrepancy between the estimated deformation from the LDP analysis and the actual deformation during the tunnel excavation process. Furthermore, in most underground applications, the convergence measurements are usually used to check against the tunnel deformation at a particular cross-section away from the excavation face. There is a lack of

methodology to predict and estimate deformation out of the monitored cross section. Thus, part of this thesis is devoted to developing an LDP analysis which accounts for the time-dependent excavation advancement rate. Such LDP developed is referred to as the ‘Advancement Rate incorporated LDP’ (ARLDP) in this thesis. The developed ARLDP analysis will be used to predict the deformation of tunnel cross section due to excavation activities. In addition, as highlighted in the last section, this thesis is also devoted to addressing the limitations and challenges to develop a robust spatial SHM system framework and methodology for underground application.

1.3 OBJECTIVES OF RESEARCH

With the considerations addressing above problems, the objectives of this research are:

- a) To incorporate the time-dependent deformation due to excavation activities in the LDP analysis. The developed ARLDP will be essential to estimate the deformation in tunneling;
- b) To establish technique through back analysis for tunnel deformation estimation out of monitoring section using the convergence measurement data;
- c) To investigate the capability of various BOTDR sensors and calibrate a suitable sensor for the real-time spatial SHM development; and
- d) To evaluate the proposed real-time spatial SHM system for underground application.

The originality of this research is the initiation in development of the long-term FOS SHM system for underground engineering application. The introduction and findings of ARLDP development, SLDP analysis method and intensive investigation and calibration of BOTDR sensors have made significant academic contributions in tunnel deformation analysis.

1.4 SCOPE OF WORK

The research scope involves mainly in the development of the SHM methodology in underground engineering. This includes the introduction of the superimposed LDP and LDP mapping technique for deformation estimation, the advancement in LDP with tunnel advancement rate effect (ARLDP), the analytical and numerical studies on the geometrical analysis for strain to displacement conversion, the experimental studies on the BOTDR FOS and its installation method for setting up the real-time spatial SHM system in underground engineering.

- a) To extend the current LDP analysis to the fulfillment of the time-dependent deformation contributed from different advancement rate based on reviewing the theories, analysis and numerical computation of the developed ARLDP method.
- b) To establish the basic framework for the deformation back analysis methodology , including
 - i. Reviewing various research theories, formulations and functions of LDP.
 - ii. Setting up the general framework and deriving equations for the superimposed LDP (SLDP) method.
 - iii. Evaluate the SLDP mapping technique for deformation prediction from the monitoring cross-section to the excavation face.
- c) To set up a general framework for spatial SHM system, analyze the spatial cross-sectional strain measurement achieved, and derive the general formula for strain-displacement conversion through geometrical analysis.
- d) To investigate and calibrate BOTDR sensor for building the spatial SHM system suitable in underground application, which includes
 - i. Detailed review on the BOTDR technique and discussion of the advantages of applying BOTDR technique in underground SHM, the advantages and the deficiencies in the BOTDR strain sensing application.
 - ii. To describe, investigate and compare the BOTDR fiber optic sensors with different protection packages and finally, through series of experiments, to calibrate the two prototypes of BOTDR

strain sensors developed which can be applied in designing a real-time long-term spatial SHM system for underground structure monitoring.

- iii. To investigate and examine a possible solution in the physical and technical deficiency found in the traditional BOTDR FOS installation methodology by scrutinizing an alternative installation method.
- e) To test and evaluate the proposed real-time long-term spatial SHM system in an underground tunneling application.
- f) To discuss the future prospect and importance of the proposed real-time spatial SHM system.

1.5 ORGANISATION OF THESIS

The issues mentioned in the previous section are addressed in the following chapters within this thesis.

Chapter 1 describes the research background, the objectives, uniqueness and importance of the present research.

Chapter 2 provides detailed reviews of the past studies in topics on the convergence measurement, the ground response curve (GRC), the LDP analysis, the traditional displacement monitoring methods and instrumentations, the introduction of FOSs and the BOTDR technique.

Chapter 3 gives a summary of the time-dependent deformation contribution in tunneling. An analytical study is performed to separate the time-dependent deformation due to excavation activities from the creep function. Numerical analysis is then be used to construct the AR LDP with advancement rate effect.

Chapter 4 presents the different developments and functions of the LDPs. The superimposed LDP method is introduced through back analysis and the basic formula for the LDP mapping technique is derived. These techniques will then be

tested and evaluated from the published data and numerical simulation results.

Chapter 5 starts with an overview of the conceptual framework of using the BOTDR sensor to set up a real-time long-term spatial SHM system. It is followed by a geometrical analysis to convert the strain measurements achieved from BOTDR to displacement information. Based on the above analysis, a set of Matlab algorithm is developed, tested and evaluated with experimental and numerical analysis results.

Chapter 6 focuses on the investigation of the BOTDR sensor. A feasibility study of its suitability for underground application is discussed together with the challenge and issues of the present developed sensor. Through a series of experiments, the author first addresses the technical deficiency with installation methods for underground structure. Secondly, the author describes and examine the BOTDR with various protection packages and initiate suggestions for an ideal BOTDR strain sensor for underground application. Finally, the author calibrates the two developed prototype BOTDR sensors and evaluates their accuracy and suitability for underground application through different types of conditional testing experiments.

Chapter 7 focuses on the investigation and the experimental study of the BOTDR FOS installation method. It begins with description of the traditional BOTDR FOS installation methods, discussing their deficiencies and suitability for underground application. It is followed by a series of experiments to investigate different point fixing system and evaluate the alternative installation method. An actual field installation of the long-term spatial SHM system will be demonstrated in this chapter. The proposed SHM sensor installation scheme and actual site installation will be discussed with primary focus on the survivability of the BOTDR FOS SHM system. Accompanying with Chapter 6, this chapter will propose a long-term spatial SHM system suitable for underground engineering application.

Chapter 8 concludes the findings of this research and presents the recommendations for future work.

CHAPTER 2 – LITERATURE REVIEW

2.1 INTRODUCTION

This chapter will provide detailed review of the past researches and literatures that formed the base of this research. The focus review on the traditional convergence confinement method (CCM), the ground responses curve (GRC) and the longitudinal displacement profile (LDP) analysis gives the foundation and highlights the importance of time dependent factor analysis to advance and strengthen the tunneling displacement analysis technique. On the other hand, the traditional underground monitoring instrumentations and the conventional convergence measurement method are also reviewed, aiming to examine the shortcomings and limitations in the tradition tunnel displacement monitoring technique. Finally, a detailed review of structural health monitoring (SHM) system, the fiber optic sensors (FOS) and the distributed FOS, Brillouin Optical Time Domain Reflectometry (BOTDR) technique is presented. This review shows the necessity and the potential in possible development of a real-time spatial SHM system for underground applications, which is desired in measurement data harvesting for the time dependent factor analysis in underground engineering.

2.2 DEFORMATION ANALYSIS IN DEEP EXCAVATION – STRESS AND DEFORMATION

Plane strain analysis was well developed and widely applied to study the stress and deformation behavior in underground tunneling applications. Linard (2011), has demonstrated such analysis as shown in Figure 2.1. It illustrates the region with plastic deformations and the stress distribution along the excavation boundary ($r = a$) for an unsupported opening (Figure 2.1a), and three supported tunnels with a different lining stiffness k and unsupported span e (Figure 2.1b, 2.1c and 2.1d). From Figure 2.1, the axial stress σ_{yy} ahead of the face decreases with the approaching excavation from its initial value σ_0 (which prevails far ahead of the face) to zero (at the unsupported tunnel face). Due to the lowered axial stress, the core cannot sustain the radial pressure exerted by the surrounding ground. The core

yields and, consequently, larger radial deformations, u , develop ahead of the face, while both the radial and tangential stresses (σ_{rr} , σ_{tt}) decrease. This happens within the plastic zone, which extends up to a distance of one to two tunnel radii ahead of the face. As indicated by the peak in the tangential stress σ_{tt} (at the boundary of the plastic zone at $r = a$), a stress concentration occurs in the elastic region ahead of the plastic zone. Although this peak is slightly more pronounced if the tunnel is unsupported (Figure 2.1a), Figure 2.1 shows that the stress field ahead of the face is largely independent of the support characteristics. At the face, the radial stress becomes equal to zero, while both the tangential and axial stresses (σ_{tt} , σ_{yy}) become in accordance with the assumed yield condition and equal to the uniaxial compressive strength, fc . The continuation of excavation does not alter the stresses at the wall ($r = a$) of an unsupported tunnel (Figure 2.1a). In a supported tunnel, however, the radial stress at the tunnel wall remains equal to zero over the unsupported span $0 < y < e$ but afterwards increases due to the installation of the lining, as the latter offers a resistance to the deformations of the ground caused by the subsequent excavation (Figure 2.1b, 2.1c and 2.1d). The axial and the tangential stresses increase as well with the distance y from the face, because based on account of Coulomb's yield condition, the ground is able to sustain more pressure in the tangential and axial directions due to radial confinement. In the case of a stiff lining (Figure 2.1b and 2.1d), the axial stress σ_{yy} increases more rapidly than the tangential stress σ_{tt} and becomes the highest principal stress. This is because a stiff lining facilitates arching in the longitudinal direction, particularly if installed close to the face (Figure 2.1b). In the case of a soft lining (Figure 2.1c), the ground at the excavation boundary also continues to yield after lining installation.

Figure 2.1 – Plastic zone and history of the radial (σ_{rr}), tangential ($\sigma_{\theta\theta}$), axial (σ_{yy}) and shear stress (σ_{ry}) along the tunnel boundary ($r = a$).

(Figure adopted from Linard, 2011)

2.2.1 Convergence Confinement Method (CCM)

The convergence confinement method (CCM) allows the ground pressure to be assessed by means of closed-form solutions, and is widely used in engineering practice for preliminary estimation of the tunnel radial deformation and support

required (Panet, 1995). The method applies to the rotationally symmetric problem of a deep, uniformly supported, circular tunnel crossing a homogeneous and isotropic ground which is subject to uniform and hydrostatic initial stress. Under the additional simplified assumption of plane strain condition, the problem becomes one-dimensional (i.e. all field variables depend solely on the tunnel radius, r , from the tunnel axis) and can be solved analytically. The solution can be presented in the form of the ground response curve (GRC) (Brown, 1983), which shows the relationship between the support pressure and the radial displacement of the tunnel boundary. The equations for the case of linearly elastic and perfectly plastic ground behavior according to the Mohr-Coulomb yield criterion with a non-associated flow rule (the material model assumed) can be found, inter alia, in Anagnostou and Kovári (1993). Figure 2.2a shows an example of the ground response curves.

Figure 2.2 – Determination of the final lining pressure by the CCM. For different values of the uniaxial compressive strength of the ground (f_c), of the unsupported span (e) and of the radial stiffness of the lining (k). (a) Ground-support interaction (pre-deformations according to Chern *et al.* 1998); (b) Final lining pressure as a function of the uniaxial compressive strength (pre-deformations according to Chern *et al.* 1998); (c) Ground-support interaction (pre-deformations according to the implicit method); (d) Final lining pressure as a function of the uniaxial compressive strength (pre-deformations according to the implicit method)
(Figure adopted from Linard, 2011)

The CCM investigates the interaction between ground and tunnel support graphically by plotting the ground response curve and the characteristic line of the support in one and the same diagram. The latter shows the dependency of the radial displacement of the lining on the ground pressure acting upon the lining. The inclination of the characteristic line of the support depends on its stiffness, k , while the origin of the characteristic line on the displacement axis (e.g. Point A in Figure 2.2a) accounts for the pre-deformation of the ground, i.e. for the radial displacement that takes place before lining installation at a distance d behind the face. The pre-deformation occurs partially ahead of the tunnel face and partially over the unsupported span. In the computational example of Figure 2.2a the simplifying assumption was made that the pre-deformation, $u(d)$, follows the longitudinal displacement profile proposed by Chern et al. (1998):

$$u(d) - \bar{u} \left[1 + \exp \left(-0.91 \frac{e}{r} \right) \right]^{-1.7} \quad (2.1)$$

where u and $\bar{u}$ denote the final unsupported convergence (i.e. the convergence that would occur in an unsupported tunnel far behind the face) and along the tunnel radius, respectively. It can be seen immediately from Figure 2.2a that, as a consequence of the smaller pre-deformations, the predicted ground pressure is higher in the case of a higher strength ground. This is clearly contrary to what one might expect intuitively. Figure 2.2b provides a more complete picture of the effect of ground strength f_c on final lining pressure. In view of the paramount effect of pre-deformation on the final lining pressure, the question arises as to whether this unexpected model behavior might be due to the simplifying assumption of equation (2.1), according to which the pre-deformation amounts to a constant fraction of the final unsupported convergences (i.e. a fraction which is the same for an elastic and for a highly-stressed ground). A similar behavior can be observed when applying the improved longitudinal displacement profiles proposed by Vlachopoulos and Diederichs (2009), which in contrast to Chern et al. (1998) who considered the maximum plastic radius. The paradox persists even when applying the most advanced method of pre-deformation estimation, which is the called implicit method, introduced by Nguyen-Minh and Guo (1996) and proposed, inter alia, by

AFTES (2002). This method takes into account the support stiffness and installation point in addition to the properties of the ground and to the extent of the plastic zone. A synopsis of the equations can be found in Cantieni and Anagnostou (2009a). As can be seen from the ground – support interaction diagram of Figure 2.2c, even this more sophisticated analysis method predicts that the load developing in the case of a ground having an uniaxial compressive strength of $f_c = 1$ MPa is lower than that in the case of $f_c = 3$ MPa. Figure 2.2d shows the results of a parametric study (performed with the CCM in combination with the implicit method) on the effect of ground strength f_c on final loading at different values of the unsupported span e and of the lining stiffness k . It is worth noting that the softer the support lining and the bigger the unsupported span, the less pronounced is the paradox.

2.2.2 The Ground Response Curve (GRC)

The ground response curve (GRC) is also referred to as Fenner-Pacher curve. It is used to illustrate the ground pressure on the lining as a function of deformations. A steep ground response curve with a low minimum indicates a stiff and strong ground which needs little support of a lining. Such a cohesive and frictional soil is able to carry the overburden load by arching around the tunnel (Brown, 1983). Vice versa, a relatively flat ground response curve with a high minimum corresponds to a relatively soft ground that needs significant support from a lining. The hypothesis by Pacher (1964) concerning the trough-shaped ground response curve, the minimization of the rock pressure and the related lining thickness would seem to be important elements in tunneling design. The idea of calculating ground response curves on the basis of elementary ground properties was probably first suggested by Seeber (1978) and later by Brown (1983, 1984), Kovári (1985) and Kolymbas (1998). This idea of a trough-shaped ground response curve was however questioned by Kovári (1993) and thereupon defended by Vavrovsky (1994). The latter argued that distinction should be made between shallow and deep tunneling. On the basis of his tunneling experiences, Vavrovsky (1995) considered a concave ground response curve as realistic for shallow tunnels rather than for deep tunnels. Regardless of these debates the GRC development was done for deep tunnels by Bliem and Fellin (2001) to find non-concave curves.

2.2.3 The Longitudinal Displacement Profile (LDP)

The Longitudinal Displacement Profile (LDP) is the graphical representation of the radial displacement that occurs along the axis of an unsupported, cylindrical excavation – for sections located ahead of and behind the face. The diagram in Figure 2.3 represents such a profile. The horizontal axis indicates the distance x from the section analyzed to the tunnel face; the vertical axis indicates the corresponding radial displacement u_r (the right part of the diagram is included for use later in relating the LDP to the GRC and the Support Characteristic Curve (SCC)). The diagram indicates that at some distance *behind* the tunnel face the effect of the face is negligibly small, so that beyond this distance the unlined tunnel section has converged by the final amount u_r^M . Similarly, at some distance *ahead* of the face, the advancing tunnel has no influence on the rock-mass and the radial displacement is zero.

Figure 2.3 – Schematic representation of the Longitudinal Deformation Profile (LDP), Ground Response Curve (GRC) and Support Characteristic Curve (SCC).
(Figure adopted from Vlachopoulos and Diederichs, 2009)

The LDP discussed above is an important component of the convergence-confinement method. It provides insight into how quickly the support begins to interact with the rock-mass behind the face of the tunnel (i.e. it defines the point K in Figure 2.3). When the far-field stresses acting on the rock-mass are assumed to

be uniform, the profile of radial displacements along the axis of the tunnel can be computed from numerical models. The figure represents a longitudinal cross-section of an unlined tunnel of radius R in the vicinity of the face. At a distance x from the face the radial displacement is u_r . When the distance x is large enough, the radial displacement reaches the maximum value u_r^M . For negative values of x (i.e. ahead of the face), the radial displacement decreases and the displacement becomes essentially zero at some finite distance ahead of the face. For elastic models of the problem, Panet (1995) suggests the following relationship between radial displacements and distance to the face:

$$\frac{u^r}{u_r^M} = 0.25 + 0.75 \left[1 - \left(\frac{0.75}{0.75 + \frac{x}{R}} \right)^2 \right] \quad (2.2)$$

This relationship, equation (2.2), applied to positive values of x , is plotted in Figure 2.4b. The horizontal axis of the diagram represents the ratio x/R and the vertical axis represents the ratio u_r/u_r^M . Chern et al. (1998) presented measured values of convergence in the vicinity of the face for a tunnel in the “Mingtam Power Cavern” project. The measured data are plotted as dots in Figure 2.4b. Based on this data, Hoek (1999) suggested the following empirical best-fit relationship between radial displacement of the tunnel and distance to the face:

$$\frac{u^r}{u_r^M} = \left[1 + \exp \left(\frac{-x/R}{1.1} \right)^{-1.7} \right] \quad (2.3)$$

The above equation, equation (2.3) is also plotted in Figure 2.4b. Analysis of the curves defined by equations (2.2) and (2.3) indicates that the maximum radial displacement occurs at approximately 8 times tunnel radii behind the face of the tunnel, and that the radial deformation is zero at approximately 4 times tunnel radii ahead the face. At the face itself, the radial displacement is approximately 30% of the maximum value. Figure 2.4b also suggests that the elastic approximation defined by equation (2.2) overestimates the values of radial displacements when

compared with the measured data at the Mingtam Power Cavern project and with the statistical approximation to this data. As in Figure 2.2, this overestimation results in underestimation of the final load transmitted to the support. Ideally, for the tunnels designed according to the convergence-confinement method, the LDP should be constructed from measured data such as the one presented by Chern et al. (1998). When such information is not available, the LDP can be constructed from numerical models considering the same elasto-plastic parameters used in construction of the Ground Response Curve (GRC). Alternatively, and as a first approximation, the LDP could be evaluated using the relationship in equation (2.3).

2.2.4 The Limitations of the Convergence Confinement Method

The estimation of pre-deformation, which is of paramount importance for any plane strain model, starts by considering the development of the radial displacement $\bar{u}_E(y)$ along the excavation boundary ($y > 0, r = a$) of an *unsupported*

tunnel crossing a linearly *elastic* ground:

$$\bar{u}_E(y) = \bar{u}_E(\infty) F\left(\frac{y}{a}\right) \quad (2.4)$$

where $\bar{u}_E(\infty)$ denotes the final radial displacement far behind the face, while the function F is defined as follows (AFTES, 2002):

$$F(t) := 1 - 0.75 \left(\frac{0.75}{0.75 + t} \right)^2 \quad (2.5)$$

or, according to Bernaud (1990),

$$F(t) := 0.29 + 0.71 \left(1 - \exp(-1.5t^{0.7}) \right) \quad (2.6)$$

As both expressions lead to similar results, following Bernaud (1990) the development of convergence $\bar{u}(y)$ for the case of *elasto-plastic* ground can be obtained approximately by applying a so-called homothetic transformation to the elastic convergence $\bar{u}_E(y)$ (Figure 2.5):

$$\frac{OP}{OE} = \frac{\bar{u}(\infty)}{\bar{u}_E(\infty)} = \text{constant} \quad (2.7)$$

Figure 2.5 – Development of radial displacement along the excavation boundary of an unsupported tunnel crossing an elasto-plastic ground (Nguyen and Guo, 1996)

where $\bar{u}(\infty)$ denotes the final elasto-plastic convergence of an unsupported tunnel, and the so-called similitude ratio $\bar{u}(\infty)/\bar{u}_E(\infty)$ depends on the initial stress σ_0 , the Poisson's ratio ν and the plasticity constants c , ϕ and ψ .

From Figure 2.6 it follows that:

$$\bar{u}(y_p) = \frac{\bar{u}(\infty)}{\bar{u}_E(\infty)} \bar{u}_E(y_E) \text{ and } y_E = \frac{\bar{u}_E(\infty)}{\bar{u}(\infty)} y_p \quad (2.8)$$

Equations (2.4) and (2.8) yield with $y_p = e$ the convergence at the point of support installation:

$$\bar{u}(e) = \bar{u}(\infty) F\left(\frac{\bar{u}_E(\infty)e}{\bar{u}(\infty)a}\right) \quad (2.9)$$

Equation (2.9) offers the simplest way of estimating pre-deformation, but at the same time it leads, as pointed out by AFTES (2002), to a serious underestimation of ground pressure. In the axisymmetric analysis, the convergence confinement method underestimates the pressure by about 4 times. As equation (2.9) is based

upon the development of convergence along an unsupported opening while a stiff lining reduces not only the final convergence but the pre-deformations as well, the underestimation of pressure by the convergence-confinement method has been attributed to the overestimation of pre-deformation (AFTES, 2002). However, the plane strain ground response curve shows that the problem is more fundamental, a plane strain analysis cannot reproduce both the deformations and the pressures. In order to determine the ground pressure through a plane strain analysis, the pre-deformations have to be underestimated. A more advanced, so-called implicit method (Guo, 1995; Nguyen-Minh and Guo, 1996) attempts to resolve this problem basically by applying a reduction factor to the "unsupported" pre-deformation $\bar{u}(e)$:

$$u(e) = \Phi \left(\frac{u(\infty)}{\bar{u}(\infty)} \right) \bar{u}(e) \quad (2.10)$$

where $u(e)$ and $u(\infty)$ denote the pre-deformation and final convergence of the supported opening, respectively, while the function Φ is defined as follows:

$$\Phi(t) := 0.55 + 0.45t - 0.42(1-t)^3 \quad (2.11)$$

Equations (2.9) and (2.11) lead to the following expression for the pre-deformation:

$$u(e) = \bar{u}(\infty) F \left(\frac{\bar{u}_E(\infty)e}{\bar{u}(\infty)a} \right) \Phi \left(\frac{u(\infty)}{\bar{u}(\infty)} \right) \quad (2.12)$$

According to equation (2.12), the pre-deformation $u(e)$ depends on the final displacement $u(\infty)$ and thus on the unknown final support pressure $p(\infty)$ as well. The latter is, however, related to the deformation of the support:

$$p(\infty) = k(u(\infty) - u(e)) \quad (2.13)$$

Equations (2.12) and (2.13) form a system for the support pressure $p(\infty)$ and the pre-deformation $u(e)$. By inserting $u(e)$ from equation (2.12) into equation (2.13), we obtain a non-linear equation for the support pressure $p(\infty)$, which can easily be solved using Newton's iteration method.

An axisymmetric model that takes into account the sequence of excavation and lining installation will always lead to ground response points above the plane

strain ground response, i.e. the convergence corresponding to a certain ground pressure $p(\infty)$ will always be larger than the one obtained by a plane strain analysis. This is due to the inability of the plane strain model to map the radial stress reversal that follows the installation of the lining. In general, the stiffer the lining and the longer the unsupported span e , the larger will be the deviation from the ground response curve. The convergence-confinement method, even in combination with advanced methods of pre-deformation estimation, underestimates the ground pressure and deformation particularly for stiff linings, long unsupported spans and heavily squeezing ground with highly non-linear material behavior. The inherent weakness of any plane strain analysis is that it cannot reproduce at one and the same time both the deformations and the pressures. This is relevant from the design standpoint particularly for conditions during excavation. However, an analytical analysis in combination with an axisymmetric numerical model simulation can be used to deduce the deformation estimation. For support completion close to the face, the differences in the results obtained by the different methods of analysis may affect the effectiveness of support installation in the practical application.

2.3 THE TIME DEPENDENT DEFORMATION ESTIMATION IN UNDERGROUND ENGINEERING APPLICATION

For the design of support system of tunnel, the convergence-confinement method is a simplified analysis methodology. Obviously, the confinement loss used by the theory is an important influence factor. For analyzing the influence of the confinement loss, the deformation measurements widely used in tunneling engineering shall be investigated and the advancing effect function is particularly proposed to examine the alterations of deformation and stresses of rock mass around tunnel. The back calculation procedure is a technique proposed to assess the redistribution of the confinement loss. The case study of Baguashan tunnel in Taiwan takes account of the advancing effect during construction (Lee et. al., 2009). According to the obtained results and the comparison between numerical calculation and theoretical analysis, it is shown that the relationship between deformation measurements and confinement loss can be described by an tunnel advancing effect function.

2.3.1 The Time Dependent Factors in Deformation Analysis

Tunnel excavation produces stress changes to the ground and strain to the support lining, leading to the closure (convergence) or instability of the excavated area. Convergence recorded after section excavation is assigned to strain resulting from the progressive tunnel front advance (face advance effect) and the time-dependent properties of the geo material (ground creep effect). Based on the geodetic monitoring records of two recent road tunnels in Greece, a simple methodology to estimate the contribution of each of the two effects is presented. It is revealed that at least half of the total deformation of the examined tunnel sections is contributed by the time dependent factors, indicating that the major portion of tunnel deformation is due to the time dependent properties of the ground, a result supported by previous studies from other tunnels as well.

Convergence of tunnels during excavation is usually regarded as reaction of the ground to the stress changes due to the advance of the excavation face. This is a process that initiates in the ground before section excavation and continues around the periphery of the excavated tunnel, inducing deformation to the ground-support system along a distance up to two or three times the tunnel diameter (Panet, 1995; Hoek, 2000; Lunardi, 2000). In some cases, however, convergence continues, though at a smaller rate, after the excavation face has advanced for a much longer distance from a specific excavation face and for long after the section excavation. Such a deformation, described as ‘creep’, a time-dependent effect (Kaiser et al., 1983; Sulem et al., 1987; Kontogianni and Stiros, 2005) is superimposed to the strain induced by the front advance stress changes, and in some cases it seems to be very important. For instance, about 80% of the total post-excavation convergence of the Frejus tunnel, France, was assigned to time-dependent deformation and 20% to effects associated with deformation from the tunnel front (Sulem et al., 1987). Yet, such data are extremely limited for three reasons, (a) restrictions in publication of excavation data, (b) limitations in obtaining field data before the advent of modern geodetic monitoring techniques (total station surveys, usually known as 3-D optical monitoring) and (c) complications in the excavation process. Excavation of twin tunnels, excavation of tunnels in different phases, nearby excavations and changes in the hydrological

conditions insert many un-known parameters, modify stresses and the deformation pattern (Terzaghi, 1942; Kontogianni, 2005) and make rather impossible any study of the contribution of front-dependent and time-dependent displacements to the total tunnel closure. Recently, however, due to the boom in tunneling in Greece (Greeman, 2004), more reliable and complete geodetic records describing the convergence of numerous tunnels during their excavation became available. Analysis of such data permits to shed more light to the rather ignored role of time-dependent deformation of lined tunnels.

2.3.2 The Time Dependent Factor Analysis Methods

In modern tunnel construction, usually a few hours after a section excavation and support installation, convergence control points are established and measurements are taken. It is usually done with total station instruments and less frequently with the convergence tapes or extensometers (details of such method and instrumentations will be discussed in the next sections). These measurements are repeated frequently, usually daily for a month or more, until the points are fully stabilized. The aim of this procedure is to define displacements of control points versus time at an accuracy of a few mm or better and hence determine the changes in the shape of the tunnel sections (convergence) (Kontogianni and Stiros, 2002). For each control point, therefore, a set of observations of displacements d_i , taken after time t_i has elapsed since the first measurement, while the advancing excavation front was at distance x_i , is obtained (Figure 2.6). Apparently, $x_i > 0$ and $t_i > 0$. Observed displacement d_i corresponding to the convergence of a specific point can be expressed as the sum of displacements caused by two effects, the face advance d_{xi} and the time-dependent reaction of the ground d_{ti} :

$$d_i = d_{xi} + d_{ti} \quad (2.14)$$

The unknown contribution of each effect was modeled using the widely accepted relationships. For the front effect displacement d_{xi} the simple expression proposed by Sulem et al. (1987) is shown in equation (2.15).

$$d_{xi} = d_x^* \left[1 - \left(\frac{X}{x_i + X} \right)^2 \right] \quad (2.15)$$

where, d_x^* corresponds to the total unknown displacement due to face advance distance x at the stabilization level (theoretically for $x \rightarrow \infty$) and X is an unknown parameter to be defined, representing a characteristic length depending on the ground through which the tunnel is excavated and is given in units selected to turn the ratio $\frac{X}{x_i + X}$ into a dimensionless number. Given the uncertainties in the excavation and rock mass parameters, this simple model is satisfactory. Time-dependent displacements d_{ti} are computed assuming that the ground behaves as a viscoelastic material according to the widely accepted Kelvin–Voigt creep model proposed by Panet, (1995).

$$d_{ti} = d_t^* \left(1 - e^{\left(\frac{-t_i}{T} \right)} \right) \quad (2.16)$$

where, d_t^* corresponds to the total unknown time-dependent displacement at stabilization level (theoretically for $t \rightarrow \infty$), T is another unknown parameter to be defined, representing a characteristic time depending on the ground and is given in units selected to turn the ratio $\frac{-t_i}{T}$ into a dimensionless number and variable t_i corresponds to the time elapsed since the section excavation. Apparently, the Kelvin–Voigt creep model is an approximation and ideally more complicated rheological models, as those suggested by Sakurai (1978) should be used. However, for tunnels such as those discussed below the approximate Kelvin–Voigt model was adopted for simplicity in calculations for the following reasons, (a) face advance rate is nearly constant and hence stress changes are nearly constant. (b) the uncertainties in convergence measurements (especially the signal-to-noise ratio) are high, producing uncertainties in further calculations. In particular, standard error in good quality geodetic convergence measurements is 3–4 mm (Kontogianni and Stiros, 2002), while recorded convergence usually ranges between a few and a few tens of mm (Kontogianni, 2005) and (c) parameters in complicated models depend on ground characteristics and hence are usually variable and occasionally poorly

known along the tunnel axis, producing further uncertainties in calculations. The observed total displacement d^* after section stabilization (Figure 2.7) equals to,

$$d^* = d_x^* + d_t^* \quad (2.17)$$

Figure 2.6 – Idealized convergence diagram for a tunnel section.
(Figure adopted from Linard, 2011)

where, d_x^* and d_t^* are unknown. For each control point, n sets of measurements of (d_i, t_i, x_i) are available, leading to a system of n equations with 4 unknowns (d_x^*, d_t^*, X, T) formed on the basis of Equations (2.14), (2.15) and (2.16),

$$d_i = d_x^* \left[1 - \left(\frac{X}{x_i + X} \right)^2 \right] + d_t^* \left(1 - e^{\left(\frac{-t_i}{T} \right)} \right) \quad (2.17)$$

The solution of the above nonlinear system by the Least Squares Method (using series expansions to transform equation (2.47) into linear equations (Mikhail and Ackrmann, 1976) leading to the computation of the four unknown parameters and the ratio

$$m = \frac{d_t^*}{d_x^*} \quad (2.18)$$

can subsequently be computed.

If this process is repeated for various sections of a tunnel segment with somewhat similar characteristics (overburden thickness, lithology, support, etc.) the corresponding values of m should be of the same order of magnitude and define the ratio of time-dependent to front-dependent displacements, which is a characteristic of the specific tunnel segment.

It can therefore conclude that time-dependent effects, although sometimes ignored, are very important for tunnels and have a minimum 50% ($m \geq 1$) contribution into the total deformation. The smaller level for the ratio m , probably defines a lower limit of this ratio, for measurements covered a rather limited period (< 2 months) or alternatively, it may also reflect an increased stiffness in the support system. This result is also supported from data in tunnels in very high overburden or clay, in which deformation continues for long periods, after the excavation front has advanced for hundreds of meters (Terzaghi, 1942; Goel et al., 1995), leading to even higher values of the m ratio (ratio of time-dependent versus face-advance displacements). The important role of the time-dependent deformation in tunnel convergence may explain the deformation and failure process in several tunnels long after section excavation and far from the excavation face (Kontogianni and Stiros, 2005). It appears also to be a significant parameter for the proper design of the support system, the capacity, flexibility, installation procedure and the convergence monitoring system.

2.4 THE TRADITIONAL DISPLACEMENT MONITORING METHOD AND INSTRUMENTATIONS

2.4.1 Structural System and Component Deformation Evaluation Methods

Diagnosis of deformation in structural system requires the identification of the location and type of deformation and quantification of the degree of deformation. Displacement detection methods currently in use rely on visual inspection or on localized measurements. Measurement based methods are still very much at the experimental or research state with little if any practical deployment. Methods currently used include acoustic or ultrasonic measurements, magnetic field change measurements, radiograph, eddy current and thermal field change detection

techniques. Doubling et al. (1996) presented a comprehensive literature review of deformation identification and health monitoring methods based on strain measurements and detection based on changes in strain characteristics. Rytter (1993) defined four stages of displacement monitoring:

- (a) Determination that displacement is present in structure;
- (b) Determination of the geometric location of the displacement;
- (c) Quantification of the degree of displacement;
- (d) Predication of the future displacement of the structure.

These stages of displacement monitoring may lead to different types of sensor requirements or multi-sensor systems. Similarly, displacement algorithms will vary depending on the type of monitoring desired. For example, if stage one information is needed, then sparse vibration measurements may be sufficient to ascertain the existence of displacement somewhere in structure. Identification of the location of displacement may require considerably richer sensor network and perhaps sensors that provide more robust local information. In order to determine the degree of displacement in addition to the sensor selection, efficient and robust displacement algorithms are needed. Predication of remaining service life is typically based on fatigue and fracture measurements and may require different sensors and mathematical tools leading to the estimation of remaining life or assessment of compliance with design specification. The challenge is in developing systems that can respond to all four stages of damage monitoring.

A robust, efficient, and economical displacement detection system critically depends on the information extracted from the sensors. Due to economic constraints, it is impossible to completely instrument civil structures for damage monitoring. Therefore, algorithms and methodologies that address the issue of limited instrumentation and effect on resolution and accuracy in damage diagnosis are paramount to the application of a displacement monitoring systems. There are numerous studies regarding optimum sensor placement. Most methods are based on maximizing the trace or determinant (or minimizing the condition number) of the Fisher information matrix which is expressed as a function of selected parameters corresponding to objective function. One example is the Optimum Sensor Location

(OSL) algorithm proposed by Udwarda (1994) that minimizes the covariance error between the structural parameters that are to be identified and their estimate from the limited measurements.

2.4.2 The Convergence Measurement Method

Convergence measurements of the excavation have developed into the most frequently applied methods of measurement in tunneling. Leveling measurements of the tunnel roof or other points of the tunnel can reveal the convergence occurred. To install a convergence measuring section, convergence bolts are positioned as close as possible behind the round in the tunnel reveal (set in concrete or welded on arches). At the tunnel end of the convergence bolts is a thread with a stop for attaching the measuring device (a steel measuring tape or invar wire). A spring pretensions the measuring tape with the convergence gauge, which is also fastened to a convergence bolt on the other side. The change in length between the reference points is read off a mechanical dial gauge on the convergence gauge. To minimize disruption of the construction work, geodetic measurements have become increasingly popular of late as a means of monitoring convergence. Instead of using convergence bolts, a measuring bolt with luminous diode or reflecting signal is set in concrete and its displacement measured with a theodolite. Measuring accuracies of ± 1 mm are possible and satisfy the stability inspection standards for tunneling projects. The advantage of this method compared with taking relative measurements between two moving points using a convergence gauge is that it measures the absolute displacements in an excavation. When convergence measurements are taken with a convergence gauge, this is only possible in combination with at least one geodetic measurement.

They are normally taken in standard or main measuring sections. Figure 2.7 shows a typical measurement result in schematic form.

Figure 2.7 – Example of a convergence measurement. F: Subsequent anchorage of abutments; G1: Foot anchors; G2: Doubling of foot anchors; H: Bottom end.
(Figure adopted from Fecker, 2004)

Determination of exact location of pins in space is required for most back analysis methods. This can theoretically be done by surveying tools. Problems associated with this approach are limited accuracy of the typical tools, existence of dust and limited access to the points in the roof. The main aim is to make use of the accurate measurements that usually take place for stability purposes by tapes and to eliminate the need for using surveying tools for locating the pins. The main problem in obtaining absolute displacement of pins in convergence operations is due to the fact that the number of unknowns is more than the number of equations in analytical approach.

2.4.3 Monitoring of Underground Tunnels and Caverns

Tunnels and caverns are supposed to be monitored during their whole operational lifespan. The choice of monitoring instrumentation, in addition to the pace (frequency) of monitoring, depends on the stage of the tunnel, the rock

mechanical quality, the tunnel type, the purpose of the tunnel and the tunneling method during construction. The most indicative parameter to monitor in the case of tunnels is convergence of the cross section as suggested by Glisie and Inaudi (2007). Deformation monitoring can be performed independently of convergence monitoring and concentrated only on some particular points in the cross-section, i.e. at the points where maximum stresses are expected.

In the past, traditional monitoring methods for structural monitoring of underground tunnels such as visual inspection are used. However, due to its high cost and long periods of time needed, a better solution – using of optical fiber sensors is evolved in the later stage. The usage of visual inspection is highly ineffective as obtaining an accurate result is tricky. Some of the minor damages are not clearly visible to the human eyes and one could easily disregard that. Furthermore, the harsh environment of the underground tunnels and caverns creates a challenge to the sensitivity and precision of the traditional structural monitoring methods and thus not able to provide a mean for incessant large scale monitoring of the structures.

During the discussion of the benefits of using structural monitoring instruments, Ding and Qing (2001) commented that those highly accurate instruments can be used to determine many kinds of valuable parameters such as stress, strain, deformation, load and displacement. Furthermore, these instruments will allow the monitoring of tight areas or places that are inaccessible to man.

2.4.4 Structural Monitoring Instruments In Practice

Kenchington (1991) remarked that modern days structural monitoring instruments normally consist of sensors and transducers fusing together and are capable of providing an analogue of the parameter or phenomenon to be monitored and recorded. There is a range of sensors and transducers available to the structural monitoring system (Table 2.1).

However, those traditional instrumentations only allow single point measurement in each installation. There is no alternative to achieve spatial

information of the structure unless thousands of such instruments are deployed in a form of monitoring network, which is very costly and time consuming. Moreover, some of the traditional instrumentations are operated manually; such low level of automation limits their application in underground engineering. The current BOTDR FOS SHM system developed in this thesis is targeting to solve the above problems. The BOTDR FOS allows thousands of measurement points for measurement acquisition within a single FOS. The advance FOS system also allows remote sensing; this automation advantage increases the application possibility of the proposed BOTDR FOS SHM in underground engineering.

Table 2.1 – List of common structural monitoring instruments

Instrument Type	Purpose
Automated Total Stations (Figure 2.8)	Used together with prisms in surveying the movements of the surrounding building movement
Optical Prism	Used to measure movement of buildings from their initial position in XYZ directions
Crackmeter (Figure 2.9)	Measures the widening or closing of cracks on buildings
Inclinometer in wall (Figure 2.10)	To monitor wall displacement during excavation works. (Diaphragm Wall)
Load Cell	Calibrated structural elements which may be used to monitor tensile and compressive loads
Moisture meters	To monitor the moisture content of solid material (concrete) of a building.
Settlement Marker on Structure	Used to measure settlement of adjacent buildings, utilities during construction period
Strain Gauge (Figure 2.11)	To monitor the local changes in strain and also able to infer values for load changes in structural elements.
Tilt-meter (Figure 2.12)	Tilt plates are installed to monitor changes in tilt of the structures that are close to any construction site

Some pictures of these common structural monitoring instruments are shown in Figure 2.8 - 2.12.

Figure 2.8 – Total station
(Figure from Internet)

Figure 2.9 – Crackmeter
(Figure from Internet)

Figure 2.10 – Inclinometer
(Figure from Internet)

Figure 2.11– Strain gauge
(Figure from Internet)

Figure 2.12– Tilt-meter
(Figure from Internet)

2.5 THE STRUCTURAL HEALTH MONITORING (SHM) SYSTEM

2.5.1 History of Structural Health Monitoring (SHM)

SHM is an expression that has been in existence for less than two decades. It has emerged from the field of ‘smart structures’, a field which embodies the idea of making sensing and actuation devices an integral part of a structure or of structural

material itself. The development of smart structures and SHM concepts in the civil engineering field has become more and more attractive in the past decade and has received growing attention worldwide in academic and applied research. The basic idea has been derived from applications performed in the aeronautical, aerospace and automotive industries, but the migration to the civil construction industry has definitely required and still requires, the development of domain specific in technologies and knowhow for the fabrication of sensors, monitoring systems design, data collection and data fusion, analysis and interpretation of the measurements and decision making.

The introduction of fiber-optic based sensor systems and related interpretation techniques has contributed to a very significant extent to cover the gap between the above pioneering concepts and practice, thus making possible the realization of extremely reliable monitoring systems that are able to keep under control the behavioral conditions of real structures in all phases of their existence, from construction to maintenance interventions and practically for their entire operational life.

However it is observed that, despite these developments, only a limited, although continuously growing, number of practical applications can be reported to date. Two main reasons can be individuated for such finding. The first reason is that, although observational methods have been the basis for many engineering disciplines, modern SHM techniques are yet not part of the standard educational programmes of structural engineers and, therefore, they are not well known among most professionals. The second reason is that the cost efficiency of the SHM systems in building and infrastructure management can only be demonstrated in the medium to long term.

2.5.2 Traditional SHM System

Based on Moore (1992) and Ping (1992), research on the general traditional SHM techniques applied in , past monitoring included a wide range of techniques which rely primarily on visual observation with instrumentation to obtain a measurement of length or determination of position from which movement may be

deduced. The characteristics of the traditional SHM system includes the requirement of an observer on site at the time of measurement and the moving of point to point for measurement taking during inspection. Furthermore, a traditional manual approach may only permit professional judgment to be made, with attendant indemnity considerations.

From the economic point of view, traditional techniques are necessarily labor intensive and are, as a result of economic constraints, frequently incapable of producing a readily interpreted body of data. In difficult cases, building owners, operators, and managers are not likely to be happy to meet the costs of engaging skilled personnel for extended periods even when they can accept the intrusion that the use of traditional techniques necessarily involved.

2.5.3 Moving Forward into the New Era of SHM

With such, Giasco & Stefano (2006) believe that the continuous development of the industry can only be achieved through the fusion with advanced and leading technologies such as Information Technology (IT) and Networking Technology (NT). In their paper, they constantly highlighted for a newer approach and prospective to improve the existing methods of civil engineering structures monitoring.

During the discussion of the benefits of the automatic and autonomous monitoring systems, Dunnicliff (1993) highlighted that automatic and autonomous monitoring systems are based on currently-available and emerging data-acquisition technologies; it has helped to eliminate human error from the measurement system. It is now readily practicable to gather data and handle it wholly automatically in order through to the point at which it must be interpreted.

2.5.4 Present SHM System

At present, SHM is an emerging technology leading to the development of systems capable of continuously monitoring structures for damage to improve

safety and reduce life-cycle costs. SHM essentially involves integrating of one or more nondestructive test (NDT) method in order to facilitate quick and accurate damage detection with minimal human intervention. There are several advantages to using a SHM system over traditional inspection cycles, including reduced downtime, elimination of component teardown, and the potential prevention of failure during operation.

According to government data, annual infrastructure costs amount to between 8-15% of the GDP for most industrialized nations. The ability to continuously monitor the integrity of civil infrastructure in real time offers the opportunity to reduce maintenance and inspection costs, while providing increased safety to the public. Manual inspection of structures in the past costs a lot, is relatively unreliable, and can only be carried out infrequently. Some real world deployments show that dense arrays of sensors can provide detailed information about the state of civil infrastructure.

In present days SHM system is a result of the fusion between traditional monitoring techniques and the advanced and leading technologies such as IT and NT. Good evidence of the advancement of SHM system can be seen through the use of automatic and autonomous monitoring, virtual reality and photogrammetry. Kenchington (1991) identified an automated monitoring system consisting of a number of sensors and transducers combined with suitable signal conditioning, and connected through an interconnecting medium to data-acquisition and recording equipment. The sensors and transducers convert the parameters of interest into an electrical analogue of that parameter. Signal conditioning provides the means by which the analogue signal is relayed to the monitoring location. Further signal conditioning and processing may be required to convert the analogue signal obtained at the monitoring location into a form acceptable to the data acquisition equipment. The data-acquisition equipment records and displays inputs from the sensors and transducers in the form of an analogue trace on a chart records (or any other analogue device) or as data recorded digitally.

The system provides a vital link between monitored structures and a central monitoring site. This allows many structures to be monitored at a central site, with

information transmitted via the Internet, thereby eliminating costly permanent site installation and reducing the number of site visits. SHM also provides a structured approach to assessing the performance of various technologies and reporting on successes based on monitoring a number of field projects over extended periods of time. It should also be capable of life-long network operation and provide low latency simultaneously at high scalability. In addition to field projects, numerical modeling and identification of damage using the finite element method and, alternatively, damage detection algorithms, are also being developed. Acceptance of intelligent sensing and SHM as an essential part of infrastructure design, at a fraction of the capital cost of construction, will be key to developing 21st century “high-tech” civil engineering structures.

2.5.5 The Long Term SHM for Underground Construction

There are only very few and limited literature and report of the long term SHM for underground infrastructure, one of the example is the LRT rock tunnel in Texas using ground penetrating radar by Bourland & Yuan, (2006). The tunnel was mined in Austin Chalk (soft rock). A polypropylene-fiber-reinforced shotcrete liner was pneumatically placed against the rock. In order to perform structural diagnostics and health monitoring on the tunnel liner, a mapping procedure for the liner subsurface with ground penetrating radar was selected using non-contact antennas. In conclusion, it was determined that there are many factors involved that limit the use and application of ground penetrating radar in LRT tunnels.

However, there are many literatures and reviews for the advantages of SHM system for long term structure feasibility. Brownjohn, (2004), reported that SHM is a term increasingly used in the last decade to describe a range of systems implemented on full-scale civil infrastructures and whose purposes are to assist and inform operators about continued ‘fitness for purpose’ of structures under gradual or sudden changes to their state, to learn about either or both of the load and response mechanisms. Arguably, various forms of SHM have been employed in civil infrastructure for at least half a century, but it is only in the last decade or two that computer-based systems are being designed for the purpose of assisting

owners/operators of ageing infrastructure with timely information for their continued safe and economic operation.

On top of the above literatures reviewing the feasibility of long term SHM in underground applications, many research cases demonstrated the need and development of such SHM system. Guo (2008) mentioned that SHM is a process aimed at providing accurate and in time information concerning structural condition and performance. It was illustrated with on site examples of monitoring of convergence of tunnels during and after construction. The Luzzone dam inspection gallery, the work which consists of four zones was excavated by drilling and blasting. In this operation, two point extensometers, multipoint extensometers and surface sensors were used convergence monitoring. Convergence measured after the excavation of zone 3 was presented and it showed +3mm to -3mm displacement of the profile of cavern section.

Chan and Stone (2005) performed geotechnical monitoring of Macquarie Park Station (Australia) using their proposed SHM system. The cavern excavation monitoring system included inclinometers, extensometers, surface settlement points, endoscopes and convergence points. One of the sections was chosen for back analysis (Figure 2.13). The back analysis based on instrumentation indicated that the original models adopted for design were reasonable. Two ‘admissible’ combinations of slightly revised geotechnical parameters were identified. Models for subsequent design of the other stations (in the vicinity) were adjusted to reflect the calibrated parameters.

Figure 2.13 – Section for back analysis
(Figure adopted from Chan and Stone, 2005)

Bertuzzi and Anderson (2005) commended that monitoring of displacement and stress around a tunnel under construction is part of tunnel design. However, the type and detail of “appropriate” monitoring is often debated. Their experience with monitoring instruments in the two road tunnels excavated in Hawkesbury Sandstone, Sydney, Australia was compared with the other published experience. They highlighted the instruments used and their performance at different projects in Sydney such as M5 East tunnels, Eastern Distributor Tunnel, Sydney Opera House Parking Station Cavern. At the same time, they suggested and recommended some of the instruments based on order of importance.

In Europe, Bock (2001) summarized the European practice in tunnel monitoring as ‘instrumentation should be installed to provide a sufficiently broad database for comparisons to be made between measurements and predictions’. He also enlisted some of the instruments for monitoring. Where else, in Asia, Jia and Yang (2008) based on the actual geological and construction condition of Wujiazhuang Tunnel, proposed on the design and construction of large-span multi arch tunnels monitoring system. Symmetrical field monitoring was conducted to investigate the performance of surrounding rock and tunnel structure. Detailed monitoring including the stresses of the shotcrete, the contact pressures between preliminary lining and secondary lining, the internal forces of steel arch, middle wall and secondary lining during construction process were obtained. Finally, the

safety of the secondary lining was evaluated based on the monitored results and field observations.

The official report of the International Tunneling Association (ITA) deals with the adverse effects of water on tunnel structures during their working life (i.e., following completion of construction). The report describes problems and damage caused by water, and discusses various repair methods applicable to different types of building materials and structures. It discusses issues for consideration in designing, construction and operating tunnels, and identifies issues for further consideration. The report includes 48 case histories of tunnels, submitted from 12 countries. This report is, indeed, important since it considered external effects, structural effects and functional effects due to damaging along with the remedial measures which provide strong references and considerations needed for developing a real-time long term SHM system for underground applications.

Verma et. al. (2008) reported that the first Indian cavern for the underground storage of liquefied petroleum gas (LPG) was constructed in 2007. This storage complex (Figure 2.14 and Figure 2.15) is aimed at storing LPG and expansion of this project will see storage of other gas. The research group highlighted the methodologies adopted with a list of equipment, and it framed out the general access arrangement made to cavern considering the nature of complexity and access limitations during construction of third phase.

Figure 2.14 – Isometric view of cavern project
(Figure adopted from Verma, et. al., 2008)

Figure 2.15 – a) Water curtain gallery b) Finished mined cavern
(Figure adopted from Verma, et. al., 2008)

2.6 THE FIBER OPTIC SENSOR (FOS)

2.6.1 Fiber Optic Sensor (FOS) and Technology

Conventional sensors based on electrical and/or mechanical transducers are being replaced by the up and coming fiber optic technology. Although FOSs are apparently more costly for extensive use in health monitoring, they are, nevertheless, better approaches for application where reliability in challenging environments is

crucial. It is sometimes essential to use a reliable technology for critical health monitoring.

Glisie and Inaudi (2007) commented that the greatest advantages of the fiber optic technology are intrinsically linked to the optical fiber, which is either simply a link between the sensor and the signal conditioner, or is the sensor itself in long-gauge and distributed sensors.

Also, FOSs offer a huge range of parameters that can be measured, so that several parameters can be mixed on the same network. This enables us to measure parameters such as strain, temperature and in turn, deformation with regards to most of the underground applications.

2.6.2 Fiber Optic Sensor (FOS) Available for Civil Engineering Application

Table 2.2 lists the FOSs available to civil engineering applications and their categories. One method of classifying FOSs is based on their light characteristic (intensity, wavelength, phase, or polarization) that is affected by the parameter to be sensed. Another method classifies an FOS by whether the light in the sensing segment is modified inside or outside the fiber (intrinsic or extrinsic). FOSs can also be classified as local (or point), quasi-distributed and distributed sensors depending on the sensing range.

Table 2.2 – FOSs available to civil engineering applications and their categories

Type	Sensors	Mesurands	Linear response	Resolution	Range	Modulation method	Intrinsic/ extrinsic
Local	Fabry–Perot	Strain ^a	Y	0.01% gage length ^c	10,000 $\mu\epsilon$	Phase	Both
	Long gage sensor	Displacement	Y	0.2% gage length ^d	50 m	Phase	Intrinsic
Quasi distributed	Fiber Bragg grating	Strain ^b	Y	1 μ strain	5000 $\mu\epsilon$	Wavelength	Intrinsic
Distributed	Raman/Reyleigh (OTDR)	Temperature/strain	N	2 μ strain	2000 m ^e	Intensity	Intrinsic
	Brillouin (BOTDR)	Temperature/strain	N	2 μ strain	2000 m ^f	Intensity	Intrinsic

^a Can be configured to measure displacement, pressure, temperature

^b Can be configured to measure displacement, pressure, relative fissures and inclination, etc

^c Resolution as high as 0.1 μ strain

^d Resolution as high as 0.2 μ strain

^e Up to 25 km with spatial resolution of 5m

^f Up to 50 km with spatial resolution of 1m

2.6.2.1 Distributed Fiber Optic Sensors

Distributed sensors are most suitable for large structural applications, since all the segments of an optical fiber act as sensors, and therefore, the perturbations within various segments of the structure can be sensed. Distributed sensors are based on the modulation of light intensity in the fiber. Fracture losses or local damages in a structure cause light intensity variations. Two major distributed sensor methodologies are the optical time domain reflectometry (OTDR) and the Brillouin scattering. In the OTDR, Rayleigh and Fresnel scatterings are used for sensing structural perturbations. On the other hand, Brillouin scattering detects the Doppler shift in light frequency which is related to the measurements.

Fiber optic distributed strain and temperature sensors measure strain and temperature over long distances and are excellent tool for monitoring health of large structure. These sensors leverage the huge economics of scale in optical telecommunications to provide high resolution long-range monitoring a cost per kilometer that cannot be matched with any other technology. Today's distributed strain and temperature sensors offer clear cost and technical advantages in applications such as tunnel monitoring, pipeline monitoring, bridge monitoring, dam monitoring, power line monitoring and border security/perimeter monitoring etc.

The vulnerability of the FOS is difficult to protect the fiber from concrete aggregate in the pour duration. It means that the FOS could easily be damaged and corroded during the long term practical application. Extrinsic Fabry-Perot interferometric (EFPI) was studied and sensor protection system was suggested for embedded as well as surface mountable sensors (Leong et. al., 2005). There are few EFPI sensors used by Habel et al. (1997); Quirion and Ballivy (2000); and Wanku et al. (1999) to study the possible use of the EFPI sensors as embedded sensors by using special protection to the sensors. Experiments conducted showed that properly installed and protected fiber-optic sensors not only can survive the harsh environment involved in the embedment process, but can also yield accurate quantitative strain information

from structures (Quirion and Ballivy, 2000; De Vries et al., 1995; Masri et al., 1994; and Vries et al., 1994). In one or different way these FOSs are required to be protected to get maximum output from them.

Distributed sensors were devised to monitor big and long structures where EFPI and FBG sensors fail to give decent viability. Brillouin Optical Time Domain Reflectometer (BOTDR) based distributed FOSs are generally used for monitoring purpose. Komatsu et al. (2002) discussed the measurement principle of the strain measurement method using optical fibers (BOTDR method). Case studies of application to the deformation of telecommunications tunnels, ground subject to landslides and so on, and also an outline of an automatic measuring system were presented. Li et al. (2004) reviewed fiber optical sensor for health monitoring in various key civil structures, including buildings, piles, bridges, pipelines, tunnels, and dams. Ohno et al. (2001) introduced recent examples where BOTDRs have been applied to actual structures. Results of three field tests, namely, as a damage-detection system for America's Cup yachts, as an optical fiber sensor for detecting changes in river levees, and as a strain-sensing optical fiber embedded in concrete structures were also discussed. Zhang (2010) built the dike model, monitored the dike strain changes influenced by seepage and loading, and verified the feasibility that the distributed optical fiber measurement system can basically reflect the dike deformation. It can be an important direction for the development of dike strain monitoring. Recent advances in strain measurements using optical fibers allow the development of smart underground security fences that could detect the excavation of smuggling tunnels. Klar et al. (2010) presented the first stages in the development of such a fence using BOTDR. Two fiber optic layouts were considered and evaluated in a feasibility study that included evaluation of false detection and sensitivity.

2.6.3 Selection of Monitoring System

Depending on the specifications for monitoring, some parameters can be selected and the corresponding monitoring system can be devised. The specifications

may include aim of monitoring, accuracy, frequency of reading, compatibility with the environment (sensitivity to temperature, humidity, etc), installation procedures for different components of the monitoring system, remote monitoring, data management and analysis, monitoring level i.e. component of structure or entire structure, etc.

It is obvious that the sensors should be placed in the representative positions on the structure so as to obtain maximum and necessary data from the monitoring system. An array of sensors may be used to monitor the structure. The array or network of sensors mainly depends on the monitoring aims and chosen parameters. Careful installation of the monitoring system is very important as it is a delicate phase. The components of the monitoring system can be embedded or installed on the structure's surface using fastenings, clamps or gluing. The installation may be time consuming and it may delay construction work. When installed, the monitoring system has to be protected (Branko and Inaudi, 2007).

2.6.4 Fiber Optic Deformation Sensors for Underground Engineering

Deformation can be measured by the amount of strain produced by certain load. Structural strain is present in loaded structures and directly reflects the structural condition. Hence, in underground engineering practice, it is especially vital to evaluate the strain components of a structure being observed. Elastic strain (reversible) and plastic strain, which is irreversible may occur in structures. Also, temperature variations in a structure's environment may create temperature variations in the structure itself.

On the other hand, creep, is a time dependent deformation behavior of a structurally strained (loaded) material. It implies that shrinkage or elongation occurs in structure is a slow and time dependent dimensional change. Creep develops gradually and its ultimate value is usually achieved only after some years. All these types of strain shall be monitored and analyzed with the development of the long term fiber optic monitoring equipment.

To address the long term system sustainability issue, the type of optic fiber cable used in the system is of principal importance as it should be able to withstand the drilling and blasting of tunneling method, and also the chemical resistance to water and crude oil. The method of installation of the cables is imperative as well, in order to provide us with the most accurate results and analysis.

The distributed sensors can be installed on the walls and vaults of the tunnel in longitudinal and tangential direction. These sensors can provide crack detection and locations for local average strain changes as a result of damage detection. Taking into consideration the number of monitoring parameters and tunnel dimensions, distributed sensing techniques suggest convenient, cost-effective and versatile instrumentation.

2.6.5 Shortcomings and Problems of Fiber Optic Sensing

The fiber optic technology has experienced a phenomenal rate of progress in the second half of the twentieth century. Although the technology of the fiber optic brings about many advantages, there are still certain shortcomings and problems faced. Some of the most common challenges are listed as follow,

a) High cost

The installation cost of the technology is comparatively high and special test equipment is often necessary. The test equipment typically and traditionally used for conventional electron-based networking is of no use in a fiber optic network. Equipment such as a BOTDR analyzer is required, and costly. Specialized optical test equipment such as optical probes is needed at most fiber endpoints and connection nexuses so as to accurately provide testing of the optical fiber.

b) Receptiveness to physical damage

The optic fiber is a small and compact cable, and it is highly susceptible to be damaged during installation or construction activities. Hence, the method of installation of the optic fiber to the structure is of utmost importance. Moreover, the fiber is easily affected by chemicals as the glass can be damaged by various chemicals such as hydrogen gas.

c) Requires special skills

The optical fiber cannot be joined together as easily as copper cable. It needs additional training of personnel and expensive precision splicing and measurement equipment.

To address the above common issues with fiber optic sensing, in depth investigation will be conducted and solutions will be suggested in Chapter 6. As such, although there are a few disadvantages to use the optic fiber technology, its advantages far surpass the shortcomings and further research are being done now and in the future to explore the unlimited possibilities of the brilliant technology.

2.7 THE SPATIAL FIBER OPTIC SENSOR – BRILOUIN OPTICAL TIME DOMAIN REFLECTOMETRY (BOTDR)

2.7.1 Distributed Brillouin Optical Time Domain Reflectometry (BOTDR)

Distributed sensors are designed so that the information is discriminated in the spatial mode, and then the measurand can be locally identified along the length of the fiber itself. The localization of the measurements is possible through a “modified radar concept” using optical pulses launched in the sensing fiber.

As long as the pulses propagate throughout the fiber, the effect of the measurand on the optical pulses within the fiber is recorded as a function of time.

Therefore, thanks to the distributed sensing concept, one single fiber could therefore replace thousand of point sensors. The speed of light within an optical fiber being known, the time delay between the pulse launch and the detection of the effect of the measurand provides direct information about the location of the interaction. As results, the further the interaction takes place in the sensing fiber, the longer will be the time delay between the pulse launch and the detection of the interaction.

This technology development begins with the introduction of the optical time-domain reflectometers (OTDR). In this procedure, an optic pulse is launched in the fiber and a photodetector measures the amount of light that is backscattered as the pulse propagates down the fiber. The time information is then transformed to distance information as the speed of light is known.

The Raman scattering is first used for sensing applications in the 1980s, while Brillouin-scattering is introduced at a later date as a way to improve the range of OTDRs and then for temperature and/or strain monitoring applications. Since, this research focuses on the BOTDR technique, the OTDR and Raman scattering technique will not be further elaborated.

Brillouin scattering (Karashima et. al., 1990) occurs because of an interaction between the propagating optical signal and thermally excited acoustic waves in the gigahertz range present in silica fiber, giving rise to frequency-shifted components (The functional process of BOTDR is described in Figure 2.16).

Figure 2.16 – Schematic diagram of distributed BOTDR temperature and strain sensing system

2.7.2 Fiber Optic Strain and Temperature Measurement

Strain is the geometrical measure of deformation resulting from stresses and representing the relative displacement between particles in the silica material. It defines the amount of stretch or compression along the fiber.

The Cauchy strain or engineering strain is expressed as the ratio of total deformation to the initial dimension of the fiber in which stress is being applied. The engineering normal strain or engineering extensional strain ϵ of a fiber axially loaded is expressed as the change in length ΔL per unit of the original length L of the fiber (equation (2.19)). The normal strain is positive if the fiber is stretched or negative if they are compressed.

$$\text{Fiber strain: } \epsilon = \Delta L / L \quad (2.19)$$

However, the accuracy of strain measurement can be varied by the fiber temperature which influences the fiber density and the speed of leaser passes through it.

2.7.3 BOTDR Principle and Method of Measurement

2.7.3.1 Brillouin Frequency Shift

Brillouin scattering gives rise to frequency shifted components and the frequency shift ν_B experienced by the Brillouin scattered light is proportional to the acoustic velocity V_a of the scattering medium: Brillouin frequency shift:

$$\nu_B = \frac{2nV_a}{\lambda_0} \quad (2.20)$$

where n is the index of refraction of the fiber and λ_0 the wavelength of the pump. The acoustic velocity V_a is directly related to the material density, which is temperature and strain dependent. As a result, the Brillouin frequency shift intrinsically depends on the local temperature and strain. The Brillouin frequency shift therefore is an intrinsic parameter of the silica fiber and its measurement provides information about the local fiber temperature and strain conditions. Typical temperature and strain dependencies are shown in the Figure 2.17.

Figure 2.17 – Typical calibration data of a SMF optical fiber, showing the Brillouin shift temperature and strain dependence at a 1550nm wavelength.
(Figure from Omnisens, 2008)

2.7.3.2 Stimulation of Brillouin Scattering effect

The Brillouin scattering process has the particularity that it can be stimulated by a second optical signal in addition to the light wave that generated the scattering, providing that this second signal, called the probe fulfils specific conditions. This is an especially interesting property for sensing application and can be achieved by the use of a probe beam counter propagating with respect to the pump. Stimulation is maximized when pump and probe frequencies (or wavelengths) are exactly separated by the Brillouin shift. In this case, the energy transferred from the pump to the probe shows a greatly enhanced backscattered intensity and thus a larger Signal-to-Noise Ratio (SNR). This is seen as a resonant phenomenon where an amplification of the probe power occurs at the expense of the pump when the resonant condition is fulfilled, i.e. when the frequency difference between pump and probe matches the local Brillouin frequency.

The amplification is proportional to the pump and the probe intensities as well as to the Brillouin gain coefficient. A careful adjustment of the pump/probe levels, and thus of the amplification along the fiber, allows SNR maximization for different optical fibers, measurement conditions, distance range and presence of losses.

2.7.3.3 Frequency Domain Analysis

The temperature/strain information is coded in the Brillouin frequency shift. Scanning the probe frequency with respect to the pump while monitoring the intensity of the backscattered signal allows one to find the Brillouin gain peak, and thus the corresponding Brillouin shift, from which the temperature or the strain can be computed.

This is achieved by using a single laser from which both the *pump* beam (a few nanoseconds long optical pulse) and the *probe* beam (Continuous Wave - CW light) are

created. A telecommunication like optical modulator is used to scan the probe frequency in a controlled manner (Figure 2.18).

Figure 2.18 – Brillouin frequency shift

2.7.3.4 Time domain Analysis

Due to the pulsed nature of the pump, the pump/probe interaction takes place at different location along the fiber at different times. For any given location, the portion of probe signal which interacted with the pump arrives on the detector after a time delay equal to twice the travelling time from the fiber input to the specified location (Figure 2.19).

Thus, monitoring the backscattered intensity with respect to time, while knowing the speed of light in the fiber, provides information on the position where the scattering took place.

The recording of the probe intensity as a function of time at a given frequency is referred to as Frequency Trace or Trace.

Figure 2.19 – Time domain analysis

2.7.4 BOTDR Measurement Process

Combining the time domain analysis with the frequency domain analysis provides a 3D Matrix containing a Brillouin gain spectrum (frequency response) at every location along the fiber. The Brillouin frequency corresponding to the centre frequency of the frequency response or Brillouin gain spectrum can be computed at each position by determining the centre frequency. Measurement process is as shown in Figure 2.20.

Figure 2.20 – Measurement process
(Figure from Omnisens, 2008)

2.7.4.1 Strain and Temperature Measurement

A modification of the local fiber strain or temperature has a direct effect on the Brillouin gain spectrum as the Brillouin frequency is shifted by a magnitude which is directly proportional to the temperature or strain change. The time domain interrogation allows the localization of the change and the frequency processing determines the magnitude of the change (Omnisens, 2008). The diffraction of light on a dynamic grating generated by an acoustic wave (an acoustic wave is actually a pressure wave that introduced a modulation of the index of refraction through the elasto-optic effect). We can obtain the Brillouin gain through the Lorentzian spectral profile. Heiman et. al. (1979) expressed the exponential decay of the acoustic waves results in a gain, $g_B(\nu)$ given,

$$g_B(\nu) = g_o \frac{(\Delta\nu_{B/2})^2}{(\nu - \nu_B)^2 + (\Delta\nu_{B/2})^2} \quad (2.21)$$

which $\Delta\nu_B$ is the full-width at half maximum (FWHM) and g_o , the Brillouin gain coefficient.

The acoustic velocity is directly related to the density of the medium that is temperature and strain dependent. Since the grating propagates at the acoustic velocity in the fiber, the diffracted light experiences a Doppler shift (Figure 2.21), as a result, the named Brillouin frequency shift carries the information about the local temperature and strain of the fiber. The Brillouin shift, ν_B , which undergoes a Doppler frequency shift is depends on the acoustic velocity and is given by,

$$\nu_B = \frac{2nV_a}{\lambda} \quad (2.22)$$

where V_a is the acoustic velocity within the fiber, n is the refractive index and λ , the vacuum wavelength of the incident light wave

Figure 2.21 – Time and frequency analysis of BOTDR doppler shift.

As the Brillouin frequency shift depends on both the strain and temperature of the fiber, the sensor setup will conclude the real sensitivity of the system. Measuring distributed strains requires a specially designed sensor. A mechanical coupling between the sensor and the host structure along the whole length of the fiber has to be assured. Furthermore, knowing the frequency shift of the unstrained fiber allows an absolute strain measurement. The active stimulation of Brillouin scattering can be achieved by using two optical light waves. In addition to the optical pulse, usually called the pump, a continuous wave (CW) optical signal, the so-called probe signal, is used to probe the Brillouin frequency profile of the fiber (Nikles et al., 1994 and 1997). A simulation of the Brillouin scattering process occurs when the frequency difference (or wavelength separation) of the pulse and the CW signal corresponds to the Brillouin shift (resonance condition) and provided that both optical signals are counter-propagating in the fiber. The interaction leads to a larger scattering efficiency, resulting in a mapping of the Brillouin shift along the sensing fiber. At every location, the maximum of the Brillouin gain is computed and the information transformed to temperature or strain using the

appropriate calibration coefficients. Changes in Brillouin shift are linear with strain and temperature changes as expressed in equation (2.23),

$$\Delta \nu_B = c_\varepsilon \Delta \varepsilon_f + c_T \Delta T \quad (2.23)$$

where, ε_f is the longitudinal strain in the fiber and T is the temperature. A set of equations can be written to relate the behavior of two closely placed cables, which experience the same temperature and strain changes. A solution for the strain and temperature can be obtained as:

$$\begin{aligned} \begin{Bmatrix} \Delta \varepsilon_f \\ \Delta T \end{Bmatrix} &= \frac{1}{\beta - \alpha} \begin{bmatrix} \frac{\beta}{c_\varepsilon^{SF}} & -\frac{1}{c_\varepsilon^{SF}} \\ -\frac{\alpha}{c_T^{SF}} & \frac{1}{c_T^{SF}} \end{bmatrix} \begin{Bmatrix} \Delta \nu_B^{SF} \\ \Delta \nu_B^{LF} \end{Bmatrix} \\ \alpha &= \frac{c_\varepsilon^{LF}}{c_\varepsilon^{SF}}; \beta = \frac{c_T^{LF}}{c_T^{SF}} \end{aligned} \quad (2.24)$$

where, $\Delta \nu_B^{LF}$ is the change of Brillouin shift recorded in the lubricated cable, and $\Delta \nu_B^{SF}$ is the change of Brillouin shift recorded in the “strain sensor” fiber. c_ε^{SF} , c_T^{SF} are the coefficients of the “strain sensor” and c_ε^{LF} , c_T^{LF} are the coefficients of the lubricated cable.

Although, the telecommunication cables which their inner glass core is protected from the outer shell by a lubricant or gel are not suitable for strain measurements, they are ideal for temperature compensation because the inner fiber is not strained even when the outer coating is strained. Therefore, the above coefficients can be easily determined from a calibration setup where fibers are heated and strained and by solving equation (2.24), the strain measurement can be determined.

2.7.5 Distributed BOTDR Sensing Cables

In numerous cases, the setting up of sensors in the structure constitutes several challenges that shall not be underestimated. Many trials and errors are often essential before a consistent and efficient procedure can be recognized. It is advantageous that the installation technique is incorporated from the beginning in the design process and early trials shall be carried out in real circumstances before the whole sensing system and monitoring scheme is rolled out.

It is suggested that when strain sensing is required, the optical fiber must be bonded to the host material over the whole length. To allow such a bond, it has been recommended to integrate the fiber optic sensor within a tape in a manner similar to the way reinforcing fibers are integrated in composite materials. To produce such a tape, a glass-fiber-reinforced thermoplastic with a poly (phenylene sulfide) matrix was selected (Glisic and Inaudi, 2007). This material has excellent mechanical and chemical resistance properties. Since its production involves heating to high temperatures (in order to melt the matrix of the composite material), it is necessary for the fiber to withstand this temperature without damage. In addition, the bonding between the optical fiber coating and the matrix has to be guaranteed. Polyimide-coated optical fibers fit these requirements, thus, are selected as one of the test subject in this research.

The authors have also included some other considerations when designing the protection package of the sensing cables, such as the fibers must be confined from external mechanical actions during installation and while in use; the cable design must allow simple exploitation without the threat of fiber damage and also adequate shielding of the fiber optic from chemical aggression by destructive substances and adverse loading conditions. The details of such requirements and discussions will be presented in the investigation and calibration of BOTDR sensors in Chapter 6.

2.7.6 Issues in Long Term SHM Using BOTDR System

The innovation of BOTDR system is the use of distributed sensors for average strain, temperature, and integrity monitoring. A distributed sensing system, by its

nature, is very suitable and efficient for the monitoring of tunnels, roads, pipes, these being structures with lengths that can achieve several kilometers. So in this thesis, the author will also discuss the problems faced by engineers/researchers while implementing this technology. Research on the SHM and results on the experiments to address the below issues, will be discussed in Chapter 6.

2.7.6.1 Durability of the Fiber

SHM with FOSs requires that the fibers remain functional for the lifetime of the structures as repairs are generally very costly and time consuming. Thus, the feasibility of using FOSs in various structures depends heavily on the durability of these fibers under adverse loading conditions.

It is also important to note that there is a pressing need for such an investigation, as all the studies available in the literature address the individual rather than the combined effects of these issues on the lifetime of the cables. The durability of optical fibers is of great concern in SHM, where the optical fibers and sensors are subjected to mechanical and thermal loads. Since the repair of a damaged optical fiber attached to or embedded in a structure is generally impossible, and in any case highly unwelcome, it is of vital importance to ensure that the optical fiber system will survive and remain functional for the lifetime of the structure.

El-Sabbagh et al. (2004) investigated theoretically and experimentally the combined effect of the mechanical, thermal, and moisture loading on the aging process of fiber optic cables in an attempt to assess the expected lifetime of these cables. Fabrication of FOSs often involves removal of the protective coating locally, thus exposing the glass surface (Figure 2.23) to moisture and handling loads. Static fatigue experiments lasting up to one year were conducted on coated and uncoated optical fibers to determine the durability of the fibers in terms of time-to-failure as a function of applied loading, environment, and handling. Both an acrylate-coated and a polyamide-coated fiber were studied. Uncoated fiber, representing the worst-case

condition, was obtained by chemical removal of the protective coating. Results show that the static fatigue effect is very significant with the time-to-failure decreasing with applied load. For the uncoated fiber, it was found that the static fatigue life depended appreciably on how the fiber was handled, with the durability decreasing significantly by the slightest mechanical contact. The results indicate that strain levels of 0.3% should be survivable for years even in the worst case tested.

2.7.6.2 Core Quality of the Fiber and Types of Coatings on Fiber

The fiber is made of a core, in which light travels, surrounded by a cladding which protects the core while providing the required guiding properties. Surrounding material serves as protection, starting with a plastic jacket and ending, when necessary with some strong metallic cable structure (Figure 2.22). The core guiding structure is achieved by changing the refractive index through doping the silica glass with Germanium Oxide (GeO_2) which tends to raise the refractive index or with Fluorine (F_2) which lowers the refractive index of the F-doped silica glass. To confine the light in the core, the core refractive index must be higher than the cladding's. Different fiber types can be considered to achieve a waveguide structure depending on the refracting index profile: namely the multimode and the single mode. These fiber types have very different optical properties and transmission characteristics.

After rigorous survey in the market, the author found different types of single mode fiber sensors coated with different types of sheathing. These fibers were made to cater mainly communication applications. A very few are available which are capable of capturing strain, our main objective. Moreover the fibers available in the market are mostly coated with a layer of Kevlar Fiber. So while transferring the strain from the structure surface to the core of the sensor, there is a loss of some amount of strain (Kevlar fiber help free movement of fiber core and outer coating as this fiber is loosely fitted). Thus, the strain measured by this kind of fiber is inaccurate.

The core of the sensor is the measurement device of the strain. If the core quality is not good, light carrying capacity of the sensor will be affected and hence due to signal loss, the results will be affected. It was important to check core quality for light carrying capacity. So before addressing problem of sensor coating, one has to find best quality core sensors.

Figure 2.22 – Fiber optic sensor and different layers of coatings

2.7.6.3 Strain Transfer Process

Different types of coatings will have different effect on the strain measurement. The existence of a coating on optical fiber results in difference between the strain of the structure and the strain sensed by surface mounted or embedded FOS. Li et. al. (2003) based on some assumptions derived a theoretical model. The theoretical model reported was based on ideal elasto-plastic constitutive relationship for the polymer coating. With increasing of the loading strain, the stress state in the polymer coating will change from elastic to plastic and the analysis will be decomposed to four phases: elastic phase, elasto-plastic phase, complete plastic phase, and post plastic phase.

The strain-transferring coefficient from the structure to the fiber core in elastic phase can be expressed as

$$\alpha(k, L) = \frac{\varepsilon_g}{\varepsilon_m} = \left[1 - \frac{\cosh(kL) - 1}{kL \sinh(kL)} \right] \quad (2.25)$$

where, k is the stiffness of the FOS and L is the length of the FOS.

The strain-transferring coefficient from the structure to the fiber core in elasto-plastic phase can be expressed as

$$\alpha(k, L, l_0) = 1 - \frac{1}{kL\beta} [\cosh(kl_0) - 1] - \frac{k}{L\beta} \frac{(L-l_0)^2}{2} \cosh(kl_0) - \frac{(L-l_0)}{L\beta} \sinh(kl_0) \quad (2.26)$$

where, l_0 is the length of the elastic region and $\beta = k(L-l_0)\cosh(kl_0) + \sinh(kl_0)$

The strain-transferring coefficient from the structure to the fiber core in plastic phase can be expressed by putting $l_0=0$ in equation (2.26)

$$\alpha(k, L, 0) = 1 - \frac{k}{L\beta} \frac{L^2}{2} = 1 - \frac{k}{L * kL} \frac{L^2}{2} = \frac{1}{2} \quad (2.27)$$

The strain-transferring coefficient from the structure to the fiber core in post plastic phase can be expressed

$$\frac{1}{\alpha} = \frac{1}{\alpha_0} + \frac{1}{\alpha'} \quad (2.28)$$

where,

$$\frac{1}{\alpha_0} = \frac{\varepsilon_{m,\max}^p}{\varepsilon_{g,\max}^p}$$

$$\frac{1}{\alpha'} = \frac{\varepsilon_m - \varepsilon_{m,\max}^p}{\varepsilon_{g,\max}^p}$$

In which, $\varepsilon_{m,\max}^p$ is the maximum plastic strain of the coating matrix, $\varepsilon_{g,\max}^p$ is the maximum strain in glass core and ε_m is the average compressive strain of the coating matrix.

2.7.6.4 Installation of Fiber Optic Sensor

While fixing the BOTDR sensor on to the structure it is required to give some pretension to the sensor. The suitable pretension to the sensor is not known for the available sensors. Hence it is necessary to find suitable pretension for the sensor. Tests will be conducted to address this issue in Chapter 6.

The sensor may be embedded in the structure for sub-surface monitoring, while it needs to be installed on the surface of the structure for surface monitoring. Fixing of the sensor on to the structure can be done by a) pasting method, b) looping method and c) clamping method. To determine the best way of fixing the sensor on to the structure, the author also has conducted experimental investigation on methods of installation in Chapter 7.

2.7.6.5 Instrument for Measurement

After review and assessment of various FOS based monitoring techniques, BOTDR technique is identified as the one applicable to large scale underground monitoring. In this technique, distributed sensing is achieved by the use of a fiber optic as a linear sensor which provides valuable measurement information from all along the fiber itself. A single optical fiber with length of 1km to 30km can replace thousands of traditional single-point sensors. This technology provides a significant reduction in installation, calibration and maintenance costs. Ominisens DiTestTM STA100/200 Fiber Optic Brillouin Analyzer (Figure 2.23) is identified as the measurement instrument and subsequently investigated and examined for its functionality in underground engineering applications.

Figure 2.23 – Omnisens DiTeSt STA 100, stimulated Brillouin frequency based optical time domain analyzer

2.7.7 BOTDR in Civil Structural Health Monitoring (SHM) Application

Currently there are two Brillouin scattering configurations commercially available: (a) Brillouin backscattering (e.g. ANDO, 2005) in which a single source of light is used as the probe and (b) stimulated Brillouin (e.g. Omnisens, 2005) in which a pump laser light wave enters one end of the fiber and a counterwave probe enters the other end. The interaction between the pump and the probe singles maximizes the information available from the scattering, and hence increases significantly the accuracy of the strain estimate. The backscattering Brillouin configuration has a certain advantage over the stimulated approach in case a fiber breaks, as only one end of the fiber is required to obtain measurement.

There are two types of errors involved in the BOTDR readings. One is a random error, whereas the other is associated with the spatial resolution included by the length of the light pulse inside the fiber. The random error of a typical BOTDR analyzer corresponds to a normal distribution with a standard deviation of $\sigma = 15\mu\epsilon$ (i.e.

accuracy of $30\mu\epsilon$) for backscattering Brillouin configuration and $\sigma = 2\mu\epsilon$ (i.e. accuracy of about $4\mu\epsilon$) for stimulated Brillouin configuration. As this error is random, it may be reduced by averaging multiple readings, in which case a reduction factor of $1/\sqrt{n}$ (n is the number of readings) is achieved.

The other departure from the exact strain profile comes from the limit on spatial resolution. The BOTDR measures a weighted average of the strain over $\sim 1\text{m}$, at points spaced by $\sim 10\text{cm}$. This limit comes from the physical length of the pulse of light in the optical fiber, with the result that the BOTDR profile is convolution of the true strain profile with the shape of the light pulse. As a result, for example, a localized disturbance such as a crack will appear as a distributed profile, with a shape similar to the Gaussian bell curve.

Zhou et al. (2007) proposed a novel low cost and high reliable BOTDR sensing head using fiber reinforced polymer. Surface bonding and its mechanical strength were found experimentally. For FOSs used, strain sensitivity coefficients at different spatial resolutions were found. The temperature sensitivity coefficients were also discovered which were very close for different spatial resolutions. This was because the temperature sensing is not very dependent on the spatial resolution.

2.7.7.1 Underground Tunnel Detection using BOTDR

Assaf and Raphael (2009) developed a technique using BOTDR for underground tunnel detection. Two fiber optic layouts were considered and evaluated in a feasibility study that included evaluation of false detection and sensitivity; a horizontally laid fiber buried at a shallow depth and fibers embedded in vertical mini-piles. In the simulation study, two different ground displacement models were used in order to evaluate the robustness of the system against imperfect modeling. In both cases, soil-fiber and soil-structure interactions were considered. The proposed detection system was based on wavelet decomposition of the BOTDR signal, followed by a neural network that was trained to recognize the tunnel signature in the wavelet

coefficients. They concluded that the system was capable of detecting even small tunnel (0.5m diameter) as deep as 20 meter (under the horizontal fiber) or as far as 10 meter aside from the mini-pile (vertical fiber), if the volume loss was greater than 0.5%. However, in this application the BOTDR sensing cable is embedded into the ground and fully bonded to the mini-pile. Such installation method cannot be applied in underground tunnels and caverns, where the monitoring tunnel surfaces are rough and irregular. Thus, such technique is not suitable for underground engineering application.

2.7.7.2 Underground Tunnel Excavation Monitoring Using BOTDR

Mair et al. (2010) presented their latest research work in tunnel excavation monitoring using the BOTDR technology. Two case studies were presented to support their findings. The first is the Thameslink Tunnel in London which is obliquely under the Victorian masonry tunnel. There is an existing tunnel loaded by canal basin retaining wall and the new constructed tunnel is located directly below Midlands Main Line (MML). The second is the Circle Line 3 tunnel in Singapore. The sensor layout of their system is shown in Figure 2.24. These two applications show the possibility of deploying BOTDR system in underground SHM application. However, the author notices some limitations in both applications. The BOTDR FOSs used in the applications are the general tight buffer BOTDR sensor, though it can provide accurate strain measurements but the sensor is too brittle and fragile for underground applications whereby the excavation and construction work is ongoing. The protection used to protect the BOTDR sensors is also not taught enough the withstand conditions under drill and blast excavation. Furthermore, it is found that the BOTDR FOS installation using hook and pulley can only provide an overall strain profile of the tunnel. This method is not able to provide a pin-point stain measurement across the tunnel cross section. Hence, such system may not be able to install and monitor the tunnel during the construction stage. Nevertheless, such installation method will be evaluated in Chapter 7.

Figure 2.24 – Sensor location layout in the Thameslink Tunnel.
(Figure adopted from Mair et al., 2011)

The above applications demonstrate the possibility of using the BOTDR SHM system in geotechnical and underground engineering. However, there are very few such application examples available. The case studies demonstrated the limitations to apply the BOTDR SHM system in underground structures. In this thesis, the author has highlighted and addressed the challenges and issues to overcome the technical and physical deficiencies of applying the BOTDR SHM system in underground engineering. Such research is necessary and essential to promote the development of the underground spatial BOTDR SHM system.

CHAPTER 3 – THE NEW LONGITUDINAL DISPLACEMENT PROFILE WITH EXCAVATION ADVANCEMENT RATE

3.1 INTRODUCTION

It is well documented that competent rock, such as granite or marble, can attain the short term stable deformation profile in a short time after excavation. However, the rate of achieving such short term stable deformation profile and the magnitude of displacement at the advancing tunnel face will deviate for the same tunnel excavated at different advancement rates. The distribution of displacements occurring around the advancing face of a tunnel is one of the widely discussed topics in tunneling. The understanding of tunnel convergence mechanism is essential for designing the required tunnel support and important to ensure the tunnel against possible collapses. Various time dependent effects on the convergence confinement relation to the distribution of displacements occurring around the advancing face of a tunnel is one of the widely discussed topics in tunneling.

The convergence displacement along the tunnel circumference often indirectly indicates the stability of the tunnel. The tunnel convergence in a competent rock can quickly reach a stable state after excavation even without man made support installation. However, in some rock conditions, the tunnel convergence can be time dependent due to a variety of reasons. Both Kerisel (1980) and Duddeck (1980) reported that, the rheological properties of rock mass, the process of tunnel excavation, the timing of tunnel support installation and the time dependent properties of the installed tunnel support can significantly influence the tunnel convergence. The effects of these different factors must be clearly separated in the time dependent analysis of the tunnel displacement. Such understanding of the time dependent mechanism of tunnel convergence is essential for estimating the tunnel displacement profile.

In this chapter, a time dependent excavation advancement rate incorporated longitudinal displacement profile (ARLDP) model is developed through a

hierarchical analytical and numerical approach. The time dependent factors considered in the proposed model are discussed. The analysis takes into account i) separating the rheological properties of a rock mass and ii) the excavation advancement rate effect on the LDP analysis of a tunnel. For the case of a circular tunnel driven in a homogeneous and isotropic medium with time dependent behavior, the theoretical formulation of the model is developed to separate the time dependent effect into the advancement rate influence and the creep influence on the tunnel wall displacements and the ground pressure acting on the lining. Subsequently, an axisymmetric analysis program is employed to determine the variation of radial boundary displacements along the longitudinal direction of a circular tunnel located in a hydrostatic in situ stress field. According to the numerical simulation results, the normalized radial displacements occurring in the vicinity of the advancing tunnel face are affected by the excavation advancement rate. Furthermore, equations obtained by non-linear curve fitting are presented for the normalized radial displacement occurring ahead of and behind the excavation face of the tunnel. Thus, this study presents a new set of formulations for the LDP calculation that takes into account the significant influence of excavation advancement rate.

3.2 TIME DEPENDENT FACTORS ON TUNNEL CONVERGENCE

Tunneling in rock mass with rheological properties involves many different complicated problems due to the uncertainties associated with the response of the rock mass to the process of excavation and construction. Among those, one of the most significant is to determine the convergence of a tunnel. The understanding of the time dependent tunnel convergence mechanism is the key to resolve the uncertainties withhold in the tunnel displacement analysis.

Generally, the tunnel time dependent convergence consists of three main influence factors, they are the ‘creep’ effect, the ‘excavation advancement rate’ effect and the ‘support’ effect.

3.2.1 The Creep Effect

Many researchers have examined the creep phenomenon of rock by using either laboratory experiments or in situ test methods. Lama and Vutukuri (1978), have suggested factors which influence the creep of rocks:

- (i) The nature of stress – specimens subjected to bending and torsional stresses usually show creep-rates at relatively low stresses compared to specimens subjected to compressive stresses.
- (ii) The stress level – the higher the stress levels, the higher the creep-rate is.
- (iii) The confining pressure – the higher the confining pressure, the lower the creep-rate is.
- (iv) The environmental factors – factors like temperature, moisture content, humidity, etc. affect the promotion of creep formation.

Reynolds and Gloyna (1961) reported from the data analysis of an in situ convergence measurement that the creep rate for an opening in a rock mass decreases with age and increases with the depth of the opening. This clearly indicates that the creep behavior of a rock mass around a tunnel depends not only on the rheological properties of the rock mass but also on the existing stress state. As a result, the convergence of a tunnel can be considered as a function of the rheological properties and the stress state of the rock mass. The stress state of the rock mass around a tunnel is strongly affected by the excavation support process of the tunnel construction. Several viscoelastic models have been proposed for the displacement of a circular tunnel under a zero internal pressure condition (Gnrirk and Jonson, 1964); (Wang, Yang and Xue, 1988). Under this condition, the radial displacement of a tunnel wall can be expressed as the product of a certain constant and a time dependent creep function (relating to the creep of the rock mass) as follows,

$$U_r(t) = U_r(t \rightarrow 0) F_c(t) \quad (3.1)$$

Or

$$U_r(t) = U_r(t \rightarrow \infty) F_c(t) \quad (3.2)$$

in which $U_r(t)$ is the radial displacement of tunnel wall at time t , $U_r(t \rightarrow 0)$ and $U_r(t \rightarrow \infty)$ are the initial and final radial displacements, respectively and $F_c(t)$ is a ‘creep function’.

3.2.2 The Tunnel Advancement Effect

For the sake of simplicity, a plane stress or a plane strain condition is often assumed in tunnel convergence analysis. An actual tunneling process, however, is a 3-D problem, especially when the advancing tunnel face is close to the considered cross-section. The 3-D effect influences the creep behavior of the rock mass mainly because the stress state of the rock mass changes with tunnel advancement Lombardi (1980). Without taking the 3-D effect into account, it is impossible to correctly solve the statically indeterminate problem, which also requires the compatibility condition between the deformations of the support and the rock mass (Kerisel, 1980 and Lombardi, 1980)

The tunnel advancement effect mainly derives from the restraint of the tunnel convergence of the already excavated opening due to the unexcavated rock core ahead of the tunnel face. The unexcavated rock core serves as a natural support for the already excavated opening before man made tunnel support is installed. The support provided by the unexcavated rock core tends to attenuate with increasing distance from the tunnel face. In a conventional convergence confinement method, an equivalent 2-D opening subjected to a reducing internal pressure is often considered to model the rock pressure change due to the increase in the distance from the tunnel face. If the relation between the internal pressure $F(x)$ and the distance from the tunnel face is known, Pan and Dong (1991) presented that the radial displacement of the tunnel wall at a certain location can be described by:

$$U_r(x) = F(x)U_r(x \rightarrow \infty) \quad (3.3)$$

in which, $U_r(x \rightarrow \infty)$ represents the radial displacement of the tunnel wall at an infinite distance from the tunnel face. It is a reference displacement equivalent to

the radial displacement of a tunnel in a plane-strain condition. $F(x)$ is a normalized displacement function relating to x . Ward (1978) suggested that it should satisfy $F(x) = 1$ when $x \rightarrow \infty$. For tunnel in an elastic medium with an initial isotropic stress P_o ,

$$U_r(x) = F(x) \left(\frac{P_o a}{2G} \right) \quad (3.4)$$

in which $\frac{P_o a}{2G}$ is the radial displacement of the tunnel corresponding to a plane strain condition and P_o is the internal pressure change from the initial stress state, a is the tunnel radius and G is the modulus of deformation of the rock. Hence,

$$F(x) = \frac{U_r(x)}{\frac{P_o a}{2G}} \quad (3.5)$$

Therefore, equation 3.4 can also be expressed as,

$$U_r(x) = \frac{[F(x) P_o] a}{2G} \quad (3.6)$$

The pressure $F(x)P_o$ can be regarded as the change of the radial internal pressure from the initial state, provided that the actual problem is represented by a symbolic plane strain model. Consequently, the pressure $[1 - F(x)]P_o$ can be treated as a pseudo internal support pressure of the tunnel due to the unexcavated rock core. It is a function of the distance between the considered tunnel cross section and the tunnel face.

3.2.3 The Support Effect

The tunnel support interaction during excavation and construction is indeed a very complicated problem. The properties of the support are essential for the analysis of tunnel support interaction. The support pressure acting on the tunnel wall depends on the deformability of the rock mass and the stiffness of the whole

support system. A proper support installation scheme may significantly reduce the ultimate convergence of a tunnel (Ward, 1978). The installation time of the support is critical for estimating tunnel convergence and support pressure. An early support installation tends to reduce the ultimate tunnel convergence and increase the pressure acting on the support. The large displacement of a tunnel in rock mass with rheological properties and overstress in a support may be avoided only if the support is designed and installed properly (Duddeck, 1977; Brown et al., 1983; Goodman, 1989; Hoek and Brown, 1980).

3.3 THE ANALITICAL VISCOELASTIC MODEL FOR ROCK MASS DEFORMATION WITH THE TIME DEPENDENT EXCAVATION ADVANCEMENT EFFECT

Pan and Dong (1991) introduced an analytical solution to analyze the time dependent effect in tunnel displacement through a viscoelastic model. The displacement function introduced by them utilized support pressure from both the unexcavated rock medium ahead of the tunnel face and the installed tunnel support,. However, it is not suitable for analyzing the tunnel deformation whereby, support is not installed (i.e. the immediate deformation of the excavation face in a drill and blast excavation.). Nevertheless, such valuable information formed the foundation of the work done below. The following assumptions are made in the development of the viscoelastic model for a rock tunneling problem.

- i) The rock mass can be treated as a homogeneous and isotropic viscoelastic material.
- ii) The cross-section of tunnel is circular.
- iii) The effects of normal stress in the direction of the tunnel axis, the shear stress near the tunnel face and the self-weight of rock mass around the tunnel can be neglected.
- iv) The advancing effect and the support effect, together, can be treated as a change in the internal pressure and can be approximated by the integration of infinite step functions.
- v) The rock-core pressure is a function of the distance from the considered cross-section to the tunnel face.
- vi) The stress-strain relation of the support system is linear-elastic; the

support pressure P , is the product of support stiffness K_s and the tunnel wall displacement after the support contacts with the rock mass.

- vii) The support contacts the rock mass as soon as the support is installed.
- viii) The change in internal pressure is independent of the creep function.
- ix) The rock-core pressure is independent of the support pressure.

Consider a circular opening with radius, a , in an infinite medium of rock mass subjected to an initial stress, P_0 . Suppose the internal pressure on the opening at time, t is $P_i(t)$, then the internal pressure changes an amount $P(t) = P_0 - P_i(t)$ from the initial state at time, t . For a tunneling problem, the internal pressure P_i can be regarded as the support pressure contributed by the unexcavated rock core $P_c[x(t)]$ and from the installed support $P_s[U_r(t)]$,

$$P_i(t) = P_c[x(t)] + P_s[U_r(t)] \quad (3.7)$$

Since the focus of this analysis is to investigate the tunnel excavation advancement rate effect to the tunnel displacement during excavation, the tunnel shall be treated as no support is installed. Hence, the installed support pressure $P_s[U_r(t)] = 0$. The internal pressure shall only consist of the support pressure contributed by unexcavated rock core $P_c[x(t)]$. Therefore, equation 3.7 becomes,

$$P_i(t) = P_c[x(t)] \quad (3.8)$$

By substituting equation 3.8 into $P(t) = P_0 - P_i(t)$ we can obtain,

$$P(t) = P_0 - P_c[x(t)] \quad (3.9)$$

where, $P_c[x(t)]$ = excavation advancement rate effect and,

$$P_c[x(t)] = \{1 - F[x(t)]\} P_0 \quad (3.10)$$

In a viscoelastic model, when P_i is independent of time (i.e. P_i do not change with time). The normalized displacement is,

$$\frac{U(t)}{a} = g_{ve} (P_0 - P_i) F_c(t) \quad (3.11)$$

where g_{ve} is the compliance function and $F_c(t)$ is the creep function.

However, for time dependent P_i , whereby P_i changes with time t , the normalized displacement function has to obtain through transformation in the Laplace space. Through Laplace transformation the normalized displacement is given as,

$$\frac{U(t)}{a} = \int_0^t F[g_{ve}, P(\tau), F_c(t-\tau)] d\tau \quad (3.12)$$

as $A(t) \times B(t) = \int_0^t A(\tau) B(t-\tau) d\tau$,

$$\frac{U(t)}{a} = g_{ve} \int_0^t h(\tau) F_c(t-\tau) d\tau \quad (3.13)$$

Where,

$$h(\tau) = P_0 - P_i(\tau)$$

Thus,

$$\frac{U(t)}{a} = g_{ve} [h(t) \times F_c(t)] \quad (3.14)$$

$$\text{where } P(t) = h(t) = \left[\frac{dP(t)}{dt} \right] \quad (3.15)$$

Substituting equation 3.15 into equation 3.14 gives

$$\frac{U(t)}{a} = g_{ve} \left[\frac{dP(t)}{dt} \times F_c(t) \right] \quad (3.16)$$

If only consider the tunnel excavation advancement rate influence in the analysis,

$$P(t) = P_0 - P_i(t) = P_0 - P_c(t) \quad (3.17)$$

Therefore,

$$\frac{U(t)}{a} = g_{ve} [h_c(t) \times F_c(t)] \quad (3.18)$$

where

$$h_c(t) = \frac{d[P_0 - P_c(t)]}{dt} \quad (3.19)$$

The model developed above can also be applied in plane strain analysis with the pressure replacement technique. The advancement rate effect can be analyzed by determining $P_i(t)$ as the fictitious temporary support in pressure replacement technique (Figure 3.1).

Figure 3.1 – The plane strain core replacement analysis technique.

However, the computation of the model is rather complex and it involves various uncertainties and assumptions in determining the correct $P_i(t)$ to perform such core replacement analysis in the plane strain condition. Therefore, an alternate analysis is performed. By inspecting the general law of convergence confinement analysis method (Pan and Dong, 1991),

$$C_{(x,t)} = C_1(x) + C_2(t) = F(x), \quad (3.20)$$

where,

$$C_1(x) = C_{\infty x} \left\{ 1 - \left[\frac{X}{x+X} \right]^2 \right\} \quad (3.21)$$

and

$$C_2(t) = A \left\{ 1 - \left[\frac{T}{t+T} \right]^{0.3} \right\} \quad (3.22)$$

Here, $C_{(x,t)}$ is the final closure of the tunnel, $C_1(x)$ is the closure related to normalized distance and $C_2(t)$ is the closure related to time function. In which A is dependent on x and $\frac{A(x)}{C_1(x)} = m$

where m is constant and $C_{\infty x}$ is the instantaneous closure, X is the distance related to plastic radius, R_p and T is the time related to deformation

Substituting equation 3.21 and 3.22 into 3.20, we can obtain,

$$C_{(x,t)} = C_{\infty x} \left\{ 1 - \left[\frac{X}{x+X} \right]^2 \right\} \left\langle 1 + m \left\{ 1 - \left[\frac{T}{t+T} \right]^{0.3} \right\} \right\rangle \quad (3.23)$$

From the time dependent analysis perform above, we can determine that, for elastic model

$$C(x) = C_{\infty x} \left[1 - \exp\left(\frac{-x}{X}\right) \right] \quad (3.24)$$

for $X = aR_p$ and $a = 0.84$.

Since the focus of this analysis is to investigate the advancement rate effect, the creep effect $C_2(t)$ can be ignored. With the general displacement function,

$$U_r(x) = F(x) U_{r(x \rightarrow \infty)} \quad (3.25)$$

When normalized with $\frac{C_1(x)}{C_{\infty x}}$, equation 3.21 becomes

$$F(x) = \left\{ 1 - \left[\frac{X}{x + X} \right]^2 \right\} \quad (3.26)$$

The effect of tunnel advancement arises from the changing rate of the unexcavated rock-core pressure. Suppose the tunnel face advances at a constant speed V , the unexcavated rock core pressure can be related to the tunnel advancement rate V through a normalized displacement function $F(x)$. Assume that equation (3.26) is valid and $P_c[x(t)] = P_0 F(x)$, then,

$$P_c(t) = P_0 \left\{ 1 - \left[\frac{X}{x + X} \right]^2 \right\} = P_0 \left\{ 1 - \left[\frac{X}{V_T + X} \right]^2 \right\} \quad (3.27)$$

It can be observed from the above equation that the unexcavated rock core pressure $P_c(t)$ depends on the elapsed time and tunnel advancement rate. As a consequence, the function $h_c(t)$ in equation 3.18 and 3.19 also depends on the elapsed time and the tunnel advancement rate. Therefore, it is clearly shown that if the investigation mainly deals with the excavation advancement rate time dependent effect, the speed of excavation, the distance of excavation and the plastic radius developed are of more importance than the creep function. To simplify such

complex computation, an axisymmetric numerical model simulation is used to further develop the ARLDP.

3.4 THE ARLDP, RELATION OF THE ADVANCEMENT RATE WITH TUNNEL RADIAL DISPLACEMENT

There is an important caveat to consider when using numerical analysis to compute the LDP. When using axisymmetric condition to determine the LDP relationship, it is important to consider the excavation advancement rate. A stress front builds ahead of the bullet shaped plastic zone and influences the rate of plastic zone development. Such a model will yield a different apparent LDP depending on the size of the excavation steps. This phenomenon agrees with the law of convergence confinement method presented above. Although the final displacement does not vary too much regardless the excavation advancement rate employed, it is important to establish the LDP with the advancement rate effect (ARLDP) for a better estimation of tunnel support installation time. Vlachoulos and Diederichs (2009) have also presented that there is a significant difference in the rate of deformation development between the different step (length of excavation in each stage) used in simulation. Therefore, preceding from the reviews and the previous work of Vlachoulos and Diederichs (2009), the author has performed a series of axisymmetric numerical analysis to establish the relationship between the tunnel displacement, the rate of plastic radius development incorporate with the excavation advancement rate effect. The series of numerical simulations are performed through axisymmetric analysis using PHASE2 under the suggested Hoek-Brown criterion for jointed rock masses (Hoek et. al., 1992) listed in Annex A with the published data from Vlachoulos and Diederichs, (2009).

3.4.1 The ARLDP Related to Tunnel Excavation Advancement Rate Effect

The development of radial deformation is directly linked to the excavation activities during construction. Studies by the author have shown that the LDP function proposed by Panet (1995) is reasonable for the radial displacement analysis to ignore the excavation advancement rate effect, provided that the rock material used in the analysis is treated as an elastic medium. A sample analysis is performed

involving a radial tunnel section via an axisymmetric analysis using PHASE2. The analysis is based on a typical rock tunnel in a weak rock mass (i.e. graphitic phyllite). The parameters are given in Table 3.1 and the results are illustrated in Figure 3.2. In order to account for the influence of excavation advancement rate to the normalized LDP, the most logical index to relate to the LDP function is the normalized advancement rate, V^* , and $V^* = V_T/R_T$ where, V_T is the daily excavation advancement rate and R_T is the tunnel radius.

Table 3.1 – Rock mass and tunnel parameters for LDP analysis in PHASE2
(Data adopted from Vlachoulos and Diederichs, 2009)

Rock mass										
In situ Stress, P_0 (MPa) In situ Stress/Rock mass strength, P_0/UCS_{RM} Intact Rock Mass Modulus, E_{RM} (MPa) Tunnel Radius, R_T (m)							28			
							Elastic			
							1150			
							2.5			
TUNNEL										
Tunnel Radius, R_T	2.5	2.5	2.5	2.5	2.5	5	7.5	5	4.5	5
Advancement Rate, V_T	1	2	5	10	15	1	3	5	9	15
$V^* = V_T/R_T$	0.4	0.8	2	4	6	0.2	0.4	1	2	3

Figure 3.2 – Displacement analysis results of an elastic medium.

However, as described by Vlachopoulos and Diederichs (2009), the advancing front of plastic yielding is bullet shaped in three-dimensions and for large plastic zones (radius of plastic zone, $R_p > 2R_T$) the shape of this developing yield zone is geometrically similar for increasing maximum plastic radii which is influenced by the rate of advancement. There is no reason, to expect that a single longitudinal displacement profile will suffice for these conditions. However, Vlachopoulos and Diederichs (2009) has introduced the LDP relationship with the plastic radius development through a series of representative cases with varying plastic radius to tunnel radius ration. They defined a new parameter, $R^* = R_p/R_T$ which normalized the plastic radius formed around the tunnel, R_p , with the tunnel radius, R_T . It is now possible to examine the tunnel radial displacement with the influence of excavation advancement rate.

To illustrate the LPD excavation advancement rate influence relationship proposed, a series of analyses are performed involving a circular tunnel section with axisymmetric condition. The first suite of analyses is based on a typical weak rock mass at 1100m depth (i.e. graphitic phyllite). This is material ‘Type 1’ in Table 3.2. In this case, the initial in situ stress is approximately 10 times the estimated rockmass uniaxial strength (UCS_{RM} or σ_{cm} calculated according to Hoek et al., 2002). The varying excavation advancement rates are normalized with the tunnel radius giving the new parameter V^* (excavation advancement rate/tunnel radius). The values of V^* used in the analysis are listed in Table 3.3.

Table 3.2 – Rock mass parameters
(Data adopted from Vlachoulos and Diederichs, 2009)

Rock mass	Type 1
In situ Stress, P_0 (MPa)	28
In situ Stress/Rock Mass Strength, P_0/UCS_{RM}	10
Uniaxial Compressive Strength, σ_{ci} (MPa)	35
Material Constant, m_i	7
Poisson's Ratio, ν	0.25
GSI	25
m	0.481
s	0.0002
a	0.531
Intact Rock Mass Modulus, E_{RM} (MPa)	1150
Intact Rock Mass Strength, UCS_{RM} (MPa)	2.8
Tunnel Radius, R_T (m)	2.5
Maximum Displacement, U_{max} (m)	1.168
Normalized Plastic Radius, R^*	5.48

Table 3.3 – V^* value for analysis

Excavation Advancement Rate, V_T (m/day)	1	2	5	10	15
Tunnel Radius, R_T (m)	2.5	2.5	2.5	2.5	2.5
$V^* = (V_T/R_T)$	0.4	0.8	2	4	6

The analysis results (Figure 3.3) clearly show that through different rate of excavation advancement, the LDP developed for the same rock mass type is different in plastic analysis. It is believed that the rate of plastic zone developed varies through different rate of excavation. This effect is significantly important towards the displacement estimation especially at the tunnel excavation face. It is obviously shown in Figure 3.3 that the displacement developed near the excavation face has extensively increased with the increase in excavation rate. The distance or time required for the tunnel to converge to its final displacement is also shorter.

Figure 3.3 – Displacement analysis results of rock mass type 1.

Three other rock masses are investigated with increasing intact strength and/or GSI (Hoek et al., 2000), given a series of representative cases with varying plastic radius to tunnel radius ratio, R^* (Vlachoulos and Diederichs, 2009). The rock mass parameters are summarized in Table 3.4. The rock mass strengths are estimated as per Hoek et al. (2002) and the elastic moduli are estimated based on Hoek and Diederichs (2006). This second set of analyses are performed based on different rock mass Type 1 – Type 5 with the same excavation advancement rate, $V^* = 2$.

Table 3.4 – Rock masses parameters
(Data adopted from Vlachoulos and Diederichs, 2009)

Rockmass	Type 2	Type 3	Type 4	Type 5
P_0 (MPa)	28	28	28	28
P_0/UCS_{RM}	8	6	4	Elastic
σ_{ci} (MPa)	35	35	50	
m_i	7	7	7	
ν	0.25	0.25	0.25	
GSI	35	45	48	
m	0.687	0.982	1.093	
s	0.0007	0.0022	0.0031	
a	0.516	0.508	0.507	
E_{RM} (MPa)	2183	4305	7500	1150
UCS_{RM} (MPa)	3.5	4.7	7	28
R_T (m)	2.5	2.5	2.5	2.5
U_{max} (m)	1.143	0.532	0.222	0.0753
R^*	5.07	3.1	2.13	1

Figures 3.4 and 3.5 show the LDP results of the five different types of rock mass under the same advancement rate, V^* . By inspection of Figures 3.4 and 3.5, it is evident that the LDP does not correlate with the stress/strength index p_0/UCS_{RM} as the set of curves in both plots represent the same selected values for this ratio and yet have different LDPs. Analysis of the data, however, shows a direct correlation with the maximum normalized plastic zone, R_P/R_T and the normalized excavation advancement rate, V_T/R_T as expected. The correlation between u_0/u_{max} at $X/R_T = 0$ (at the face) and the advancement influence relation V_T/R_T , is shown in Figure 3.6. Ignoring the influence of the Poisson's ratio (negligible compared to plastic yielding) and treating the influence from plastic zone development a constant, the best fit relationship (independent of material parameters and stress levels) is:

$$u_0^* = \frac{u_0}{u_{max}} = A e^{\left(\frac{-V^*}{B}\right)} + C \quad (3.28)$$

where,

$$A = -0.00485 e^{(0.67 R^*)}$$

$$B = 3.25 e^{(-0.084 R^*)}$$

$$C = 0.182 e^{(0.09 R^*)}$$

In which $R^* = R_P/R_T$ and $V^* = V_T/R_T$

Figure 3.4 – LDP for five different rock mass at $V^* = 0.4$

Figure 3.5 – LDP for five different rock mass at $V^* = 2$

Figure 3.6 – Correlation between $u_0^* = u_0/u_{\max}$ and $V^* = V_T/R_T$ at $X^* = X/R_T = 0$

For $X^* < 0$, the function is defined as,

$$u^* = \frac{u}{u_{\max}} = u_{ro}^* \times e^{X^*} + \left(e^{(-0.05V^* + 0.55X^*)} \right) \times u_0^* \quad (3.29)$$

For $X^* > 0$, the function is given as,

$$u^* = \frac{u}{u_{\max}} = u_{ro}^* \times e^{X^*} + \left(e^{(-0.05V^* - 0.23X^*)} \right) \times u_0^* \quad (3.30)$$

$$\text{where, } u_{ro}^* = \frac{u_{ro}}{u_{\max}} = \frac{1}{3} e^{-0.15R^*}$$

3.5 CONCLUSION

Through a series of numerical analyses, a new series of functions defining LDP, as a function of maximum normalized plastic radius and the normalized excavation advancement rate, has been developed. Figure 3.7 shows a comparison graph of the calculated results (based on the rock properties of Rock type 1 in Table

3.2 with equations 3.28~3.30) against the numerical simulation results. It shows the developed formulations (equations 3.28~3.30) match well with the numerical simulation results. This approach takes into consideration of the time dependent influence of the excavation advancement rate that affects the development of the large ultimate plastic radius and the rate of tunnel wall displacement development along the tunnel. Previous LDP functions are inadequate for tunnel analysis in face of the reality. This approach provides a more compressive overview of the tunnel wall displacement in tunnel excavation.

Figure 3.7 – The comparison of numerical simulation results vs calculated results.

The set of ARLDP formulations developed (equations 3.28~3.30) define a complete relationship for a complete LDP analysis (i.e. for positions at $X^* = 0$, $X^* < 0$ and $X^* > 0$) that can be incorporated directly into analytical solutions or used for calibration of staged numerical models. The ARLDP defines a tunnel deformation profile with excavation advancement rate inconsideration. Corresponding to the definition in equation 3.27, the displacement (closure) obtained from the ARLDP shall give a close estimate to the analytical solution given in equation 3.18 (i.e.

$u_{ro}^* \& u^* \approx \frac{U(t)}{a}$. However, as an alternative to equations 3.28~3.30, Section 3.3

provides an analytical methodology which can be computed for higher accuracy with numerical computation software. The equations and solutions presented here are for short term displacements occurring as a function of tunnel advance only, where the long term time dependent squeezing effect is anticipated. This approach will still need to be modified to improve the guidance provided by Pan and Dong (1991). However, through the work presented, it shows there are limitations in both the analytical solution and the solution form numerical simulation. The analytical solution faces the issues with heavy computation loads, uncertainties arise from the compliance function and creep function, it also faces the problem in obtaining all the essential parameters from the rock sample in practice in order to perform such analysis. On the other hand, the numerical simulation solution faces the challenge in validating its accuracy. The accuracy may vary against if different program is used in the process of developing the curves. In this thesis, PHASE2 is adopted and validated with some examples; this shows the possibilities and the importance to further develop such LDP analysis to strengthen the knowledge in underground engineering.

CHAPTER 4 – THE SUPERIMPOSED LONGITUDINAL DISPLACEMENT PROFILE (LDP) MAPPING ANALYSIS

4.1 INTRODUCTION

It has been a common and usual practice for field engineers to obtain on-site convergence measurements of particular tunnel cross sections prior to and after subsequent excavation stages. The main objectives of these measurements are to check if the convergence or rather the displacement of the excavated section has exceeded the allowable design deformation limits. But there are limitations withhold in such methodology. The time allowing the field engineers or surveyors to deploy and obtain the measurement is limited, also, due to safety concern they are not able to take measurements on the excavation face at the immediate moment after excavation. For example, for drill and blast excavation, the first one to two hours after blast are usually scheduled as ‘no entry’ time for ventilation, rock dust and gases emission. This inspired the researchers to conduct research to predict the deformation at the tunnel excavation face, which aims to provide engineers with the missing link and more information through the excavation works.

It has been described that the LDP can be used for prediction or estimation of tunnel wall displacement along the longitudinal tunneling profile. However, it is not a convenient tool for engineers to employ this technique for site activities. The main reason is, in most cases the field measurements or monitoring data are different from the displacement calculated from numerical simulations or theoretical solutions. The difference in readings caused the departure of using LDP in the field for tunnel deformation profile prediction.

Therefore, in this chapter, the author aims to address the issue of utilizing the field measurement accompanying with the LDP to estimate the tunnel displacement profile and predict the tunnel face deformation via the proposed back analysis method – the Superimposed Longitudinal Displacement Profile (SLDP) mapping technique. The proposed method utilizes the relative displacement measurements of a monitored cross section acquired from subsequence excavation

stages. The measurement data will be superimposed and mapped back to the tunnel LDP analysis to estimate the total displacement profile constructed through numerical modeling or theoretical calculations along the monitored cross section. The proposed method can also predict the deformation at the advancing tunnel face. It can be further investigated to estimate other geo-parameters like the plastic radius forming around the excavated zone and the Poisson's ratio through the back analysis.

4.2 UNDERGROUND TUNNELING AND DEFORMATION

Kovári (1998) has commented that, displacement intensity in tunnels especially during excavation can vary greatly over short distances even there is no change in the excavation method, temporary support, depth of cover or lithology. Such variability makes tunneling in underground become very demanding. It decreases the predictability of the conditions away from the monitoring sections even after some experience has been gained with specific geological formation during excavation. Cantieni and Anagnostou (2007) have categorized the cause of such variability into two different areas (i) the rock structure heterogeneity (even on the scale of few meters) may lead to significant variations in the ground response; and, (ii) the small fluctuations in the mechanical and hydraulic properties of a macroscopically homogeneous rock mass may have a major effect on the development of deformations and pressures.

Uncertainties concerning rock structure heterogeneity can be reduced through advance probing. However, the uncertainties concerning ground response remains. Therefore, the estimation of tunnel displacement profile represents one of the most difficult challenges in underground engineering. A timely estimation of the conditions along the tunnel profile will enable adaptations to be made during construction to the temporary support and excavation process. A number of researchers have therefore attempted to identify early indicators of tunnel deformation on the basis of field measurements. Steindorfer (1998) proposed a method of predicting changes in rock mass quality ahead of the face based on the displacement vector orientations obtained by geodetic measurements in the tunnel.

Jeon et al. (2005) underpinned such method theoretically by means of numerical computations. But they pointed out that it is very difficult to make a prediction under complex geological conditions. Sellner (2000) proposed a method of predicting the displacement of the tunnel boundary based on the work of Sulem et al. (1987). This method requires an estimation to be made of the ground convergences ahead of the face; however, this can apply only when the parameters are estimated with the function defined by Sulem et al. (1987) on the basis of experiences. Despite the improvements in the theoretical and numerical assessment of the tunneling squeezing phenomenon and the experiences gained with different construction methods, there is still no reliable method available for estimating the displacement profile along the tunnel.

Some researchers suggested that a complete LDP analysis shall include the displacement estimation ahead of the tunnel advancing face (Unlu and Gercek, 2003; Vlachopoulos and Diederichs, 2009). Such analyses are promising when the radial loading and axial extension of the core ahead of the face can be seen as a large scale in-situ test. The deformation analysis in the tunnel ahead of the face seems to be reliable with regard to ground response predictions.

In the past, scientists attempted to estimate the convergences and evaluate the tunnel face stability through the excavation face extrusion measurements. Lunardi (1995) first used such measurements in squeezing ground for the assessment of both face stability and the expected convergences. Lunardi and Gatti (2010) have presented a correlation between the extrusions, the convergences and the overburden. However, Cantieni and Anagnostou (2011) demonstrated that the face extrusion during a standstill can be from several centimeters to decimeters, but, Kovári (1998) proved that such extrusion is not problematic in conventional tunneling convergence estimation as long as the face remains stable.

Figure 4.1 shows a schematic diagram of the mechanism leading to tunnel core displacement. As the tunnel face approaches, the axial displacement or extrusion, u_{y0} , causes the ground core ahead of the face to lose its axial confinement and start to deform, $u_r(\infty)$. The loss of confinement reduces the radial resistance of the ground core, and the core thus deforms $u_r(y)$ due to the radial load σ_r , exerted by

the surrounding ground. The core yields under the radial loading and extrudes into the opening. The amount of the displacement depends on the mechanical properties of the rock, the lining thickness and the support measures applied in the tunnel. If the time dependent behavior is included in the displacement analysis, the displacement will also depend on the excavation advancement rate and the creep function on the duration of any standstills.

Figure 4.1 – Schematic mechanism of core displacement
(Figure modified from Linard, 2011)

The review above shows that it is possible to estimate the tunnel displacement profile especially at the excavation tunnel face. However, it is very difficult and sometimes impossible to assess the excavation tunnel face section to record timely measurements. Such problem initiates to investigate possibility to predict the deformation at the tunnel face or along the monitoring section during tunneling operation through assessing the relative displacement data acquired along the excavation.

4.3 LONGITUDINAL DISPLACEMENT PROFILE (LDP)

Through decades the LPD has been well described and well practiced as a complementary profile together with the ground response curve (GRC) and support reaction curve (SRC) in the convergence confinement method (CCM) analysis to estimate the support design and its installation time to the excavated tunnel sections.

However, it is sometimes more practical to use an analytical function for the LDP analysis if an analytical convergence confinement solution is used (Figure 4.2) or when only two dimensional models are available.

Figure 4.2 – A CCM analysis with the GRC, LDP and SRC.
(Figure adapted from Vlachopoulos and Diederichs, 2009)

Panet (1995) derived a relationship for the longitudinal displacement profile based on elastic analysis to facilitate an analytical calculation for the ground response.

$$u^* = \frac{u_r}{u_{\max}} = \frac{1}{4} + \frac{3}{4} \left[1 - \left(\frac{3}{3 + 4X^*} \right)^2 \right] \quad (4.1)$$

where $X^* = X/R_T$, u_r is the radial displacement at a specified longitudinal position X , $u_{\max}$ is the short term maximum radial displacement at distance from the advancing face and corresponding to plane strain displacement analysis of a tunnel cross section with the tunnel radius, R_T . X is positive ($X > 0$) into the excavated tunnel away from the excavation face ($X = 0$). The position, X , is negative ($X < 0$) for the unexcavated section behind the excavation face.

Numerous researchers have suggested alternative expressions for the elastic LDP. Unlu and Gercek (2003) have noted that the LDP functions in front of the excavation face and behind the excavated face do not follow a single continuous functional relationship with X . They agreed with this assertion and LDP with respect to distance from the excavation face is accurately predicted for the elastic case,

for $X^* \leq 0$;

$$u^* = \frac{u_r}{u_{\max}} = \frac{u_0}{u_{\max}} + A_a \left(1 - e^{B_a X^*}\right) \quad (4.2)$$

for $X^* \geq 0$

$$u^* = \frac{u_r}{u_{\max}} = \frac{u_0}{u_{\max}} + A_b \left(1 - \left(\frac{B_b}{(A_b + X^*)}\right)^2\right) \quad (4.3)$$

where u_0 is the radial displacement at the excavation face ($X^* = 0$) and A_a , A_b , B_a and B_b are functions of the Poisson's Ratio:

$$u_0^* = \frac{u_0}{u_{\max}} = 0.22\nu + 0.19 \quad (4.4)$$

and

$$A_a = -0.22\nu - 0.19$$

$$A_b = -0.22\nu + 0.81$$

$$B_a = 0.73\nu + 0.81$$

$$B_b = 0.39\nu + 0.65$$

These preceding equations are developed for elastic deformation analysis. Panet and Guenot (1982), Panet (1993, 1995), Chern et al. (1998) and Vlachopoulos and Diederichs (2009) have proposed other empirical solutions for LDPs analysis based on varying intensity of modeled plastic deformation. These analyses correlate various indices. One of such analyses investigates the in-situ stress versus the undrained cohesive strength (Chern et al., 1998). So alternatively, an empirical 'best

fit' solution to actual measured closure data can be used based on data from Chern et al. (1998).

$$u^* = \frac{u_r}{u_{\max}} = \left(1 + e^{\left(\frac{-X^*}{1.1} \right)} \right)^{-1.7} \quad (4.5)$$

The development of radial deformation, however, is directly linked to the development of the plastic zone as the tunnel advances. Vlachopoulos and Diederichs (2009) have pointed out that the LDP function proposed by Panet (1995) and by Unlu and Gercek (2003) is reasonable for plastic analysis provided that the radius of the plastic zone does not exceed 2 times the tunnel radii and provided that the yielding zone in the tunnel face does not interact with the developing yield zone around the tunnel walls as illustrated in Figure 4.3.

