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Surface Settlements due to Tunnel Excavation at the Construction Site Stuttgart 21 - Main Station South

Master‘s Thesis

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Graz, October 2018

Statutory declaration

I declare that I have authored this thesis independently, that I have not used other than the declared sources/resources, and that I have explicitly marked all material which has been quoted either literally or by content from the used sources. The text document uploaded to TUGRAZonline is identical to the present master‘s thesis.

Date	Signature

Principle of equality

Due to reasons of legibility, this work does not include gender-specific formulations. However, the used male expressions stand for both genders.

Acknowledgement

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Graz, 29.10.2018 Bernhard W. Hopf

Abstract

This thesis deals with development of settlements due to tunnel excavation, groundwater level lowering and unstable ground conditions at the construction site Stuttgart 21 - Main Station South in Stuttgart. It addresses the assumed ground behaviour, settlements predicted in the design phase by numerical simulations and the construction process itself. The huge amount of monitoring data is sorted and important data are shown. This is the basis for the evaluation of the settlement behaviour at the construction site.

The evaluation is done at six tunnel cross-sections along the emergency access tunnel. These sections match with the design tunnel-cross sections for numerical analyses. For each section the geological conditions are explained first. Afterwards the numerical results are presented and compared stepwise with predicted settlements after Fillibeck (2012), defined warning and alarm values, measured settlements, comparative settlements due to groundwater level lowering and measured displacements. Additionally the measured settlements are referenced to construction processes, construction stops and events that took place during construction. Geological deviations along the tunnel axis and groundwater level lowering of poorly known groundwater bodies are mainly responsible for the differences. Additionally installed support did not work as efficiently as assumed. These evaluations show causes and connections which influenced settlements most.

Kurzfassung

Diese Arbeit beschäftigt sich mit den aufgetretenen Setzungen in der Umgebung des Bauloses Stuttgart 21 - Hauptbahnhof Süd zufolge von Ausbruch, Wasserhaltung und stark wechselnden Gebirgsverhältnissen in Stuttgart. Die Baustelle umfasst den Vortrieb der Rettungszufahrt, die Vortriebe der Tunnelröhren 801, 901, 802 und 902 in Richtung Hauptbahnhof, Zwischenangriff Ulmer Straße und Filder sowie den Ausbruch von vier Pfeilerstollen. Sie hinterfragt die vorangegangene numerische Setzungsberechnung auf Basis der Erkundungsprogramme. Die wichtigsten und maßgebenden Messdaten werden dargestellt und bilden die Basis der Auswertung.

Die Auswertung erfolgt an sechs Tunnelquerschnitten. Diese Querschnitte stimmen mit jenen der numerischen Berechnung überein. Für jeden Querschnitt werden zuerst die geologischen Verhältnisse und die numerischen Berechnungsergebnisse dargestellt. Im Anschluss werden schrittweise die numerisch errechneten Setzungen mit Prognosewerte nach Fillibeck (2012), Warn- und Alarmwerten, gemessenen Setzungen, Vergleichswerten der Setzung zufolge Grundwasserabsenkung und Verscheibungen im Tunnel verglichen. Zusätzlich werden die gemessenen Setzungen mit Vortriebsstand, Vortriebsunterbrechungen und Baumaßnahmen verglichen um etwaige Zusammenhänge zu erkennen.

Die im Rahmen der Vortriebsarbeiten gewonnenen Daten und Erkenntnisse zeigen starke Abweichungen der geologischen Modelle von den tatsächlichen Baugrundverhältnissen. Die ausgeführten Gegenmaßnahmen, um die Setzungen unter Kontrolle zu bringen, waren weitestgehend Verstärkungen des Ausbaus und der Einbau von zusätzlichen Stützmitteln, zeigten jedoch weniger Wirkung als angenommen. Die Grundwasserhaltung hat einen stärkeren Einfluss auf das Setzungsverhalten als die Vortriebsarbeiten.

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Abbreviations

DB	.....	Deutsche Bahn
EAT	.....	Emergency Access Tunnel
FEM	.....	Finite Element Method
FP	.....	Tunnel Portal Filder
GWL	.....	Groundwater Level
GWLL	.....	Groundwater Level Lowering
IH	.....	Intermediate Heading
MS	.....	Main Station
PPS	.....	Planning Permission Section
RMP	.....	Rock Mass Properties
TM	.....	Tunnel Meter (Chainage)
UCS	.....	Uniaxial Compressive Strength
US	.....	Ulmer Street

1 Introduction

Increasing standards of design, construction and operation require increasing quality of methods to predict the influence of projects on humans, property and the environment. This is unavoidable, especially for urban areas with high building density. Underground construction has a strong and immanent influence on the environment due to uncertainties making prediction and design difficult. This is valid for surface settlements caused by tunnelling too.

The prediction of settlements due to tunnel excavation can be done by empirical or numerical methods. