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Trigger Values for tunnel monitoring in SCL shallow tunnels

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Abstract

The rapid increase of the world population in the last view decades poses an enormous challenge for big cities like London, Singapore or New York. The problems concerning mobility which are linked to this make the upgrade and development of the public transport system like undergrounds and high-speed railways essential. Additionally often problems with surrounding conditions like high building density and difficult geological conditions are linked to this. To make the construction of projects which are in urban environment with low overburden possible trigger values as part of the risk management are required. In addition these projects are often within the direct zones of very sensible structures like gas pipelines, existing underground structures or basements of skyscrapers. This makes it necessary to use these trigger values to guarantee save conditions for everybody and a constant advance of the construction sequence.

The first chapter of this master thesis should give a general overview over the construction method SCL-Sprayed Concrete Lining/NATM for shallow tunnelling in soft ground and urban environment. After that the main focus should be put on the monitoring and minimization of risks of structures like this and show how “Trigger Value - systems” which have been developed in the past. Furthermore it should be shown how they have been applied and refined at projects like Farringdon Station – Crossrail (London UK) and Bank Station Capacity Upgrade. It should also be presented where innovations are possible, mistakes have been done, compromises have to be assumed and what is important at the application of safety and monitoring systems like this. In the last chapter which is the main part of this thesis it will be shown how it is possible to implement a 3D model into a 2D model in the finite element code Phase 2 in two different methods, and where there are problems and differences concerning the calibration. After the calibration of the 2D model the same determination of trigger values that was carried out for a project in 3D will be executed for two different tunnel sizes to see if it is possible and delivers similar results and if it is an opportunity to save money and time for future projects.

Kurzfassung

Der rasante Anstieg der Weltbevölkerung, vor allem in den letzten Jahrzehnten, stellt Großstädte wie London, Singapur oder New York vor immer größer werdende Herausforderungen. Die damit verbundenen Probleme bezüglich Mobilität machen den Ausbau und die Entwicklung der dort vorherrschenden öffentlichen Verkehrsmittel wie U-Bahnen oder Hochgeschwindigkeitszüge unumgänglich. Zu diesem Problem kommen oft schwierige Rahmenbedingungen wie eine hohe Bebauungsdichte und besondere geologische Bedingungen hinzu. Um die Errichtung komplexer Projekte in urbaner Umgebung mit oft sehr geringer Überlagerung und in direkter Umgebung sehr sensibler Bauwerke wie Gaspipelines, bestehender U-Bahnlinien oder Gründungen von Hochhäusern für alle Beteiligten so sicher wie möglich zu gestalten und um konstanten Fortschritt gewährleisten zu können, werden im Untertagebau sogenannte „trigger values" als ein Teil des Risikomanagements eingesetzt.

Das Erste Kapitel dieser Masterarbeit soll einen generellen Überblick über das Konstruktionsverfahren Spritzbeton im Tunnelbau (SCL - Sprayed Concrete Lining) für oberflächennahe Tunnelbauwerke in weichem Boden und urbaner Umgebung geben. Das Hauptaugenmerk soll dann auf die Überwachung und Risikominimierung solcher Bauwerke gelegt werden und zeigen, wie sich solche „Alarm Wert Systeme" entwickelt haben und in unterschiedlicher Ausführung an namhaften Projekten wie Farringdon Station – Crossrail (London UK) oder Bank Station Capacity Upgrade angewendet wurden. Des Weiteren soll gezeigt werden wo Innovationen möglich sind, wo Fehler begangen wurden, Kompromisse eingegangen werden müssen und worauf es bei der Anwendung solcher Sicherheits- und Messsysteme ankommt. Im letzten Kapitel soll dann mit zwei verschieden Methoden gezeigt werden wie ein solches System von 3D in 2D umsetzbar ist und untersucht werden wo die Probleme und Unterschiede bezüglich der Kalibrierung liegen. Nach der Kalibrierung wird die gleiche Bestimmung von „trigger values" wie sie für ein Projekt in 3D durchgeführt wurde für zwei verschiede Tunnelgrößen in 2D gemacht. Dadurch soll untersucht werden, ob es überhaupt möglich ist und ähnliche Resultate liefert beziehungsweise ob dadurch bei zukünftigen Projekten Geld und Zeit eingespart werden können.

“Hope for the best, but prepare for the worst“

(Benjamin Disraeli, 1804-1881)

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List of abbreviations

AITES/ITA	International Tunnelling and Underground Space Association
CIRIA	Construction industry research and information association
CLC	Capacity Limit Curve
DIN	Deutsche Industrienorm
DRM	Daily Review Meetings
DSP	Dr. Sauer & Partners
EC7	Eurocode 7
EC2	Eurocode 2
FE	Finite element
FOS	Fibre optic sensor
GL	Ground Load
ICE	Institution of Civil Engineering
KPI	Key Performance Indicators
NATM	New Austrian Tunnelling Method
ÖGG	Österreichische Gemeinschaft für Geotechnik
RESS	Required Excavation and Support Sheet
SCL	Sprayed Concrete Lining
SEM	Sequential Excavation Method
SLS	Serviceability Limit State
UK	United Kingdom
ULS	Ultimate Limit State
USDA	United States Department of Agriculture
WL	Water load

List of symbols

A1	[-]	Action factor (EC 7, 2.4.7.3.4.2)
D_1	[m]	Diameter
E_u	[N/m²]	Undrained elastic soil modulus
E	[N/m²]	Young´s modulus
f_c	[N]	Compressive force
f_{cu} (t)	[N/m²]	Compressive strength of concrete over time
H	[N]	Horizontal Force
h	[h]	Height of overburden
K_0	[-]	Factor of earth pressure at rest
l	[m]	Length
M1	[-]	Material resistance (EC 7, 2.4.7.3.4.2)
Pe	[m]	Perimeter
P	[N/m²]	Pressure
P´	[N/m²]	Inside tunnel pressure
P_0´	[N/m²]	Outside tunnel pressure
R1	[-]	Resistance factor (EC 7, 2.4.7.3.4.2)
r	[m]	Radius
u_o	[N/m²]	Pore water pressure
V	[N]	Vertical Force
V_t	[m³]	Ground loss
V_s	[m³]	Settlement volume
z_0	[m]	Distance between tunnel center and surface

Greek symbols

β	[-]	stiffness reduction factor
λ	[-]	relaxation factor
φ	[°]	Friction angle
γ_{Soil}	[N/m^3]	Soil unit Weight
$\mathcal{E}$	[-]	Total strain rate
$\mathcal{E}_{el}$	[-]	Elastic strain rate
$\mathcal{E}_{pl}$	[-]	Plastic strain rate
$\mathcal{E}_{ve}$	[-]	Viscous strain rate
$\mathcal{E}_{f}$	[-]	Flow strain rate
$\mathcal{E}_{3}^{P}$	[-]	Minor elastic strain
$\mathcal{E}_{cp}^{P}$	[-]	Plastic peak strain
$\mathcal{E}_{c}$	[-]	Concrete strain
δ_1	[m]	Perimeter deformation
η	[-]	Safety level factor
σ_{vo}	[N/m^2]	Vertical stress according to overburden
$\sigma(t)$	[N/m^2]	Applied sprayed concrete stress
σ_3	[N/m^2]	Compressive load
σ_c	[N/m^2]	Compressive strength

1 Introduction

The rapid expansion of big cities like London or New York in the past decades causes overcrowded underground stations and train coaches nearly every day and forced these cities to increase their capacity as quick as possible. As a consequence many big projects have been undertaken in the last view years to increase the capacity of public networks and big underground stations. Most of the time these projects are right in high profile urban environment with many existing structures and utilities which represents a very big challenge for designers and construction companies.

To deal with these challenges so-called trigger values are implemented in the design and construction process of underground projects. In the past some approaches have been used and developed in different projects. In this thesis firstly a rough overview of the sprayed concrete lining method will be presented. After that different approaches for the determination of trigger values will be explained and the occurrence of deformations and different measurement methods will be explained.

After this a 2D finite element model of a pilot tunnel construction with following platform tunnel enlargement will be calibrated to a 3D model. The purpose is to see which assumptions and changes have to be made and to investigate the behaviour of the construction in 2D compared to 3D resulting from variations of different soil parameters. Also a comparison of two different finite element approaches and the difference of the results will be presented and discussed. All these calculations will be executed in the 2D finite element program Phase 2. The objective of this thesis is to see how a 3D model can be calibrated into a 2D software and which results according to deformation and lining stress relationship can be achieved in a 2D model compared to 3D model.

2 SCL tunnelling method

This chapter should give a rough overview of the history and the development of the New Austrian Tunnelling Method/Sprayed Concrete Lining method (NATM/SCL) and what distinguishes this method from other methods which are used to build an underground space. The technique of this method was pioneered by the Austrians Ladislaus von Rabcewicz, Leopold Müller and Franz Pacher in the last decades of the twentieth century rather for applications in rock. The reason why this milestone of the tunnelling history was done especially in Austria is that the country is dominated by the Alps which are difficult to pass and where high overburdens in combination with rapidly changing geologic conditions are no rarity. In order to deal with these problems engineers had to find new solutions to face them and to make tunneling applicable for almost all conditions. The name of the method which can be understand also as a design philosophy and a construction method was born at the thirteenth Geomechanics Colloquium in Salzburg by Professor Ladislaus von Rabcewicz in 1962 (ÖGG, 2009). Today the method is also known in the USA as the Sequential Excavation Method (SEM) and in the United Kingdom as the Sprayed Concrete Lining Method (SCL).

After sprayed concrete was used in some engineering projects for rock support at the beginning of the nineteenth century the SCL method was first applied in soft ground and urban environment for the construction of a metro in Frankfurt in Germany in 1968 and many others followed after the successful completion. Many innovations could be achieved in the following years by practical experiences of different projects. In this thesis the main focus will be put on the application of the NATM in the United Kingdom, especially London, where the method after some successful applications suffered a set back because of the collapse at the Heathrow Express Rail Link Station in 1994. Today this construction method is very common for big projects all over the world. Currently this method is used for projects in the UK like the Bank Station Capacity Upgrade and Crossrail (both in London) which is one of the biggest infrastructure projects in Europe at the moment.

2.1 Construction method and main properties

In addition to the before mentioned problems of shallow overburden in combination with difficult ground conditions and high building density designers often have to deal with an already very busy underground environment of existing metro stations and running tunnels. According to these conditions designers had to make adaptations in the theory, construction and design of the original method in rock to make it suitable for challenging environment in soft ground. In this context it should be mentioned that there are many different approaches of NATM design around the world because of differences in geology, groundwater conditions, overburden, geometry of stations as well as waterproofing systems and equipment used during construction. So there is no general policy which can describe this method (DSP, 2004).

The main invention of the traditional method was that both, the tunnel and the surrounding rock/soil, create an interacting support structure. This should be achieved by a controlled deformation of the surrounding rock/soil in combination with inspection of ground conditions in advance and continuous monitoring which will be discussed in detail later. After Thomas the ground can be grouped in three main categories: hard rock, blocky rock and soft ground (see Table 1). This is a very simplified categorization and it is well known that there is not a clear defined transition between this three ground types and most of the time it is a combination of them. In the further discussion with soft ground it is meant clay because of the predominance of the so-called London Clay in the region around London. A categorization of clay according to the United States Department of Agriculture is shown in Figure 1.

The definitions of rock and soil according to the guideline of the ÖGG (Österreichische Gemeinschaft für Geotechnik) for the geotechnical design of underground structures with conventional method are as follows.

Rock: Aggregate, consisting of minimal components, developed from natural processes, characterized by the types and amount of the minerals and grain structure.

Soil: Accumulation of anorganic solid varigrained particels with occasional organic admixtures, the properties are predominately governed by granulometric composition, the compaction and the water content.

category	soft ground	blocky rock	hard rock
description	soil and weak rocks	weak to moderately strong rock	massive strong rocks
behaviour	continuum	discontinuum	continuum
strengths	$\sigma c < 1$ MPa	$1 \leq \sigma c \leq 50$ MPa	50 MPa $\leq \sigma c$
examples	sands, clays, chalk	limestone, sandstone	basalt, granite

Table 1: Simplified categorization of ground types (Thomas, 2009)

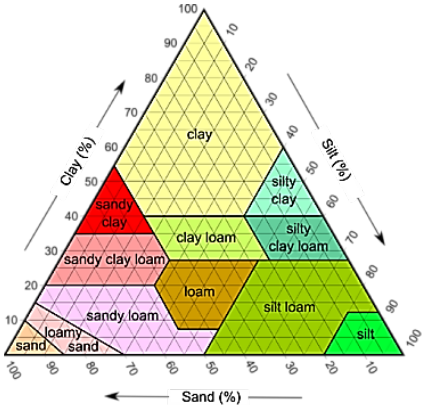

Figure 1: Soil types by clay, silt and sand composition (USDA)

In urban environment acceptable deformations due to tunnelling are limited because of existing utilities, existing underground structures and the possible disturbance of third parties. Normally soft ground or weak rock is acting like a single unisotropic mass with a strength of approximately 0 - 10 MPa. According to this and the normally very short stand up time, like for example of the London Clay 18-24h, the ground needs full support immediately and an early ring closure as soon as possible to control deformations (Thomas, 2009).

To achieve these, primary support systems like bolting, lattice girder, wire mesh in combination with sprayed concrete are installed according to the load deformation characteristic of the surrounding ground. In special cases where the open face is straight next to flowing ground or sand lenses additional grouting, spiling, ground freezing, dewatering or other modifications can be added. In tunnels where settlements have to be limited to a minimum, full-face excavations are often not suitable and for this the face can be split into smaller parts either in horizontal or vertical direction or combined (DSP, 2005). This flexibility allows building cross sections and junctions in almost every size and profile and makes it very favourable for applications in soft ground conditions in combination with limited space like a complex underground station upgrade which is shown in Figure 2.

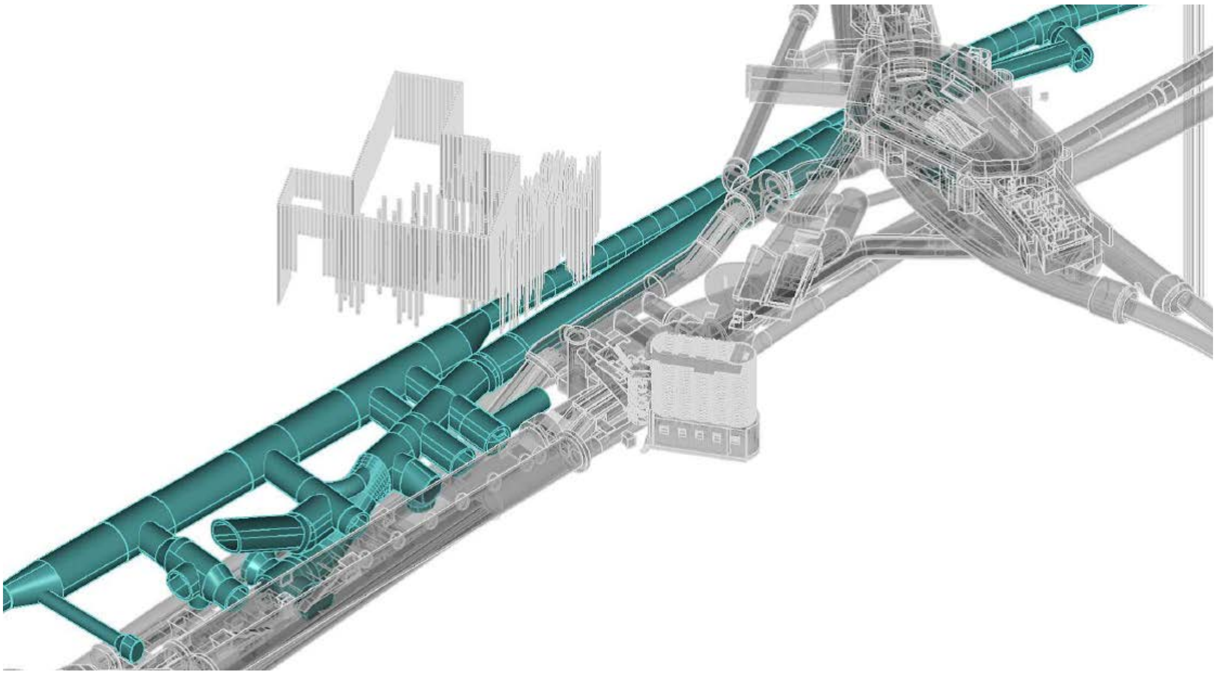

Figure 2: Underground station upgrade London UK (Dr. Sauer & Partners, 2015)

For the construction there are many different sequences for face subdivisions like side galleries, pilot tunnel in connection with later enlargement and so on. Probably the most usual methods are the vertical division in top heading – bench – invert and the side gallery method which are shown in cross and longitudinal section in Figure 3 und Figure 4.

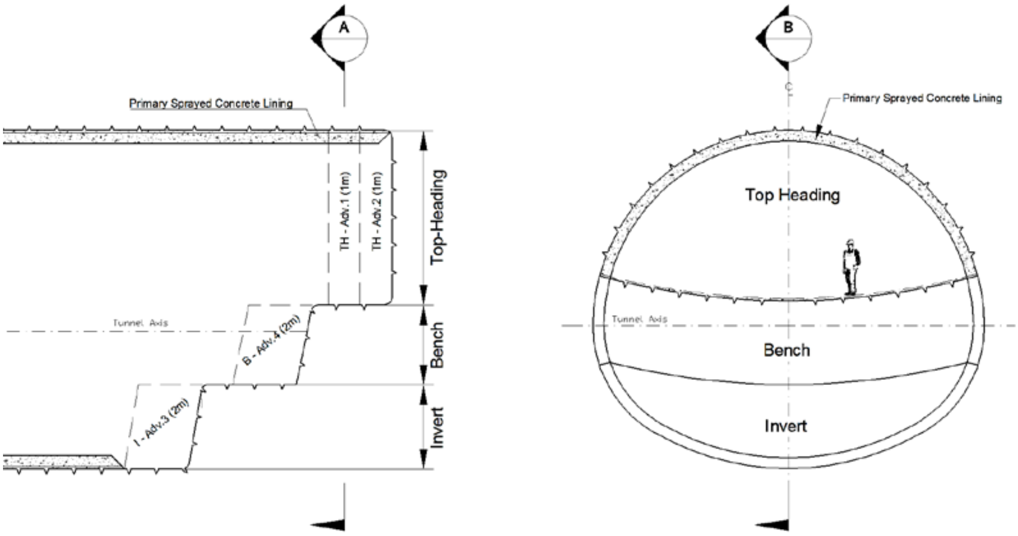

Figure 3: Excavation sequence top heading – bench – invert (DSP, 2016)

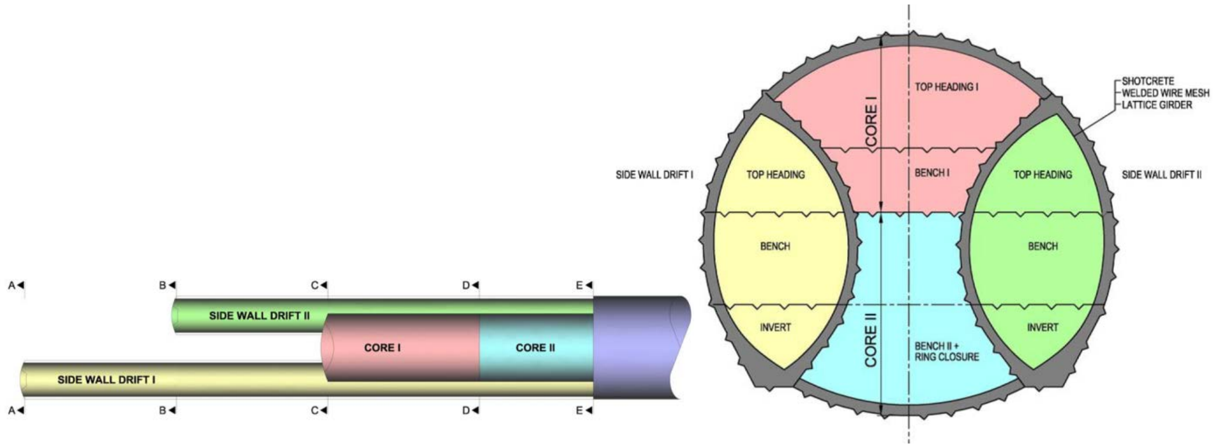

Figure 4: Side wall drift excavation method (The Austrian Practice of Conventional Tunnelling, 2009)

As seen in the longitudinal section the excavation advances incrementally in small steps and with immediately support afterwards. To keep settlements to acceptable limits the excavation rounds are often reduced to 1m for the top heading and 2m- 4m for bench and invert excavation but can be significantly different for different ground conditions and settlement predictions. To reduce the settlements as well as the stresses in the lining the distance between top heading and following bench and invert ring closure the excavation sequence has also been related to the quality and early strength distribution of the young sprayed concrete.

2.2 Performance of the lining

Karl Terzaghi (1946) was the first who investigated the so-called arching effect and load on the lining in defect rock ground conditions in his paper “Rock defects and loads on tunnel support”. A schematic illustration of the investigated case can be seen in Figure 5.