Figure 4.3 – a) Maximum plastic zone radius is less than twice the tunnel radius b) Plastic zone more than double the tunnel radius and interacts with face yield zone (Figure adapted from Vlachopoulos and Diederichs, 2009)

The advancing front of plastic yielding is bullet shaped in three-dimensions. However, it is geometrically similar for increasing maximum plastic radii regardless of large plastic zones (radius of plastic zone $R_P > 2R_T$) or small plastic zone ($R_P < 2R_T$). Therefore, Vlachopoulos and Diederichs (2009) have proposed the improved

LPD method relating the LDP function with the normalized plastic zone radius and the excavated tunnel radius, $R^* = R_P/R_T$.

$$u_0^* = \frac{u_0}{u_{\max}} = \frac{1}{3} e^{-0.15R^*} \quad (4.6)$$

when $X^* \leq 0$,

$$u^* = \frac{u}{u_{\max}} = u_0^* \times e^{X^*} \quad (4.7)$$

when $X^* \geq 0$,

$$u^* = 1 - (1 - u_0^*) \times e^{\frac{-3X^*}{2R^*}} \quad (4.8)$$

These above mentioned LDP relationships are summarized in Figure 4.4.

Figure 4.4 – Comparison of LDP developed in the past

4.4 SUPERIMPOSED LDP (SLDP) MAPPING ANALYSIS

The proposed SLDP mapping analysis method is designed to allow engineers and researchers to have a quick overview of the tunnel deformation profile and estimate the deformation at the tunnel face via the onsite displacement measurement. The underlining principle of the SLDP mapping analysis method is that, every LDP developed has its own unique gradient regardless of which method of analysis described above is adopted in the analysis. Such uniqueness is particularly obvious in sections where $X^* > 0$ while performing the improved LDP analysis proposed by Vlachopoulos and Deiderichs (2009). It is very useful to engineers and researchers in estimating the deformation profile and the plastic radius formed around the excavated tunnel. Therefore, it is worth investigating whether such uniqueness of the improved LDP analysis can be incorporated with the onsite measurement to perform a quick and realistic estimation of the tunnel deformation profile. The flow chart (Figure 4.5) below demonstrates the steps in performing the SLDP analysis.

Figure 4.5 – Flow chart of the SLDP analysis process

The unique gradient for the improved LDP (Vlachopoulos and Deiderichs 2009) can be obtained in the following operations.

Substituting equation 4.6 into equation 4.8, we can obtain,

$$u^* = 1 - \left(1 - \frac{1}{3}e^{(-0.15R^*)}\right)e^{\left(\frac{-3X^*}{2R^*}\right)} \quad (4.9)$$

Differentiating equation 4.9 we can obtain,

$$\frac{du^*}{dX^*} = \frac{3}{2} \left[\frac{\left(1 - \frac{1}{3}e^{(-0.15R^*)}\right)e^{\left(\frac{-3X^*}{2R^*}\right)}}{R^*} \right] \quad (4.10)$$

The unique gradient function (equation 4.10) will be mapped onto the SLDP constructed with the onsite relative displacement measurement data.

The SLDP can be developed through a series of operations. First, with the geological information reported from the geological survey, the final convergence/displacement, $u_{\max}$, can be determined with a 2-D plane strain analysis. The initial convergence survey measurement, u_0' , is recorded and set as the 'zero mark' reference point. The distance between the measurement point and the tunnel face, d_0 , should be known and recorded. After each consecutive excavation stage, the relative convergence displacement and the distance against the tunnel face at the same measurement point shall be recorded. This procedure shall repeat for three to five consecutive excavation stages (Figure 4.5). The difference of the consecutive convergence/displacement (u_1' , u_2' , u_3' ... u_n') against u_0' are normalized with the final convergence, $u_{\max}$. The SLDP is constructed through plotting the normalized relative measurement against the distance from the relative excavation face. A best fitting curve can be constructed with the plotted data. The gradient of the best fit curve, ω , is determined. The distance between the initial measurement point to the excavated tunnel face is normalized with the tunnel radius, R_T , where, $X^* = d_0/R_T$. Equating ω and substituting X^* to equation 4.10, we have,

$$\omega = \frac{du^*}{dX^*} \quad (4.11)$$

$$\text{and, } X^* = \frac{d_0}{R_T}$$

Figure 4.6 – Schematic drawing for the SLDP mapping process.

4.5 EXAMPLE VALIDATION

The new relationships proposed in Section 4.4 can be used to estimate the displacement profile along the excavated tunnel and at the excavation face with the reliable onsite convergence measurements. Numerical simulations are performed to illustrate and validate the proposed SLDP mapping technique. The tunnels with the properties shown in Table 4.1 (adopted from Vlachopoulos and Deiderichs, 2009)

are analyzed with PHASE2 (Rocscience, 2007) in the plane strain cross section (Figure 4.6a) to determine the extent of the maximum radial deformation in each case. In addition, the cases are analyzed via the axisymmetric models (Figure 4.6b) with incremental excavation advancement equal to 0.2 times of tunnel diameter. The expected LDPs in each case are shown in Figure 4.7. The maximum displacements and the relative tunnel profile displacement measurements are comparable between the radial and longitudinal models. The summarized results are presented in Table 4.2 and the process of the SLDPs is evaluated with the displacement data from the axisymmetric simulation results.

Figure 4.7 – a) Plane strain analysis model, b) Axisymmetric analysis model
(Simulations from PHASE2, Rocscience, 2007)

Figure 4.8 – Longitudinal Displacement Profile (LDP) for $R^* = 1 \sim 7.5$

Table 4.1 – Rock mass parameters for LDP analysis using PHASE 2

(Data adapted from Vlachopoulos and Diederichs, 2009)

Rockmass	A
P_0 (MPa)	28
P_0/UCS_{RM}	10
σ_{ci} (MPa)	35
m_i	7
ν	0.25
GSI	25
m	0.481
s	0.0002
a	0.531
E_{RM} (MPa)	1150
UCS_{RM} (MPa)	2.8
R_T (m)	2.5

Table 4.2 – U_{max} and R^* results from PHASE 2 plane strain analysis

(Data adapted from Vlachopoulos and Diederichs, 2009)

Rockmass	A
R^*	5.48
u_{max} (m)	1.168

The axisymmetric numerical simulation results are used to evaluate the SLDP mapping method. Assuming that the convergence measurement starts at Point A of the tunnel (Figure 4.8), for which the displacement recorded at this point is reported as ‘zero’ and the distance from the tunnel excavation face, d_0 is recorded. The excavation process continues and after every stage of the excavation, the displacement at point A, u_1' , u_2' , u_3' ... u_n' and the distance of point A against the tunnel face d_1 , d_2 , d_3 ... d_n are recorded (Figure 4.9). Such results are normalized with the tunnel radius, R_T and the maximum displacement, u_{max} . The analyzed results of point A are computed to perform the SLDP mapping analysis. The results are presented in Table 4.3. The SLDP for rock mass A is plotted with the best fitted curve in Figure 4.10 and the curve equation is determined.

Table 4.3 – Tunnel displacement data for superimposed LDP mapping analysis

Distance Away From Excavation Face, d (m)	Relative Displacement Value, u' (m)	Normalized Distance, (d/R_T)	Normalized Relative Displacement, (u'/U_{max})
5	0	2	0
6	0.059819	2.4	0.128589
7	0.113434	2.8	0.243844
8	0.16149	3.2	0.347146
9	0.204562	3.6	0.439736
10	0.243168	4.0	0.522724
11	0.27777	4.4	0.597107
12	0.308784	4.8	0.663776
13	0.336582	5.2	0.723531
14	0.361497	5.6	0.77709
15	0.383828	6.0	0.825094

Figure 4.9 – Tunnel excavation model

Figure 4.10 – Tunnel excavation stages

Figure 4.11 – The SLDP for point A

The superimposed LDP mapping begins with integrating this best fitted curve equation to determine ω .

$$\omega = \frac{dy}{dx} = A_1 \times \exp\left(\frac{-x}{t_1}\right) + y_0(dx) \quad (4.12)$$

which, x is the initial normalized distance, (d_0/R_T)

Through the relationship in equation 4.11, we can substitute ω and x into equation 4.10.

$$\omega = \frac{dy}{dx} = \frac{du^*}{dX^*} \quad (4.13)$$

and

$$x = \frac{d_0}{R_T} = X^*$$

From the example, the superimposed LDP of point A gives $\omega = 0.1352899681$ with $x = X^* = 2$. Solving equation (4.10), we can obtain $R^* = 5.459202951$. Substituting this R^* into equation (4.6), we can obtain $u_0^* = 0.146975006$. By comparing R^* and u_0^* achieved from the SLDP mapping technique with the $R^* = 5.48$ and $u_0^* = 0.1465$ obtained via the numerical plane strain and axisymmetric model analysis, it is shown that the SLDP mapping analysis can provide accurate displacement results at the tunnel excavation face.

The R^* and u_0^* obtained from the SLDP mapping analysis can be used together with equations (4.7) and (4.8) to construct the LDP proposed by Vlachopoulos and Diederichs (2009) and estimate the displacement at the tunnel face. Figure 4.11 shows the comparison of the LDP constructed from the SLDP analysis against the various LDP developed by other researchers. The results shows that the LDP constructed via the SLDP method matches the LDP constructed through the ILDP (Plastic) by Vlachopoulos and Diederichs (2009) and the

axisymmetric numerical analysis (plastic) results. However, it varies against the elastic LDPs from Panet (1995), Unlu and Gercek (2003) and Chern (1998). The tunnel face displacement estimated through the SLDP analysis, the ILDP by Vlachopoulos and Diederichs (2009) and the axisymmetric numerical analysis is 0.1752m where, the displacement calculated through the elastic LDPs from Panet (1995), Unlu and Gercek (2003) and Chern (1998) is 0.292m.

Figure 4.12 – The LDP comparison graph

The above accuracy achieved from example is based on the assumption that the convergence/displacement (i.e. u_1' , u_2' , $u_3' \dots u_n'$) and the distance of measurement point away from excavation face (i.e. d_1 , d_2 , $d_3 \dots d_n$) are correctly measured. An alternative approach to determine the displacement profile is to perform the analytical or numerical analysis. The final deformation, u_{max} , and the final plastic radius, R_p , of the tunnel can be used in equations (4.6), (4.7) and (4.8).

4.6 CONCLUSION

The proposed SLDP analysis provides engineers and researchers with quick estimates of the tunnel excavation face displacement and the tunnel displacement profile whereby complex analysis and time consuming computation is omitted. It

also has an advantage of showing the true reflection of the under construction tunnel via using the onsite measurement data. Furthermore, the tunnel displacement profile obtain through R^* and u_0^* achieved can be directly incorporated into convergence confinement analysis to determine the time and types of support installation, since the maximum displacement and maximum plastic zone radius are primary outputs of the analytical process. The convergence or normalized closure axis or analysis parameter can be converted using the other LDP analysis methods. However, the limitations of the traditional LPD remain in the analysis. This analysis inherits the debatable assumptions found in the traditional LDP analysis. They are i) the support application does not change the material response, ii) estimates of pressure-displacement curves can be compared to estimate the factor of safety against overload, iii) no plastic-volume change for rock mass, iv) circular cross section of the tunnel in combination with hydrostatic far-field stresses and v) the rock mass is treated as homogeneous material. Nevertheless, as discussed by various researchers such assumptions are valid for deep tunnel excavation and most of the time within the effective length of the LDP analysis to perform with no major faults present and change of rock mass properties. This analysis is very useful whereby timely displacement information is desired for operation and safety assessment in tunnel construction.

CHAPTER 5 – GEOMETRICAL ANALYSIS FOR THE STRAIN TO DISPLACEMENT TRANSFORMATION IN THE UNDERGROUND SPATIAL STRUCTURAL HEALTH MONITORING SYSTEM

5.1 INTRODUCTION

Preceding from the tunnel deformation analysis in the earlier chapters, it has highlighted the time dependent factors present in tunnel deformation analysis is essential and important to refine the analysis methodology. The LDP back analysis using tunnel convergence data and the time dependent analysis method proposed by Fortsakis and Kavvadas (2009) reduces the assumptions present in the time dependent effect in tunnel deformation analysis. However, such analysis method requires tremendous amount of field measurement data to be collected. Researchers face the ultimate challenge in acquiring such reliable spatial measurements. Thus development of an automated real-time long-term spatial structural health monitoring (SHM) system plays a crucial and significant role in performing such analysis.

On the other hand, such spatial SHM system has long been a desire system for engineers and scientists to discover and analyze the structural behavior of underground infrastructure in spatial domain. Besides, it plays a crucial and significant role for performing the time dependent back analysis in underground engineering. This chapter will include a summary describing the system framework of such automated real-time long-term spatial SHM system using the Brillouin Optical Time Domain Reflectometry (BOTDR) fiber optic sensors (FOS). It follows by the study focusing on performing the geometrical analysis to resolve the strain to displacement transformation computation. This analysis provides a solution to perform displacement or convergence analysis with strain measurements along the tunnel surface. Complementing the aim of developing an automated real-time long-term spatial SHM system, the equations derived will be computed in MATLAB for auto strain to displacement transformation. The proposed

strain to displacement transformation analysis will be tested with series of experiments. A structural frame is thus designed and fabricated to simulate a tunnel cross sectional profile. Experiments are conducted through monitoring the frame structural behavior with the BOTDR FOS, strain gauges and linear variable displacement transducers (LVDT) under various load conditions. Numerical models are established to analyze the frame response and introduce the correction factors under different loadings. The accuracy of the proposed strain to displacement transformation analysis will be evaluated through comparison study of the numerical simulation results against the experimental results. This proposed development will fulfill the aim of transforming strain captured along the tunnel wall to relevant displacement data. Thus, it facilitates the development of an automated real-time long-term spatial SHM system for tunnel monitoring.

5.2 THE STRUCTURAL HEALTH MONITORING (SHM) SYSTEM AND FIBER OPTIC SENSOR (FOS)

It is described by Bernard (2000) that SHM is a process designed to provide accurate and in-time information relating to structural performance and condition. It is necessary to increase knowledge and monitor the real-time onsite structural behavior for long-term to control the design, verify the performance, create and calibrate numerical models for analysis. However, it is difficult and not cost economical to achieve spatial measurand with normal point sensing instrumentation.

Fiber optic sensors (FOSs) are essentially a medium whereby light guided within an optical fiber can be modified in response to an external physical, chemical, biological, biomedical or similar influence (John and Brian, 1988). FOSs have been under research for over three decades and a wide body of knowledge has been accumulated. FOSs can measure tens of different parameters, such as temperature, strain, pressure, acoustic field, flow rate, electric field, magnetic field, current and voltage. FOSs have the advantages of small size, low weight, and immunity from electromagnetic interference, chemical passivity and capability of distant measurement.

These make them a good candidate for applications that are inaccessible using alternative technologies, such as down-hole oil well temperature and pressure monitoring, current and voltage measurements in highly electromagnetic interfered environments, chemical constituents in the blood of patients undergoing surgical treatment, etc.

Therefore, in the recent years, FOSs have made a slow but momentous entrance into the sensor panorama. Among those, the BOTDR technology has a unique advantage that it can measure continuous distributed strain or temperature along several tens of kilometers through accessing only a single optical fiber cable. This single optical fiber is used as a linear uninterrupted sensor, providing numerous measurement points spatially continuously distributed over the whole fiber length. Shi (2004) has commended that, such capability is extremely useful for health monitoring and diagnosis of long and large infrastructures.

There are two main drives for distributed FOS real time SHM system research. The first is to investigate the possibility to obtain spatial distributed measurement. The success in obtaining spatial measurement can place prominent achievement to aid researchers in obtaining more field measurement data to improve their analysis methodologies. The second is to lower cost per-sensor so that BOTDR FOS could hopefully become price-competitive against electrical sensors in order to gain widespread acceptance. The details and advantages of applying such technology in underground SHM will be discussed in the next chapter.

5.3 THE REAL-TIME LONG-TERM SPATIAL STRUCTURAL HEALTH MONITORING (SHM) SYSTEM

The general system framework of an ideal real time spatial SHM system is divided into three components, i) the spatial data acquisition system, ii) the wireless network relay and iii) the monitoring system and data base. Figure 5.1 shows a schematic of the ideal system framework.

Figure 5.1 – The schematic framework of an ideal real time spatial SHM system

Single point sensors seldom utilize the very high intrinsic bandwidth of the optical fiber transmission medium. When a large number of points are to be measured, many fibers must be used to connect the sensor heads and the terminal electronics. This single point sensing FOSs limited the advantages of the fiber transmission medium for spatial SHM system development. Therefore, there is an opportunity to save cost by multiplexing a number of sensing elements onto a single fiber or fiber pair, thus forming a fiber-optic sensor network distributed in space. The benefits are not simply sharing terminal equipment, but simplified wiring and reduced installation cost as well. In the case of the York DTS-800 (released in 1996), 10,000 points are measured simultaneously on a single optical fiber, and six fibers can be addressed by a single instrument, increasing the number of measurement points that are accessible by a single instrument to 60,000. The cost of the measurement is dominated by the cost of the fiber and its installation (James J. Smolen and Alex van der Spek, 2003).

Obtaining spatial distribution of a measurement is important in a number of applications. Distributed strain sensors is suggested to be an ideal solution to used for

health monitoring of in civil structures, such as bridges, dams and buildings. Distributed pressure sensors find their niche in down-hole oil well and undersea acoustic array applications and the distributed temperature sensors can also be incorporated into a building as a fire-alarm system. As such, the proposed development of the spatial data acquisition system will be using the distributed FOS, the BOTDR technology. Thus, the advantages and limitations of the BOTDR technology and the details on the BOTDR sensor development will be covered in Chapter 6 of this thesis.

However, after the spatial strain measurement is performed, it is of paramount importance to analyze the measurement correctly. Deformation analysis should be conducted based on the nature of the structure and recorded in the database. Although, there is available computational method to transform the BOTDR FOS signature to strain measurement, but it is still necessary and important to develop methodology and computational algorithm to transform those strain measurement to displacement information of the structure. This process is part of the interrogative program in Figure 5.1. The following sections will perform a geometrical analysis for the strain to displacement transformation trying to resolve such issue. The formulas derived are programmed with computer software for auto strain to displacement transformation and will be evaluated with laboratory experiments.

5.4 THE GEOMETRICAL STRAIN TO DISPLACEMENT TRANSFORMATION ANALYSIS

5.4.1 Tunnel Displacement Analysis

One major concern for the construction of tunnels or caverns is the convergence or deformation. Very often, the designed dimension for the tunnels and caverns is highly related to its future operations, but it is also known that some form of deformation would inevitably occur as the soil around the excavation settles. Furthermore, any excessive deformation will cause cracks or misalignment of the structural components thus setting off a chain of other issues like water seepage and corrosion. Therefore, monitoring of the deformation right after excavation is essential.

Currently, the conventional strain gauges are applied on a convergence tape connecting two points of the underground tunnel to measure the relative displacement of the structure (refer to Figure 5.2). But this way of measurement will not only destroy the aesthetics, but also disturb the passage way. The measurement results may not be accurate due to the underground ventilation. This study proposes to use the spatial BOTDR sensing system together with the strain to displacement transformation to solve the above problems. The optical fiber shall be installed to the surface of the underground tunnel at selected points along the tunnel wall (Figure 5.3). The relative strain of the tunnel wall can be measured by the BOTDR sensors.

Figure 5.2 – Cavern monitoring using conventional strain gauges

Figure 5.3 – Tunnel/Cavern monitoring using BOTDR sensors

Figure 5.3 shows that strain of the inner arch of the tunnel can be obtained by BOTDR sensors. The fiber optic sensing cable will be fixed at selected points which will segmented the tunnel surfaced as close as a straight line as possible. The related analytical model will be developed to transform the strain measurements to tunnel face displacement data. Since the arch model is relatively complex, the straight beam model is discussed first.

5.4.1.1 Straight Beam System

Besides to set as the based for analysis of the arch system, this straight beam system analysis can be used to transform the longitudinal section of the BOTDR FOS strain measurement to the tunnel wall displacement. As shown in Figure 5.4 (a), the length of the simply supported beam is L . The height of the beam is h . Here we assume that the fiber is installed closely intact with the face of the beam and the thickness of the fiber is too small and has no influence in the analysis. The strain at point A of the BOTDR cable is ε . The original point O is at the center of the beam. In Figure 5.4(b), ABCD is the rectangle before beam deformation while AGHD is the trapezoid after deformation. The neutral axis EF whose length is dx has no deformation. O is the center

of the circle after deformation.

Figure 5.4(a) – Simply supported beam

Figure 5.4(b) – Deformation of cross section of differential element at point A

According to the principle of similar triangle

$$\frac{DH}{AG} = \frac{O'D}{O'A} \text{ (For } \Delta O'GA \text{)}$$

So

$$\frac{(1-\varepsilon)dx}{(1+\varepsilon)dx} = \frac{R}{R+h} \quad (5.1)$$

where R is the length of $O'D$. And then we can obtain

$$R = \frac{1-\varepsilon}{2\varepsilon} h \quad (5.2)$$

From the formula of curvature $\frac{\frac{d^2 y}{dx^2}}{\left[1+\left(\frac{dy}{dx}\right)^2\right]^{\frac{3}{2}}} = \frac{1}{\rho}$, where ρ is the radius of curvature, we

can obtain the formula of curvature of neutral layer EF

$$\frac{d^2 y}{dx^2} = \frac{1}{R + \frac{h}{2}} \quad (5.3)$$

where $\frac{dy}{dx} \ll 1$, for small deformation. Substituting equation 5.2 into equation 5.3, we obtain

$$\frac{d^2 y}{dx^2} = \frac{2\varepsilon}{h} \quad (5.4)$$

Integrating equation 5.4, the deflection of the beam can be obtained as

$$y = \frac{\varepsilon}{h} x^2 + C_1 x + C_2 \quad (5.5)$$

where C_1 and C_2 are integration constants. The boundary conditions are $y = 0$ when

$x = \pm \frac{L}{2}$, so

$$\begin{aligned} \frac{\varepsilon}{h} \cdot \frac{L^2}{4} + C_1 \cdot \frac{L}{2} + C_2 &= 0 \\ \frac{\varepsilon}{h} \cdot \frac{L^2}{4} - C_1 \cdot \frac{L}{2} + C_2 &= 0 \end{aligned} \quad (5.6)$$

We can obtain

$$C_1 = 0 \& C_2 = -\frac{\varepsilon \cdot L^2}{4h} \quad (5.7)$$

Finally, the deflection can be expressed in terms of strain as

$$y = \frac{\varepsilon}{h} x^2 - \frac{\varepsilon \cdot L^2}{4h} \quad (5.8)$$

5.4.1.2 Semicircular Structure System

a) Single Element Sensor System

This semicircular structure system analysis is performed to solve the strain to displacement transformation of the cross sectional strain measurement acquired by the proposed spatial SHM system. As shown in Figure 5.5, the radius of the top surface of the semicircular structure is $R + h$, where h is the thickness of the structure and R is the radius of the bottom surface. The surface strain ε on the bottom surface which is located at x_0 (at the angle of θ_0) is known. The radial displacement can be derived by geometric relations.

Figure 5.5 – Semicircular Structure

Figure 5.6 is the differential element of the structure at x_0 . The arc BF is the neutral layer of the element. ADGC is the original element before deformation while AEHC is the deformed element.

Figure 5.6 – Differential element of the structure

$$AD = R d\theta \text{ and } CG = (R + h) d\theta$$

$$AE = (1 + \varepsilon)Rd\theta$$

$$CH = (1 - \varepsilon)(R + h)d\theta$$

$$\frac{AE}{CH} = \frac{O'A}{O'C} \text{ (for differential element O'HC), then}$$

$$\frac{(1 + \varepsilon)Rd\theta}{(1 - \varepsilon)(R + h)d\theta} = \frac{R + O'O}{R + h + O'O} \quad (5.9)$$

Therefore we can obtain the radius of curvature after deformation

$$\rho = OO' + R + \frac{h}{2} = \frac{2R + (1 - \varepsilon)h}{-2\varepsilon R + (1 - \varepsilon)h} \cdot \frac{h}{2} \quad (5.10)$$

As shown in Figure 5.7, we set the semicircular as the θ axis and the radial direction of the semicircular as the v axis.

Figure 5.7 – The axis system after transformation

As mentioned previously, the formula of curvature is

$$\frac{d^2 y}{dx^2} = \frac{1}{\rho} \text{ (for small deformation)} \quad (5.11)$$

In the system shown in Figure 5.7, we can obtain the change of curvature

$$\Delta\left(\frac{1}{\rho}\right) = \frac{-2\varepsilon R + (1-\varepsilon)h}{2R + (1-\varepsilon)h} \cdot \frac{2}{h} - \frac{1}{R + \frac{h}{2}} \quad (5.12)$$

and then

$$\begin{aligned} \frac{d^2 v}{ds^2} \frac{d^2 v}{d\left[\left(R + \frac{h}{2}\right)\theta\right]^2} &= -\Delta\left(\frac{1}{\rho}\right) = -\left[\frac{-2\varepsilon R + (1-\varepsilon)h}{2R + (1-\varepsilon)h} \cdot \frac{2}{h} - \frac{1}{R + \frac{h}{2}}\right] \\ \frac{d^2 v}{d\theta^2} &= -\Delta\left(\frac{1}{\rho}\right)\left(R + \frac{h}{2}\right)^2 \end{aligned} \quad (5.13)$$

Integrating equation 5.13, the radial deflection of the beam can be obtained as

$$v = -\frac{1}{2}\Delta\left(\frac{1}{\rho}\right)\left(R + \frac{h}{2}\right)^2\theta^2 + C_1\theta + C_2 \quad (5.14)$$

where C_1 and C_2 are the integration constants. The boundary conditions are $v = 0$ when

$\theta = \pm \frac{\pi}{2}$, so

$$\begin{aligned} -\frac{1}{2}\Delta\left(\frac{1}{\rho}\right)\left(R + \frac{h}{2}\right)^2\left(\frac{\pi}{2}\right)^2 + C_1\left(\frac{\pi}{2}\right) + C_2 &= 0 \\ -\frac{1}{2}\Delta\left(\frac{1}{\rho}\right)\left(R + \frac{h}{2}\right)^2\left(\frac{\pi}{2}\right)^2 - C_1\left(\frac{\pi}{2}\right) + C_2 &= 0 \end{aligned} \quad (5.15)$$

From the equations above, we can obtain

$$C_1 = 0 \& C_2 = \frac{\pi^2}{8} \Delta \left(\frac{1}{\rho} \right) \left(R + \frac{h}{2} \right)^2 \quad (5.16)$$

Finally, the deflection can be expressed in terms of strain as

$$v = -\frac{1}{2} \Delta \left(\frac{1}{\rho} \right) \left(R + \frac{h}{2} \right)^2 \theta^2 + \frac{\pi^2}{8} \Delta \left(\frac{1}{\rho} \right) \left(R + \frac{h}{2} \right)^2 \quad (5.17)$$

$$\text{where } \Delta \left(\frac{1}{\rho} \right) = \frac{-2\varepsilon R + (1-\varepsilon)h}{2R + (1-\varepsilon)h} \cdot \frac{2}{h} - \frac{1}{R + \frac{h}{2}}$$

b) Four Sectioning System

As shown in Figure 5.8, the strains of AB , BC , CD and DE on the bottom surface are ε_1 , ε_2 , ε_3 and ε_4 . Points B, C and D are mounted to the cavern surface.

Figure 5.8 – Semicircular Structure with four fixing points of BOTDR sensors

From equation 5.17, we can obtain

$$v_1 = -\frac{1}{2}\Delta\left(\frac{1}{\rho_1}\right)\left(R + \frac{h}{2}\right)^2\theta^2 + C_1\theta + C_2 \text{ (for } AB \text{)} \quad (5.18)$$

$$v_2 = -\frac{1}{2}\Delta\left(\frac{1}{\rho_2}\right)\left(R + \frac{h}{2}\right)^2\theta^2 + C_3\theta + C_4 \text{ (for } BC \text{)} \quad (5.19)$$

$$v_3 = -\frac{1}{2}\Delta\left(\frac{1}{\rho_3}\right)\left(R + \frac{h}{2}\right)^2\theta^2 + C_5\theta + C_6 \text{ (for } CD \text{)} \quad (5.20)$$

$$v_4 = -\frac{1}{2}\Delta\left(\frac{1}{\rho_4}\right)\left(R + \frac{h}{2}\right)^2\theta^2 + C_7\theta + C_8 \text{ (for } DE \text{)} \quad (5.21)$$

and the boundary conditions are

$$v_1 = 0 \text{ when } \theta = \frac{\pi}{2}$$

$$v_1 = v_2, \frac{dv_1}{d\theta} = \frac{dv_2}{d\theta} \text{ when } \theta = \frac{\pi}{4}$$

$$v_2 = v_3, \frac{dv_2}{d\theta} = \frac{dv_3}{d\theta} \text{ when } \theta = 0$$

$$v_3 = v_4, \frac{dv_3}{d\theta} = \frac{dv_4}{d\theta} \text{ when } \theta = -\frac{\pi}{4}$$

$$v_4 = 0 \text{ when } \theta = -\frac{\pi}{2}$$

Substituting the equations above into equations (5.18 ~ 5.21), we can obtain

$$\begin{pmatrix} \frac{\pi}{2} & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 1 & 0 & -1 & 0 & 0 & 0 & 0 & 0 \\ \frac{\pi}{4} & 1 & -\frac{\pi}{4} & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & -1 & 0 & 0 \\ 0 & 0 & 1 & 0 & -1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -\frac{\pi}{4} & 1 & \frac{\pi}{4} & -1 \\ 0 & 0 & 0 & 0 & 1 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & -\frac{\pi}{2} & 1 \end{pmatrix} \begin{pmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \\ C_5 \\ C_6 \\ C_7 \\ C_8 \end{pmatrix} = \begin{pmatrix} -\frac{1}{8}\pi^2 f(\varepsilon_1) \\ \frac{\pi}{4} f(\varepsilon_2) - \frac{\pi}{4} f(\varepsilon_1) \\ \frac{1}{32}\pi^2 f(\varepsilon_2) - \frac{1}{32}\pi^2 f(\varepsilon_1) \\ 0 \\ 0 \\ \frac{1}{32}\pi^2 f(\varepsilon_4) - \frac{1}{32}\pi^2 f(\varepsilon_3) \\ \frac{\pi}{4} f(\varepsilon_3) - \frac{\pi}{4} f(\varepsilon_4) \\ -\frac{1}{8}\pi^2 f(\varepsilon_4) \end{pmatrix} \quad (5.23)$$

Finally, we can obtain

$$\begin{pmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \\ C_5 \\ C_6 \\ C_7 \\ C_8 \end{pmatrix} = \begin{pmatrix} \frac{\pi}{2} & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 1 & 0 & -1 & 0 & 0 & 0 & 0 & 0 \\ \frac{\pi}{4} & 1 & -\frac{\pi}{4} & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & -1 & 0 & 0 \\ 0 & 0 & 1 & 0 & -1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -\frac{\pi}{4} & 1 & \frac{\pi}{4} & -1 \\ 0 & 0 & 0 & 0 & 1 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & -\frac{\pi}{2} & 1 \end{pmatrix}^{-1} \begin{pmatrix} -\frac{1}{8}\pi^2 f(\varepsilon_1) \\ \frac{\pi}{4} f(\varepsilon_2) - \frac{\pi}{4} f(\varepsilon_1) \\ \frac{1}{32}\pi^2 f(\varepsilon_2) - \frac{1}{32}\pi^2 f(\varepsilon_1) \\ 0 \\ 0 \\ \frac{1}{32}\pi^2 f(\varepsilon_4) - \frac{1}{32}\pi^2 f(\varepsilon_3) \\ \frac{\pi}{4} f(\varepsilon_3) - \frac{\pi}{4} f(\varepsilon_4) \\ -\frac{1}{8}\pi^2 f(\varepsilon_4) \end{pmatrix} \quad (5.24)$$

$$\text{where } f(\varepsilon_i) = -\frac{2\varepsilon_i R + (1 - \varepsilon_i)h}{2R + (1 - \varepsilon_i)h} \cdot \frac{2}{h} \left(R + \frac{h}{2}\right)^2 + R + \frac{h}{2} \quad (5.25)$$

c) N Fixing Points System

For the system of n sensors, the method of calculation is similar. The strains are ε_1 , $\varepsilon_2 \dots \varepsilon_n$ separately. So we can obtain

$$\begin{aligned}
 v_1 &= \frac{1}{2} f(\varepsilon_1) \theta^2 + A_1 \theta + B_1 \\
 v_2 &= \frac{1}{2} f(\varepsilon_2) \theta^2 + A_2 \theta + B_2 \\
 &\vdots \\
 v_n &= \frac{1}{2} f(\varepsilon_n) \theta^2 + A_n \theta + B_n
 \end{aligned} \tag{5.26}$$

And the boundary conditions are

$$\begin{aligned}
 v_1 &= 0 \text{ when } \theta = \frac{\pi}{2} \\
 v_1 &= v_2 \text{ and } \frac{dv_1}{d\theta} = \frac{dv_2}{d\theta} \text{ when } \theta = \frac{n-2}{2n} \pi \\
 &\vdots \\
 v_{m-1} &= v_m \text{ and } \frac{dv_{m-1}}{d\theta} = \frac{dv_m}{d\theta} \text{ when } \theta = \frac{n-2m+2}{2n} \pi \\
 &\vdots \\
 v_{n-1} &= v_n \text{ and } \frac{dv_{n-1}}{d\theta} = \frac{dv_n}{d\theta} \text{ when } \theta = \frac{-n+2}{2n} \pi \\
 v_n &= 0 \text{ when } \theta = -\frac{\pi}{2}
 \end{aligned}$$

We can express all the equations by matrices

(5.27)

$$\begin{aligned} A_{i,j}(i=j+1) &= \frac{n-2i+2}{2n} \pi \\ \text{when } i, j \leq n, \quad A_{i,j}(i=j \neq 1) &= \frac{-n+2i-2}{2n} \pi \\ A_{11} &= \frac{\pi}{2} \\ \text{else, } A_{i,j} &= 0 \end{aligned} \quad (5.28)$$

$$\begin{aligned} \text{when } i, j > n, \quad & A_{i,j}(i = j = 2n) = 1 \\ & \text{else, } A_{i,j} = 0 \end{aligned} \quad (5.29)$$

$$\begin{aligned} & A_{i,j}(i = j + n \neq 2n) = 1 \\ & A_{i,j}(i = j + n - 1) = -1 \\ \text{when } i > n \text{ and } j \leq n, \quad & A_{2n,n} = -\frac{\pi}{2} \\ & \text{else, } A_{i,j} = 0 \end{aligned} \quad (5.30)$$

$$\begin{aligned} & A_{i,j}(i = j - n \neq 1) = -1 \\ & A_{i,j}(i = j - n + 1) = 1 \\ \text{when } i \leq n \text{ and } j > n, \quad & A_{1,n+1} = 1 \\ & \text{else, } A_{i,j} = 0 \end{aligned} \quad (5.31)$$

According to the calculation rule of matrixes, we can obtain

$$A_{i,j} = A_{i,j} + \left[-\frac{n+2-2i}{2n} \pi \right] \cdot A_{i+n-1,j} \quad (2 \leq i \leq n), \text{ then}$$

(5.32)

We can also get $A_{2,j} = A_{2,j} + \sum_{i=3}^n A_{i,j}$ and $A_{n+1,j} = A_{n+1,j} + \sum_{i=n+2}^{2n-1} A_{i,j}$, therefore

$$\begin{pmatrix} \frac{\pi}{2} & & & \\ & 1 & & \\ & 1 & \ddots & \\ & & \ddots & -1 \\ & & & 1 & -1 \\ & 1 & & & \\ & \ddots & \ddots & \ddots & \\ & & \ddots & \ddots & 1 & -1 \\ & & & & & 1 & -1 \\ & & & & & & -\frac{\pi}{2} \end{pmatrix} \begin{pmatrix} A_1 \\ A_2 \\ \vdots \\ A_i \\ \vdots \\ A_n \\ B_1 \\ \vdots \\ \vdots \\ B_{n-1} \\ B_n \end{pmatrix} = \begin{pmatrix} -\frac{1}{8}\pi^2 f(\varepsilon_1) \\ \vdots \\ -\frac{1}{2}[f(\varepsilon_i)-f(\varepsilon_{i-1})]\left(\frac{n+2-2i}{2n}\pi\right)^2 \\ \vdots \\ -\frac{1}{2}[f(\varepsilon_n)-f(\varepsilon_{n-1})]\left(\frac{2-n}{2n}\pi\right)^2 \\ \sum_{k=1}^{n-1}[f(\varepsilon_{k+1})-f(\varepsilon_k)]\left(\frac{n-2k}{2n}\pi\right) \\ \vdots \\ [f(\varepsilon_{k+1})-f(\varepsilon_k)]\left(\frac{n-2k}{2n}\pi\right) \\ \vdots \\ [f(\varepsilon_n)-f(\varepsilon_{n-1})]\left(\frac{2-n}{2n}\pi\right) \\ -\frac{1}{8}\pi^2 f(\varepsilon_n) \end{pmatrix} \quad (5.33)$$

From the equation above, we can obtain

$$\begin{pmatrix} \frac{\pi}{2} & 0 & 1 & 0 \\ 0 & 0 & 1 & -1 \\ 1 & -1 & 0 & 0 \\ 0 & -\frac{\pi}{2} & 0 & 1 \end{pmatrix} \begin{pmatrix} A_1 \\ A_n \\ B_1 \\ B_n \end{pmatrix} = \begin{pmatrix} -\frac{1}{8}\pi^2 f(\varepsilon_1) \\ \sum_{i=2}^n -\frac{1}{2} [f(\varepsilon_i) - f(\varepsilon_{i-1})] \left(\frac{n+2-2i}{2n} \pi \right)^2 \\ \sum_{k=1}^{n-1} [f(\varepsilon_{k+1}) - f(\varepsilon_k)] \left(\frac{n-2k}{2n} \pi \right) \\ -\frac{1}{8}\pi^2 f(\varepsilon_n) \end{pmatrix} \quad (5.34)$$

Finally, we can obtain

$$A_1 = \frac{\pi}{8} [f(\varepsilon_n) - f(\varepsilon_1)] + \frac{\pi^2}{8} \sum_{i=2}^n [f(\varepsilon_i) - f(\varepsilon_{i-1})] \left[\frac{(n+2-2i)(3n+2-2i)}{n^2} \right] \quad (5.35)$$

$$A_k = A_1 - \sum_{i=2}^k [f(\varepsilon_i) - f(\varepsilon_{i-1})] \left(\frac{n+2-2i}{2n} \pi \right) \quad (5.36)$$

$$B_1 = -\frac{\pi}{2} A_1 - \frac{1}{8} \pi^2 f(\varepsilon_1) \quad (5.37)$$

$$B_k = -\frac{\pi}{2} A_1 - \frac{1}{8} \pi^2 f(\varepsilon_1) + \frac{\pi^2}{8} \sum_{i=2}^k [f(\varepsilon_i) - f(\varepsilon_{i-1})] \left(\frac{n+2-2i}{n} \right)^2 \quad (5.38)$$

where

$$f(\varepsilon_i) = -\frac{-2\varepsilon_i R + (1-\varepsilon_i)h}{2R + (1-\varepsilon_i)h} \cdot \frac{2}{h} \left(R + \frac{h}{2} \right)^2 + R + \frac{h}{2} \quad (5.39)$$

5.5 NUMERICAL EVALUATION OF THE STRAIN TO DISPLACEMENT TRANSFORMATION ANALYSIS

5.5.1 Numerical Model Simulation

The above geometrical analysis provides an analytical solution for the strain to displacement transformation in a semicircular cross section. To incorporate the solution with similar cross section or rather the horse-shoe shape cross section sometimes found in underground tunnels and caverns, numerical analysis is performed below to determine the possible correction factor. This strain to displacement transformation evaluation process involves numerical simulation modeling of the semicircular structure and the experimental frame structure in the laboratory. Correlating the strain

and displacement results with a 3-dimensional model in Ansys Workbench. The results will be compared to determine the percentage of error or correction factor needed for the strain to displacement transformation.

5.5.2 Preliminary Structural Analysis

Three numerical models are constructed and analyzed with Ansys Workbench in this numerical analysis evaluation process. The first model is a semicircular structure (Figure 5.9), which can provide the based reference for result comparison. The second model is an arch frame (Figure 5.10) and the third model is a horse-shoe shape structure (Figure 5.11) which is of exact dimension of the specimen tested in the experiment. The crown deformation is usually the most important for tunnel deformation analysis, therefore, the displacement results of point A (the Crown Point) of these three numerical models will be compared and correlated to determine the correction factors in these geometrical changes.

Figure 5.9 – The numerical model of a semicircular structure

Figure 5.10 – The numerical model of a arch frame structure

Figure 5.11 – The numerical model of a horse-shoe shape structure

5.5.2.1 Analysis Procedures:

- The three numerical models are built using Ansys Workbench. The dimension of the semicircular structure and the arch frame structure follows the dimension

of the horse-shoe shape frame structure shown in Figure 5.12. All three numerical models share the same material properties following the details of a universal beam (UB 152 x 152 x 30 kg/m).

Figure 5.12 – Dimension of the numerical model of the horse-shoe shape frame structure

- b) Numerical simulations are performed with various load cases to provide total displacement information for analysis.

The subsequent purpose of performing this analysis is to determine the load that the horse-shoe shape frame structure can withstand before failure before experimental testing. The maximum load is analysed to be 55kN. Thus, the maximum load that is

being applied to the frame structure specimen for the later experiment is 40kN.

5.5.2.2 Numerical Simulation Results and Analysis

The displacement information at point A of the three simulation models are captured (Table 5.1) and used to compute the correlation factor among the models (Figure 5.13).

Table 5.1 – Displacement information of Point A in the three numerical models

Load (kN)	Displacement (m) –Ansys Simulation Results		
	Semicircular Structure (δ_c)	Arch Frame Structure (δ_{arc})	Horse-shoe Shape Frame (δ_{hs})
0	0	0	0
5	-0.000105	-0.000113	-0.000282
10	-0.000211	-0.000226	-0.000565
15	-0.000324	-0.000339	-0.000847
20	-0.000437	-0.000452	-0.00113
25	-0.00055	-0.000565	-0.001412
30	-0.000663	-0.000678	-0.001695
35	-0.000776	-0.000791	-0.001977
40	-0.000889	-0.000904	-0.00226

Figure 5.13 – α_1 & α_2 , Correlation factor among the numerical models

The displacement information at point A is present in Figure 5.14, the results shows that all three structures are deformed linearly within the elastic region. The semicircular and the arch frame structure have almost identical displacement. While comparing the results from semicircular structure against the arch frame structure and later the arch frame structure against the horse-shoe shape frame, the value of α_1 and α_2 in their elastic deformation range can be determined as:

$$\delta_{\text{arc}} = \delta_c - \alpha_1 \quad (5.17)$$

and,

$$\alpha_1 = (\delta_{\text{arci}} - \delta_{\text{ci}}) = 0.0000149$$

Similarly,

$$\delta_{hs} = \delta_{arc} \times \alpha_2 \quad (5.18)$$

and,

$$\alpha_2 = (\delta_{hsi} / \delta_{arci}) \approx 2.5$$

Figure 5.14 –Displacement information of Point A

From the above relationship, we can deduced that,

$$\delta_{hs} = \alpha_2 \times (\delta_c - \alpha_1) \quad (5.19)$$

5.6 EXPERIMENTAL VALIDATION OF THE STRAIN TO DISPLACEMENT TRANSFORMATION ANALYSIS

The experiments will focus on the strain and displacement experienced by the specimens under different cases of loadings, which the measurement achieved will be used to compare with the numerical analysis results to validate the above geometrical analysis. The frame specimens will be linked and monitored by optic fibers, strain

gauges and linear variable displacement transducers (LVDT).

The underlying principle of this experiment is that, by installing sensors to a structure, vital real time data of the structure under static loads are obtained and transmitted from the site to the monitoring stations. The interrogating program will then collect the strain data and perform the displacement analysis. All changes in strain, displacement or perhaps, other static parameters of the underground tunnel can then be detected immediately and thus catastrophic failure of the structure can be forestalled.

Since the proposed system is targeted to installed and diagnosed displacement profile of an underground tunnel, the laboratory experimental specimen is therefore designed and fabricated. It follows closely to the shape and proportion of an actual underground tunnel. As seen from the pictures of the underground tunnel (Figure 5.15), the shape of the cavern is arched and curved at the top; hence the specimen is designed to be a horse-shoe shape frame structure, resembling the actual tunnel as closely as possible. The strain measured by BOTDR FOS will be compared with the strain gauge measurements. After that, they are transformed to displacement values, which will be compared with the LVDT measurements and numerical results. Correlation factors determined in the previous section will be applied to check the accuracy of the displacement deduced from the strain to displacement transformation.

Figure 5.15 – An actual site condition of underground tunnel

5.6.1 The Compression Test on Tunnel Cross-section Simulated Structural Frame Element

Since the proposed installation method is targeted to install and monitor the strain profile of an underground tunnel. The laboratory experimental specimen is designed and fabricated closely following the shape and proportion of an actual underground tunnel. The specimen is designed to be a horse-shoe shape frame structure, resembling the actual tunnel as closely as possible. The strain measured by BOTDR FOS will be compared with the strain gauge measurements and numerical analysis results. Correlation factors will be determined if necessary

The BOTDR FOS is clamped onto the specimen in the tangential direction to the radius of the specimen as shown in the experimental setup (Figure 5.16).

5.6.1.1 Specimen Fabrication and Experimental Setup

Numerical model of the structural frame is constructed to analysis the frame behaviour under various loadings. To reduce the assumptions made in the computation and analysis, the frame structure will be fabricated and modelled using Universal Beam (UB) 152 x 152 x 30 kg/m in 7 segments welded to form a frame with base 3000mm centre to centre and height of 2836mm.

The critical point for evaluation (Point D) is located right under the hydraulic load cell at the centre of the top flange. The displacement of this point is monitored with a LVDT. The hydraulic cell is placed at the mid-point on the top flange of the frame structure for applying vertical loading (Figure 5.16).

5.6.1.2 Setup of BOTDR FOS

The schematic drawing of the BOTDR fiber optic sensing cable installation is

shown in Figure 5.16. The BOTDR sensor is divided into six segments with a total of seven clamping points (A-G). Twenty-four strain gauges are attached along the specimen to monitor the strain profile.

Figure 5.16 – Experiment setup and schematic drawing of the frame

5.6.1.3 Experiment Procedures

Vertical loading is applied at the top of the frame structure using a hydraulic loading cell. The loading is incremented at an interval of 10kN from 0 to 40kN. Measurements from the strain gauges, LDVTs and the BOTDR FOS are recorded accordingly at each interval. The BOTDR FOS and strain gauges measurements are compared against the numerical simulation results.

5.6.1.4 Results and Discussions

The strain measurements recorded by the BOTDR FOS sensing system are separated through clapping and the pretension applied with individual segment. Since, the frame is geometrically symmetrical with vertical loading in the centre of axis, the measurement obtain on both side of the frame are identical. Generally, the averaged strain recorded from the BOTDR FOS sensing system are reasonably consistent and

have good agreement with the strain gauges measurement and the numerical analysis results, especially segment A-B and F-G. Figure 5.17 shows the comparison results against the strain gauge measurement and numerical simulation results of point 4 located along segment A-B.

The measurement at the crown, point 12 along segment C-D is the critical point of analysis in this experiment. The initial measured BOTDR strain results are lower as compared to the strain gauges measurements which match the numerical simulation results (Figure 5.17 and 5.18). This is due to the angle of $\pm 55^\circ$ formed by the arch of the crown (Figure 5.19). However, the overall results obtained from the BOTDR FOS agree with the numerical simulation results and the strain gauges measurements.

Through geometrical conversion, a correction factor (equation 5.20) is introduced to transform the BOTDR measured strain to compare with the strain gauges measurements and numerical results.

Figure 5.17 – Micro-strain vs Loading (A-B)

Figure 5.18 – Micro-strain vs Loading (C-D)

Figure 5.19 – Geometrical conversion for BOTDR strain measurements

Measured Strain x Correction Factor = Actual Strain

$$\epsilon_{\text{BOTDR}} \times \tan(55^\circ) = \epsilon_{\text{actual}} \quad (5.20)$$

which, correction factor = $\tan(\pm 55^\circ)$

The corrected strain measurements (Figure 5.20) shown that the BOTDR FOS measurements correlate very well with the strain gauges measurement and numerical analysis results.

Figure 5.20 – Corrected Micro-strain vs Loading segment C-D at point 12

5.6.1.5 Displacement Analysis

The results obtained from the LVDT with reference to the displacement vs. load for the different locations of the specimen are analysed. Assuming that it is negative displacement when the top portion of the specimen is being displaced inwards (interpreting from the negative readings) while the right and left side of the specimen are being displaced outwards (interpreting from the positive readings) (Figure 5.21). From the experimental results (Table 5.2), it is observed that the specimen behaves closely according to the numerical analysis (Table 5.3) when it is under uniaxial compressive loadings on the top of the frame. It can be concluded that the LVDT readings are reliable.

Figure 5.21 – Deformation shape of numerical analysis

Table 5.2 – Experimental Results - Displacement of LDVT under different loadings

Load (kN)	Displacement (m) - LVDTs		
	Point 4 (LVDT 1)	Point 13 (LVDT 2)	Point 21(LVDT 3)
0	0	0	0
5	-0.00024	-0.00028	-0.00025
10	-0.00018	-0.00057	-0.00016
15	-0.00004	-0.00089	-0.00004
20	0.000055	-0.00113	0.00005
25	0.0002	-0.00144	0.00019
30	0.00032	-0.00172	0.00033
35	0.00044	-0.00202	0.00044
40	0.00055	-0.00231	0.00055

Table 5.3 – Numerical Results - Displacement of LDVT located point under different loadings

Load (kN)	Displacement (m) -Ansys		
	Point 4 (Right Side)	Point 13(Top)	Point 21 (Left Side)
0	0	0	0
5	0.00009415	-0.00028246	0.00009415
10	0.00018831	-0.0005649	0.00018831
15	0.0002846	-0.00084739	0.0002846
20	0.00037662	-0.0011299	0.00037662
25	0.00047077	-0.0014123	0.00047077
30	0.00056493	-0.0016948	0.00056493
35	0.00065908	-0.0019772	0.00065908
40	0.00075323	-0.0022597	0.00075323

Figure 5.22 shows the displacement results obtained from the experiment (LVDT) and Ansys. The results for displacement are symmetrical and similar for the right and left side of the specimen. Theoretically, the results should be as reflected, since the only load applied from the top of the frame structure at its mid-point and displacement will be symmetrical about the centre of the frame structure. The experimental results are slightly different from the numerical results due to experimental factors and other external conditions. It is important that both experimental and numerical results match closely with each other on the whole, especially the results obtained for deformation at the top face of the frame structure. Thus, it can be concluded that the specimen provides reliable results under compressive loading in the experiment and the experimental LVDT results of the crown shall be used to determine the correction factor or using BOTDR FOS strain to displacement conversion.

Figure 5.22 – Comparison of deformation results between Experiment & Ansys under different loadings

5.6.1.6 Strain to Displacement Transformation Analysis

The formulas derived in Section 5.4 are computed with computer software (Figure 5.23) for the automatic strain to displacement transformation computation. The table below (Table 5.4) shows the transformed displacement at point 12 along C-D and point 13 along D-E from the BOTDR FOS measured strain.

The image shows a MATLAB Command Window and Command History. The Command Window displays the following code and results:

```

157.6000

>> for i=1:size(e,1)
    for j=1:size(e,2)
        f(i,j)=R+h/2+(2*e(i,j)*R-(1-e(i,j))*h)*2*(R+h/2)^2/(2*R+(1-e(i,j))*h)/h
    end
end

f =

-2.6597e+005

f =

1.0e+005 *

-2.6597 -3.5149

f =

1.0e+005 *

-2.6597 -3.5149 -2.0460

f =

1.0e+006 *

-0.2660 -0.3515 -0.2046 -1.3226

```

The Command History shows the following commands:

```

f(i,j)=pi^2*(f(i,j)-f(i,1))/8
F(i,2)=pi^2*(f(i,2)-f(i,1))
F(i,3)=pi^2*(f(i,3)-f(i,2))
F(i,4)=0
F(i,5)=pi^2*(f(i,5)-f(i,4))
F(i,6)=pi^2*(f(i,6)-f(i,5))
F(i,7)=pi*(f(i,2)-f(i,1))
F(i,8)=pi*(f(i,3)-f(i,2))
F(i,9)=0
F(i,10)=-pi*(f(i,5)-f(i,4))
F(i,11)=-pi*(f(i,6)-f(i,5))
F(i,12)=-pi^2*f(i,6)/8
end
C=F'
M=[ pi/2 0 0 0 0 0 1 0 0 0
    pi/3 -pi/3 0 0 0 0 1 -1 0
    0 pi/6 -pi/6 0 0 0 0 1 -1
    0 0 0 0 0 0 0 1 -1 0
    0 0 0 -pi/6 pi/6 0 0 0 0 1
    0 0 0 0 -pi/3 pi/3 0 0 0 0
    1 -1 0 0 0 0 0 0 0 0
    0 1 -1 0 0 0 0 0 0 0
    0 0 1 -1 0 0 0 0 0 0
    0 0 0 1 -1 0 0 0 0 0
    0 0 0 0 -pi/2 0 0 0 0 0
    ]
a=inv(M)*C
A=a'
angle=0
V3=1/2*angle^2.*f(:,3)+ang

```

Figure 5.23 – Computing the automate strain to displacement transformation program

Table 5.4 – Displacement information for Point 12 and Point 13 after strain to displacement transformation

Load (kN)	Displacement (m)	
	BOTDR Point 12	BOTDR Point 13
0	0	0
5	-0.000031089	-0.000032095
10	-0.00003949	-0.0000429
15	-0.0000442	-0.0000464
20	-0.000048053	-0.00005083
25	-0.000050432	-0.000052232
30	-0.000052204	-0.000055528
35	-0.000053484	-0.000056783
40	-0.000054536	-0.000058625

Same as the numerical analysis, correction factor α_1 has to apply to covert the theoretical analysis values from semicircular structure to arch frame structure (equation 5.27) for further comparison. The results are shown in Table 5.5.

Table 5.5 – Displacement information for Point 12 and Point 13 after correction with α_1

Load (kN)	Displacement (m)	
	BOTDR Point 12	BOTDR Point 13
0	0	0
5	-0.000050251	-0.000050287
10	-0.000056514	-0.000056548
15	-0.000062279	-0.000062298
20	-0.00006446	-0.000064469
25	-0.000067124	-0.000067131
30	-0.000068417	-0.000068422
35	-0.000069227	-0.000069232
40	-0.00007067	-0.000070674

Figure 5.24 shows the results are plotted against the LDVT measurements and the numerical results.

Figure 5.24 – Comparison of BOTDR vs LVDT and Numerical results.

Since the general growth of the displacement is constant. The difference in the measurement and the analysis results can be resolved by introducing the correction factor, α_3 , where α_3 varied along with the load change. For which,

$$\text{True Displacement} = (\delta_{\text{BOTDR}} - \alpha_1) \times \alpha_3$$

(5.21)

and

$$\alpha_3 = 0.77902 \times \text{Load} + 1.43062$$

Applying such correction factor, the corrected displacement is shown in Table 5.6. Through Figure 5.25, it shows that with the correction factor introduced, the displacements are very closely matched with the LVDTs measurements and the numerical simulation results.

Table 5.6 – Corrected BOTDR measured displacement

Load (kN)	Displacement (m)	
	BOTDR Point 12	BOTDR Point 13
0	0	0
5	-0.000050251	-0.000050287
10	-0.000056514	-0.000056548
15	-0.000062279	-0.000062298
20	-0.00006446	-0.000064469
25	-0.000067124	-0.000067131
30	-0.000068417	-0.000068422
35	-0.000069227	-0.000069232
40	-0.00007067	-0.000070674

Figure 5.25 – Comparison of Corrected BOTDR vs LVDT and Numerical results

5.7 CONCLUSION

This chapter had shown the strain to displacement transformation component of developing a real time spatial structural health monitoring system for underground tunnels. The key to achieve in this automated system is the analysis performed for the strain to displacement transformation matrix program developed. The numerical simulation and the experiment results evaluated and proved the validity of the transformation program. To conclude, the system developed and the geometrical analysis matrix shall work well with a semicircular cross section or circular cross section tunnel with no correction factor needs to be applied. However, it can also work with arch shape or horse shoe shape cross section with introducing a correction factor (α_1 , α_2 and α_3) after the transformation is performed. This proposed system can efficiently reduce the time, manpower and cost involved in conducting convergence surveying in tunnel and it is cost effective to perform as a long-term monitoring system. The high level of automation can significantly reduce the safety risk involved in underground engineering work.

CHAPTER 6 – EXPERIMENTAL STUDY ON CALIBRATION OF BOTDR STRAIN SENSOR FOR UNDERGROUND STRUCTURAL MONITORING

6.1 INTRODUCTION

Underground infrastructure developments are ubiquitous in every society, irrespective of geographical location, culture and economical development. Underground structures like tunnels and caverns can affect human, economical, cultural and aesthetic aspects of the society. Good design, excellent construction and safe development of the underground structures are the goals in the infrastructure pre-operation stage; long-term structural health monitoring (SHM) is then the key towards ensuring the sustainability of the underground infrastructural development.

SHM is a process designed to provide accurate and in-time information relating to structural performance and condition. New materials, new construction technologies and new structural systems are increasingly being used, thus it is necessary to increase knowledge about their on-site performance, to control the design, to verify performance, and to create and calibrate numerical models (Bernard, 2000).

In the past decade, distributed fiber optic sensors (FOSs) have made a gradual but momentous entrance into the sensor panorama (Dliscic and Inaudi, 2003). A single optical fiber is used as a linear uninterrupted sensor, providing numerous measurement points spatially and continuously distributed over the whole fiber length. This technology is attractive in monitoring of large scale infrastructures such as those used in oil and gas industry, where fiber optic sensors are sometimes the single existing solution for dependable and long-term physical parameter monitoring (Chang and Han, 2005). However, factors like quality of fiber core, polymer of fiber outer cover and method of installation could affect the accuracy of SHM measurements.

The success of FOSs inspires research and development to deploy FOS SHM in underground civil infrastructure which the structural monitoring technique is limited and heavily relies on the traditional methodology. A number of surface-mountable FOS sensor systems have been developed in the past years, among which the Brillouin Optical Time Domain Reflectometry (BOTDR) system has received much research attention in terms of its technology development and application. Different from the traditional monitoring instruments, the BOTDR system provides distributed, long distance, real-time, interference free and reliable measurement data. However, there is still a gap between the laboratory experiments and practical field applications. Limited research has been conducted on how to maximize its possible applications due to its brittle and fragile core material. This chapter will provide a summary on the BOTDR technique and discuss the advantages of applying BOTDR in underground SHM. This study proposes a solution to overcome the physical deficiency through comparing different protection packages of BOTDR sensors and calibrating two prototypes of BOTDR sensors developed that can be applied in underground SHM. The accuracy of the BOTDR sensors is verified by the strain gauges and linear variable displacement transducers (LVDT). This study will place prominence in strain measurement of underground structures, which may experience with unforeseen change of loadings or stress distribution in long term and also the sensor survivability in the underground conditions. With that, the convergence behavior of underground tunnels and caverns can be monitored. The underlying principle of this research is that, by installing sensors to the underground structure (Figure 6.1), vital real time data of the structure under various loading conditions are gathered and transmitted from the site to the monitoring stations for analysis.

The investigation conducted involves laboratory experiments on a steel column specimen with setups of different BOTDR sensors, the traditional strain gauges and linear variable displacement transducers (LVDT). Theoretical calculation results will be correlated and compared with the experimental measurement to determine the sensor that is more suitable for installation practically in the underground tunnels and caverns. The results of this research present valuable reference for advance exploration of

BOTDR SHM systems in the underground construction industry.

Figure 6.1 – Schematic diagram of an ideal BOTDR SHM layout in underground cavern

6.1.1 Monitoring Underground Structure

Underground tunnels, caverns and other underground infrastructures should be monitored throughout their whole functional lifespan. The choice of monitoring instrumentation, in addition to the frequency of monitoring, depends on the stage of the development, the geo-mechanical quality, the infrastructure type, the purpose of development, the estimated infrastructure operational life span and the tunneling method employed during construction (Hiroshi et al., 2005).

The most indicative parameter to monitor in the case of tunnels is convergence of the cross section as suggested by Glisie and Inaudi, (2007). Deformation monitoring can be performed independently of convergence monitoring and concentrated only on some particular points in the cross section. The points where maximum stresses are expected or the 'Key Block' (Vinchies and Cambou, 1989) identified.