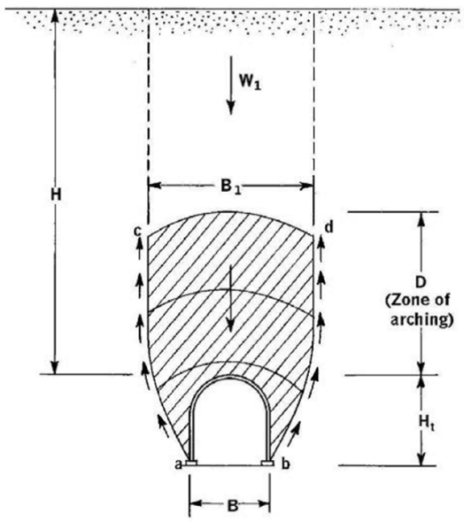

Figure 5: Loading on the tunnel lining (Terzaghi, 1946)

This approach can also be used as estimation for the loading on the lining of a tunnel in soft soil. In the case of a shallow urban tunnel in soft clay with normally consolidated conditions we can assume that the full weight of the overburden acts on the lining of the tunnel and that the horizontal pressure reaches as minimum the pressure in normal consolidation "in rest". The horizontal pressure highly depends on the flexibility of the lining and the compressibility of the surrounding ground. Therefore the average lining pressure lies in between the following values explained in Formula 1 and 2 (Brand; Brunner, 1981).

$$P = \sigma_{vo'} + u_o = \sigma_{vo} \tag{1}$$

$$P = \sigma_{vo'} * (1\text{-}\sin \varphi') + u_o \tag{2}$$

P pressure on the lining
$\sigma_{vo'}$ effective vertical stress
u_o pore pressure
σ_{vo} total vertical stress
sin φ´ effective friction angle

Over the last decades there have been significant developments of SCL tunnels with respect to the lining. The trend went from the "double shell" to the "single shell" to the present "composite shell" method (Su, 2013). The last one is currently used in most of the projects and typically consists of 100 – 300mm of primary sprayed concrete with or without fibre reinforcement, a sheet waterproofing membrane or sprayed waterproofing layer and a secondary lining of sprayed or cast in situ concrete. Although this method is currently one of the most popular ones in the construction of SCL tunnels there are still uncertainties associated with it. One of the biggest is the behaviour of the primary and secondary lining in view of the interface with the waterproofing membrane. This issue should already be respected in the design of the structure because of the long term loads on the linings. The different loading conditions for a "double shell" and a "composite shell" tunnel lining can be seen in Figure 6. In the calculations and sensitivity analysis only the primary lining of the SCL pilot tunnel and platform tunnel with a thickness of 200 and 300mm will be considered.

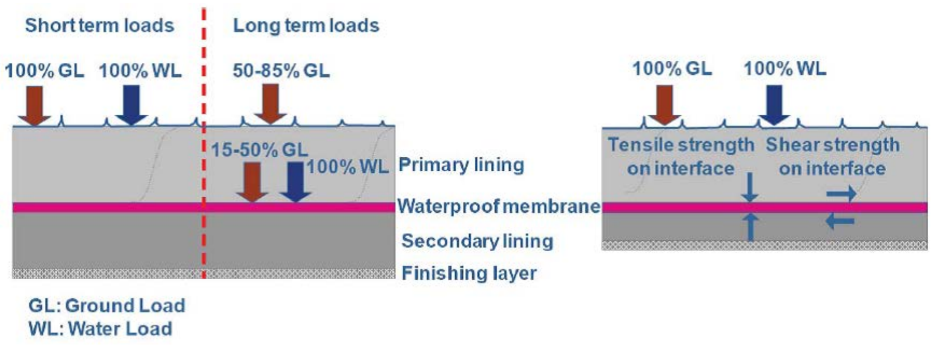

Figure 6: Load conditions for double shell and composite shell tunnel lining (Su, 2013)

Fenner (1938) and Pacher (1964) have been one of the first who investigated the loadings on tunnel linings and who developed the so-called ground response curve which represents the deformations of the tunnel crown in response of pressure on the tunnel lining. In Figure 7 different ground response curves can be seen for shallow tunnels and varying ground conditions. A very stable and strong ground where no support is needed is shown in Curve I. Meanwhile Curve II shows a moderately stiff ground where some support has to be applied Curve III represents the curve of a very soft ground where significant support is needed to achieve equilibrium. For this simplification of the ground response curve some assumptions like initial stress field (K_0 = 1), circular tunnel, constant lining thickness and isotropic ground behaviour have to be postulated (Duddeck, 1979).

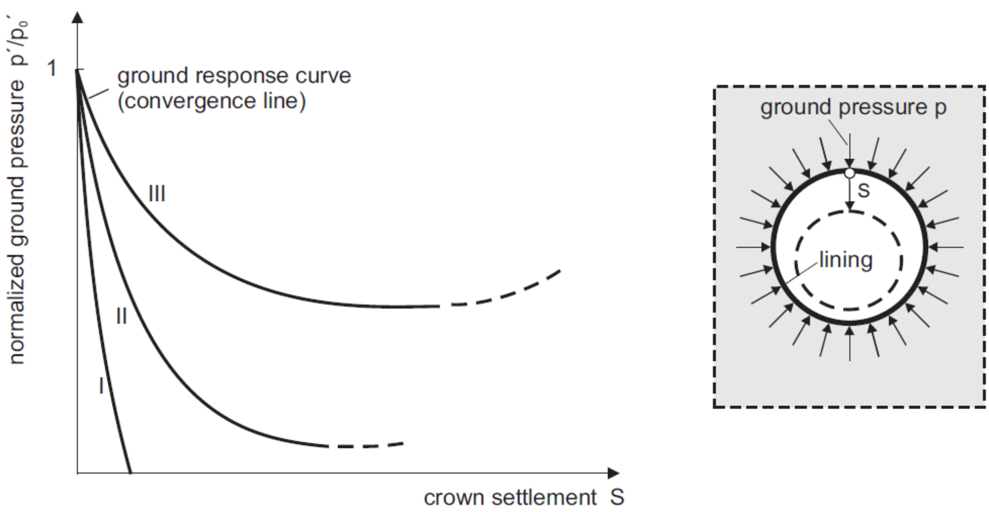

Figure 7: Ground response curves for different ground types after Pacher (1964)

In urban areas where ground deformations and surface settlements are limited the lining support has to be installed very quickly to provide stability. Therefore the lining has to carry more loads which is represented by line b in Figure 8. On the other side line a in Figure 8 represents a case of a deep tunneling where larger displacements can be allowed to reduce ground pressure on the lining and in the following save lining thickness.

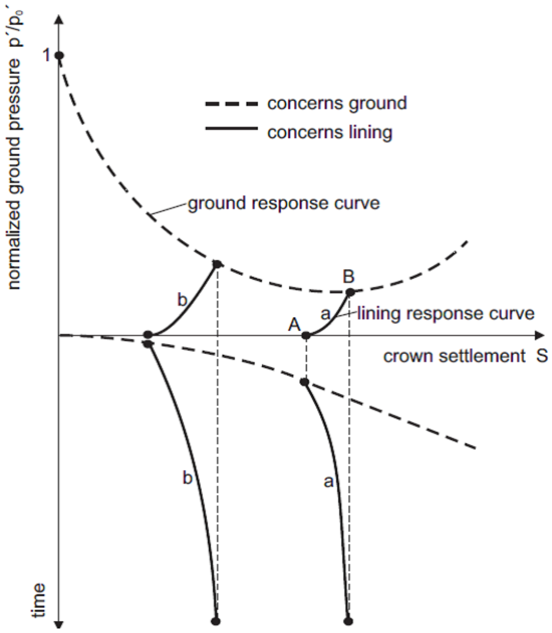

Figure 8: Relationship of ground pressure and timing of support installation after Pacher (1964)

2.3 Material properties

“Sprayed concrete is concrete which is conveyed under pressure through a pneumatic hose or pipe and projected into place at high velocity, with simultaneous compaction.” (DIN 18551, 1992). Typically the product consists of the same basic ingredients like normal concrete which are water, cement and aggregate. Additional to this sprayed concrete needs some various composition refinements like accelerator, plasticizer, stabilizer, microsilica, fly ash and a higher water/cement ratio to make it pumpable and easier to spray.

Today the use of fibre reinforced sprayed concrete gets more and more important because conventional steel reinforcements are often not suitable for sprayed concrete works especially for difficult intersections. Generally there are two processes of sprayed concrete called the wet and dry mix processes which both have their advantages and disadvantages.

Wet mix: This means that the sprayed concrete is delivered in ready workable mix on the site. To apply the shotcrete on the surface only air and accelerators have to be added. The main advantages of this method are the higher output capacity (up to 25 m^3/h), less rebound and dust generation as well as higher quality and compressive strength in the long term. As disadvantages the higher work intensity and the fact that the concrete gets waste if it is not applied within a certain time can be mentioned (Thomas, 2009; Schlumpf, Höfler, 2004).

Dry mix: For this process the shotcrete is transported ready mixed but without water on the site. To apply it on the surface compressed air conveyed the mixture to the nozzle where water and accelerator are added controlled by the nozzleman. This procedure is often used in smaller tunnels where smaller outputs are required and the focus is put on high early strength. The advantages are the high early strength, small space requirement and the fact that it can be stored in silos on the site which means no concrete waste at the end of the day. The disadvantages are the high generation of rebound and dust and that the quality of the shotcrete mixture itself highly depends on the skills of the nozzleman (Thomas, 2009; Schlumpf, Höfler, 2004).

2.3.1 Strength in compression

In tunnel engineering the parameter of strength in compression is most of the time the main criteria for the application of the material mixture. In tunnelling especially in urban areas with limited stand up time of the surrounding ground the early strength, which means not older than 24 hours of young sprayed concrete, is of particular significance (ÖVBB). The difference of strength distribution between wet mix and dry mix sprayed concrete of different age can be seen in Table 2.

Age	Dry mix sprayed concrete	Wet mix 6% alkali-free accelerator	Dry mix 6% alkali-free accelerator
6 minutes	0.95	0.5	-
1 hour	1.3	1.0	-
1 day	23.0	15.0	17.0
56 days	41.0	61.0	39.0

Table 2: Difference of wet and dry mix sprayed concrete (Lukas et al. 1998)

The strength development of young applied sprayed concrete which also has a high influence on the dust and rebound amount can be classified in three categories J1, J2, J3 (EN 14487-1) which can be seen in Figure 9.

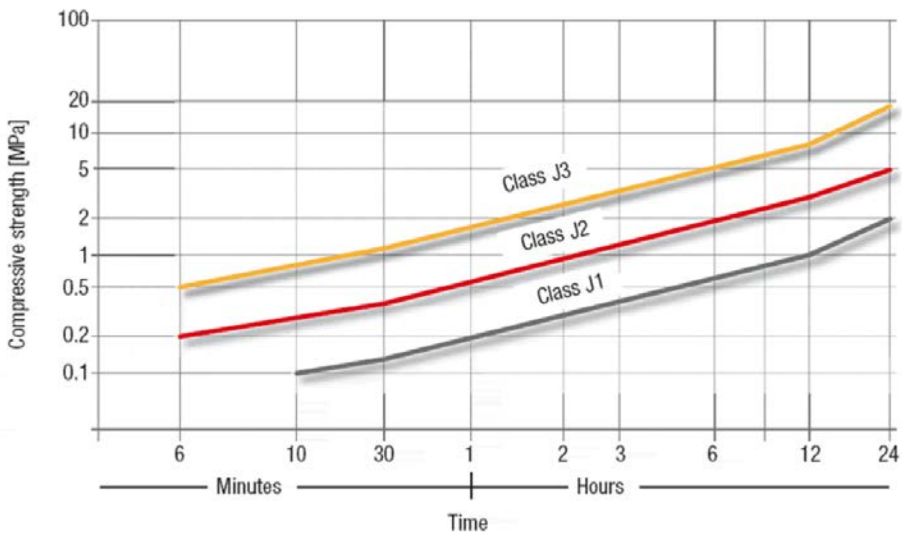

Figure 9: Early age strength development of sprayed concrete (Sika, 2012)

The strength of young sprayed concrete highly depends on the mixture and on the imperfections of the cement-aggregate interaction including pores and micro cracks (Thomas, 2009). At this point it has to be mentioned that especially the very early strength gain of young sprayed concrete is highly influenced by the temperature of the sprayed concrete (Jones, Li, Ahuja, 2014). It is widely accepted that for the calculation of the compressive strength and strength in tension of normal concrete and sprayed concrete the same parameters can be used. From this follows the following equation for the compressive strength of fibre reinforced sprayed concrete.

$$f_{cd} = \frac{\alpha_{cc} * f_{ck}}{\gamma_c} \qquad \text{(EN 1992-1-1,3.15) (EC 2) (3)}$$

α_{cc} is the coefficient taking account of long term effect on the compressive strength and unfavourable effects resulting from the way the load is applied

f_{ck} cylindrical compressive strength of concrete

γ_c is the partial safety factor for concrete

A typical stress – strain curve according to the Eurocode 2 (EN 1992-1-1) of concrete for uniaxial loading in compression and stress-strain curves of sprayed concrete of different ages are shown in Figure 10.

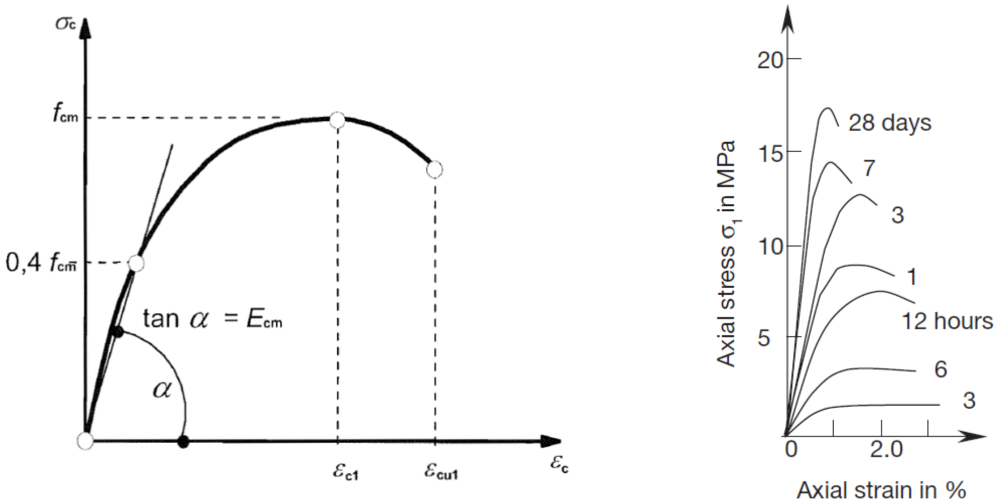

Figure 10: Stress strain relation (EC 2) & sprayed concrete of different age (Thomas, 2009)

For the purpose of this study it was assumed that the steel fibre reinforced material has a linear-elastic perfectly plastic behaviour in compression. In tension it is assumed that the material has a linear-elastic perfectly plastic behviour first and changes to a constant behaviour afterwards. The used model for SCL with linear-elastic perfectly plastic behaviour in compression as well as the behaviour in tension can be seen in Figure 11. The detailed material data of the SCL used in the FE Analysis will be shown later in chapter 5 in connection with all other parameters of the calculations.

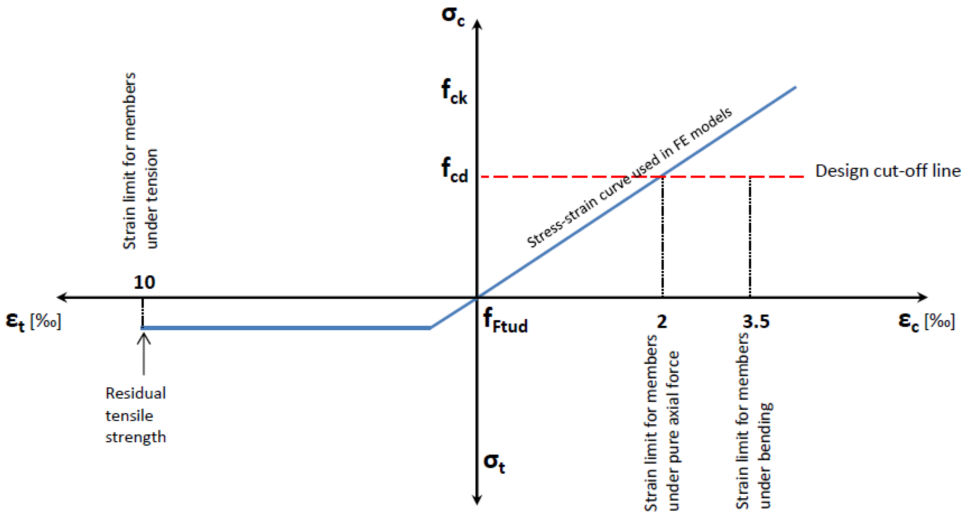

Figure 11: Stress-strain curve for steel fibre reinforced concrete (DSP, 2015)

2.3.2 Strength in tension

The tensile strength of concrete normally plays a secondary role during the design process because of the very small capacity compared to the compressive strength. To compensate the very brittle behaviour of concrete and increase the ductility most of the time steel fibres or steel meshes are added to the mixture. Figure 12 shows the influence of steel fibre application and the behaviour in tension of sprayed concrete of different age. It can be seen that shortly after the application (up to 4 hours) the sprayed concrete behaves plastically with a limit strain up to 0.5%. With age this decreases rapidly and reaches only 0.05% after five hours (Thomas, 2009). For the consideration of the tensile strength of fibre reinforced sprayed concrete in the design the Eurocode considers the following equation.

$$f_{\text{ctd}} = \frac{\alpha_{\text{ct}} * f_{\text{ctk, 0.05}}}{\gamma_c} = \frac{\alpha_{\text{ct}} * 0.70 * 0.30 * (f_{\text{ck}})^{2/3}}{\gamma_c} \qquad \text{(EN 1992-1-1, 3.16) (EC 2) (4)}$$

α_{ct} is a coefficient taking account of long term effects on the tensile strength and of unfavourable effects, resulting from the way the load is applied

$f_{ct,\ 0.05}$ nominal 5% fractile value of concrete tensile strength

γ_c partial safety factor for concrete

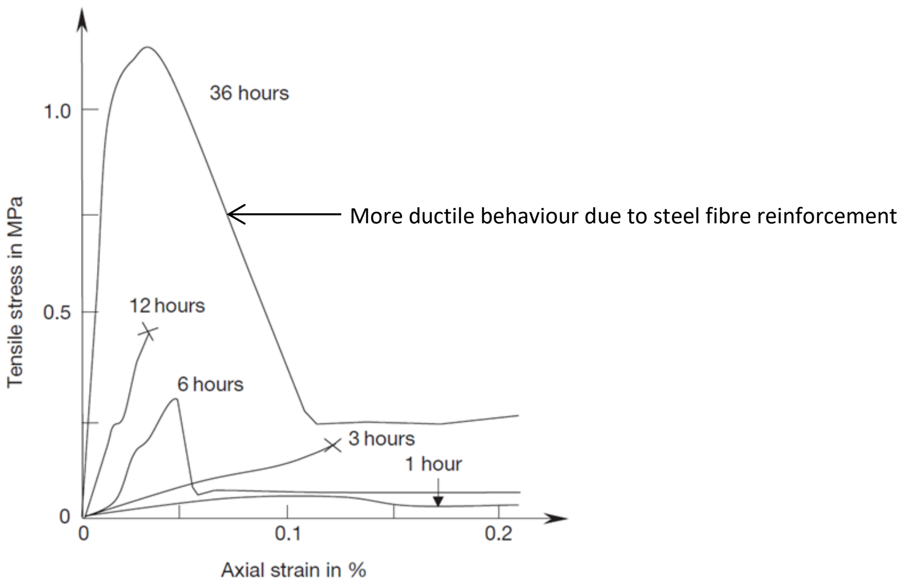

Figure 12: Tension behaviour at different ages (Euram, 1998)

2.4 Design approaches – observational method

The reason that geotechnical engineers often have to deal with high uncertainties of subsurface geological conditions and support systems like SCL or compensation grouting makes it necessary for them to base their design on conservative interpretation of the parameters of the existing ground. The so called observational method makes the use of monitoring and observation to one of the most important factors during construction to check the behaviour in correlation to the predicted one. Prerequisite is to define the predicted behaviour of the structure in terms of what can be measured later during construction. Nicholsson et al. 1999 described the method as a continuous managed and integrated process of design, construction control, monitoring and review, enabling appropriate preciously defined modification to be incorporated during design.

Karl Terzaghi and Ralph E. Peck were the first who recognized the design approach in 1967 in the idea of an efficient cost saving construction without compromising safety. The idea was to base the design on the most probable not on the most unfavourable conditions and deal with the remaining risks with monitoring and observation. According to this Peck (1969) identified the following eight steps for the application of the observational method.

I. Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail

II. Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions

III. Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions

IV. Selection of quantities to be observed as a construction proceeds and calculation of their anticipated values on the basis of the working hypothesis

V. Calculation of the values of same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions

VI. Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis

VII. Measuring of quantities to be observed and evaluated on actual conditions

VIII. Modifications of design to suit actual conditions

3 In tunnel monitoring

During the construction of tunnels and other underground structures like caverns deformations cannot be avoided. According to this and the existence of sensitive structures in the direct vicinity as well as the uniqueness of every underground structure monitoring is an integral part of the design and construction process of underground infrastructure especially in urban environment with shallow overburden. The objectives to be monitored during tunnelling in urban environment are different from those in mountain areas. In a mountain environment the main purpose is to control the ground pressure to avoid a collapse due to roof collapse or bottom heave. In urban environment on the other hand the main objective is to prevent structures and utilities of third parties from ground deformations and impact of the construction. Due to the fact that urban tunnels are usually shallow loadings on the lining are small compared to those in mountain environment (Kavvadas, 2003). The fact that the deformations have to be kept very small means that the precision of the instruments have to be very high and they should be installed as early as possible.

Before the focus will be put on monitoring in detail it well be shortly discussed where, why and how settlements induced by tunnelling occur. The settlements of the surrounding structures in response of settlements of the ground depends on the geological, hydro-geological conditions of the ground, geometry and depth of the tunnel as well as geometry, construction and the conditions of the subsurface structure itself (ITA, 2006). In urban tunnelling with soft and cohesive soils especially instability of the tunnel face, day lighting collapse, support failure, surface settlements and drawdown of ground water table leading to settlements play the major roles which have to be observed during and after the construction (ÖGG, 2014). The response of the ground according to the changes of the initial situation of the ground lead to horizontal and vertical displacements and in further consequence to the development of a settlement trough as shown in Figure 13.

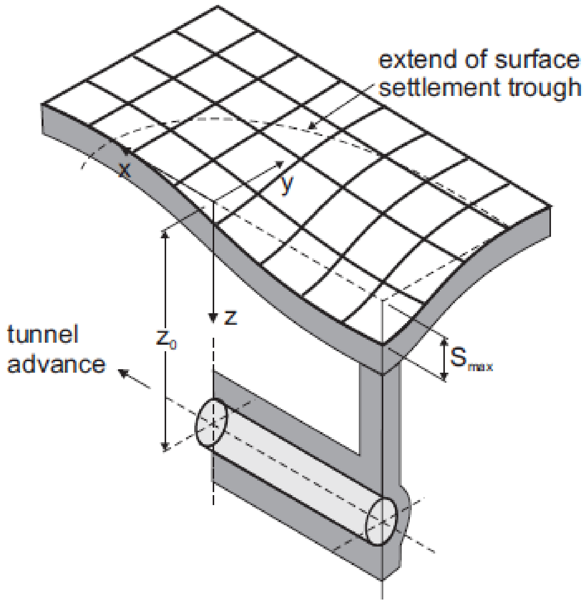

Figure 13: Three dimensional induced settlement trough (Attewell, 1986)

It is well known that the distribution of ground movements in cross and longitudinal directions can be predicted by the Gaussian curve which was shown first for the transverse settlement trough by Schmidt & Peck (1969) and later by Attewell and Goodman (1982) for the longitudinal trough.

In tunnelling the monitoring of those behaviour plays a key role and is done by instrumentations installed at the surface and within the tunnel. The main difference between them is that surface instruments can already be installed before the beginning of the construction and several zero measurements can be executed. On the other hand the instrumentation operating within the tunnel like optical reflector targets and borehole rod extensometer etc. can be installed at the earliest when the tunnel advances and the tunnel face is 2-4m ahead to avoid interferences with construction works for SCL and excavation (Kavvadas, 2003). At this time a major part of the deformations one tunnel diameter ahead and 1,5 diameter behind the tunnel face have already occurred which can be seen in Figure 14. The dimension of the deformations which already took place highly depend on the ground conditions, quantity of support and the construction sequence and can exceed more than 50% for specific soil conditions. For this reason the designers have to consider this at the very beginning of their design and finite element calculations.

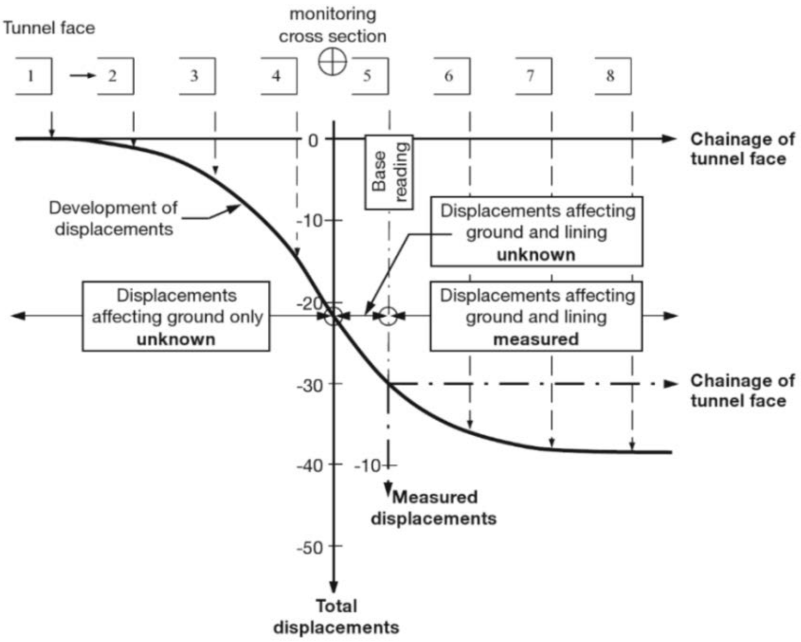

Figure 14: Displacement development ahead of the tunnel face (Stärk, 2009)

3.1 Design of instrumentation and monitoring program

In this thesis the focus is on the case of shallow urban tunnels and measurements which can be carried out inside the tunnel. Instrumentation and monitoring of structures can roughly be separated in during and after construction and includes in general the following measurements (Dunnicliff, 1993).

[1] Convergence of the tunnel wall, and usually crest settlement and spring-line closure
[2] Deformations at ground surface including settlements and tilts of surface structures
[3] Deformations in the ground around the tunnel

The instrumentation and monitoring system required for specific projects depends largely on the used design, hazard sensitiveness and the risk level which can be taken. In high-risk urban environments like underground metro stations in combination with the before discussed observational method and SCL construction method monitoring plays an important part of the whole project as part of the risk management process as well as construction control and has to be clearly specified at the beginning of the project. Unclear specifications like the frequencies of measurements or the responsibility of data evaluation can lead to accidents, higher costs, or the lack of adequate detection of anomalies and trends during the proceeding of the project (ITA, 2015). Especially critical sections like junctions or penetration into existing structures like running tunnels have to be monitored carefully. However the measurements should be kept as simple as possible to facilitate the analysis process. According to the ICE – Institution of Civil Engineering a monitoring system should correspond to the following listed project specific functional requirements of which the most important ones are describe below:

[1] Extent of the area to be monitored
[2] Frequency of monitoring
[3] Accuracy
[4] Precision
[5] Density of monitoring
[6] The range of measurements to be undertaken
[7] System robustness and reliability
[8] Requirements for system recovery after a failure
[9] Requirements for data processing and usage

[1] The areas to be observed can be splitted into the "Vigilance Zone" and the "Active Zone" and are given in the tender/contractual documents. In the vigilance zone background monitoring and close-out monitoring are carried out. The dimension of this area depends mainly on the surrounding conditions (greenfield or urban environment). In the active zone which is the part of the vigilance zone where the excavation is carried out, active monitoring is applied (ITA, 2015).

[2] The monitoring frequency can be understood as the slowest frequency between acquisition, transmission and storage frequency (ITA, 2012) and depends on the distance of the measurement points to the tunnel face and the following factors (ICE, 2011).

[1] Rate at which change is expected to develop
[2] Feedback requirements for practical control of a construction process
[3] Requirements of any contingency or emergency plans
[4] Stage of the work (different during background monitoring and main construction phase)
[5] Undertakings given to satisfy third parties

It has to be kept in mind that high frequency readings are not always advantageous because of the reason that too much information can lead to problems in the evaluation and the management of data (ICE, 2011).

[3] and [4] Accuracy of a system is the degree of correctness of measurements or how close they are to the true quantity. Precision is expressed in ± x units and is the degree to which measurements under unchanged conditions show the same results (Standing, 2008). These requirements highly depend on the usage of the collected data. In the case of this thesis these two characteristics of measurement systems play a major part in the definition of trigger values because of their direct influence if a value is exceeded or not. According to this the designer has to assess the degree of ground movements and specify the applied system (ICE, 2011). For a better understanding see Figure 15.

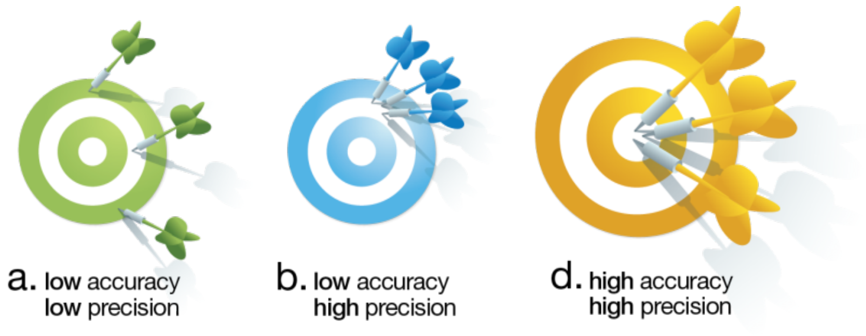

Figure 15: Description of Accuracy and Precision (Exelate, 2014)

The main requirements for the in tunnel monitoring deformations are the observation of convergences of the tunnel walls, the stresses, strains, cracks in the lining and the pore water pressures as well as deformations in the surrounding ground (ITA, 2011). In this thesis the focus is put only on the instrumentation which is of interest for this special topic. In this case this is the observation of the total tunnel wall displacements with installed robotic total stations in combination with optical reflector pins.

3.1.1 Convergences of tunnel walls

Generally there are two types to measure convergences inside the tunnel. On the one hand it can be executed with tape extensometers which measure distances and on the other hand with 3D optical measurements in combination with robotic or manually total stations to measure 3D coordinates. The main disadvantages of tape extensometers are in fact that only movements along a specific line can be measured and that it obstructs construction works during the time of measurements. However the high accuracy (0.2mm up to 10m), easy application and maintenance make it suitable for some application where the disruption of traffic can be accepted. For specific urban projects where the space is already very limited and continuous monitoring is required 3D optical measurements by robotic stations have replaced tape extensometers. Today real time monitoring is state of the art and is used in most of the underground projects. The robotic total stations are installed on special bases on the wall or crown of the tunnels as seen in Figure 16 and read the installed reflector targets in before defined intervals. The collected data is then sent to an online platform where the designers and site engineers can interpret them and make decisions. Normally five to seven reflector pins are installed every cross section in intervals of 10-20m with increasing density at difficult areas like transitions and intersections with other tunnels (Riaz, 2015).

By interpreting the collected data, environment conditions like temperature, dust and the long term deformation of the tunnel have to be considered and the measurements corrected if necessary. To get absolute coordinates the measurements have to be referenced to fixed points which are typically located outside the tunnel.

Figure 16: Installed reflector pin left, installed robotic station right (ÖGG, 2014)

4 Trigger values for in-tunnel monitoring in shallow tunnels

4.1 Definition

Trigger values which are a synonym for “hazard warning levels” or “response levels” are a common way in geotechnical engineering to show the actual behaviour of the structure in comparison of what was predicted from pre-calculations. The systems normally include the time in which detections have to be made and reviews, decision and modification have to be carried out. Normally the systems are designed for explicit project conditions and updated during construction when more data is available. Basically they are pre-defined values and can be set on every measurable parameter. A very likely solution is to set them on parameters like tilt, deformation, water pressure and strain. In this thesis it will be focused only on the determination of trigger values which can be set on in tunnel measurable deformations. The definition of this trigger values is challenging because of the reason that the designers have to make estimations respectively to the thickness of the tunnel lining and the behaviour of the surrounding ground. In tunnelling it is usual to use a so-called “traffic light system” during monitoring to assess the behaviour of the underground structure and define levels of response and the importance to react or modify the design. At this point it has to be said that if triggers are exceeded and there is not a clear process for responsibilities they are “useless”. There has to be a clear flow with time limits for the allocation of information between all involved participants. How such a system can be assembled according to the ÖGG Guideline for Geotechnical Monitoring in Conventional Tunnelling (2014) is shown in Figure 17.

How these systems are established depends primarily on the environment where the tunnel has to be built. In greenfield conditions the starting position is completely different than in urban environment. While in the greenfield only the SLS - Serviceability Limit State of the structure itself is important in the urban environment especially the influence of third parties in the vicinity of the structure often impose tighter movement criteria. Additionally to this usually the systems are formed of two, three or four levels/zones but this depends on the different approaches which will be presented in chapter 4.2. The most common one includes green, amber and red levels and were proved to be very robust and clear for everybody.

Green represents that everything is fine or even more favourable than expected. This means that construction can proceed as usual. Normally this value is based on the results of finite element calculations.

Amber shows that there are deviations from the expected behaviour. In this zone which is also called the decision making zone modification plans are put in place and additional measurements with an increased frequency are carried out.

Red symbolizes the alarm level. If this level is reached unexpected behaviour happens but a safety margin still exists. This means that all the planed modifications are activated, all affected parties should be notified and work has to stop immediately. The reach of this value/level does not mean that the structure is collapsing but often it is connected with tolerable damage of other structures which have been set with third parties before construction started (Burland et al. 1977). The only correct way to define the red level is the implementation of a sensitivity analysis which has to be related to the most influencing parameters, the so-called KPI – Key Performance Indicators. The definition of the KPI highly depends on the experience of the designer and the existing ground conditions.

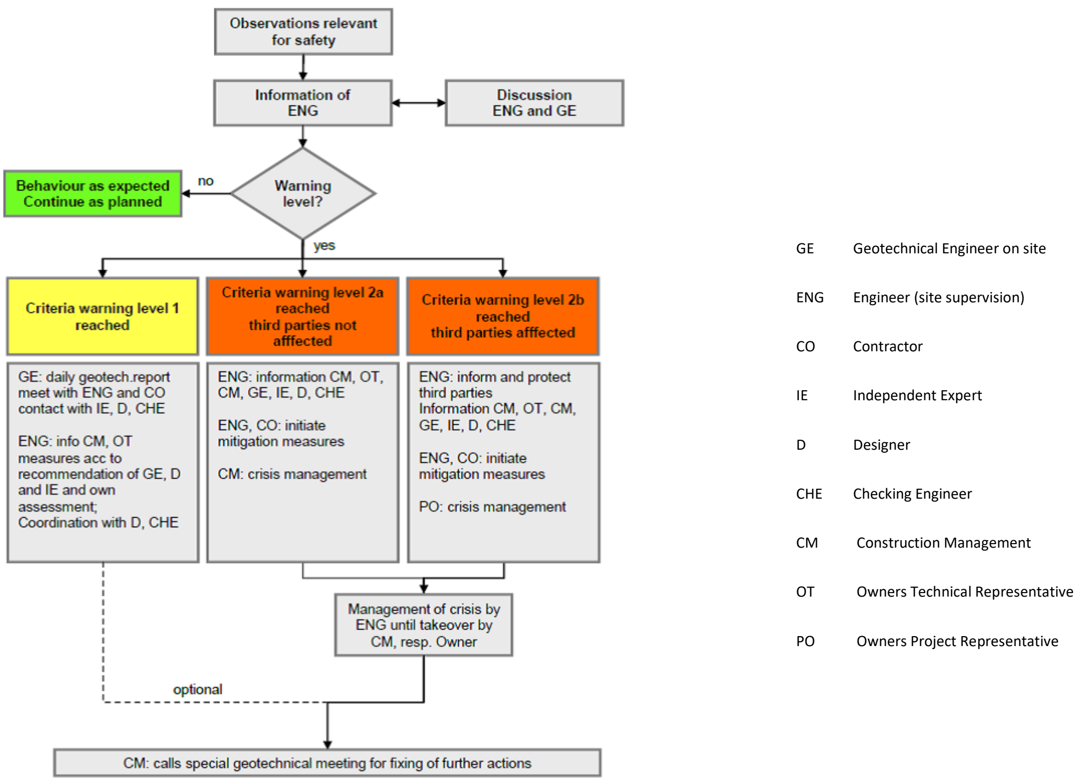

Figure 17: Geotechnical safety management plan (ÖGG Guideline, 2014)

Often more important than the trigger values itself is the so-called "trend" or rate of change which can be seen as the movement over time. This is especially the case for NATM / SCL construction sequences where there are no intermediate trigger values for the different construction steps. The main advantage of the trend is that it can be already observed at a very early stage of the construction which means immediate identification of abnormal changes and more time for reaction and modifications. A trend curve for convergences of a) most probable conditions of a previous tunnel section and b) with worse ground conditions, large convergences and a very steep gradient can be seen in Figure 18 (CIRIA, 1999).

a)

b)

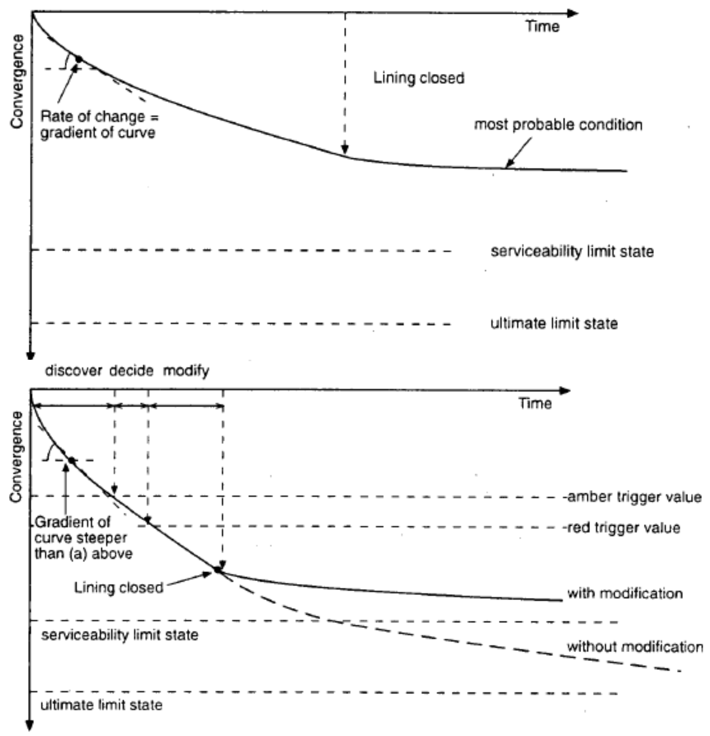

Figure 18: Trend curves: a) "most probable" conditions b) worse ground conditions (CIRIA, 1999)

The instrumentation data in comparison with the defined trigger values, the trend curves and the current geotechnical conditions are reviewed and discussed with all involved parties like contractor, engineers and design representatives in so-called "Daily Review Meetings" (DRM). Also possible modifications for support and excavation sequences as well as requirements of additional measurements are reviewed. The information flow of a DRM is illustrated in Figure 19 (Thomas, 2009). The conclusion and instructions for the next section are written down in so-called RESS "Required Excavation & Support Sheets" after the DRM´s and agreed by all parties (see appendix). The RESS sheets should include the section to which the RESS is applicable, the support to be installed, excavation sequence, monitoring to be installed in the tunnel section, soil conditioning, reference to relevant design drawings and so on (ICE, 2010).

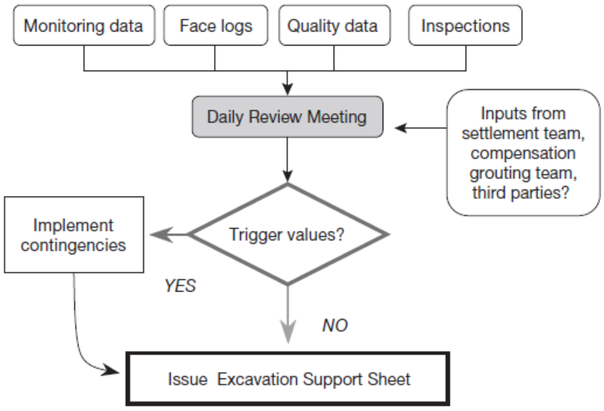

Figure 19: Information flowchart of DRM (Thomas, 2009)

4.2 Approaches

Before the different approaches of defining trigger values or zones are presented it should be mentioned that there are many different alternatives to do this and the procedure highly depends on the experience of the designer and the particular project requirements as well as the predominant failure mechanism.

4.2.1 Eurocode 7

The Design Approach 1 of the Eurocode 7 for characteristic ground parameters which can be seen in the Formula 7 considers safety factors for actions (1.4) and ground strength parameters (1.5). This approach was developed for different geotechnical problems and not for the definition of trigger values in particular. In the application for trigger values this gives combined a safety factor of 2.1 (1.4 x 1.5) for the green trigger value.

Combination 1: A1 “+” M1 “+” R1 (7)

Where “+” implies: “to be combined with” (EC7)

For the amber trigger value which represents the SLS of the structure actions are respected by a factor of 1.0 and ground parameters by 1.5. This means that actions are still full developed but safety factors are only added to the ground parameters.

The red trigger value demonstrates that a critical behaviour is reached and therefore a factor 1.0 for action and 1.1 for ground parameters is considered. This means that there is still a 10% safety margin before it is expected that the structure behaves instable.

4.2.2 Alun Thomas (2009)

In 2009 Alun Thomas presented in his introduction about sprayed concrete lined tunnels an approach to determine trigger values derived from convergences. To correlate tunnel deformations with the prediction of the settlement he compared volume loss and FE calculations. From the flowchart of his approach presented in Figure 20 it follows that he multiplied displacements of the green trigger with factors 1.4 and 2.0 for the amber and red triggers. Consequential this leads to questionable trigger values because of the non-consideration of the nonlinearity of the relationship between stresses and displacements. The results of this are trigger values which are highly underestimated in comparison of the potential lining capacity. For the trigger system of the Farringdon Station – Crossrail UK exactly this approach was applied which lead to the effect that trigger levels were often exceeded and many questions about the safety of the construction appeared.

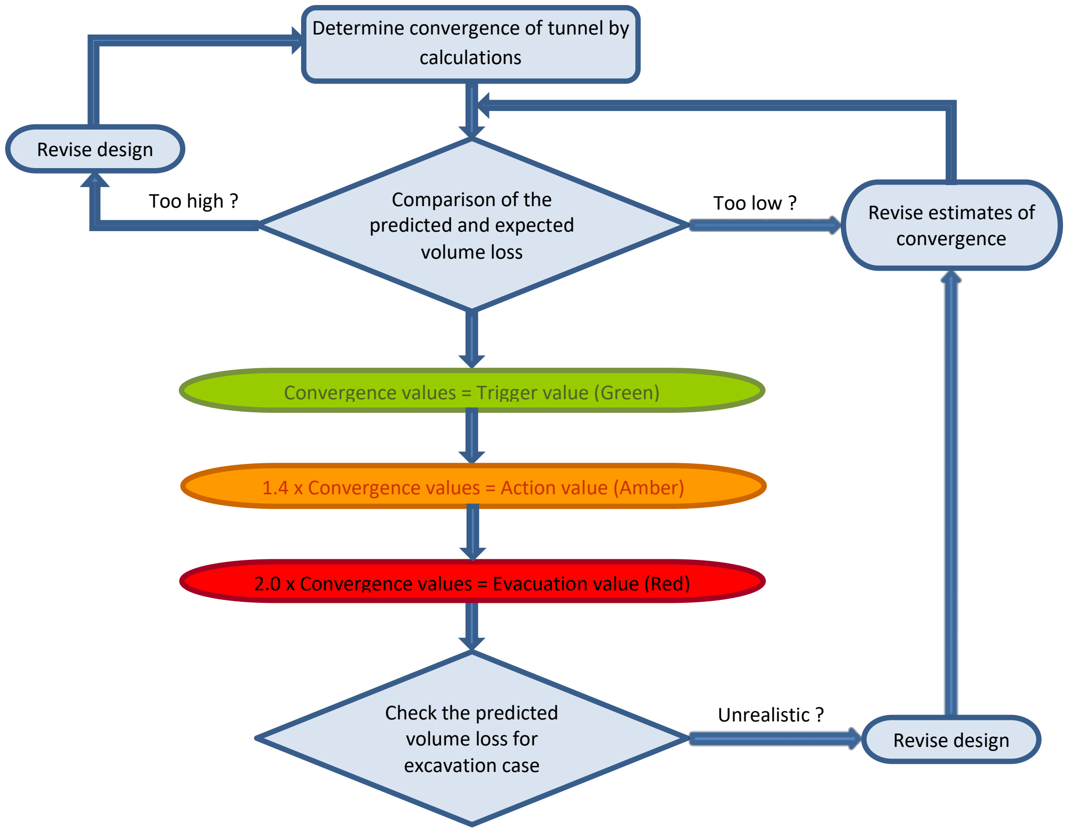

Figure 20: Approach for trigger values by Thomas (2009)

4.2.3 J. Jäger (2005)

Another practice was presented by Jäger in 2005 were he suggested a solution to determine trigger values on lining strains according to lining stresses instead of basing them on displacements in the tunnel. In his approach he used the following four steps:

[1] Measuring displacements in the tunnel
[2] Strain calculation according to the measured displacements
[3] Determination of the characteristic of the project specific sprayed concrete
[4] Correlation of the calculated strains with the condition of strain development in the sprayed concrete to evaluate the safety factor (not the absolute strain is important but the progress of the strain over time)

For his approach some simplifications like the constant stress distribution of the sprayed concrete stress and strain over the whole thickness of the lining as well as constant strain between two measured points have to be assumed. Especially the first part is very questionable because of the very complex distribution of the sprayed concrete stress and strain distribution over the thickness from higher hydration on the air side to the decreasing hydration at the center. It also has to be scrutinized how accurate the strains can be calculated by measuring the displacements of the pins at the tunnel wall. In his analysis for the sprayed concrete he used a 1-d constitutive law (Hooke´s law) where he assumed isothermal conditions, without shrinkage strains. The formulas for this were based on Hellmich (1999) and Sercombe (2000).