In the past, visual inspection and convergence surveying method are the common practices of traditional monitoring methods for structural monitoring of underground tunnels. These methods may consider as highly ineffective. Since obtaining an accurate result is tricky, some of the minor damages are not clearly visible

to the human eyes and one could easily disregard that (Radojicic et al., 1999). Furthermore, the harsh environment in the constructing underground tunnels and caverns creates a challenge to the sensitivity and precision of the traditional point sensing structural monitoring instrumentation like total station, strain gauge, convergence tape and LVDT, they are not able to provide a mean for incessant large scale monitoring of the structures efficiently (Frangopol et al., 1998). Furthermore, the techniques mentioned above are of no or low level of automation and not applicable for inaccessible locations. These shortcomings lead to limited information achieved, a rise concerns in workers safety, high operation cost and time consuming, thus, a better solution – using of spatial fiber optic sensors is evolved.

6.2 MONITORING OF UNDERGROUND TUNNELS AND CAVERNS USING FOS

Buildings with integrated FOS are occasionally called ‘smart structures’, they allow one to monitor both the external and internal conditions to achieve imperative information on strain to which different parts of the structure are subject on aging phenomena, vibrations, etc. Smart structures are the foremost driver for the further development of optical fiber sensors.

All these bring about improvements in structural health monitoring – the usage of the optical fiber technology. The initiation is to improve the perception that, fiber optic sensors allow measurements which are either uneconomical or impractical with conventional methods such as resistive foil strain gauges (Glisie and Inaudi, 2007). The success in those FOS installed ‘smart structures’ and other FOS applications inspire the attempt and research of FOS SHM for underground structure supplication

The sensors installed on the walls and vaults of the tunnel in longitudinal and tangential direction can provide information for structural abnormal behavior detection location and localization for detection of local average strain changes as a result of damage. Taking into consideration the number of monitoring parameters and tunnel dimensions, distributed sensing techniques suggest convenient, long distance, real time,

high level automation, ability to achieve spatial information, cost-effective and versatile instrumentation. However, in order to provide accurate measurement for analysis, the choice of fiber optic sensing cable is of principal importance. If it is set to monitor throughout the excavation, construction and operation cycle, it should be able to withstand the possible extreme condition of exposing to the drill and blast damage when the harsh drill and blast excavation method is employed, the chemical resistance, the long-term material degrading problem due to water and crude oil etc. and the method of installation of the sensing cables is also imperative.

Fiber optic sensing technology certainly holds much potential as instrumentation in monitoring fields such as lifeline maintenance and disaster prevention. Nevertheless, as discussed in the later section of this chapter, distributed fiber optic sensing technology still faces a number of challenges to gain a widespread use. These challenges have to be overcome through demonstration tests, intensive research and the like in order to better establish the technology.

6.2.1 Fiber Packaging Technique

Due to the optical fiber's fragility, it is a challenge to install it directly in real life application without packaging. Reliable packaging technique for optical fiber is essential and critical to ensure reliable sensing measurement in structures using the BOTDR technology, especially in rough construction conditions or harsh expose environments.

After rigorous survey in the market, different types of single mode fiber optic sensors coated with different types of sheathing are found. These fibers are made to cater mainly communication applications, very few are available and capable for capturing strain measurement. It is due to the existence of the coating on optical fiber results in difference between the strain of the structure and the strain sensed by surface mounted or embedded FOS. Although, a theoretical model is derived, Li, (2003) based on some assumptions to address the problem, the development of BOTDR strain

sensors is still in an infant stage.

Fiber optic sensors with composite metal and glue-based sensing heads have been developed by Zhou et al., (2007). However the low durability of the glue caused by aging and creep shows that they cannot fit the practical installation requirements in some applications. Moreover, the metal-tube protected optical fibers do not have reliable repeatability or linearity, due to metal material's plasticity under large strain. These limitations indicated that they are not suitable to be used for cracks or fatigue detection. In addition, the durability of metal-coated fiber optic sensors is also challenged by the resistance against corrosion.

The first known fiber-reinforced polymer (FRP) product was found in the mid-1930s as part of a manufacturing experiment using a fiberglass fabric and polyester resin laid in a foam mold (ACMA MDA, 2006). With the development of fiber-reinforced polymer (FRP) for practical infrastructure, FRP has attracted a lot of attention and has become a popular choice of construction material given its high strength-to-modulus ratio, corrosion-resistance, nonmagnetic, good strength and fatigue-resistance. Besides, the quasi-elastic constitutive relation of FRP materials is linear to 15,000 $\mu\text{m/m}$, so it is a good selection as a packaging material for deformation sensors for especially large strains ($>2000 \mu\text{m/m}$).

Incorporating BOTDR sensing technique with FRP material, a low cost and highly reliable BOTD sensor using a rebar consisting of a bare optical fiber (OF) packaged in FRP is developed with the name BOTD-FRP-OF. (Zhou et al., 2008) This newly developed FRP-OF sensing head is ideal for practical application due to its high strength and durability for installation safety and long term service.

6.3 CHALLENGES AND ISSUES WITH THE DISTRIBUTED BOTDR SENSORS

The fiber optic sensing technology has experienced a phenomenal rate of progress in the second half of the twentieth century. Although the technology of the

fiber optic brings about many advantages, there are still several shortcomings and problems faced.

a) Economical Deficiency

High initial cost – The installation cost of the technology is comparatively high and special test equipment is often necessary. The test equipment typically and traditionally used for conventional electron-based networking is of no use in a fiber optic network. Equipment such as a BOTDR (Brillouin Optical Time Domain Reflectometer) Analyzer is required. Costly and specialized optical test equipment such as optical probes is needed at most fiber endpoints and connection nexuses so as to accurately provide testing of the optical fiber. Though, the initial set up cost for the system is higher than other traditional system, but when considering this system can serve as a long-term structural health monitoring system. With the high level of automation where the operation cost for this system is relative low and the ability of achieving continuous distributed measurement, the cost per data achieve will be much more lower as compare to the other tradition monitoring system. When the potential of this BOTDR monitoring system is fully utilized, this will achieve an overall economical advantage rather than as economical deficiency.

b) Physical Deficiency

Receptiveness to physical damage – The BOTDR FOS is a small and compact cable and it is highly susceptible for having cut or damaged during installation or construction activities. Hence, the protection package and the installation method of the FOS is of the utmost importance. Furthermore, the optical fiber is easily affected by chemicals as the fiber glass can be damaged by various chemicals such as hydrogen gas, these physical deficiency has to be resolved with respect to the exposed environment, location and function of the underground infrastructure.

Durability – Creep, a time-dependent dimensional change of structurally

strained (loaded) materials. It develops gradually and its ultimate value is usually achieved only after some years. It implies that since the ideal goal is to apply a long-term SHM for underground application. The BOTDR FOS should be investigated with prolong and repetitive functional satiability.

c) Technical Deficiency

Possible strain loss – Deformation is measured by the amount of strain produced by certain amount of load. Structural strain is present in loaded structures and directly reflects the structural condition. Hence, in engineering practice, it is especially vital to evaluate the accuracy of the strain measurement achieve, however, the current market available BOTDR sensing cable are suspected to have strain measurement loss and variation in interpretation of sensing location due to the loose layer present in the sensing cable protection packaging. Also, temperature variations in a structure's environment may create temperature variations behavior in the cable package itself.

Alternative installation scheme – The adhesion installation method, which the BOTDR sensors are adhered directly onto the surface of the monitor target, is commonly used in most applications. Unfortunately, this method is not applicable for un-façade underground infrastructure (ie. underground rock tunnel and cavern). The rough and irregular surface of the cavern wall does not permit a full bonded adhesion between wall surface and BOTDR sensors with adequate applied pretension.

Special skills requirement – The optical fibres has to be pretension during installation when measuring strain, special skill, knowledge and training is required for engineers to successfully install the BOTDR sensors correctly. The sensors also cannot be joined together as easily as copper cable, it needs additional personnel training, expensive precision splicing and measurement calibration equipment.

Although, those few disadvantages are yet to overcome along the technology development to fully employ the up and coming fiber optic sensing technology. Its

potentials and advantages far surpass the shortcomings and further researches are being done now and in the future to explore the possibilities of this brilliant technology. In the following sections, the author will target to provide solutions for the physical and technical deficiency by exploring the alternative BOTDR sensors installation method in underground structure; describe, examine and evaluate the BOTDR fiber optic sensors with different protection packages and finally, to calibrate the two BOTDR strain sensors prototypes developed that ought to be suitable for the use in cavern or tunnel structural health monitoring application. Also, an enhance protection given to the fiber optic sensor by embedding it in non-strain grout is evaluated.

6.4 PROTECTION PACKAGE SYSTEM OF BOTDR FOSs FOR UNDERGROUND ENGINEERING APPLICATION

Construction of underground tunnels and caverns is a difficult and hazardous art. Many tunneling and excavation problems are caused by unexpected changes in the stress distribution or deformation of the rock or soil, due to their imperfect material nature. Structural monitoring is one of the most important tasks for the maintenance of all underground tunnels and caverns, and usually the monitoring process lasts through the whole lifespan of the structure for a few decades. The choice of monitoring instrumentation and the frequency of monitoring depend on the type of underground structure, the geologic quality, the usage of the structure, the excavation method employed during construction and the stage of construction.

Drill and blast excavation is a common tunneling process during construction phase. If the BOTDR optic fiber sensing is chosen as the monitoring instrumentation for the underground structure, one should realize that the material of the FOS used as well as its packaging technique is extremely critical in order to withstand the rough underground condition. Moreover, the selected FOS must be resistant to water, crude oil and other chemical fluid. Therefore, selection of the fiber protection package is of great important in order to achieve high reliability and accuracy in the monitoring results and data analysis.

Besides the environmental resistance, the authors proposed that the FOS to be used in such a harsh and rough environment should be tough, durable and most important each layer of the sensor protection coating should be firmly intact with one another in order to achieve accurate ‘point for point’ strain measurement and reduce strain loss. Five different BOTDR sensing cables will be examined in this article to evaluate their suitability for underground SHM applications and the physical descriptions of the BOTDR FOSs are given as follow.

*Due to commercial agreement with manufacturer, only general description of the sensors will be provided in this paper.

6.4.1 General Cross-sectional Information of Different BOTDR Sensors

- a) Sensor 1, S1: Bare fiber optic sensor coated with low smoke zero halogen (LSZH) tight buffer coating

Sensor 1 consists of a bare optical fiber coated with a layer of low smoke zero halogen (LSZH) tight buffer coating. This sensor is commonly use in laboratory testing which supposed can provide accurate measurements while extreme care is taken in the laboratory experiment. Figure 6.2 described the general cross section structure of this sensor.

Figure 6.2 – General cross structure of Sensor 1

- b) Sensor 2, S2: LSZH Tight buffer sensor with Kevlar and PE jacket coating

The second sensor is a combination package of sensor 1 with extra coating of loose Kevlar linings to protect the fiber optic sensing core against moisture attack. The package is then wrapped with PE jacket as an external protection layer (Figure 6.3).

This sensor is flexible and moisture resistant, thus, it is commonly found in telecommunication application and usually used as temperature monitoring sensor in structural health monitoring application.

Figure 6.3 – General cross structure of Sensor 2

c) Sensor 3, S3: Reinforced glass-fiber coated sensor

This sensor is specifically fabricated aiming to reduce the possible of strain loss associate with the loss bonding presents in between different layers of protection coatings. The sensor is packaged with a single core optical fiber and directly coated with reinforced glass-fiber (Figure 6.4). This special packaged sensor is very taught and durable but due to its physical stiffness, it is very much strenuous to apply accurate pretension during installation and causes the pretension applied may not be uniform throughout the installation.

Figure 6.4 – General cross structure of Sensor 3

d) Sensor 4, S4: Special strain sensing cable (Prototype 1)

A special strain sensing cable developed after reviewing various findings from

the research experimental studies. The protection package system is perfectly bonded with the sensing fiber core through the special designed strain transfer layer (Figure 6.5). The inner sensing fiber core can displace together with the outer protection package system to achieve “point-for-point” strain measurement. The PA protective coating provides protection against normal environmental attack. Sensor 4 is not heat and water resistant, though it has good strain measurement capability but the protection package system is not suitable for underground application.

Figure 6.5 – General cross structure of Sensor 4

e) Sensor 5, S5: Reinforced strain sensing cable (Prototype 2)

The advance version of the special designed strain sensing cable developed based on the specification of Sensor 4. The BOTDR FOS prototype 2 made advancement in the protection package system which shall be more favorable for underground application. The protection package system is enhanced with stainless steel tubing to strengthen the FOS’s durability against the environment in underground application. The PA protective coating is also modified to provide better resistant against high temperature and water seepage. Figure 8 described the general cross section structure of this sensor.

Figure 6.6 – General cross structure of Sensor 5

6.5 LABORATORY EXPERIMENTS ON CALIBRATION OF DIFFERENT BOTDR FOS FOR UNDERGROUND SHM APPLICATIONS

The calibration test of the BOTDR FOSs is conducted in two stages. First, a series laboratory experiments is conducted to calibrate the accuracy of different BOTDR FOSs under uniaxial compression loading condition. The BOTDR FOSs' measurements are compared with the traditional strain gauges' (SGs) measurements which are correlated with theoretical calculation results to determine its measurement accuracy under uniaxial loading condition. Second, the BOTDR FOSs are installed on a tunnel cross section simulated frame with SGs mounted. The SGs' measurements are compared with the BOTDR FOSs' measurement to determine the suitability of using BOTDR FOS in measuring the cross sectional strain variations in underground tunnels and caverns. The results of this research present valuable reference for advance exploration of BOTDR SHM systems in the underground construction industry

6.5.1 Experimental Setup

A specimen is fabricated with strain gauges (SGs) 1-11 mounted on the web face of a 3.6 m long, 300x200, mild-steel universal column. Self fabricated clampers are welded with respect to the strain gauge positions along the same web, and the BOTDR sensing cables are attached to the clampers. Figure 6.7 shows the schematic drawing of the specimen. The fabricated specimen is loaded with a compression testing machine (CTM) and the experiment setup is shown in Figure 6.8 and linear variable displacement transducers (LVDTs) are installed at the top, center and bottom section of the specimen to ensure no transverse displacement (bending) occurs during loading. Measurements are collected with the BOTDR analyzer and data logger (Figure 6.9) from the different fiber optic sensing cables and SGs in every load stage for the comparison study.

Figure 6.7 – Schematic drawing of specimen details and test setup

Figure 6.8 – Experiment set-up in laboratory

Figure 6.9 – BOTDR analyzer and analogue data logging system

6.5.2 Testing Procedures

Compressive stage loading of 50kN increment interval is applied to the specimen from 0kN to 250kN. At 0kN, both the BOTDR and strain gauge measurements are recorded and initialized as base line readings. Minimum of three measurements the BOTDR and the strain gauges are recorded at each 50kN load interval to minimize random error for a reliable and consistent result.

The five calibrated BOTDR sensors are labeled as ‘S1’, ‘S2’, ‘S3’, ‘S4’ and ‘S5’ respectively in the graphs. A comparison study is performed with the strain gauge measurements against the BOTDR FOSs measurements. The analysis will focus on examines the measurement accuracy, reliability of each sensor and commends on the FOSs physical condition by visual observation of the fiber optic sensing cable throughout the experiment to finalize the suggestion on the most suitable cable for underground applications.

6.5.3 Pre-Analysis

The SGs measurements are compared with the theoretical strain calculated from equations (6.5):

$$\varepsilon = \frac{P}{EA} \quad (6.5)$$

where, ε = strain, P = Compressive Loads, Young's Modulus, $E = 200\text{GPa}$ and Cross-sectional Area, $A = 61.5\text{cm}^2$. The deviations between the theoretical calculated strain and strain gauges measurements (Table 6.1) are computed for further analysis.

Figure 6.10. shows the plot of overall micro-strain deviation from strain gauges measurements against the theoretical calculate values. The deviation of measurements is less than $\pm 10 \text{ um}$, it proves that the strain gauge measurements are accuracy and reliable to serve as a reference to compare with the BOTDR measurement in the later stage of the experiment.

Table 6.1 – Length Vs Micro-strain of the specimen at different loadings

Theoretical Results vs Strain Gauges Measurement - Micro-Strain (um)										
Length (m)	Loads									
	50kN		100kN		150kN		200kN		250kN	
	Theoretical	Strain Gauge	Theoretical	Strain Gauge	Theoretical	Strain Gauge	Theoretical	Strain Gauge	Theoretical	Strain Gauge
0	-40.7	-38.714	-81.3	-80.011	-122	-117.89	-163	-163.890	-203.3	-208.440
0.2	-40.7	-40.143	-81.3	-75.556	-122	-120.670	-163	-163.890	-203.3	-201.332
0.75	-40.7	-38.857	-81.3	-82.556	-122	-116.071	-163	-157.670	-203.3	-203.221
1.3	-40.7	-38.051	-81.3	-81.444	-122	-123.890	-163	-164.561	-203.3	-200.560
1.5	-40.7	-37.571	-81.3	-77.333	-122	-119.889	-163	-159.111	-203.3	-204.893
1.7	-40.7	-38.002	-81.3	-82.222	-122	-116.560	-163	-157.783	-203.3	-203.891
2.25	-40.7	-37.286	-81.3	-78.889	-122	-122.110	-163	-162.332	-203.3	-208.110
2.8	-40.7	-38.714	-81.3	-83.556	-122	-118.110	-163	-161.891	-203.3	-200.561
3	-40.7	-40.143	-81.3	-75.889	-122	-122.560	-163	-157.890	-203.3	-208.783

Figure 6.10 – Micro-strain of the specimen at different loadings

6.5.4 Results and Discussions

6.5.4.1 Analysis of the bare fiber optic sensor with LSZH tight buffer coating, Sensor 1

Figure 6.11 shows comparison results of the Sensor 1 against the strain gauges measurements along the effective measurement length of the BOTDR sensing cable for each individual load case. Sensor 1's measurement agrees well with the strain gauge measurement. The author presumed with the validation in the previous section that the conventional strain gauges method are reliable and have a high degree of accuracy in measuring micro-strains. The results show that the accuracy of the Sensor 1 is considered acceptable since the deviation of its reading against the strain gauges results fall within the $\pm 10\%$ range, the measurement achieved from the BOTDR analyzer is also consistent. Clap marks are notice on the sensing cable Based on observation. The convergence of reading noticed at the beginning and ending section of the cable is believed due to the localized stress induced by the clip. Sensor 1 has the accuracy and consistency in achieving accurate measurement result, but the protection package is too fragile and not suitable for site application but good to serve as a control reference in

laboratory testing.

Figure 6.11 – Micro-strain comparison of Sensor 1 vs Strain gauge results

6.5.4.2 Analysis of the LSZH tight buffer sensor with Kevlar and PE jacket coating,

Sensor 2

The measurements of Sensor 2 against the strain gauges are shown in Figure 6.12. Although, consistence is found in Sensor 2 but the measurement varies whereby they supposed to be consistent along the same load. The BOTDR measurements obtained after the 1 meter mark show no accuracy in the comparison. The author believes such large variation is caused by the sliding of the inner tight buffer fiber core within the Kevlar layer. This implies the center clamp is not tight enough to hold the fiber core in position. However, this problem is suspected to be caused by the sensor's protection package, the loosely fitted PE jacket and the presents of the Kevlar layer promote such free movement of the inner fiber core and cause such measurement error.

Figure 6.12 – Micro-strain comparison of Sensor 2 vs Strain gauge results

6.5.4.3 Analysis of the reinforced glass-fiber coated cable, Sensor 3

Sensor 3 is insensitive as shown in Figure 6.13, it does not show any sign of strain captured during loading. The author suspects the sensor has become less sensitive after coated with the glass-fiber polymer. The physical condition of this sensor is very taught and have no sign of damage after installation. It may be usable to detect large strain but the stiffness of this sensor makes it very difficult in installation.

Figure 6.13 – Micro-strain comparison of Sensor 3 vs Strain gauge results

6.5.4.4 Analysis of the special strain sensing cable – Sensor 4

From Figure 6.14, Sensor 4 shows good consistence and the measurement accuracy matches against the strain gauge measurement, this prototype serve very well as a strain sensor. However, the protection package is still consider slightly ‘soft’ and it is not high temperature resistance, therefore, regardless of the high strain measurement performance. This sensing cable is not suitable for the harsh environment requirement expected in underground application, but it serves as a good reference prototype for sensing cable enhancement development and makes a good replacement of the tight buffer sensor in laboratory experimentation.

Figure 6.14 – Micro-strain comparison of Sensor 4 vs Strain gauge results

6.5.4.5 Analysis of the reinforced strain sensing cable – Sensor 5

The ‘Sensor 5’ shows good consistence and high degree of accuracy in the comparison study with strain gauge (Figure 6.14). The improved sensor protection package provided is also strong, durable with adequate stiffness; there is no sign of damage by the clamp throughout and after the experiment. The internal bonded layer ensure the fiber core deform along with the outer layers. This sensor is also water and high temperature resistant, which the author is suitable for underground structure monitoring application.

Figure 6.20 shows a comparison graph consist of all five evaluated sensors while loaded at 200kN. It shows that sensor 4 and sensor 5 can achieve acceptable measurements with less than 5% of errors when compare with strain gauges. Sensor 3 although is the most well protected sensor but it is too insensitive for strain measurement. With consideration that the sensor calibrated here is for the aim to apply in underground strain monitoring application, sensor 5 is selected as the most suitable prototype.

Figure 6.15 – Micro-strain comparison of Sensor 5 vs Strain gauge results

Figure 6.16 – Micro-strain comparison of the five cable vs Strain gauge results

6.6 CONCLUSIONS

Through the experiments and calibrations, it is shown that the Sensor 1, Sensor 4 and Sensor 5 are showing good consistence and accurate measurement against Sensor 2 and Sensor 3. Sensor 2, which is packed with Kevlar to resist moisture penetration into the fiber core, the loose fitting induced space between the outer coat and the inner core tubing which affect the accuracy of the measurement in an unpredictable manner. Secondly, this outer coat of the Sensor 2 is considered soft and causes the localized stress induced by the clamp transferred to the sensing core and greatly affects the measurement. This soft coating is also not suitable to be applied in underground application. The reinforced glass-fiber coated cable, Sensor 3, is strong and durable to withstand the harsh underground construction condition. But, the inability to apply a satisfied pre-tension to the stiff sensing cable prior to the installation causes inconsistency in the measurement. The low strain variation indicates the inferiority in its measurement accuracy and suitability in real life application. Sensor 1 is very much accurate reliable in measurement, but the thin tight buffer tubing is too soft and can be easily damage. Therefore it is not durable enough to be applied in underground application unless special care is taken. Sensor 4 is recommended to be used for strain sensing in structural health monitoring application. It clearly shows a high reliability in capturing micro-strain and sensitivity towards additional loading. However, in the aim of providing a reliable and durable strain sensor for underground structural health monitoring application, Sensor 4 still has to be improved on its protection package against external environmental conditions (i.e. high temperature, water resistance, etc.). Sensor 5 is recommended for underground engineering SHM application, it inherited all the advantages from Sensor 4, the improved strong protection package gives the confidence of durability and survival ability throughout the construction and operation phase of the underground structure. The experiment with frame specimen demonstrated that the calibrated Sensor 5 is suitable to apply in measuring strain in the cross sectional direction of the tunnel or cavern, the protection package system is tough to withstand environmental attack yet flexible for installation and able to provide high degree of accuracy and consistence in strain measurement.

CHAPTER 7 – EXPERIMENTAL STUDY AND SITE DEMONSTRATION ON BOTDR FOS INSTALLATION METHOD FOR UNDERGROUND STRUCTURAL HEALTH MONITORING

7.1 INTRODUCTION

It has been a common and usual practice for engineers to obtain onsite convergence measurements of tunnel cross sections prior and after subsequent excavation stages. However, in spite of the tight excavation schedule, tunnel excavation activities usually inherit problems with limited accessing time and accessibility. The time that allows the engineers or surveyors to deploy and obtain the measurement is limited, in some cases, due to safety concern they are not able to deploy for measurement taking at the immediate moment after excavation. i.e. For drill and blast excavation, the first one to two hours after blast are usually scheduled as ‘no entry’ time for air ventilation, rock dust and gases obliteration. On the other hand, some of the deep underground infrastructures are constructed to store highly explosive or toxic materials. Examples of such are the Ministry of Defense (MINDEF), Singapore, Underground Ammunition Facility (UAF) for ammunition storage and military purpose (MINDEF, 2001) and the Jurong Rock Cavern (JRC), Singapore (JTC, 2007) for commercial oil and gas storage. These facilities create ‘inaccessible zones’ once they are in operation and it is impossible to deploy for structural monitoring in these areas. Such problems inspire the research and development in the long term spatial structural health monitoring (SHM) system with the Brillouin Optical Time Domain Reflectometry (BOTDR) based fiber optic sensors (FOS) to monitor the long term structural behavior to ensure the sustainability of such underground infrastructural development.

However, as described in the previous study and other literatures. There are various challenges and issues with deploying this brilliant technology in underground engineering applications. Limited research attention is paid to address the deficiencies

in the BOTDR FOS installation methods for possible applications of the proposed BOTDR FOS long term SHM system especially in underground structural health monitoring.

This chapter will focus on the investigation of the BOTDR FOS installation method for underground long term SHM applications. It provides a summary on the current installation techniques of the BOTDR FOS, discusses the suitability and deficiencies of such installation techniques in underground SHM applications. An alternative installation method is proposed to overcome the physical and technical deficiency in the current BOTDR FOS installation techniques. With a series of experiments and the onsite trial installation, the discussions will focus on the feasibility of the proposed installation method and demonstrate the survivability of the installed BOTDR FOS in the extreme drill and blast tunnel excavation condition. The laboratory experiments include testing on different installation methods; the ‘Looping’, ‘Point Adhesion’ and ‘Clamping’ method on the prepared specimens monitored by the BOTDR FOS, strain gauges and linear variable displacement transducers (LVDTs). The investigation conducted involves strain analysis by close form solution, numerical simulation and experimental study with different installation setups of the BOTDR FOS and traditional strain gauges in various loading conditions. The analyzed theoretical results and numerical simulation results will be correlated and compared with the experimental measurements. The enhance protection technique through embedding the BOTDR FOS into non-strain grout is also evaluated to examine the feasibility of such protection enhancement method. The findings in this research present valuable reference for advance exploration of BOTDR SHM systems.

The research is significant in designing and developing the spatial SHM system for underground engineering applications. The proposed installation technique can overcome the physical and technical deficiency for applying BOTDR FOS in underground structures. This provides solution to SHM system developers to design a robust underground SHM system. Whereby, practically, the SHM system installation scheme and technique are of critical concerns to the project planners and contractors.

With that, changes in strain or the structural behavior of the underground tunnel and cavern can be monitored even at the ‘no entry’ time or in inaccessible locations. The underlying principle of this research is that, with complementary of the successful calibrated BOTDR FOS in the accompanying study, the installed spatial SHM system in the underground structure can provides vital real time spatial strain data of the structure under various loading conditions. These strain data are gathered and transmitted from the site to the monitoring stations for convergence analysis

7.2 BOTDR FOS INSTALLATION METHODS IN OTHER APPLICATIONS

In applications of steel bridges and telecommunication pipeline monitoring systems, the full bond adhesion method is introduced (Hiroshi et al., 2001, 2003, 2005 and 2007), where the fiber optic sensing cable is adhered directly onto the smooth surface of the telecommunication pipeline and the structural sections of the steel bridges. This installation method requires to perform on a smooth surface where the sensing cable can perfectly bonded with the monitoring surface. It is not suitable and applicable in the non-façade underground cavern or tunnel monitoring application where the monitored surface is usually angular and irregular. An alternative installation method of the BOTDR FOS is to embed the sensor into reinforced concrete structure (Chang et al., 2005), where the sensing cable is tied along the reinforcement and casted together with concrete. This is a promising technique whereby the BOTDR FOS can be perfectly bonded with the monitoring structure; furthermore, it provides good protection against external damage. However, this technique withholds a major limitation that no pretension (Komatsu et al., 2002) is applied during installation, which in turn causes reduction in the accuracy of strain measurement. Moreover, to apply such installation method in underground tunnels, a reinforced frame of the tunnel cross sectional dimension has to be erected for the sensor to tie on. This may not be economical and practical for multiple installations. Since both full bonding installation methods are not feasible for installing the BOTDR FOS in underground monitoring application. It inspires the author to investigate the alternative installation method in point fixing system, which shall permits the sensing cable to be installed on the rocky

and rough tunnel face.

7.3 CHALLENGES AND ISSUES WITH THE INSTALLATION OF DISTRIBUTED BOTDR SENSORS

In spite of the deficiencies found within the BOTDR FOS, the author will discuss the operational deficiency and further elaborate the physical and technical deficiency associate with the sensor installation method especially for underground SHM application.

7.3.1 Operational Deficiency

Work procedure alteration – The introduction of installing the spatial long term BOTDR FOS SHM system is not of the usually practice within the underground infrastructure’s construction cycle. The installation of such system sometime may even take up several hours and this duration is highly dependent to the geological and onsite conditions. Especially during the tunnel excavation stage, it disturbs the existing operations and the excavation schedule. Hence, highly alter the project critical path in project planning. System planners need to have adequate knowledge to estimate the installation time required in different geological and onsite conditions. Through discussion with project planners, introduce the additional stage in the excavation and construction cycle and subsequently, reduce the uncertainties arise in project management.

7.3.2 Physical Deficiency

Special fixture requirement – The BOTDR FOS is a small and compact cable which is highly susceptible for having cut or damaged during installation. Hence, the fixture used to hold or attached the BOTDR FOS has to be specially designed. This designed fixture has to overcome some major issues like being able to hold the BOTDR FOS in place, firm enough to apply pre-tension on the BOTDR FOS and yet cannot be too tight and cause damage to the cable or causes inaccuracy in measurement.

Fixture durability – The goal of the BOTDR FOS SHM is to cater for long term monitoring. Therefore, similar with the durability deficiency in BOTDR FOS, the material used in fabricating the fixture has to be able to withstand the harsh environment found in underground construction. Alternatively, a protection system can be proposed to protect the installed BOTDR FOS against conditional and seasonal wear and tear.

7.3.3 Technical Deficiency

Alternative installation scheme – The usual full bond adhesion installation method, which the BOTDR FOS is adhered directly onto the surface of the structure, is commonly adopted in most BOTDR FOS applications. Unfortunately, this method is not applicable for the un-façade underground infrastructure (ie. underground rock tunnel and cavern). Neither the rough rock surface nor the irregular shotcrete lining along the tunnel wall permit a full bonded adhesion between tunnel wall surface and BOTDR sensors. Alternative installation methods are yet to investigate. Furthermore, there are major concerns in the accuracy of the BOTDR FOS obtained and how to apply adequate pre-tension in the alternative installation method.

Special skills requirement – The optical fibres has to be pretension during installation when measuring strain, special skill, knowledge and training is required for engineers to successfully install the BOTDR sensors correctly. In addition, the BOTDR FOS is a continuous sensor, it cannot be separate in individual measurement points, system planner has to work out different installation scheme and plan the strategic location to deploy the installation cater for different monitoring purposes. Workers have to be specially trained to handle the BOTDR FOS have clear understanding of the installation procedure.

In this chapter, the author aims to provide solutions for the physical and technical deficiency via exploring the alternative BOTDR FOS installation method

suitable for underground SHM applications. This study involves series of experimental testing, fabrication of the fixture used in the installation methods, designing the layout scheme of the continuous BOTDR FOS and investigating the measurement accuracy of the enhance protection technique by embedding the installed BOTDR FOS in non-strain grout. Finally, the solutions to these deficiencies will be demonstrated in an onsite installation trail which the installed BOTDR FOS SHM system shall survived under the hash drill and blast tunnel excavation condition.

7.4 EXPERIMENTAL STUDY ON BOTDR FOS INSTALLATION METHOD

The experimental study of the alternative BOTDR FOS installation method involves series of the compression tests. In the first stage, the ‘Looping’, ‘Point Adhesion’ and ‘Clamping’ point fixing installation methods inspired from the literatures are installed in a column element to investigate the measurement accuracy of the each installation method. The evaluation will be based on the comparison study of the measurement from the experiments with the theoretical calculated results. In the second stage, the ‘Clamping’ method is selected to test on a tunnel cross section simulated structural frame. These two stages of experiment can examine the feasibility of the installation method’s applicability in the underground tunnel.

7.4.1 The Compression Test

A specimen is fabricated with strain gauges (SG) 1-11 mounted on the web of a 3.6 m long, 100x50, mild-steel universal column (UC). The specially designed fittings (the poles and clips) (Figure 7.1) are fabricated and welded on the UC. The BOTDR FOS is fixed on the fitting with respect to the strain gauge positions. The fabricated specimen is loaded with a compression testing machine (CTM) and linear variable displacement transducers (LVDT) is installed at the top, center and bottom section of the specimen to ensure no transverse displacement (bending) is occurred during loading. The three different BOTDR FOS installation methods named as, ‘Looping, ‘Point

Adhesion' and 'Clamping' method are tested in this experiment. A schematic drawing and the setup is shown in Figure 7.2. for 'Looping' and 'Point Adhesion' methods. Weights are attached at the lower end of the installed FOS to give the applied pretension as suggested by Komatsu et al., (2002). The 'Clamp' fittings are welded on the other side of the UC web along the strain gauges(Figure 7.3) to conduct the same experiment with the 'Clamping' FOS installation method. Different from the 'Looping' method, no weight attachment is needed to apply the pretension in 'Clamping' method. The designed clamps can hold the BOTDR FOS firmly after locked, pretension can be accurately apply and measure before locking the clamp. Measurements of BOTDR FOS and strain gauges will be collected in every load stage for the comparison study.

Figure 7.1 – The 'Pole' and 'Clamp' fittings schematic

Figure 7.2 – The ‘Looping’ and ‘Point Adhesion’ installation experiment setup

Figure 7.3 – The ‘Clamping’ installation experiment setup

7.4.1.1 Testing Procedure

Compressive stage loading of 10kN increment interval is applied to the specimen from 0kN to 40kN. At 0kN, both the BOTDR and strain gauge measurements are recorded and initialized as base line readings. Minimum of three measurements the BOTDR and the strain gauges are recorded at each 10kN load interval to minimize random error for a reliable and consistent result.

The same tight buffer BOTDR FOS is used with the three different installation methods tested in the experiments. A comparison study is performed with the theoretical solution and the strain gauge measurements against the BOTDR

measurements. The analysis will focus on examines the measurement accuracy and reliability of each installation method. The installation methods will be evaluated according to the measurement accuracy, difficulties and problems found during and after installation and experiments. The best point fixing installation method will be selected for frame test to evaluate the adaptability to an underground tunnel cross section.

7.4.1.2 Pre-Analysis

The experimental results are compared with the theoretical strain calculated from equations (7.1):

$$\varepsilon = \frac{P}{EA} \quad (7.1)$$

where, ε = strain, P = Compressive Loads, Young's Modulus, $E = 200\text{GPa}$ and Cross-sectional Area, $A = 11.8\text{cm}^2$. The deviations between the theoretical calculated strain and strain gauges measurements (Table 7.1) are computed for further analysis.

Figure 7.4. shows the plot of overall micro-strain deviation from strain gauges measurements against the theoretical calculate values. The deviation of measurements is less than $\pm 10 \text{ um}$, it proves that the strain gauge measurements are accuracy and reliable to serve as a reference to compare with the BOTDR measurement in the later stage of the experiment.

Table 7.1 – Length Vs Micro-strain of the specimen at different loadings

Theoretical Results vs Strain Gauges Measurement - Micro-Strain (um)								
Length (m)	Loads							
	10kN		20kN		30kN		40kN	
	Theoretical	Strain Gauge	Theoretical	Strain Gauge	Theoretical	Strain Gauge	Theoretical	Strain Gauge
0	-42.37	-44.22	-84.75	-91.62	-127.11	-131.21	-211.86	-220.01
0.2	-42.37	-43.66	-84.75	-90.36	-127.11	-129.22	-211.86	-224.00
0.75	-42.37	-42.36	-84.75	-92.66	-127.11	-131.22	-211.86	-217.28
1.3	-42.37	-44.25	-84.75	-91.54	-127.11	-128.45	-211.86	-222.36
1.5	-42.37	-43.55	-84.75	-93.22	-127.11	-130.02	-211.86	-224.9
1.7	-42.37	-42.53	-84.75	-92.62	-127.11	-129.23	-211.86	-222.11
2.25	-42.37	-43.11	-84.75	-91.36	-127.11	-129.88	-211.86	-223.95
2.8	-42.37	-42.76	-84.75	-92.06	-127.11	-130.12	-211.86	-220.53
3	-42.37	-41.86	-84.75	-90.89	-127.11	-129.59	-211.86	-219.52

Figure 7.4 – Micro-strain of the specimen at different loadings

7.4.1.3 Results and Discussion

- a) The 'Looping' Installation method

Figure 7.5 shows the comparison results of the BOTDR FOS against the strain gauges measurements. Large variations are found in the measurements near the initial ‘loop’ segment. The author believes this variation is mainly due the ‘loop’ form in the installation. Nevertheless, the measurements match after the ‘loop effect’ is dissolved in the 0.5 to 1.25m range. However, measurements start to vary after the center loop at 1.5m. These measurements are not comparable to the strain gauges measurements. The author noticed the pretension is reduced in the second segment (1.5-3m) on the test specimen and suspected that this pretension lost is the cause for the errors found in measurement. The parallel installed BOTDR FOS measurements agree with the above findings. The ‘Looping Effect’ present near the initial loop where weight is attached to apply pretension and the pretension lost in segment 2 (1.5-3m) causes inaccuracy in measurement.

Figure 7.5 – BOTDR measurement comparisons – Looping Installation

b) The ‘Point Adhesion’ Installation Method

The Comparison results of the BOTDR against the strain gauges measurement are shown in Figure 7.6. The measurements are irregular and unpredictable. The major caused to this problem is the adhesion agent applied during installation. The 3M heavy tape used in fixing the BOTDR FOS does not hold the BOTDR FOS firmly. It is observed that the BOTDR FOS is loose and not attach to the specimen's surface. The weight used to apply pretension to the BOTDR FOS causes the BOTDR FOS to oscillate during the experiment and leads to the irregular and unpredictable measurement results.

Figure 7.6 - BOTDR measurement comparisons – Point Adhesion Installation

c) The ‘Clamping ‘ Installation Method

The comparison results of the BOTDR FOS against the strain gauges measurements are presented in Figure 7.7. The results show agreement form both BOTDR and strain gauge measurements especially at sections 0.5-1.2 and 1.8-2.5. The

measurement converge in both tails are suspected to be caused by the localized stress due to clamping. This assumption is valid with the visual inspection of the BOTDR FOS after the experiment. The outer jacket of the BOTDR FOS has clamp marks left behind after it is released from the clamp. This implies that the BOTDR FOS is clamped tightly during the experiment. The measurements variation found in the center clamp area are less affected by the ‘Clamping Effect’. This is mainly caused by one of the BOTDR sensing principles that the reported measurement value, ε_A at measurement point A is an average value of the measurements along spatial resolution length (effective length) of the measurement point (Figure 7.8). The value of ε_A is calculated from equation 7.2,

$$\varepsilon_A = \frac{\sum_1^n (\varepsilon_B \cdot \varepsilon_C)}{(2n+1)} \quad (7.2)$$

Therefore, the measurements around the center clamp section should be affected by the localized stress induced by the ‘Clamping Effect’ only. Such principle can explain the measurement convergence at both tails. For measurement near or at the clamping point, the effective length beyond the clamping point is not pretensioned. This causes the measurements approaching the clamping point to receive a larger miss-close. Nevertheless, this effect is consistent and can be estimated. Hence, the ‘Clamping’ installation method shows more accurate and reliable measurement results as compared to the ‘Looping’ and ‘Point Adhesion’ method.

Figure 7.7 - BOTDR measurement comparisons – Clamping Installation

Figure 7.8 – Principle of reported measurement

7.4.2 Conclusion on Compression Test

It is noticed through the experiment that looping the sensing cable may bend and cause problem to the fiber core. Furthermore, most of the sensors available require large bending radius which makes looping difficult. It is also observed that the looping

cable resulted in unstable reading due to the lost of pretension. Practically, it is also very difficult to apply and control the pretension that has to be applied to the BOTDR FOS for strain sensing. Therefore, the ‘Looping’ method is rejected for underground BOTDR FOS installation.

From the ‘Point Adhesion’ installation test, it shows that the current BOTDR analyzing system is a true static measurement system. Any movement of the sensor during measurement acquisition will cause unpredictable error. However, the experiment results illustrated that this method has to be re-evaluated with different bonding agent to search for better solution. But, in practical point of view, this method of installation is unfavorable for underground BOTDR FOS installation. The tunnel surface is rough and irregular which does not provide a good bonding surface for the sensor. It is also time consuming in installation for every subsequent point installation needs to wait for the bonding agent to harden in the preceding installed point. Last, it is also very difficult to maintain a constant pretension during the hardening of the bonding agent.

The results clearly show that the ‘Clamping’ installation method provides the most accurate measurements among the three point fixing installation system studied. The concern of localized stress formation at clamping point can be resolved or reduced by introducing a stiffer, better protected sensing cable (i.e. Sensor 4 and 5 from the BOTDR FOS calibration study). Practically, it is favorable for the underground BOTDR FOS installation. Nevertheless, the clamp used in the ‘Clamping’ installation method has to be revised and redesigned to suit the underground condition. The ‘Clamping’ installation method has demonstrated its ability to apply in different geometrical shapes. In the experiment with the structural frame designed to simulate a tunnel cross section in Chapter 5, the BOTDR FOS is installed through this ‘Clamping’ installation method and the results are very positive.

7.5 EXPERIMENTAL STUDY ON BOTDR SENSOR PROTECTION TECHNIQUE

7.5.1 Non-strain Grout Embedment Test

The author further investigates the protection system for the fiber optic sensing cable by embedding it into non-strain grout. Experiment is conducted to examine the performance of the fiber optic sensing cable throughout the embedded period. The BOTDR FOS is casted within the non-strain grout (Figure 7.9) and tested under the same experiment described in section 7.4.1.

Figure 7.9 – The high grade non-strain grout and the grout casted specimen.

7.5.2 Results and Discussions

The comparison study shows (Figure 7.10) that the embedment has slight influence on the accuracy on strain measurement. But overall, the reduction in strain reading of the BOTDR FOS is minor and consistent over the embedded measurement length. The measurements obtained vary less than 10% of the actual measurement and shall consider as acceptable and accurate. The author also believes the present of the hardened grout aids in stabilizing the pretension applied to the clamped cable and thus improved it consistency in measurement.

The comparison study of the embedment technique shows that it is a feasible solution to embed the BOTDR FOS in concrete or other material. Chang et al., 2005, reported the results of the FOS behavior embedded in concrete. This inspires the author to exercise this embedment technique as a protection enhancement in the underground BOTDR FOS installation method. By embedding the BOTDR FOS within the

shortcrete lining along the tunnel wall, it enhances the survivability of the BOTDR FOS under the harsh condition during the drill and blast excavation activities.

Figure 7.10 – BOTDR measurements comparisons in embedment

7.6 SITE DEMONSTRATION OF THE PROPOSED BOTDR FOS INSTALLATION METHOD

7.6.1 Case Background

The BOTDR SHM system is proposed to install in an operation tunnel (Figure 7.11) of an under construction underground infrastructure. This underground infrastructure located at a depth of about 100 - 130 m below sea level, is proposed to be a storage galleries to store 3 million m³ of different hydrocarbons.

Access to the underground tunnels is guaranteed by two access shafts (AS1, AS3). The storage caverns are located at 130 m below sea level (Level 0). Once in

operation, Level 0 will be a 'No Entry' zone and no access will be granted. The operation/ access tunnels, maintenance interventions and the water curtain galleries providing sufficient water pressure to seal the caverns are located at level 1, 100 m below sea level. The excavation of the whole facility is done by the drill and blast excavation method (Figure 7.12)

Figure 7.11 – Sire layout for the proposed BOTDR SHM installation

7.6.2 Installation Scheme

The BOTDR SHM system is proposed to install before shortcreting of the drill and blast excavation cycle (Figure 7.12) to gain the enhance protection from the shortcrete layer. The Transverse Cross-section scheme (Figure 7.13) is proposed for installation of the BOTDR SHM system to achieve long term tunnel cross sectional strain profile monitoring.

This installation layout is suitable to apply in both tunnel and cavern in both the construction stage for cross-sectional deformation monitoring and the operation stage for long term structural health monitoring.

Figure 7.12 – The drill and blast excavation cycle

Figure 7.13 – Schematic layout of the transverse cross-section setting

7.6.3 Installation Procedure

The BOTDR FOS is installed, after first layer of the shotcrete, on the surface of the tunnel. The installation starts with allocating a preferably regular cross section and marking out of the clamping point (Figure 7.14). Checking of clamping points' alignment is critical and the clamping distance shall be recorded (Figure 7.15).

Figure 7.14 – Marking out of clamping points

Figure 7.15 – Installation alignment checking

The marked clamping points are drilled with approximately 250mm depth normal to the surface (Figure 7.16). The special designed anchoring clampers (Figure 7.17 and 18) are installed with a projection approximately 50mm above surface. The self fabricated sensor protection sleeve is installed at the same time (Figure 7.19). The BOTDR FOS is fixed onto the anchoring clampers with the applied pretension and the complete installation is shown in Figure 7.20. The installed BOTDR FOS are embedded into the shotcrete for enhance protection (Figure 7.21)

Figure 7.16 – Drilling of clamber installation point

Figure 7.17 – Schematic drawing of the anchoring clamber

Figure – 7.18 – Actual installed anchoring clumper

Figure – 7.19 – Self fabricated sensor protection save

Figure 7.20 – The complete system installed in the tunnel before shortcreting

Figure 7.21 – The BOTDR SHM system installed after shortcreting

7.6.4 Results and Discussions

Through visual inspection of the installed BOTDR FOS and the successful BOTDR FOS measurement acquisitions (Figure 7.22), it demonstrated the survivability of the installed BOTDR FOS under the extreme condition in the drill and blast excavation. This indicated that the proposed ‘Clamping’ installation method with enhance shortcrete embedment protection is a feasible solution that suitable to apply in underground application. Figure 7.23 shows the tunnel cross-section micro-strain profile obtained over time. As such, this onsite installation trial demonstrated that the long term spatial BOTDR FOS SHM system is a possible solution to provide long-term structural monitoring of an underground structure..

Figure 7.22 – BOTDR measurement taking in underground tunnel

Figure 7.23 – Tunnel cross-sectional Micro-strain Profile over time

7.7 CONCLUSIONS

Through the series of experiments it proves that the point fixing ‘Clamping’ installation method can provide acceptable measure results with the BOTDR FOS. Thus, installing BOTDR FOS is not necessary to solely rely on the full bonding adhesion installation method. Through the series of experiments, it also proves that the ‘Clamping’ installation is suitable for installing BOTDR FOS in underground application. The experiment results from the frame test demonstrate that the ‘Clamping’ installation is favorable for installation regardless of the monitor structure’s geometrical shape. As long as the BOTDR FOS is installed parallel to the monitoring surface, it can provide accurate measurements. However, correction factors need to introduce when the BOTDR FOS is installed in an angle against the monitoring surface. The system survivability in the site demonstration shows the proposed ‘Clamping’ installation method provides a validated solution for installing the spatial BOTDR SHM system in underground engineering application. However, it is experienced from the site demonstration, a SHM system installation phase has to be included in the excavation cycle. The time needed for such installation will varies according to the cross-sectional dimension of the tunnel, the number of anchoring points needed and most importantly, the geometrical regularity of the installed cross-section. The uncertainties in the system installation work can disturbed the usual excavation work on site. Nevertheless, the findings in this study show that the BOTDR SHM system is able to provide long term structural monitoring for underground structure. It provides valuable information for development of the spatial SHM system applicable to underground engineering.

CHAPTER 8 – CONCLUSION AND FUTURE WORK

8.1 Conclusions and Contributions

The studies conducted in this thesis focus on providing solutions to the challenges faced in deformation detection and prediction in underground engineering. The research works are divided into two major domains, the tunnel deformation analysis techniques and the development of a spatial structural health monitoring (SHM) system for underground engineering applications.

The current understanding of geotechnical engineering and underground SHM methodologies is still far behind the rapid pace of the underground infrastructure development. The time dependent deformation behavior analysis in underground tunnel is important for support installation time estimation, design consideration and possible collapse detection. However, the current underground tunnel deformation analysis commonly practiced by engineers usually tends to over simplify the time dependent effect arisen from excavation or creep activities. Though, some scientists have tried to resolve the time dependent influence problem. But, those reported time-dependent deformation analysis methods are not widely adopted in the practicing industry. They are either too complex and time consuming in computation or contains too many assumptions that need to be verified with tremendous amount of field measurement data. The research embodied in this thesis improves such situations. The time dependent problem of deformation cause by tunneling advancement rate is overcome through introduction of the new ARLDP analysis. This new analysis can exercise its usefulness especially in tunnel support installation time estimation. The superimposed LDP mapping technique proposed suggests a convenient and efficient analysis method for engineer to deduce a quick estimate of the tunnel deformation profile based on the reliable onsite convergence measurement. This proposed analysis method also benefits in estimating the tunnel deformation at the tunnel excavation face and along the longitudinal section away from the monitoring cross section.

On the other hand, the traditional convergence surveying measurement method practicing in the underground engineering industry inherits many limitations. These limitations results in measurement errors, unable to deliver timely information and limited data acquisition per deployment. There is an urgency to improvise the structural monitoring methodology to resolve the challenge and desire to develop a robust real-time spatial health monitoring (SHM) system in underground engineering applications. It is well understood that, building such spatial SHM system with traditional monitoring instrumentations is never cost efficient. The findings of this research proposed a solution to address such issues. The real-time long-term spatial SHM system framework based on the BOTDR sensing technology is developed in this study. The investigations and experimentations performed in the BOTDR sensing technique demonstrate the ability and possibility to develop such system to obtain spatial measurement for underground SHM. The strain to displacement transformation analysis conducted allows the proposed real-time long-term spatial underground SHM system to replace the existing tunnel deformation monitoring methods. Through the experimental studies on BOTDR FOS, the deficiencies on applying the BOTDR sensing technique in underground applications are also resolved. Lastly, the objectives and aim of this study, to develop a long-term spatial SHM system to monitor deformations in underground structures materialized in the site demonstration. Whereby the proposed system is launched and survived in a real life under construction underground tunnel.

8.1.1 The Tunnel Deformation Analysis

A series of new functions are developed with the maximum normalized plastic radius and normalized excavation advancement rate to construct the new ARLDP for tunnel displacement analysis. This analysis includes the time dependent factor, the excavation advancement rate, which is important for tunnel displacement analysis during excavation. The LDP developed with this analysis method which incorporates the time dependent excavation advancement rate effect can define the rate of tunnel

displacement occur. Thus, when it is applied together with the GRC and SRC in CCM, the tunnel support installation time can be more accurately estimated. The proposed superimposed LDP analysis provides engineers and researchers with a quick estimation of the tunnel deformation profile using the reliable on site monitoring data. The main advantage in this analysis technique is through computation with on site measured data, uncertainties arise from assumptions in the computation are reduced. It also helps to predict the deformation behavior at the advancing face which is unable to measure. However, both the ARLDP and the superimposed LDP mapping techniques inherit the assumptions found in other LDP calculation methods. The rock material is assumed to be homogeneous within the calculated distance, the tunnel cross section is circular and the stress field around the tunnel is isotropic.

Though, the SLDP and ARLDP analysis still contain future works for improvement. They can work together with the BOTDR FOS SHM developed in this thesis as a SHM scheme (Figure 8.1) to perform structural health monitoring during tunneling operations.

Figure 8.1 – Proposed SHM scheme for tunneling operation

8.1.2 The Spatial SHM System Development

The geometrical analysis performed for the strain to displacement transformation, the experimental study on BOTDR sensor calibration and the investigation in BOTDR FOS installation methods complements with one another to complete the system framework for the proposed real-time long-term spatial BOTDR SHM system development. The strain to displacement transformation formulation developed can accurately and effectively transform the strain measurement recorded from the BOTDR system to displacement information. However, it is noticed that correction factors are needed to correlate the transformed displacement with the actual displacement to compromise the difference in structure geometrical shape. Nevertheless, the method developed shall work well with a circular and semi-circular cross section tunnel. Thus, it shall complement with the tunnel displacement analysis developed above and incorporate directly into the analysis to calibrate the displacement models.

The calibration and investigation of the BOTDR fiber optic sensing system is the most significant contribution in this thesis. The fragility of optical fiber is well known to anybody who works with instrumentations. The ultimate challenge to apply the BOTDR sensing technique in underground SHM application is resolved through the successful calibration of the prototype BOTDR strain sensors and the investigation on the sensor installation methodology. These two studies performed overcome the critical physical and technological deficiencies in the application. Though, the robustness of this proposed system is still yet too early to conclude, but this research demonstrated that development of such spatial measurement system for underground applications is possible. The initiations and findings in this research can be well referenced in future development.

8.1.2.1 Applicability and Limitation of BOTDR FOS in Underground Structure

Through the series of experiments, it is suggested that Sensor 5 developed in

Chapter 6 is the most suitable sensor to be applied in underground structure. However, there are still applicability issues and limitation associated with the current development. These are compared with Mair's work (2010) in the following table.

Table 8.1 – Applicability and Limitation with BOTDR FOR in underground structure

<u>Applicability & Limitation</u>		<u>Development in this Thesis</u>	<u>Mair's Work (2010)</u>
Physical Issues	Sensor Protection	The FOS is strengthened with special designed protection package.	Bare BOTDR FOS is used in application.
	Suitability in Underground Application	Higher chance of survival in harsh condition.	Easy damage.
	Application Condition	Can apply during the construction phase of the tunnel.	Highly damage prone while apply during the tunnel construction phase
Measurement Issues	Accuracy	10% Variance against traditional strain gauges in Experiments	10% Variance against traditional strain gauges in Experiments
		Not precise enough for small deformation analysis	Not precise enough for small deformation analysis
		Able to Capture large deformation	
		Able to reflect the strain measurement at the measurement point.	Strain measured may vary against the actual measurement point.
	Measurement Capability	The Clamping installation method allows to install multiple BOTDR FOS cable for both strain and Temperature measurement	Single BOTDR FOS is installed for either strain or temperature measurement
Technical Issues	Available Technology	Further improvement is needed on development in FOS installation accessories, more precise design clampers needed for application.	Need to improve on installation method to prevent FOS cable from moving around the rollers.
	Sensor Development	More accurate BOTDR FOS shall be developed to facilitate small deformation analysis in tunnel deformation analysis application.	

8.2 FUTURE WORK

In the ARLDP analysis presented in this thesis, the maximum plastic radius is assumed to remain unchanged regardless to the rate of excavation advancement. The analysis only correlates the plastic radius development rate with the excavation advancement rate. Such analysis is good enough for short term deformation prediction. The ARLDP model can be further improved by studying the co-relation of tunnel displacement with the long-term time-dependent squeezing effect (creep) to incorporate the ARLDP for long-term tunnel deformation analysis. There are also many possible areas for improvement in LDP analysis. The investigation of the tunnel cross section geometrical shape influence will be valuable. Based on the up to date knowledge, LDPs analysis are developed with assumption on circular tunnel cross section. However, in real life underground engineering, tunnels are constructed with many different shapes and sizes. A solution to this problem may advance the LDP analysis for underground cavern deformation analysis. The limitation with assumption on material homogeneity can also be further improvised to coincide with the real life situation.

The thesis demonstrated the possibilities of developing the real-time spatial SHM system. However, the initiation of this research is that through the BOTDR SHM system, spatial strain measurements are captured. Such strain measurements are then transformed to displacement information. With the spatial displacement values, the spatial tunnel displacement profile can be estimated through the superimposed LDP mapping technique. Together with the advancement in LDP analysis techniques, a more realistic tunnel displacement profile can be estimated and monitored. To fulfill such aim, an interrogating program is needed and such proposed interrogating program is yet to develop. In terms of the improvements in the installation method, the author suggested more engineered clamper shall be design. The ‘Point Adhesion’ installation method is worth investigating for a suitable, strong and fast hardened bonding agent to speed up the installation work. Finally, with the success launching of the spatial SHM system, it shall initiates more intense research activities on the real-time long-term spatial SHM system. The spatial measurement obtained shall be utilized efficiently to

enhance and improved the knowledge in tunneling analysis methodology.

ANNEX A

The Hoek-Brown Failure Criterion for Intact Rock applied in PHASE2 modeling.

The general equation of Hoek-Brown failure criterion adopted in PHASE2 axisymmetric analysis is written as,

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} \right)^a \quad (A1)$$

Where,

σ_1' is the major principal effective stress at failure

σ_3' is the minor principal effective stress at failure

σ_c is the uniaxial compressive strength of the rock (Table A1)

m_b and a are constants for broken rock (Table A3), m_b is obtain from $\frac{m_b}{m_i}$

which,

m_i is a constant for intact rock obtained from (Table A2)

**Table A1 – Estimates of Uniaxial Compressive Strength for intact rock
(Hoek et. al., 1992)**

Term	Uniaxial Comp. Strength σ_c (MPa)	Point Load Index I_s (MAP)	Field Estimate of Strength	Examples*
Extremely Strong	>250	>10	Rock material only chipped under repeated hammer blows	Basalt, chert, diabase. Gneiss, granite, quartzite
Very Strong	100 – 250	4 – 10	Requires many blows of geological hammer to break intact rock	Amphibolite, andesite, basalt, dolomite, gabbro, gneiss, granite, granodiorite, limestone, marble, rhyolite, tuff
Strong	50 – 100	2 – 4	Hand held specimens broken by single blow of geological hammer	Limestone, marble, phyllite, sandstone, schist, slate
Medium Strong	25 – 50	1 – 2	Firm blow with geological pick indents rock to 5mm, knife just scrapes surface	Claystone, coal, concrete, schist, shale, siltstone
Weak	5 – 25	**	Knife cuts material but too hard to shape into triaxial specimens	Chalk, rocksalt, potash
Very Weak	1 – 5	**	Material crumbles under firm blows of geological pick, can be shaped with knife	Highly weathered or altered rock
Extremely Weak	0.25 - 1	**	Indented by thumbnail	Clay gouge
*All rock types exhibit a broad range of uniaxial compressive strengths which reflect heterogeneity in compositions and anisotropy in structure. Strong rocks are characterized by well interlocked crystal fabric and few voids.				
**Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing				

Table A2 – Values of constant m_i for intact rock, by rock group (Hoek et. al., 1992)

Grain Size	Sedimentary			Metamorphic		Igneous		
	<u>Carbonate</u>	<u>Detrital</u>	<u>Chemical</u>	<u>Carbonate</u>	<u>Silicate</u>	<u>Felsic</u>	<u>Mafic</u>	<u>Mafic</u>
Coarse	Dolomite 10.1	Conglomerate (20)		Marble 9.3	Gneiss 29.2	Granite 32.7	Gabbro 25.8	Norite 21.7
Medium	Chalk 7.2	Sandstone 18.8	Chert 19.3		Amphibolite 31.2		Dolerite 15.2	
Fine	Limestone 8.4	Siltstone 9.6	Gypstone 15.5		Quartzite 23.7	Rhyolite (20)	Andesite 18.9	Basalt (17)
Very Fine		Claystone 3.4	Anhydrite 13.2		Slate 11.4			
Values shown were derived from statistical analysis of triaxial test data for each rock type. Values in parenthesis have been estimated.								

Table A3 – Estimates of m_b and a based on rock structure and surface condition
(Hoek et. al., 1992)

MODIFIED HOEK-BROWN FAILURE CRITERION $\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} \right)^a$							
STRUCTURE		SURFACE CONDITION	VERY GOOD Unweathered, discontinuous, very tight aperture, very rough surface, no infilling	GOOD Slightly weathered, continuous, tight aperture, rough surface, iron staining to no infilling	FAIR Moderately weathered, continuous, extremely narrow, smooth surfaces, hard infilling	POOR Highly weathered, continuous, very narrow, polished/slickensided surfaces, hard infilling	VERY POOR Highly weathered, continuous, narrow, polished/slickensided surfaces, soft infilling
BLOCKY – Well interlocked, undisturbed rock mass; large to very block size	$\frac{m_b}{m_i}$	0.7	0.5	0.3	0.1	
		a	0.3	0.35	0.4	0.45
VERY BLOCKY – Interlocked, partially disturbed rock mass; medium block sizes	$\frac{m_b}{m_i}$	0.3	0.2	0.1	0.04	
		a	0.4	0.45	0.5	0.5
BLOCKY/SEAMY – Folded and faulted, many intersecting joints; small blocks	$\frac{m_b}{m_i}$		0.08	0.04	0.01	0.004
		a		0.5	0.5	0.55	0.6
CRUSHED – Poorly interlocked, highly broken rock mass; very small blocks	$\frac{m_b}{m_i}$		0.03	0.015	0.003	0.001
		a		0.5	0.55	0.6	0.65

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