To define trigger levels he assumed a constant ratio between $f_{cu}(t)$ which is the time dependent sprayed concrete strength over time and the applied sprayed concrete stress σ(t) (see Formula 8) If 100% is reached this means η = 1 and the ultimate strength of the SCL has been reached (Rokahr, Zachow 1997).

$$\eta = \frac{f_{cu}(t)}{\sigma(t)} \tag{8}$$

From the given stress path seen in Formula 9 follows a corresponding strain path Ɛ(t) with a certain safety margin which can be calculated on the basis of the constitutive relationship (Formula 10). Figure 21 below shows the strain dependency for different safety levels over time.

$$\sigma(t) = \frac{f_{cu}(t)}{\eta} \tag{9}$$

Constitutive relationship:

$$\sigma = E(\xi) * \varepsilon_{el} \tag{10}$$

Total strain consists of elastic, plastic, viscous and flow components

$$\varepsilon_{tot} = \varepsilon_{el} + \varepsilon_{pl} + \varepsilon_{ve} + \varepsilon_{f} \tag{11}$$

From this follows

$$\sigma = E(\xi) * (\varepsilon - \varepsilon_{pl} - \varepsilon_{ve} - \varepsilon_{f}) \tag{12}$$

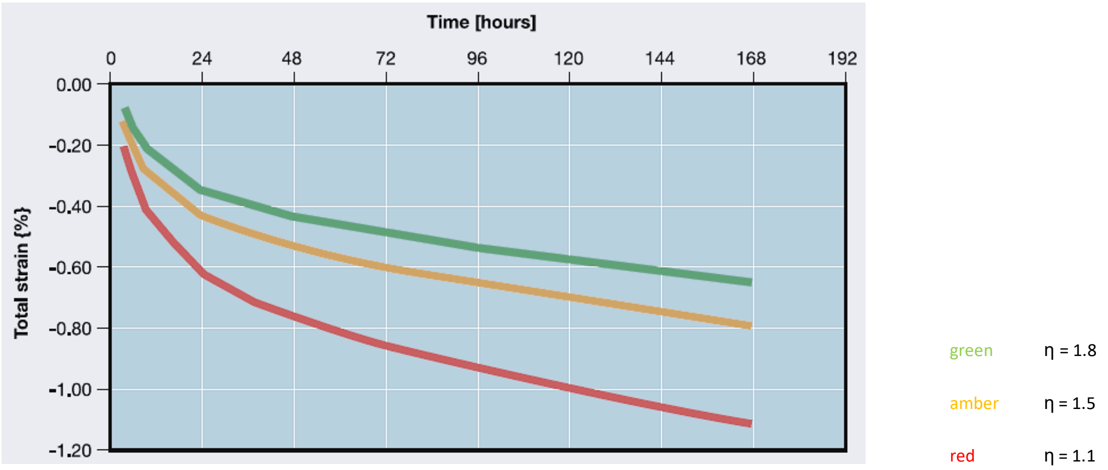

Figure 21: Strain paths corresponding to safety levels η (Jäger, 2005)

4.2.4 Nasekhian, Moldovan, Spyridis (DSP - BSCU London, 2016)

Due to the reason that the definition of too low trigger values in consequence of the determination of too conservative ground parameters can lead to significant disturbance for the project schedule, Nasekhian et al. (2016) tried to find a new approach for a new methodology to obtain trigger values for tunnel monitoring deformations.

Therefore the designers developed a system of four warning levels namely green, amber, red and black like it is illustrated in Figure 22. Within the green zone and the amber trigger it means that everything is like expected and construction can proceed. The red trigger is a transition between the expected behaviour and the ULS (black trigger) of the construction and can be set independently from the designer. At the black zone work has to stop, affected parties have to be notified, contingency plans must be applied and the construction team has to be evacuated.

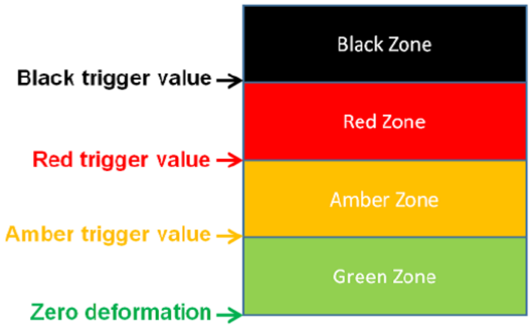

Figure 22: Developed traffic light system (Nasekhian, 2016)

For the determination of trigger values a 3D FE model of a typical pilot tunnel with a following top-heading bench and invert enlargement was used. Because of the reason that ground parameters are very uncertain at the beginning of every project and play a major role in the definition of trigger values a sensitivity analysis with before defined key performance indicators was carried out. In the analysis these were the K_0 and E_u values where K_0 is the factor of earth pressure at rest and E_u the undrained elastic soil modulus. It was identified that the ratio between the base / best estimate case (K_0 = 1.1 and E_u = 750cu) and the "worst case" leads to difference in deformation of about 30% in vertical and 50% in horizontal direction which gives on average a factor of 1.4. The results of the sensitivity analysis are shown in Table 3.

Nr.	Analysis name	Ko	Eu	Average vertical deformation_u3 [mm]	Average horizontal deformation_u2 [mm]
1	Base Case	1.1	750Cu	-14	10
2	Ko=0.6	0.6	750Cu	-15	5
3	Ko=1.2	1.2	750Cu	-13	12
4	Eu=500Cu	1.1	500Cu	-18	15
5	Eu=1000Cu	1.1	1000Cu	-10	8

Table 3: Results of 3D sensitivity analysis (Nasekhian, 2016)

After that they plotted their results of the lining section forces in dependency of the vertical and horizontal deformations against the Capacity Limit Curve – CLC which represents the characteristic strength of the lining (see Figure 23).

The principal of this capacity limit curve is to show the maximum lining design capacity in consideration of bending moment and normal force. The inspection of all cases has shown that the section forces of all cases are within the same range and that the worst case differs from the base case only by 10%. The ratio of 1.4 between base case and worst case in deformations and 1.1 in section forces is an indicator of the non-linearity between deformations and lining forces.

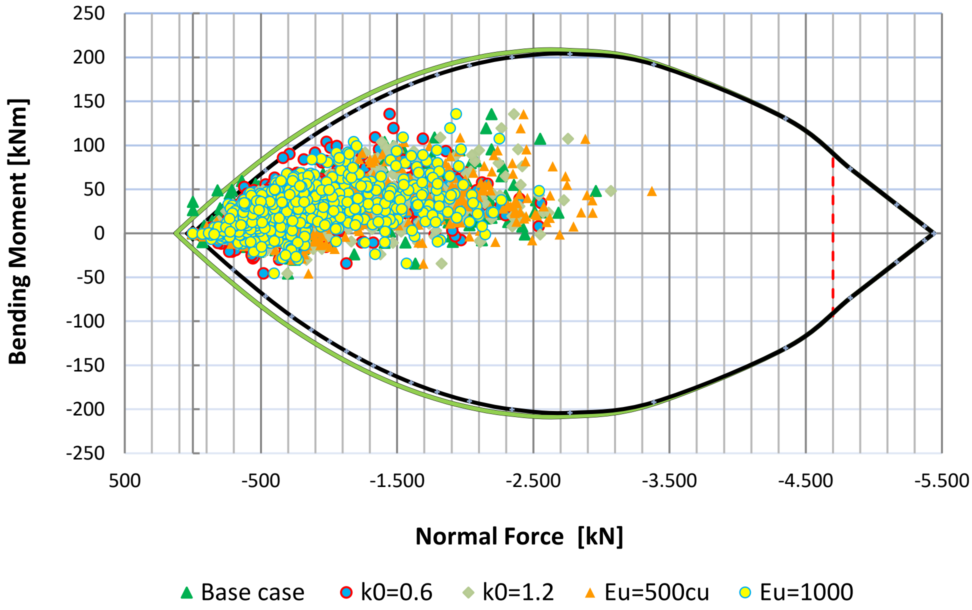

Figure 23: Bending moments and axial forces for all cases (Nasekhian, 2016)

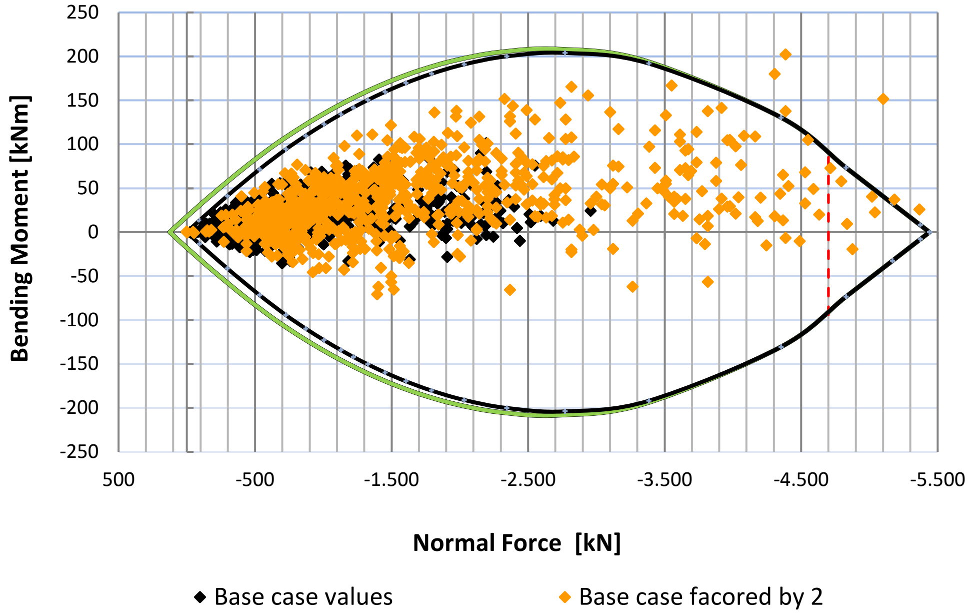

Figure 24: Base case multiplied by a factor of 2 (Nasekhian, 2016)

In order to use the whole capacity of the lining it was found out that all section forces can be multiplied by a factor of 2.0 (CLC factor) which can be seen in Figure 24. This factor as well as the factor of 1.4/1.1 for the non-linearity of deformations and lining forces was fed into the determination of the red trigger value. For the black trigger also the factors of short term loading (1.2) and material partial factor (1.5) have been considered. The whole concept is summarized in Figure 25 below.

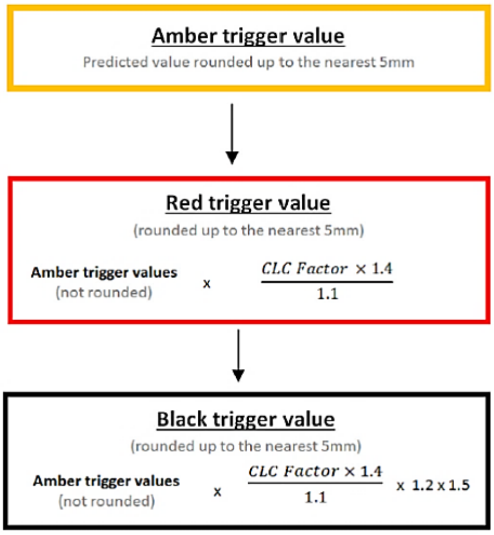

Figure 25: Determination of trigger values flowchart (Nasekhian, 2016)

In this approach it was illustrated that there is a large factor of additional lining capacity which can be considered in the design. Also the nonlinearity between stresses in the lining and deformations has been shown in a demonstrative example of a shallow tunnel in soft ground and a new approach for the determination of trigger values has been presented. From this approach it can be seen that the margins between the different zones of the expected and ULS behaviour of the construction is much higher than for example in the Eurocode 7 or the approach of Thomas (2009).

5 Conclusion literature research

In the first part of this master thesis it has been shown that monitoring is one of the key factors in geotechnical engineering especially in shallow urban tunneling which has to be considered from the very early stage on. For the successful realization of tunneling projects the designer has to cooperate with the construction team and the link between these two is the measurement team. Their task is to provide the measurement results to compare the predicted behaviour investigated in the design with those occurring in reality during the construction process. Therefore the most important instruments and methods with their limitations to measure the parameters have been presented. Also the different approaches according to the determination of trigger values which all have their advantages, disadvantages, assumptions and limitations have been introduced. All of them have their own procedure to determine limitations and reaction levels which are important later in the construction process.

Because of the time dependency and the cost aspect of generating and calculating 3D FE models with a following sensitivity analysis it will be investigated how it is possible to calibrate a 2D model by means of a 3D model and execute the same procedure in a much accelerated procedure. If the investigated results of this would be similar it would be a big advantage in future projects and could save a lot of costs and time. The calibration will be shown in two different ways which are explained in paragraph 6.1. In the following a sensitivity analysis will be carried out for a pilot tunnel (small size) as well as for a platform tunnel (big size). The reason of this is to investigate if the CLC factor and the relationship between displacements and lining forces behave in the same manner for two different sizes as well as in comparison of a 2D model to a 3D model.

The calibration of the following 2D model will be executed to the results of the 3D analysis of Nasekhian et al. (2016) mentioned in chapter 4. Also the results of the 2D determination of the CLC factor and the factor for the non-linearity will be compared to this approach to see if there are any deviations from the results in 3D.

6 Case Study

Like mentioned above this part of the thesis will show the calibration of a 2D finite element Phase 2 model to a 3D model in Abaqus. It will be shown how it is possible to simulate the 3D effects in 2D in order to achieve the same values and accuracy in a 2D model rather than in a very complex 3D simulation. Also the relationship between deformations and lining stresses will be observed to see if they behave differently or in the same manner in 2D and 3D. These will be executed for two tunnels of different sizes to see the influence of parameter changes also on different tunnel sizes. Therefore the procedure which has been carried out at a typical London underground station upgrade project to determine the trigger values in a 3D sensitivity analysis in Abaqus will be investigated and tried to understand the influence of the variation of different parameters on the results of the calculations. After that the geometry of the complex 3D model will be implemented in a 2D section and all the important parameters of the soil and concrete structures will be fed in. At this point it has also to be mentioned that all 3D models and results of the 3D sensitivity analysis which have been used for the calibration and comparison of the results already existed. In the sensitivity analysis it will be focused on the construction of a typical pilot tunnel (small size) followed by a top heading, bench and invert enlargement of a platform tunnel (large size). This case which is illustrated in Figure 26 represents an almost greenfield construction with nearly no influence on other existing structures.

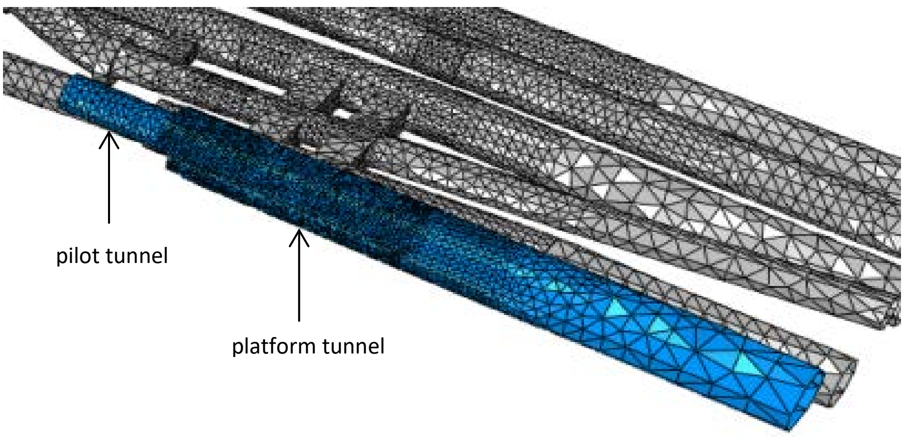

Figure 26: FE model of pilot tunnel and platform tunnel enlargement

6.1 Hand calculations

To see if a 2D hand calculation can lead to the same results than in a finite element analysis program like Simulia Abaqus or Phase 2 first hand calculations have been carried out for a simple case. Therefore a circular tunnel with a constant pressure load on the outside of the lining and the soil and concrete parameters which can be seen in Table 4 have been chosen. The hand calculations have been compared to the results of the simple finite element analysis in Abaqus and can be seen below.

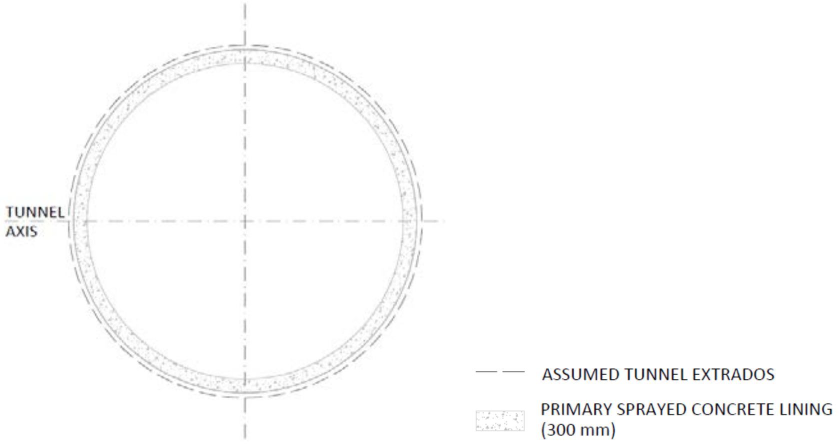

Figure 27: Used profile shape for calculation

K_0 [-]	γ_{Soil} [kN/ m³]	Overburden [m]	Young´s modulus concrete C32/40 [N/m²]	Tunnel radius [m]	Lining thickness [m]
1,0	20	20	32e9	3.88	0.3

Table 4: Parameters for stress and deformation calculation review

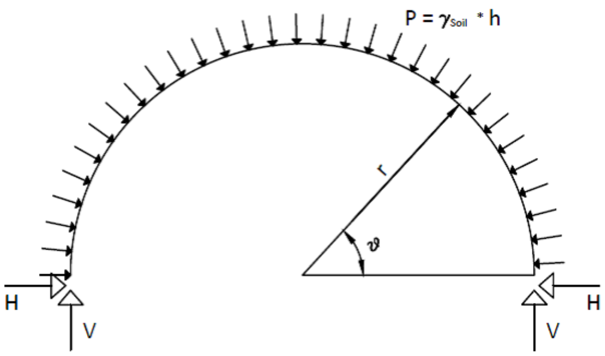

Figure 28: Lining stress system sketch

$$P = \gamma_{\text{Soil}} * h = 20 * 10 = 200 \frac{kN}{m^2} = 200\,000 \; \frac{N}{m^2}$$

$$H = 0$$

$$2V = \int_0^{\pi} \mathrm{P} * \mathrm{r} * sin\vartheta \; d\vartheta$$

$$\mathrm{V} = \left[\frac{\mathrm{P} * \mathrm{r} * \cos\vartheta}{2}\right]_0^{\pi} \rightarrow V = \frac{P{*}r}{2} + \frac{P{*}r}{2} = P * r$$

$$V = 200\,000 \; \frac{N}{m^2} * 3.88m * 1m = 776\,000 \; N$$

ABAQUS = 775 900 N✓

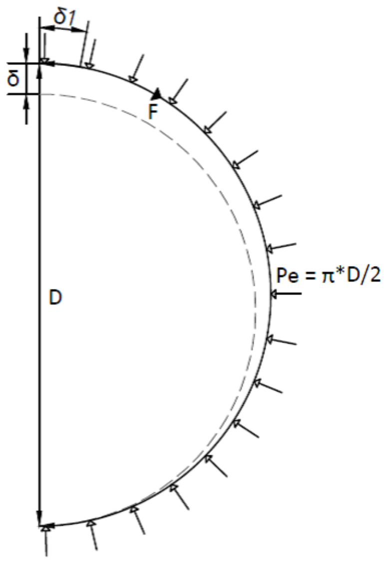

Figure 29: Lining deformation

$$\sigma = E * \varepsilon$$

$$\sigma = \frac{F}{A} = \frac{V}{1 * 0.3} = 258\;6667 \; \frac{N}{m^2}$$ → ABAQUS = 259 2000 $\frac{N}{m^2}$ ✓

$$\frac{\mathrm{dPe}}{\mathrm{dD}} = \frac{\pi}{2} \rightarrow \mathrm{CONSTANT}$$

$$\varepsilon = \frac{\delta_1}{D_1} = \frac{d\delta_1}{dD_1}$$

$$\delta_1 = \varepsilon * D_1 = \frac{Fl}{EA} = \frac{F{*}\frac{\pi{*}D}{2}}{EA} = \frac{776\,000 * \frac{\pi{*}7.76}{2}}{32e9 * 1 * 0.3} = 0{,}0009853 \; / \; \frac{\pi}{2} = 0.000627 \; m$$

→ ABAQUS = 0.000628 m ✓

The results of the hand calculations show that it is possible to achieve the same results for simple cross sections in a simple 2D hand calculation than in a complex 3D finite element program. However for more complex geometries in combination with existing structures a 2D or 3D finite element model is most of the time necessary and is applied in normally every geotechnical project.

6.2 2D finite element analysis methods

For the calibration of the 2D model in Phase 2 two common practices will be applied in this thesis. This is on the one hand the “pressure method” and on the other hand the “stiffness method”. It has to be said that the pressure method was carried out only for the calibration of the platform tunnel to see the differences of application and applied relaxation factors in order to calibrate the deformations for the base case to the model in 3D.

Before the results of the calculations are presented a short explanation of these two different methods which were used during the analysis are provided and their most important characteristics will be explained.

6.2.1 Pressure method

The method developed by Panet and Guenot in 1982 describes the relaxation prior the lining support installation and gives an estimation of the occurring volume loss. Therefore a force vector 1-λ * Po (relaxation factor λ) is applied on the excavation boundary instead of the soil to simulate the unloading effect according to the proceeding construction. The procedure of the sequence is illustrated in Figure 30. The relaxation factor value λ varies from λ=0 at the initial stage and λ=1 when the lining construction has been completed. So the internal pressure (p) at the different stages of the construction sequence is a fraction of the in-situ pressure (p0) and depends on the relaxation factor λ. The pressure for different stages can be described by the following equation (Panet, Guenot, 1982).

$$p = (1 - \lambda) * p_0 \tag{13}$$

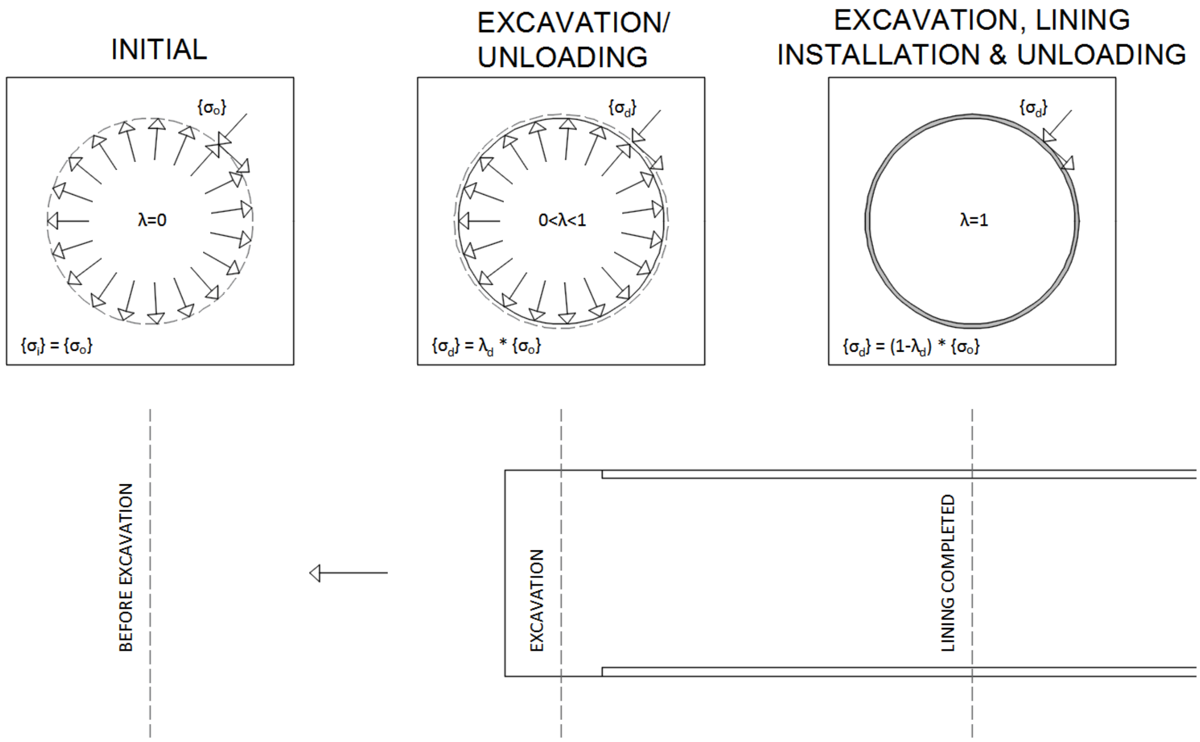

Figure 30: Pressure method (Potts, Zdravkovic, 2009)

6.2.2 Stiffness method

This approach which is also known as the core replacement method (Phase 2) was developed by Swoboda for the simulation of SCL tunneling sequence in 1979. In the procedure which is illustrated in Figure 31 the stiffness of the soil within the excavation boundary is gradually reduced by a factor β. The reduction of the stiffness leads to deformations according to excavation forces which are applied on the boundary and allows simulations of different excavation sequences like side drifts or top heading bench and invert drifts in 2D (Potts and Zdravkovic, 2009). With this method and the method mentioned above it is possible to predict the amount of tunnel deformation occurring ahead of the tunnel and prior to the moment of excavation and lining installation (Phase 2, Tutorial 18).

During the simulation of the stiffness reduction of the excavation area also the Poisson ratio ν and initial element loading has to be changed because of the connection of the parameters explained in Equation 14 and to allow the material to deform. In our case ν of the London Clay was changed from 0.49 to 0.2. Also the material type was set from plastic to elastic and the initial element loading was changed from "field stress and body force" to "none" (Phase 2, Tutorial 18).

$$\frac{E}{E_0} = \frac{(1-2\nu)*(\frac{P}{P0})}{2*(1-\nu)-(\frac{P}{P0})} \tag{14}$$

E	reduced soil stiffness
E_0	initial soil stiffness
v	Poisson ratio
P	reduced soil pressure
P_0	initial soil pressure

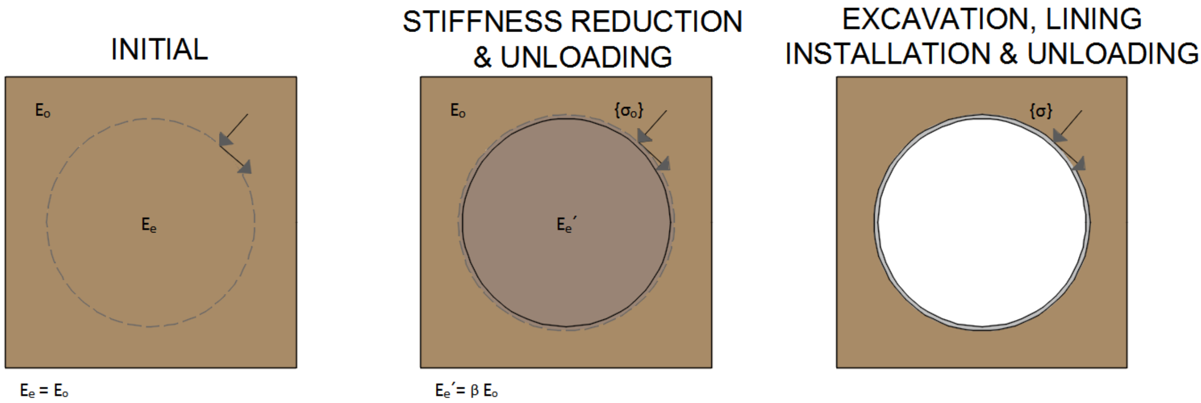

Figure 31: Stiffness method (Potts; Zdravkovic, 2009)

In all stages where a reduced stiffness is applied to the soil and initial element loading is set to none no internal stress is acting in the material. The results of each stage illustrate the deformations of one cross section step ahead or behind the proceeding tunnel face. The result of the final stage where the whole face is excavated and lining is put in place represents the case where no influence of the tunnel face on displacements and stresses can be assumed (Phase 2, Tutorial 18).

6.3 Description of the used model (coarseness, materials and shapes)

For the sensitivity analysis of the 2D models in Phase 2 all layer thicknesses and tunnel sizes in the 3D model have first been identified in size and locations. After that the same cross sections which have been used to set up the 3D model have been designed for the 2D sections. Following the cross sections of all structures were located in the right position and implemented in the 2D finite element calculation program Phase 2. It has to be said that for the calculations it was assumed that all existing structures are already in place. This assumption was made because for this thesis only the measurable deformations according to the new assets are of interest. During the analysis two different sizes of tunnels have been considered. On the one hand the construction of a pilot tunnel (small size) and followed on the other hand by a platform tunnel (big size) enlargement.

The sensitivity analysis was carried out with five different cases which have been assumed to be the most important and influencing ones according to the displacements of the tunnel lining. The factor of earth pressure of rest K_0 and the undrained elastic soil modulus E_u have been identified in previous projects as the most important and influencing factors for in tunnel deformations. During the analysis of the different case studies always one parameter change has been applied to see the exact influence on the deformations and lining stresses. The five different cases can be seen in Table 5 whereby the Base Case is the case with the best estimate values. The factors 0.6 and 1.2 for K_0 as well as 500cu and 1000cu for the elastic soil modulus are representing the upper and lower boundary of this parameter respectively.

Nr.	Case	K_0	Eu
1	Base Case	1.1	750 cu
2	$K_0 = 0.6$	0.6	750 cu
3	$K_0 = 1.2$	1.2	750 cu
4	Eu = 500 cu	1.1	500 cu
5	Eu = 1000 cu	1.1	1000 cu

Table 5: Different cases of sensitivity analysis

6.3.1 Ground model - platform tunnel (greenfield)

The established 2D section of the model in Phase 2 can be seen in Figure 32. For the model 6 noded triangle elements have been used with a coarseness of 0.3 for the whole section. The dimensions of the model have been chosen in this way to assume that there is no influence of the boundaries to the construction sequence of the tunnel.

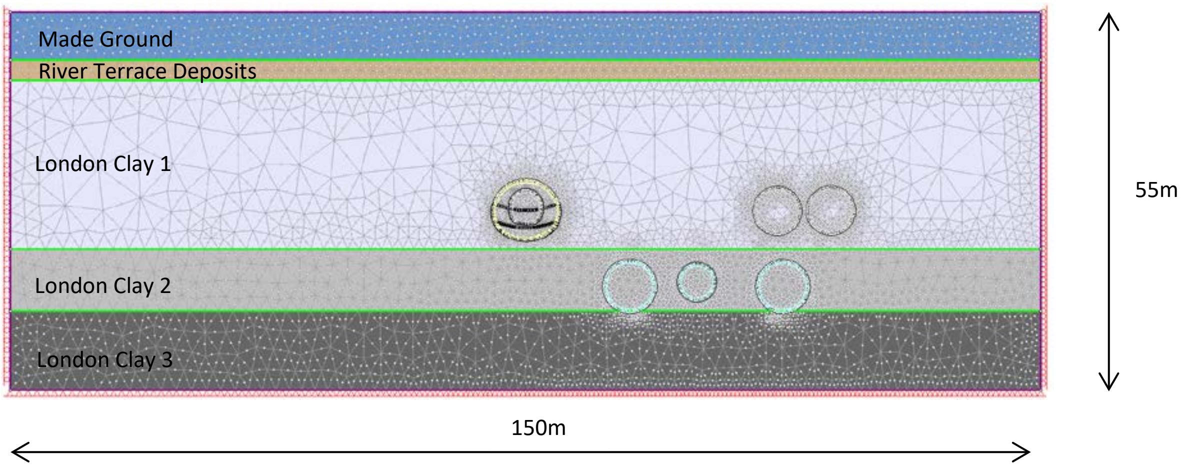

Figure 32: 2D section with existing assets and new SCL tunnel

For the 2D model the dimensions of the 3D has been maintained but it was important to move the section of interest to the middle to get less influence of the boundaries. For the purpose of this thesis and to compare the results of the 2D section with the 3D results in Abaqus only the measurable deformations at the crown (vertical) and the sidewall (horizontal) as seen in Figure 33 were used.

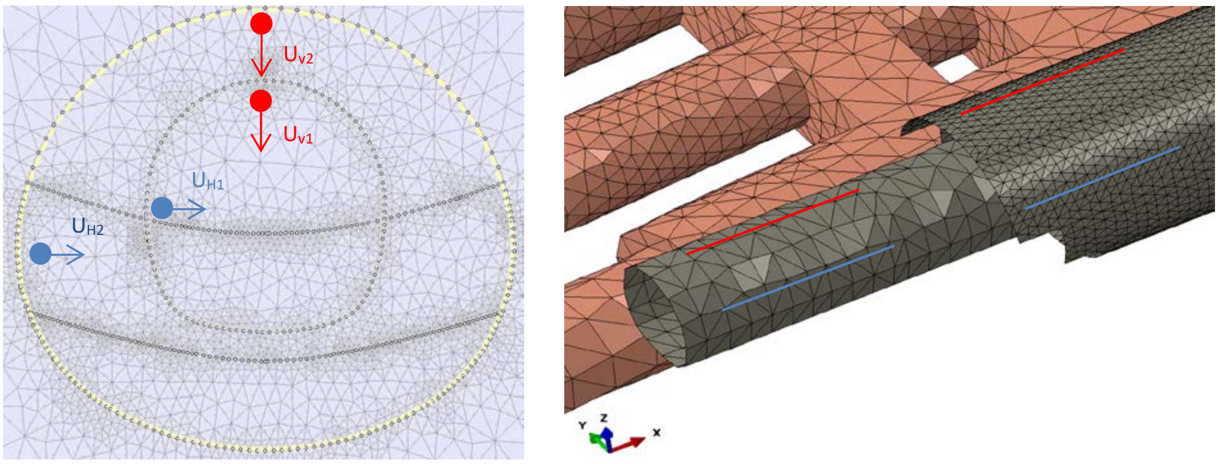

Figure 33: Areas of interest in 2D left and 3D right

For the model of the 3D analysis three soil layers namely Made Ground, River Terrace Deposits and London Clay have been implemented. It is important to mention that all tunnels, the existing ones as well as the new constructed pilot and platform tunnel are situated in the London Clay layer (see Figure 32).

The Made Ground layer was modeled with a thickness of 7m, River Terrace of 3m and the London Clay layer was assumed to reach till the bottom of the model. To model the cohesion increase of London Clay with depth in 2D three layers had to be applied (see Figure 32). Three layers have been necessary because of the different increase of the cohesion with depth which is not possible to model in one layer in 2D in Phase 2 in comparison to 3D in Abaqus. The London Clay layers have been modeled as plastic and the River Terrace Deposits and Made Ground layers as elastic. The used parameters for the soil layers of the Base Case can be seen in Table 6.

Parameters used	Symbol	Unit	Value
Made Ground			
Bulk unit weight	γ	kN/m³	19
Drained Poisson´s ratio	ν´	-	0.2
In-situ earth pressure coefficient	K_0	-	0.45
Drained Young´s modulus	E´	MPa	5
River Terrace Deposits			
Bulk unit weight	γ	kN/m³	19
Drained Poisson´s ratio	ν´	-	0.2
In-situ earth pressure coefficient	K_0	-	0.50
Drained Young´s modulus	E´	MPa	70
London Clay			
Bulk unit weight	γ	kN/m³	20
Undrained shear strength Unit London Clay 1	Cu(B)	MPa	0.082+0.006 z1
Undrained shear strength Unit London Clay 2	Cu(A3)	MPa	0.24+0.013 z2
Undrained shear strength Unit London Clay 3	Cu(A2)	MPa	0.35+0.013 z3
Angel of dilation	Ψ	°	0
Undrained Poisson´s ratio	ν_u	-	0.49/0.2
In-situ earth pressure coefficient	K_0	-	1.1
Undrained Young´s Modulus (Strain level = 0.2%)	Eu (0.2%)	MPa	750 Cu

Table 6: Used soil parameters

6.3.1.1 Mohr-Coulomb Soil Model

The used soil model was the Mohr-Coulomb linear elastic perfectly plastic model. At the elastic part the behaviour can be seen as linear elastic, for plastic behaviour it is a composite of two different criteria namely the Mohr-Coulomb criterion and Rankine surface tension cutoff criterion (ABAQUS manual). The illustration of elastic perfectly plastic material model is shown in Figure 34.

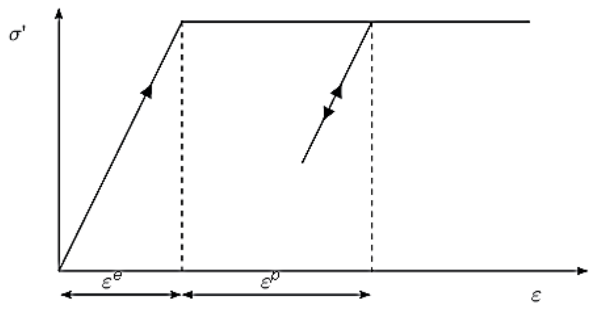

Figure 34: Main principle of linear elastic perfectly plastic model

The yield surface cone in principal stress space (c=0) is illustrated in Figure 35 and is calculated by six equations of which one is the equation shown below. In the case of this thesis the cohesion for London Clay has been modeled increasing with depth and undrained conditions were used which means that the friction angle has to be set to zero.

$$f = \frac{1}{2}(\sigma_1{}' - \sigma_3{}') + \frac{1}{2}(\sigma_1{}' - \sigma_3{}') sin\varphi' - c' cos\varphi' \leq 0 \qquad (14)$$

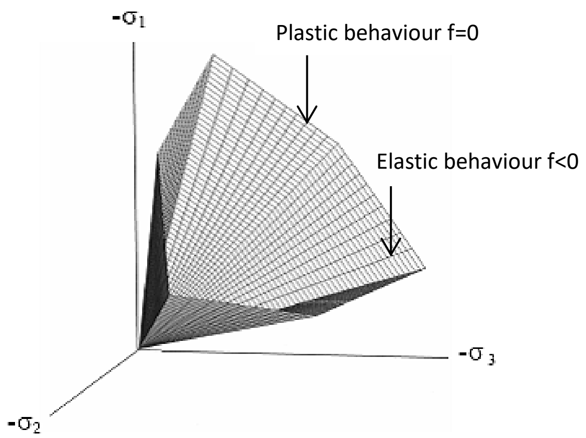

Figure 35: Mohr-Coulomb yield surface in principal stress space (c=0), (Kok Sien Ti)

The required parameters for the Mohr-Coulomb model are the Young´s modulus E [kN/m²] and Poisson´s ratio ν [-] for the elastic behaviour, Friction angle φ [°], Cohesion c´[kN/m2] and Dilatancy angle Ψ [°] for plastic behaviour (ABAQUS manual 6.13).

6.3.2 Parameters of tunnel structures

For the sensitivity analysis only the primary lining of the new pilot and SCL platform tunnel have been considered. All tunnels included in the model have been modeled with plastic behaviour. For a better understanding all existing and new assets are illustrated and described in Figure 36 and Table 7. As already mentioned before it was assumed for the analysis that the existing tunnels are already in place.

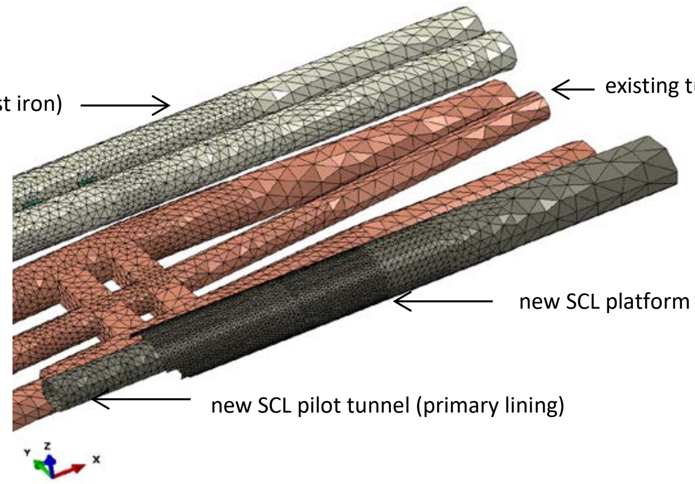

Figure 36: Existing and new tunnel assets

Parameters used	Symbol	Unit	Value
Grey Cast Iron (plastic)			
Density	γ	kN/m^3	19.85
Poisson ratio	ν´	-	0.25
Thickness	d	m	0.164
Young´s modulus	E´	MPa	66,9
Reinforced Concrete (plastic)			
Density	γ	kN/m^3	25
Poisson ratio	ν´	-	0.2
Thickness	d	m	0.25
Young´s modulus	E´	MPa	70
Primary Lining pilot tunnel (plastic)			
Density	γ	kN/m^3	25
Poisson ratio	ν´	-	0.2
Thickness	d	m	0.2
Considered Young´s modulus of C 32/40	E´	MPa	10
Primary lining platform tunnel (plastic)			
Density	γ	kN/m^3	25
Poisson ratio	ν´	-	0.2
Thickness	d	m	0.3
Considered Young´s modulus of C 32/40	E´	MPa	10

Table 7: Used parameters of existing and new assets

6.3.3 Excavation Sequences

For the calibration of the 2D model with the existing 3D excavation sequence first of all the complete modeling process and steps of the 3D model have to be reviewed and understood. Because the focus was put only on the determination of the predicted deformations according to the pilot and platform tunnel construction all stages had to be recreated in the same order than in 3D. However the values which have to correlate to the 3D are only those of the last modeled stage. The deformations of this stage represent the amber trigger value on the basis of Nasekhian et al. (2016) which is the starting point of the following determination of the red and black trigger. The excavation areas of pilot tunnel top heading bench and invert enlargement and the areas of interest after the complete excavation and complete lining installation are shown in Figure 37. The existing structures have been modelled only for completion and to see their location to the construction of the new tunnel structure. The applied stages for the used 2D models are as follows:

I. Existing assets wished in place
II. Relaxation new pilot tunnel
III. Excavation new pilot tunnel and install lining (primary lining)
IV. Relaxation top heading new platform tunnel
V. Relaxation top heading + primary lining & relax bench
VI. Relaxation top heading + primary lining, relax bench + lining & relax invert
VII. Complete Excavation + ring closure

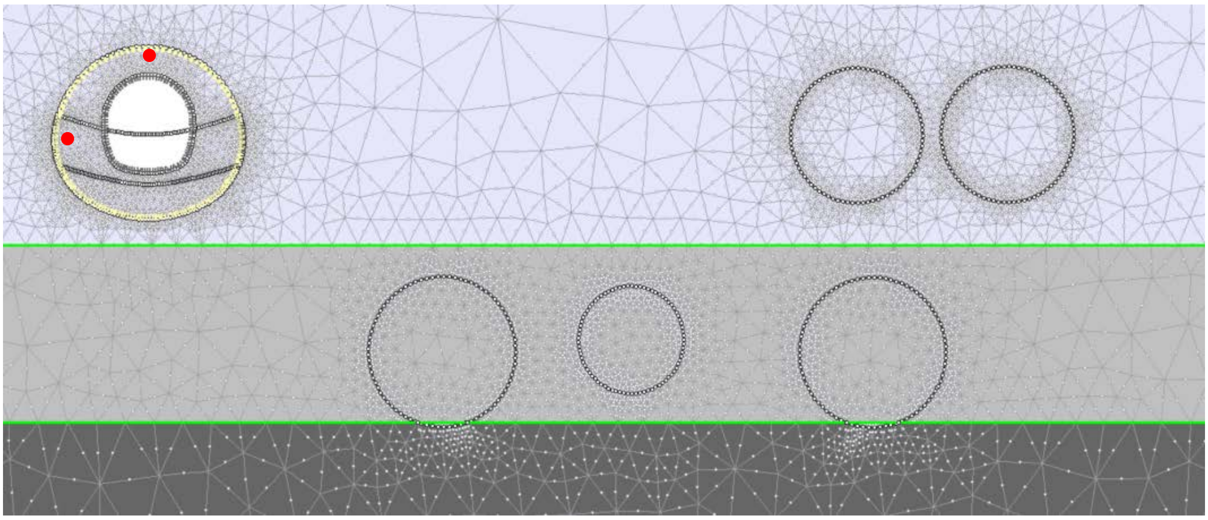

Figure 37: Position of measurement targets in platform tunnel

The exact location of the cross section in 3D is shown in Figure 38. In this illustration the vertical deformations according to the construction of the new platform tunnel are shown. For the purpose of this analysis a section in the middle of the 3D area of interest was chosen which is also shown in Figure 38. The dimensions of the two different investigated tunnel sizes are illustrated in Figure 39.

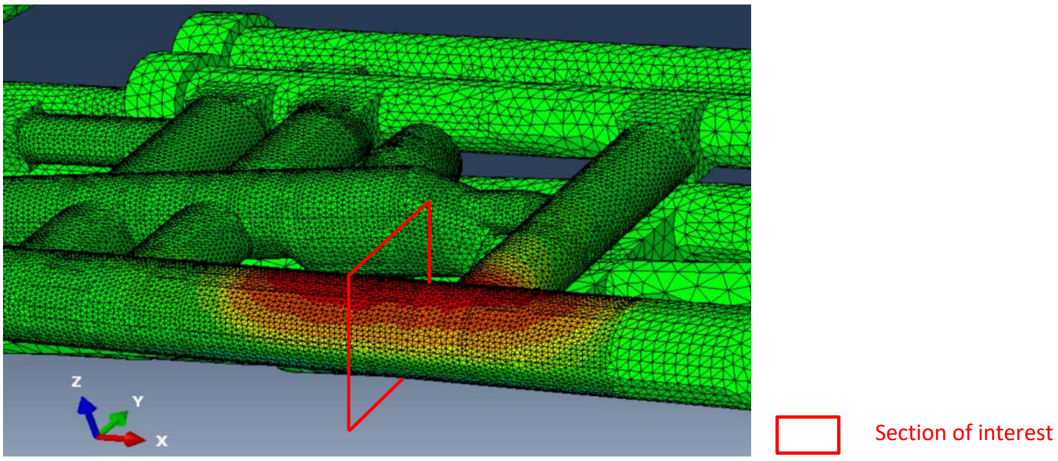

Figure 38: Section of interest in the 3D model

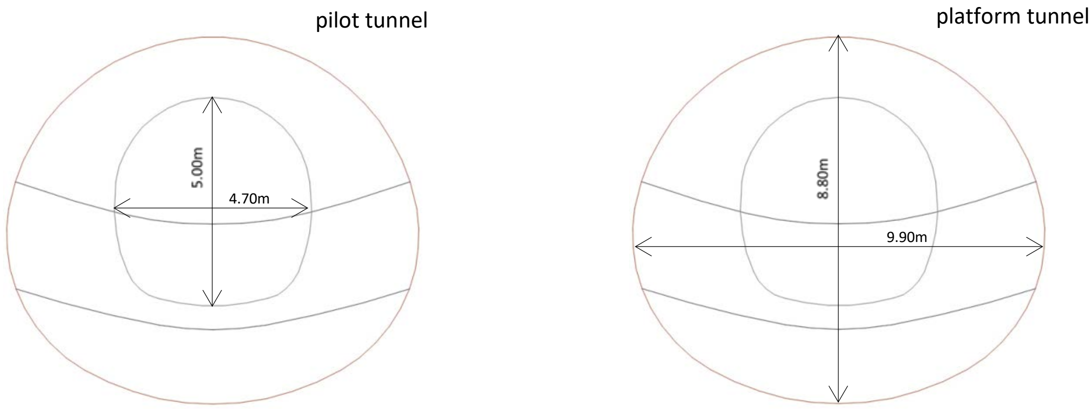

Figure 39: Dimensions of sections of interest

6.4 Calculations

At the beginning of the case studies it was tried to calibrate the 2D to the 3D FE model with very high stiffness relaxation factors of 50% and more. After many different trials it was recognizable that a convergence with these relaxation factors is not possible. According to this and because it was very difficult to converge the pilot tunnel and platform tunnel sequence in one model with the 3D sequence it was decided to go one step back, use much lower relaxation factors and converge in the first step only the pilot tunnel. After this a sensitivity analysis of different parameters for the pilot tunnel was carried out to see the differences of the results for the CLC and lining force relationship factor in comparison to the 3D. Following the platform tunnel was also calibrated with the 3D model and the same sensitivity analysis than for the pilot tunnel was executed. The calibration of the platform tunnel has been performed with the two different approaches mentioned above to see the difference of the two methods. However the sensitivity analysis for the platform tunnel was carried out only for the stiffness method.

6.5 Sensitivity Analysis Pilot Tunnel – stiffness method

To model the developing deformations during the construction of the completed platform tunnel construction first the pilot tunnel which is constructed ahead has been investigated and calibrated to 3D. The pilot tunnel with the maximum deformations in vertical and horizontal for the Base Case in 3D is shown in Figure 40 and Figure 41. For the calibration of the 2D model the maximum values have been used in vertical and horizontal direction.

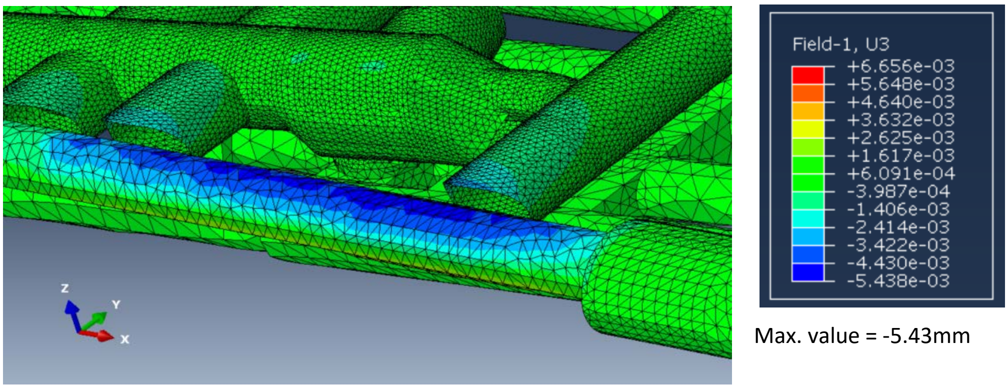

Figure 40: Vertical deformations pilot tunnel Base Case

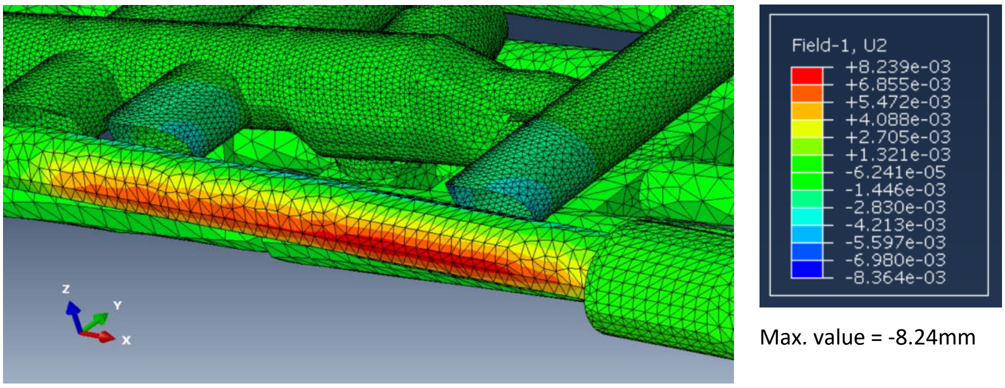

Figure 41: Horizontal deformations pilot tunnel Base Case

The best match of the 2D and 3D was achieved by a relaxation factor of 12.5% and followed by a full excavation and lining installation of the pilot tunnel. The relaxation and following excavation sequence is shown in Figure 42. The results of the converged Base Case and the following sensitivity analysis for the determination of the trigger values can be seen in Table 8.

I. All existing structures wished in place
II. Relaxation new pilot tunnel by 12.5%
III. Excavation new pilot tunnel and install lining (primary lining)

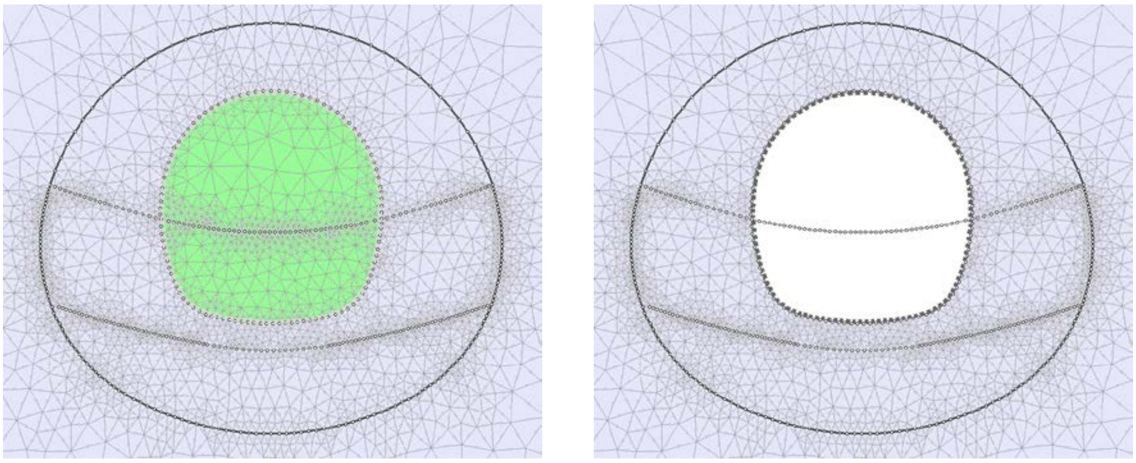

Figure 42: Excavation sequence pilot tunnel

Nr.	Case	K_0	Eu	Vertical deformation [mm]		Horizontal deformation [mm]	
				2D	3D	2D	3D
1	Base Case	1.1	750cu	-5.4	-5.4	-8.8	-8.2
2	K_0=0.6	0.6	750cu	-8.3	-6.7	-4.0	-3.2
3	K_0=1.2	1.2	750cu	-5.0	-5.2	-10.0	-9.2
4	Eu=500cu	1.1	500cu	-7.4	-7.1	-12.8	-11.5
5	Eu=1000cu	1.1	1000cu	-4.3	-4.5	-6.9	-6.5

Table 8: Results of 2D analysis and compared to results of 3D

The deformations of the different cases in vertical (measured at crown) and horizontal (measured at sidewall) direction of the different cases shown above are presented in Figure 43 and Figure 44 in normalized representation to the Base Case. Figure 43 shows that a material with a lower value of K_0 like in case two leads to much higher vertical deformations whereas in horizontal direction case four with a stiffness reduction to 500cu is significant. In the illustrations it can also be seen that case two is worse in vertical deformations but in total case four with the reduction of the stiffness to Eu=500cu is the most influencing one. The best matching in horizontal and vertical direction compared to the results of the base case is case three with an increase of K_0 to 1.2.

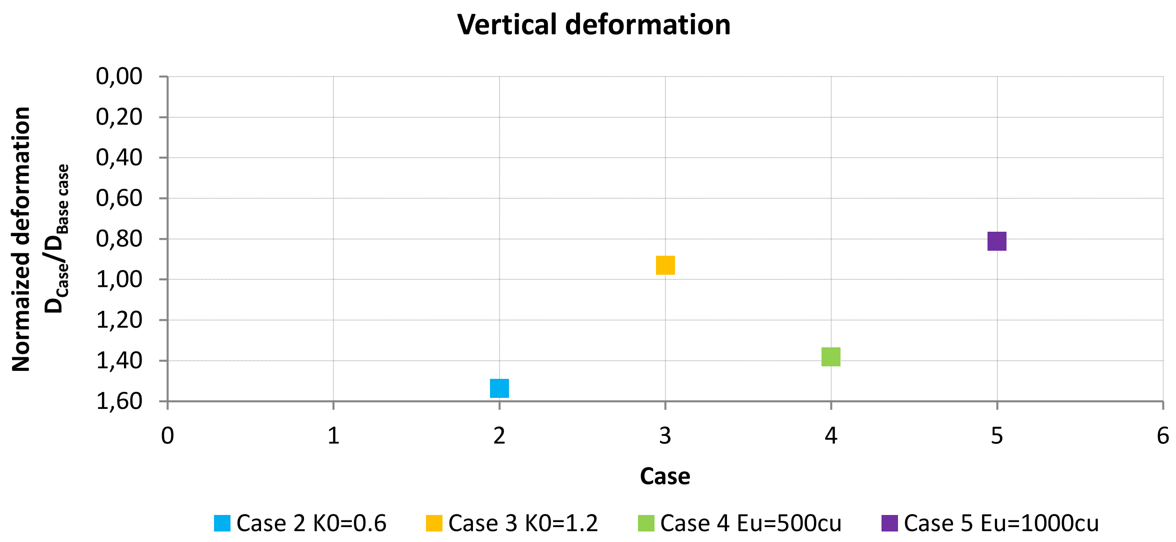

Figure 43: Normalized vertical deformations (crown) of different cases

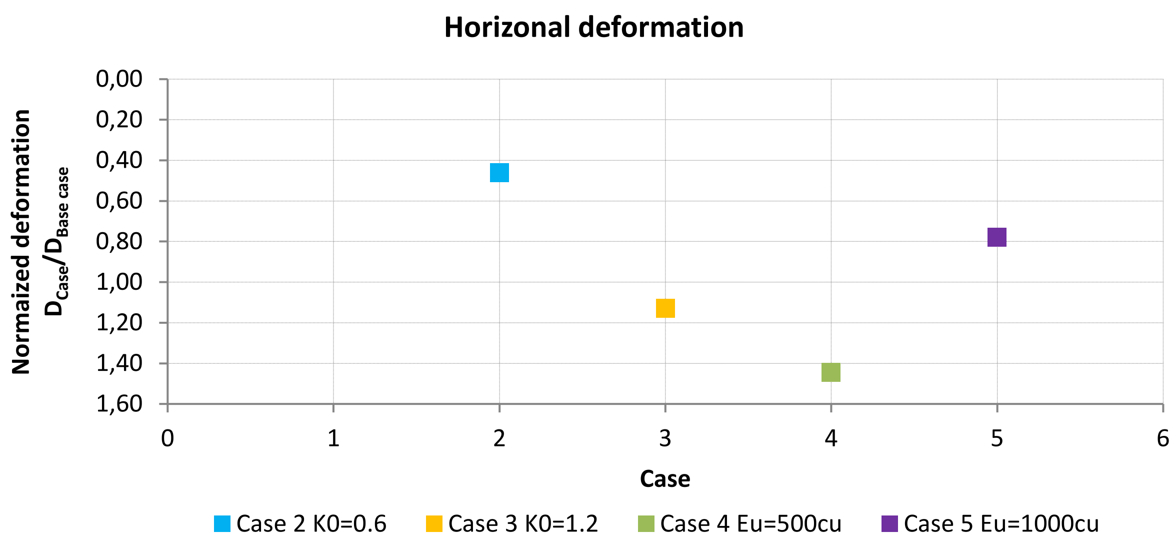

Figure 44: Normalized horizontal deformations (sidewall) of different cases

The deviation of the different cases compared to the base case are summarized and presented below.

Vertical deformations:

K_0 = 0.6 → 50% higher than base case → relevant

K_0 = 1.2 → 10% smaller than base case

Eu = 500cu → 40% higher than base case

Eu = 1000cu → 20 % smaller than base case

Horizontal deformations:

K_0 = 0.6 → 50% smaller than base case

K_0 = 1.2 → 15% higher than base case

Eu = 500cu → 45% higher than base case → relevant

Eu = 1000cu → 20% smaller than base case

6.5.1 Internal Lining Forces

To represent also the output of the model in terms of lining forces and to find out the worst case all lining force pairs where plotted against the CLC. In Figure 45 the section force pairs for all different cases of parameter variations can be seen. The used load safety factor for primary lining in short term condition was chosen with 1.2 in the analysis because of the reason that the primary lining is constructed only for a lifetime of 5 years. During this time it is assumed that no water pressure is acting on the lining and only short term loads are reacting. The new SCL works for the pilot tunnel have been modeled with the properties mentioned above and a material safety factor of 1.5 for concrete.

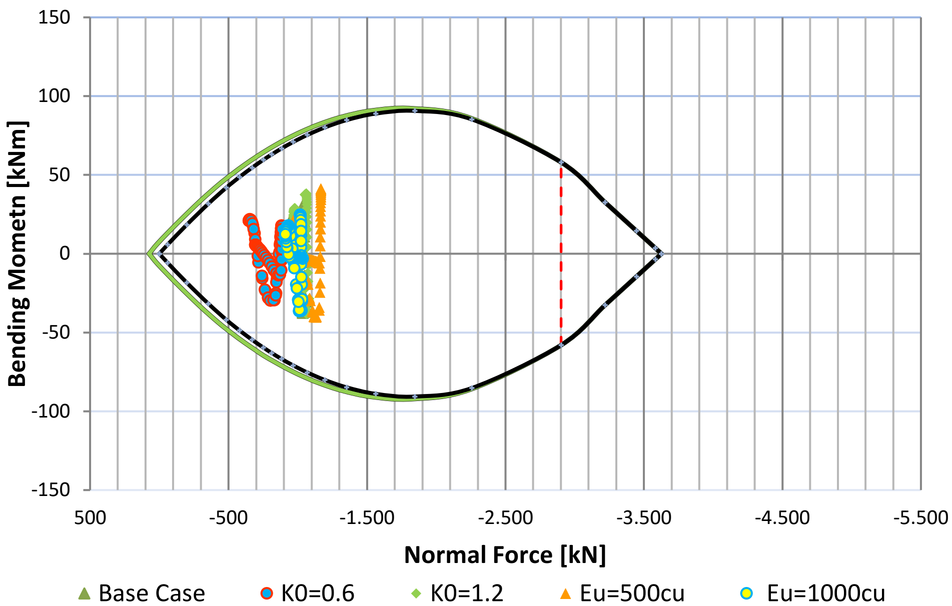

Figure 45: Bending moment vs. normal forces – All cases pilot tunnel

Figure 45 represents that all lining force pairs (bending moment + normal force) for each point of the pilot tunnel lining are within the CLC and that the difference of the soil parameter uncertainty has a more or less influence on the utilization of the lining capacity. It is visible that case four with the change of the stiffness Eu from 750cu to 500cu has the most influence on the lining forces. In order to get the same results than for the worst case the base case can be multiplied by a factor of 1.2 which is illustrated in Figure 46. Figure 45 also indicates that there is a lot of capacity left in the lining which can be utilized. Figure 47 represents that in order to use the whole capacity of the lining all values of the base case can be multiplied by a factor of 2.5.

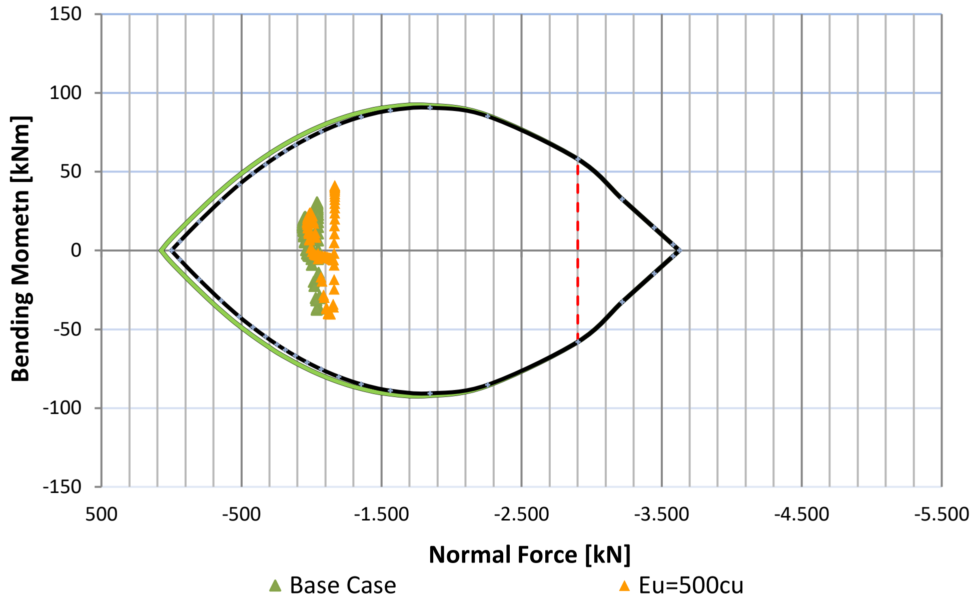

Figure 46: Bending moment vs. normal forces – base case vs. worst case

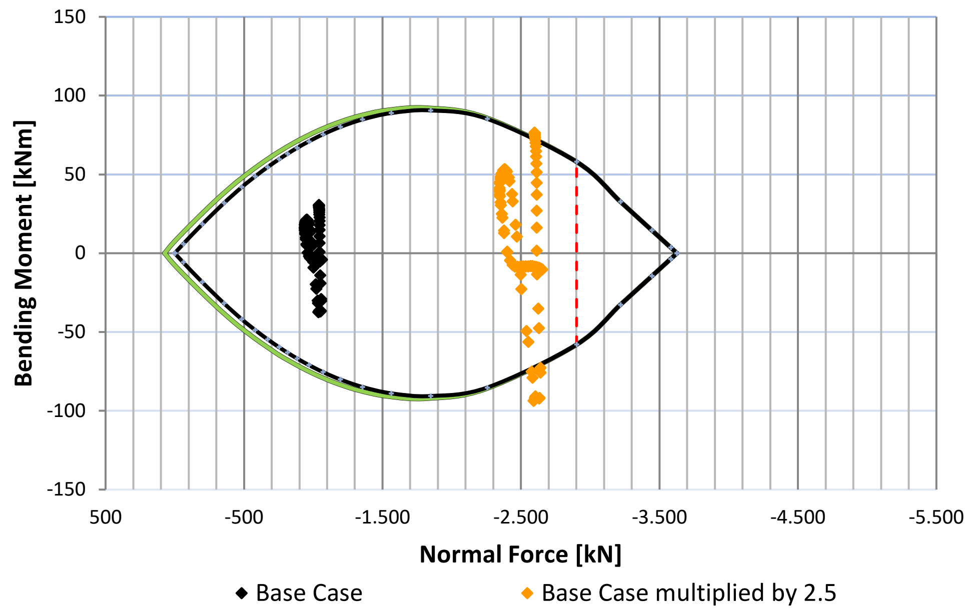

Figure 47: Bending moment vs. normal forces – base case multiplied by 2.5

In conclusion the following factors seen in the table below were summarized. The factors include all cases and scenarios in terms of displacements and lining forces of the base case to the worst case scenarios.

	Factor
Horizontal Deformation	1.4
Vertical Deformation	1.5
Internal Forces	1.2
CLC factor	2.5

Table 9: Relationship base case vs. worst case scenarios of displacements and lining forces

To obtain the trigger values for the pilot tunnel the following step methodology was used:

Amber trigger: deformations in vertical and horizontal direction derived by calibrated base case 2D FE model

Red trigger: non-rounded amber trigger multiplied by CLC-factor of 2.5 (to utilize the whole capacity of the lining) and the factor which represents the non-linearity of deformations and lining forces. (average of deformations/ internal forces → 1.4/1.2)

Black trigger: non-rounded red trigger multiplied by the factor 1.2 and 1.5 to consider the short term loading and material safety factor

The summarized values in comparison to the 3D values for base case vs. worst case of the deformations as well as for the lining forces are presented in Table 10.

	2D	3D
Horizontal Deformation	1.4	1.5
Vertical Deformation	1.5	1.3
Internal Forces	1.2	1.1
CLC factor	2.5	2.0

Table 10: Comparison of the results for the pilot tunnel with 3D

The summarized and compared results of the calibrated small size pilot tunnel to 3D have shown that the deviation of different cases behaves in the same manner than in 3D and the results of the CLC and the non-linearity factor are almost the same.

6.6 Sensitivity Analysis Platform Tunnel – stiffness method

After the calibration of the pilot tunnel with the results of the 3D Abaqus model the focus was put on the main purpose namely the construction of the platform tunnel enlargement. The contour plots of the platform tunnel deformations in 3D can be seen in Figure 48 and Figure 49. It has to be said that for the 3D sensitivity analysis and for the calibration of the 2D model to the 3D model an average value of 7 points in vertical and horizontal direction has been chosen for the amber trigger value which represents the beginning of the trigger value determination.

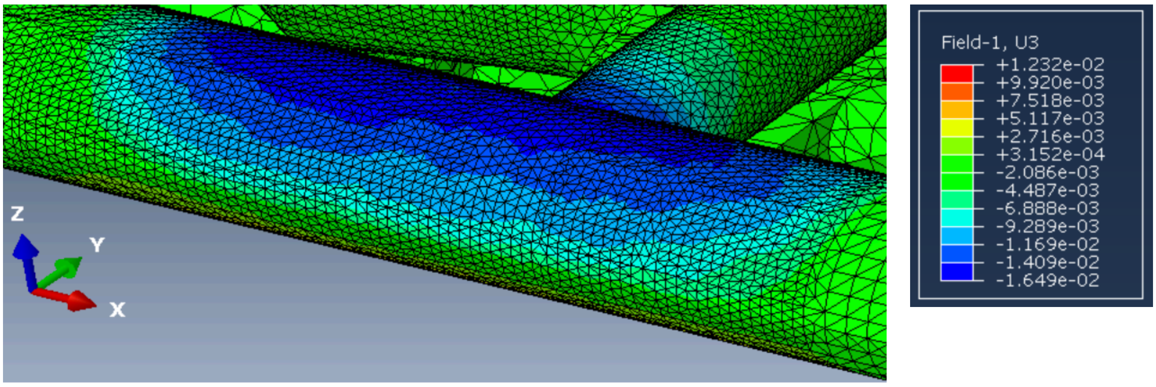

Figure 48: Vertical deformations platform tunnel base case

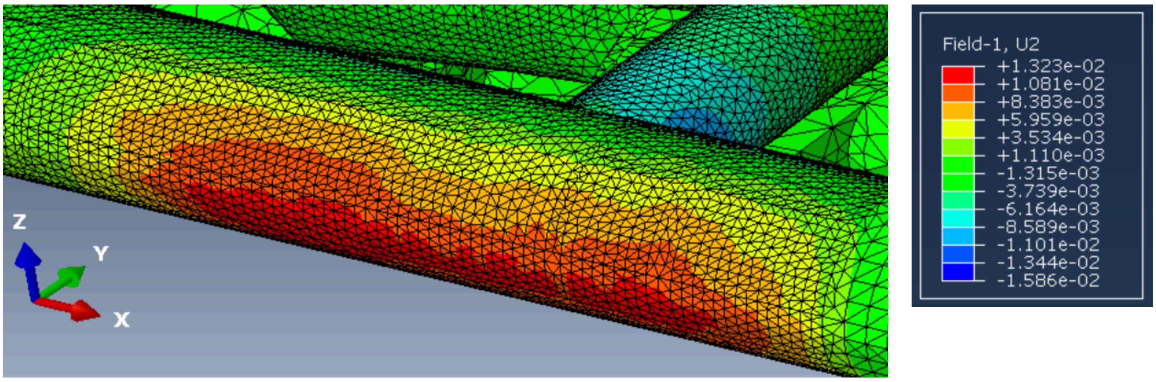

Figure 49: Horizontal deformations platform tunnel base case

It can be seen from the results in Table 11 below that the differences between all cases of the 2D and 3D are negligible. The sequence for the best calibration of the base case in 2D to the 3D is shown in Figure 50 and described below. It has to be mentioned that during the calibration the focus was only put onto the vertical deformations of the base case. The fact that the results in horizontal and vertical for all other cases are also almost equal indicates that the modeled excavation sequence and relaxation factors are simulating the 3D effects of all cases very well in 2D.

I. All existing structures wished in place
II. Relaxation new pilot tunnel 12.5%
III. Excavation new pilot tunnel and install lining (primary lining)
IV. Relaxation top heading 12.5% new platform tunnel
V. Relaxation top heading 12.5% + primary lining & relax bench 5%
VI. Relaxation top heading 12.5% + primary lining, relax bench 10% + lining & relax invert 5%
VII. Complete excavation + ring closure

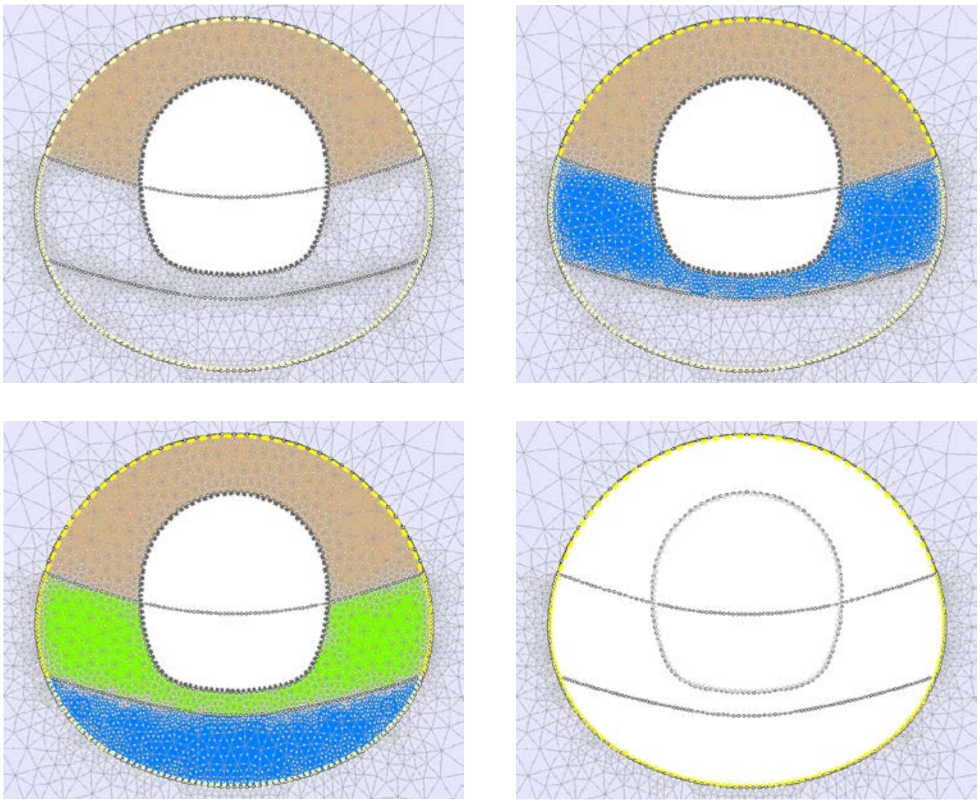

Figure 50: Excavation sequence platform tunnel

Nr.	Case	K_0	Eu	Vertical deformation [mm]		Horizontal deformation [mm]	
				2D	3D	2D	3D
1	base case	1.1	750cu	-13.9	-14.0	-10.1	-10.0
2	K_0=0.6	0.6	750cu	-16.5	-15.0	-4.2	-5.0
3	K_0=1.2	1.2	750cu	-14.1	-13.0	-11.9	12.0
4	Eu=500cu	1.1	500cu	-18.8	-18.0	-14.0	15.0
5	Eu=1000cu	1.1	1000cu	-11.4	-10.0	-8.1	-8.0

Table 11: Results of 2D analysis compared to results of 3D

The calibration of the plaftorm tunnel was also exectued with the other approach mentioned above namely the pressure method. The results of this approach have shown that these two approaches behave in a very different manner and that much higher relaxation factors have to be applied compared to the stiffness method in order to calibrate the model with this in 3D. The exact relaxation factors for the best calibrated match are illustrated in Figure 51 and listed below.

I. All existing structures wished in place
II. Relaxation new pilot tunnel 12.5%
III. Excavation new pilot tunnel and install lining (primary lining)
IV. Relaxation top heading 12.5% new platform tunnel
V. Relaxation top heading 12.5% + primary lining & relax bench 5%
VI. Relaxation top heading 12.5% + primary lining, relax bench 10% + lining & relax invert 5%
VII. Complete Excavation + ring closure

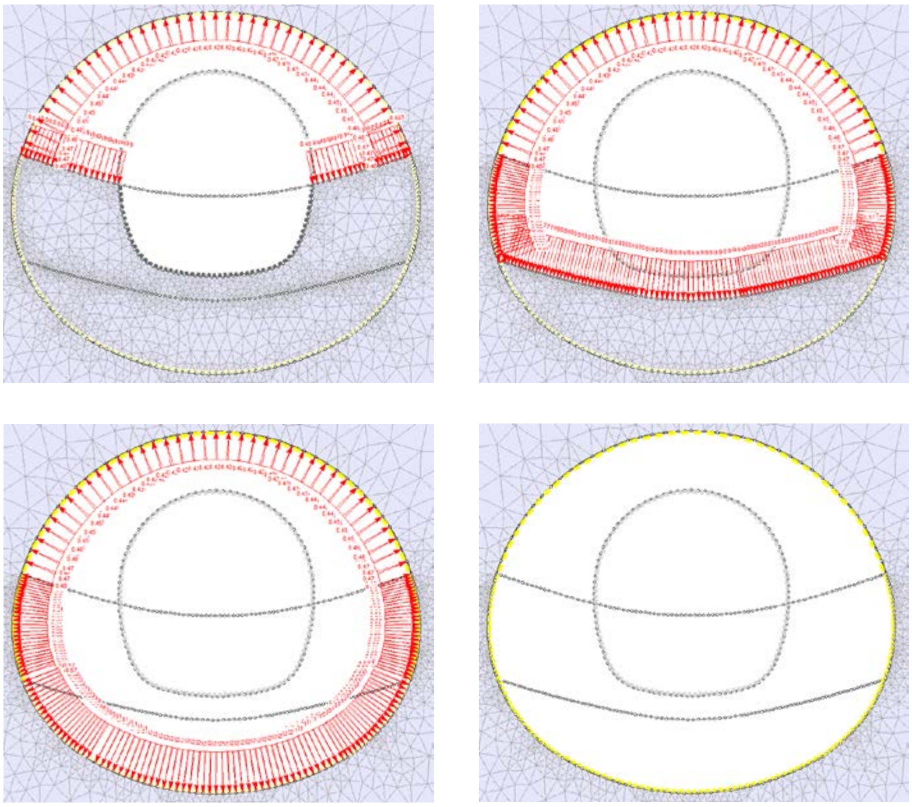

Figure 51: Excavation sequence platform tunnel convergence-confinement method

After the calibration of the model with two different approaches the sensitivity analysis was carried out only with the stiffness method to interpret and investigate the influence of different parameter changes. The results of the deformations for all cases in vertical (crown) and horizontal (sidewall) direction are shown in Table 10 above. Furthermore they are presented in normalized representation to the base case in 2D and 3D in Figure 52 and Figure 53 for the vertical and Figure 54 and Figure 55 for the horizontal direction. From the illustration of the vertical deformations it can be seen that for the platform tunnel the case with the lowest K_0 value is not the most influencing one in vertical direction like it was at the pilot tunnel sensitivity analysis. Instead case four with the lowest stiffness factor of Eu=500cu leads to the highest deviations form the base case in vertical as well as in horitzontal direction. As it could be expected from these results this case also leads to the worst results with respect to lining forces. The best matching case compared to the deformations of the base case is case three with a change of K0 to 1.2.

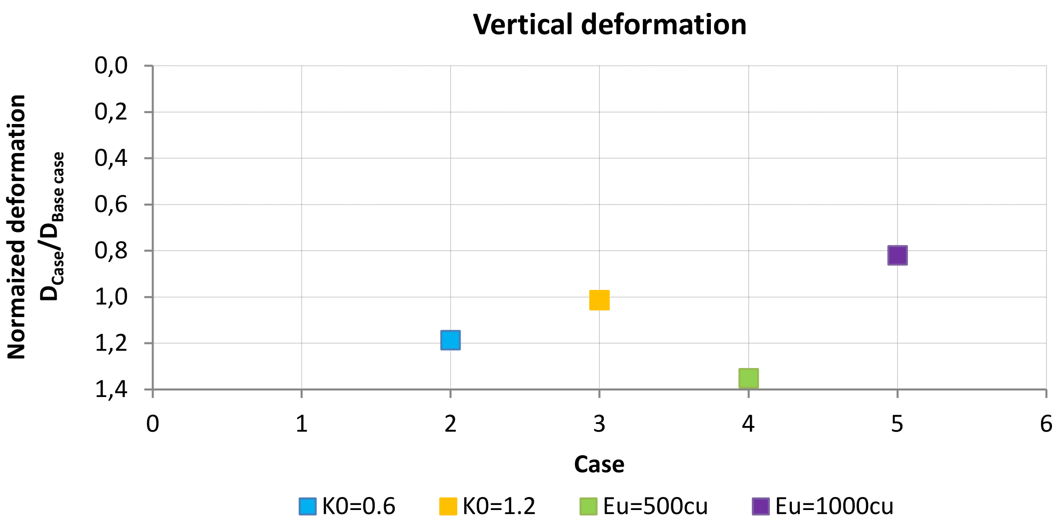

Figure 52: Normalized vertical deformations (crown) of different cases 2D

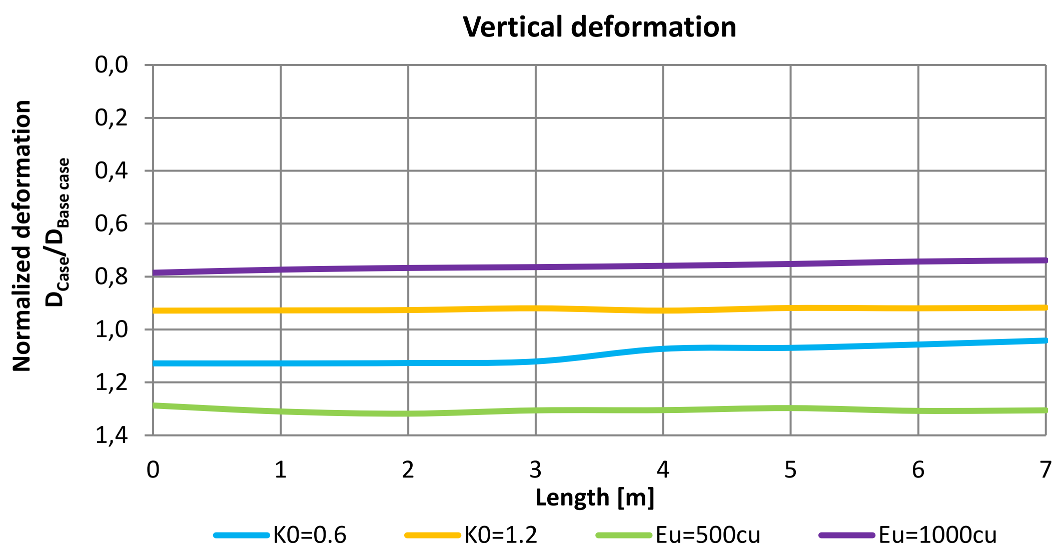

Figure 53: Normalized vertical deformations (crown) of different cases 3D (Nasekhian, 2016)

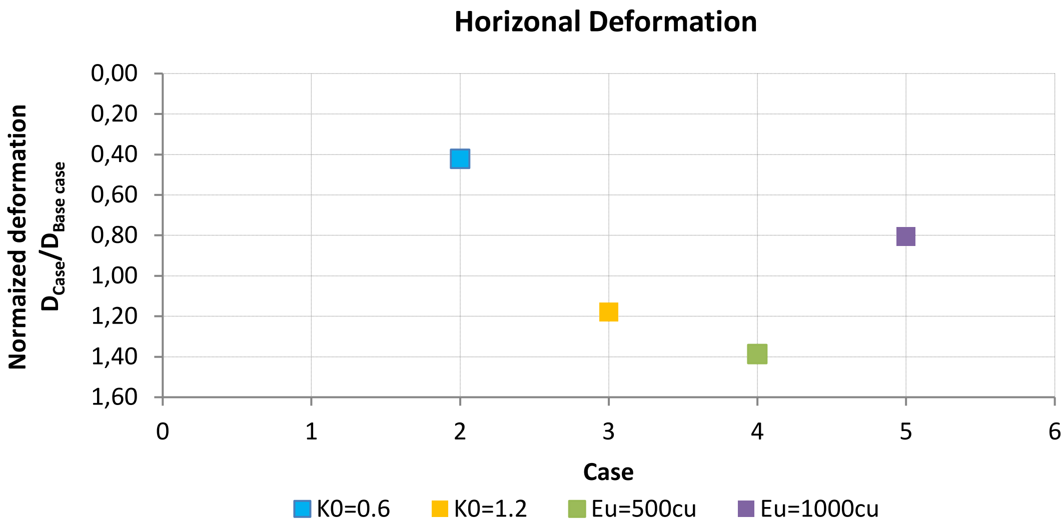

Figure 54: Normalized horizontal deformations (sidewall) of different cases 2D

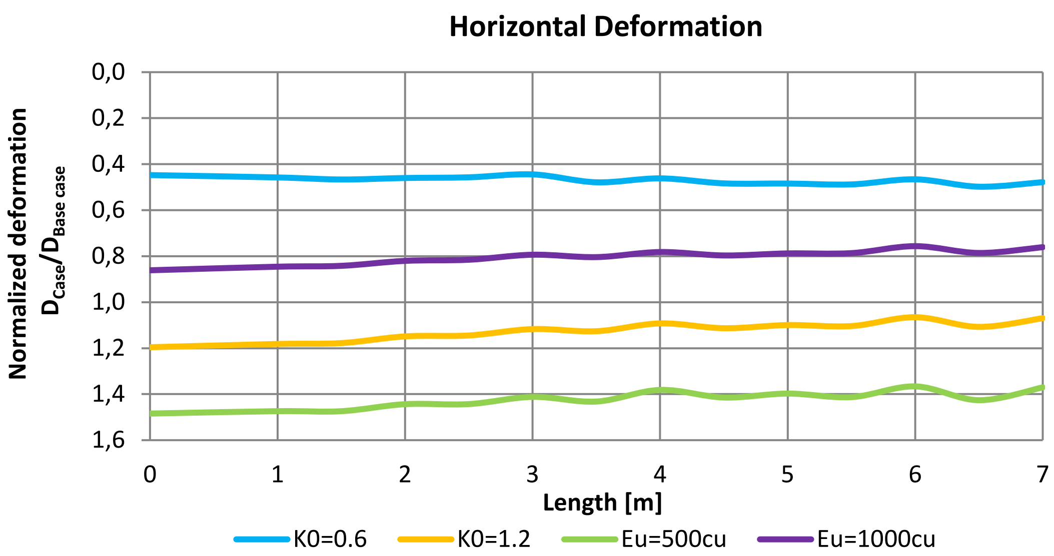

Figure 55: Normalized horizontal deformations (sidewall) of different cases 3D (Nasekhian, 2016)

A summary of the 2D results of the different cases compared to the base case is listed below.

Vertical deformations:

K_0 = 0.6 → 20% higher than base case

K_0 = 1.2 → 0% difference to base case

Eu = 500cu → 35% higher than base case → relevant

Eu = 1000cu → 20 % smaller than base case

Horizontal deformations:

K_0 = 0.6 → 60% smaller than base case

K_0 = 1.2 → 20% higher than base case

Eu = 500cu → 40% higher than base case → relevant

Eu = 1000cu → 20% smaller than base case

The results of the different cases in vertical as well as in horizontal direction show that the order and magnitude of deviation of all cases are almost equal in 2D than in 3D.

6.6.1 Internal Lining Forces

The obtained lining forces according to the five different cases were plotted against the capacity limit curve (CLC) to see the utilization of the installed lining. The section force pairs of all different cases are illustrated in Figure 56. The used load case for this sensitivity analysis was the short term load combination with a factor of 1.2 like it was for the pilot tunnel. The new SCL works for the pilot tunnel have been modeled with the properties mentioned above and a material safety factor of 1.5.

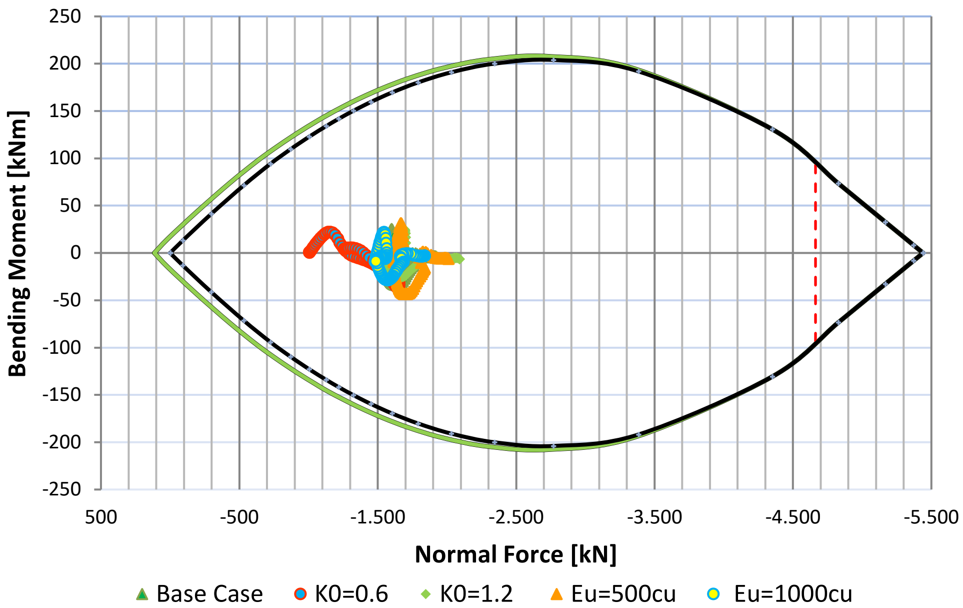

Figure 56: Bending moment vs. normal forces – All cases platform tunnel 2D

Figure 56 represents that all pairs of bending moments and normal forces for each point of the constructed platform tunnel lining are within the CLC. It is also visible that the difference of the soil parameter uncertainty has a significant influence on the utilization of the lining capacity for the bigger platform tunnel as well as for the before investigated smaller pilot tunnel. In the evaluation of the calculation results it was identifiable that on average case four with a K_0=1.1 and a stiffness of Eu= 500cu has the highest variation to the base case in comparison to lining forces. This is exactly the same case which also represented the worst case in the 3D sensitivity analysis (see Figure 57).

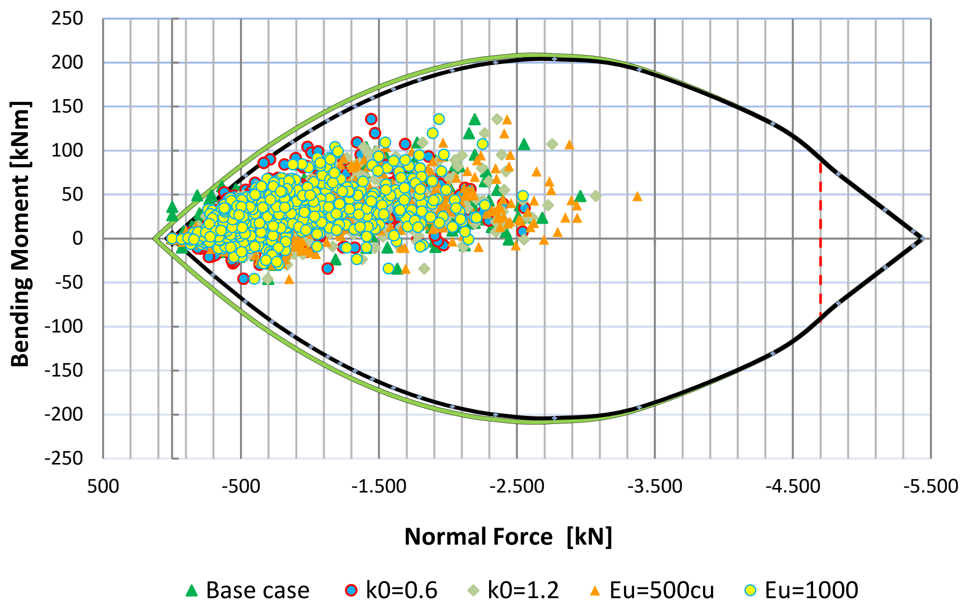

Figure 57: Bending moment vs. normal forces – All cases platform tunnel 3D (Nasekhian, 2016)

In order to find out the factor according to lining forces the base case was plotted against the worst case and it was investigated that all section forces of the base case can be multiplied by a factor of 1.1 to get the same lining forces as the worst case. The results of the multiplied section forces in 2D and 3D are presented in Figure 58 and Figure 59 below. It has to be said that in 2D only the base case vs. worst case is presented instead of the multiplied one because otherwise the difference would not be visible.

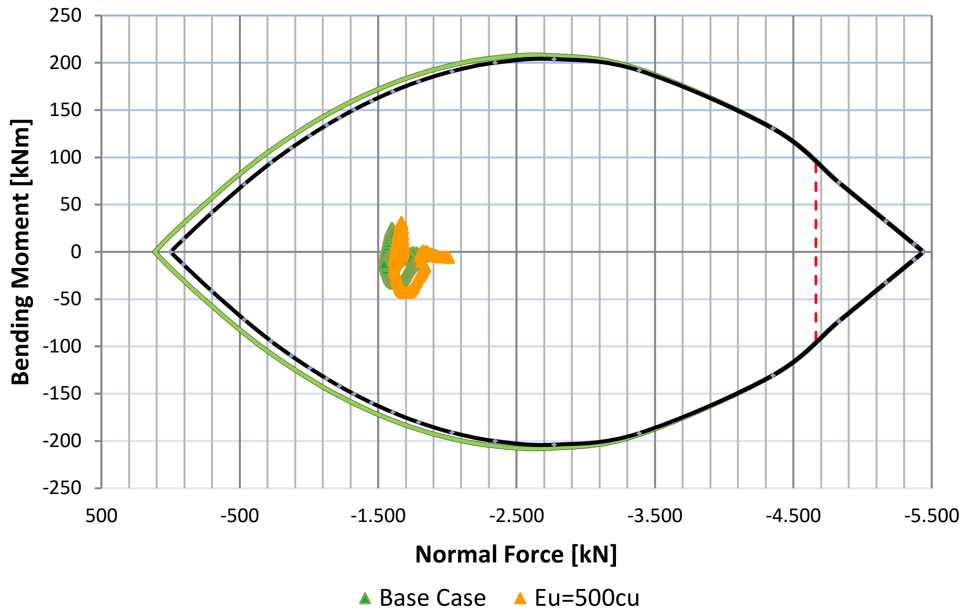

Figure 58: Bending moment vs. normal forces – base case vs. worst case 2D

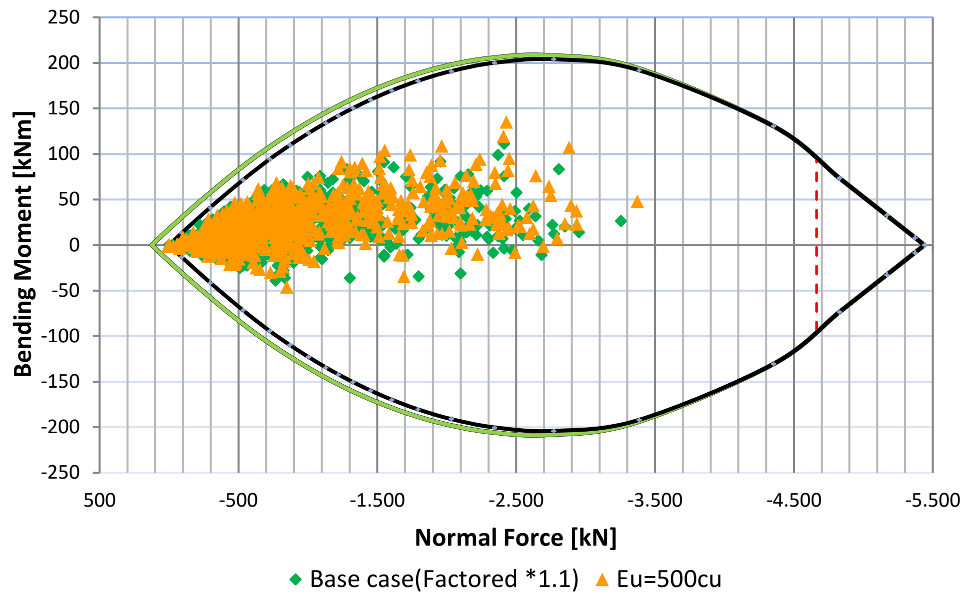

Figure 59: Bending moment vs. normal forces – base case vs. worst case 3D (Nasekhian, 2016)

Figure 60 below illustrates that in order to reach the surrounding CLC and use the full capacity of the lining the values of the base case have to be multiplied by a factor of 2.5. In 3D the full capacity of the lining was reached already by a factor of 2.0. To see the differences of the 2D and 3D the results are illustrated in Figure 60 and Figure 61.

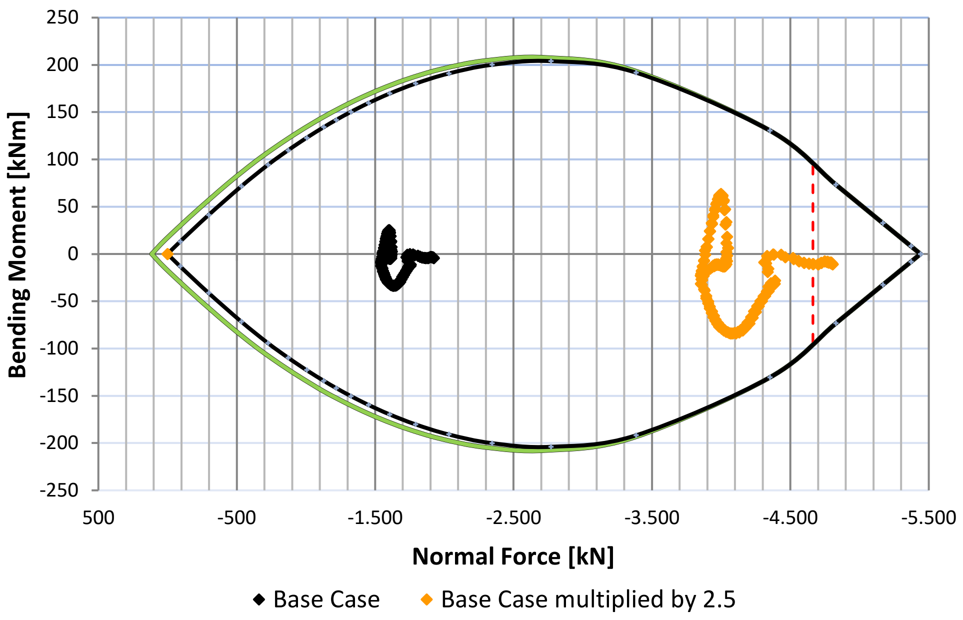

Figure 60: Bending moment vs. normal forces – base case factored by 2.5 2D

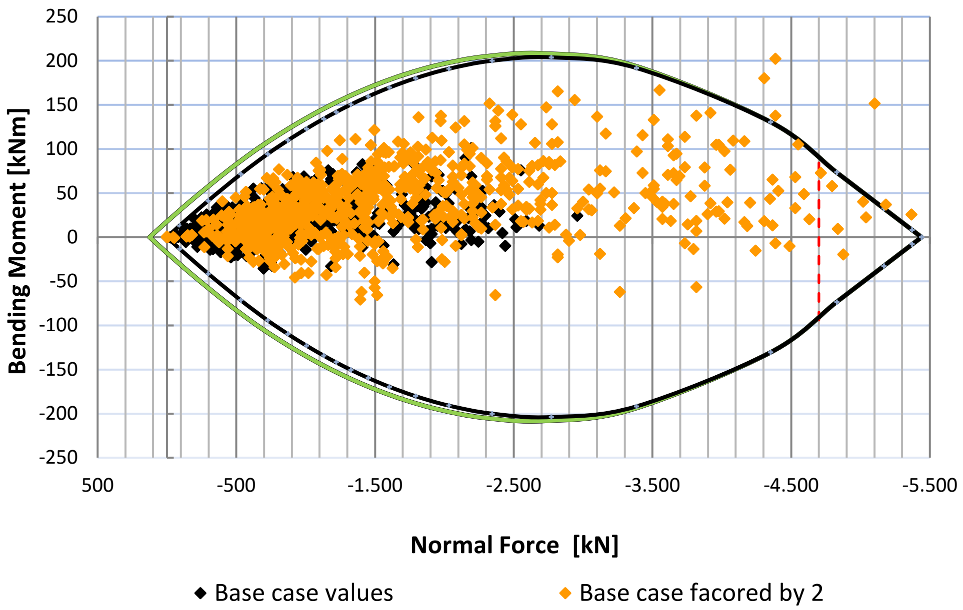

Figure 61: Bending moment vs. normal forces – base case factored by 2.0 3D (Nasekhian, 2016)

In conclusion the following factors shown in Table 12 below were summarized for the determination of the trigger values for the platform tunnel. The factors include all cases and scenarios in terms of displacements and lining forces of the base case to the worst case scenarios.

	2D
Horizontal Deformation	1.4
Vertical Deformation	1.4
Internal Forces	1.1
CLC factor	2.5

Table 12: Relationship base case vs. worst case scenarios of displacements and lining forces

To obtain the trigger values for the platform tunnel the same methodology was used as for the pilot tunnel.

Amber trigger: deformations in vertical and horizontal direction derived by calibrated base case model 2D FE model

Red trigger: non-rounded amber trigger multiplied by CLC-factor of 2.5 (to utilize the whole capacity of the lining) and the factor which represents the non-linearity of deformations and lining forces. (average of deformations/ internal forces → 1.4/1.1)

Black trigger: non-rounded red trigger multiplied by the factor 1.2 and 1.5 to consider the short term loading and material safety factor

The compared results of the 2D sensitivity analysis for the bigger size platform tunnel to the 3D results which are represented in Table 13 show that it is possible to get exactly the same results in 2D than in a very complex and time expensive 3D sensitivity analysis.

	2D	3D
Horizontal Deformation	1.4	1.5
Vertical Deformation	1.4	1.3
Internal Forces	1.1	1.1
CLC factor	2.5	2.0

Table 13: Comparison of the results for the platform tunnel results with 3D

The results of the combined 2D results of the pilot and platform tunnels and the results in 3D are summarized in Table 14. For the combined results of the lining deformations and lining forces it has to be mentioned that both have been rounded to the next digit after the comma. The factor for the lining deformations was rounded down and the factor for lining forces was rounded up which represents a conservative way concerning the determination of the trigger values.

	2D	3D
Horizontal Deformation	1.4	1.5
Vertical Deformation	1.4	1.3
Internal Forces	1.2	1.1
CLC factor	2.5	2.0

Table 14: Comparison of the combined 2D results with 3D

7 Conclusion case study

This thesis has shown that it is possible to investigate the sensitivity of a very complex underground structure also in a 2D model in an accelerated way and with much less calculation effort than in a very complex 3D model. Also two different approaches for the calibration of a 2D to a 3D model have been introduced and their differences in required relaxation factors for the calibration have been presented.

The results have shown that it is possible for future projects with high complexity to calibrate a very simple 2D to a complex 3D model with relatively small effort and execute the sensitivity analysis in a much faster and easier way in a 2D FE program. The calibration of the platform tunnel with the 3D model in two different ways made it visible that both, the stiffness as well as the pressure method work but the applied relaxation factors have to be treated very carefully in order to model the 3D effects in a 2D model.

In addition it has been shown that the values for the relationship of deformations and lining forces are very similar for tunnels of different sizes and of 2D combined to 3D analysis. The comparison of the models in 2D and 3D illustrate that the determination of the trigger values can be carried out in a “much simpler and less time expensive” way in a 2D model.

However it has to be kept in mind that for the first calibration of the 2D model to a complex task in 3D always a simple 3D model is necessary because otherwise it is almost impossible to find the exact relaxation factors which would lead to unreliable results and wrong trigger values.

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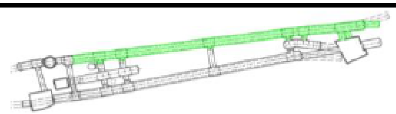
9 Appendix

Daily Review Meeting (DRM)					
Meeting No.:					
Date, Time & Location of DRM:					
RESS Issued with these MoM:					
Review of previous RESS & progress					
Review of shift reports and observations on of shaft/tunnel works (incl. handworks)					
Current ground condition/probing					
Trigger Level					
Amber	Red	Black	delete as appropriate		
Location of breach (if applicable):					
Results of surface and underground (existing structures and SCL convergence) monitoring					
Existing structures: utilities/buildings/LU assets					
SCL Monitoring:					
Sprayed Concrete early age strength results					
Profile Checks/Thickness Checks					
Scope of works for the next 24 hours					
Content of RESS briefing					
Attendees					
Name	Initial	Name	Initial	Name	Initial

RESS (Required Excavation & Support Sheet)

CP8



RESS No.: CP8-006		advance		end advance (m)	advance length (m)
Meeting No.: 768	20.06.15				
Validity from	20.06.15 15:00	TH	9	6,000	1,00
Validity to	21.06.15 15:00	B/I	10	5,000	2,00
		TH	11	7,000	1,00
		TH	12	8,000	1,00
		B/I	13	7,000	2,00
		TH	14	9,400	1,40
		B/I	15	9,000	2,00
		B/I	16	9,400	0,40

* BFK to ensure primary lining is sprayed in one continuous operation without a break in the supply of shotcrete.

* Sufficient amount of shotcrete must be available near face to spray the initial layer to the min. thickness (75mm).

Excavation and Construction in accordance with drawings:

C435-BFK-C4-DDL-M123_Z-53306	C435-BFK-C4-DDL-M123_Z-00017
C435-BFK-C4-DDL-M123_Z-53322	C435-BFK-C4-DDL-M123_Z-00003
C435-BFK-C4-DDL-M123_Z-53332	C435-BFK-C4-DDL-M123_Z-00004
C435-BFK-C4-DDL-M123_Z-53471	C435-BFK-C4-DDL-M123_Z-00005
C435-BFK-C4-DDL-M123_Z-53472	C435-BFK-C4-DDL-M123_Z-00006
C435-BFK-C4-DDL-M123_Z-53501	C435-BFK-C4-DDL-M123_Z-00007
C435-BFK-C4-DDL-M123_Z-53504	C435-BFK-C4-DDL-M123_Z-00008
C435-BFK-C4-DDL-M123_5-00150	C435-BFK-C4-DDL-M123_Z-00009

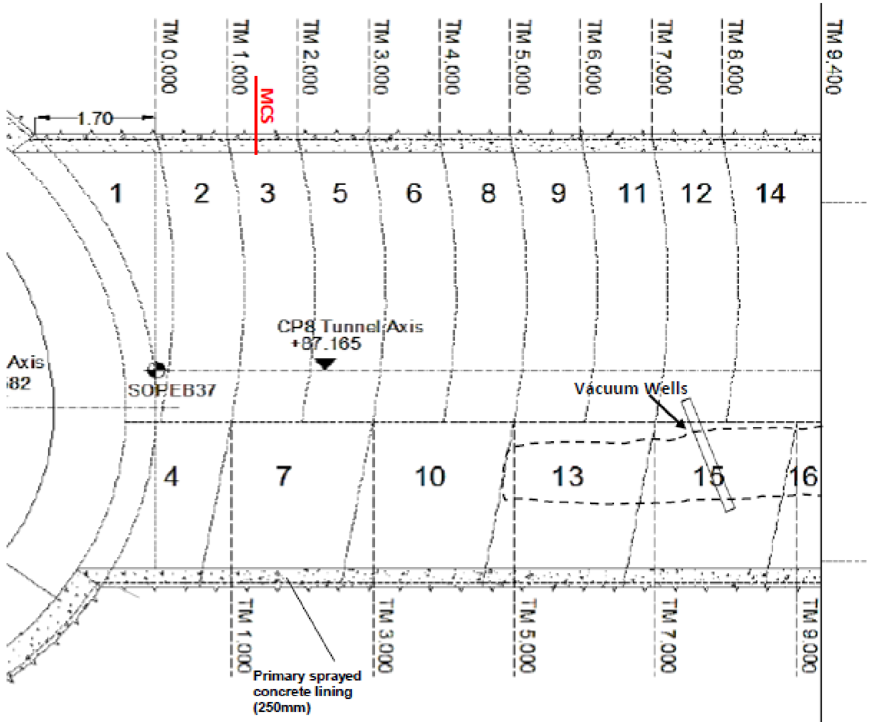
Sprayed concrete: C28/34 **Steelfibres:** 35 kg/m^3

Primary lining 250mm SFR including 75mm Initial lining

Mix design:

Tool box items/Temporary Measures:	Monitoring: installation	reading frequency
Pocket Excavation: B/I-13		
Dewatering: B/I-15	MCS 1.200	

Notes:



Sketch not to scale

Sheet 1/4

Signatures: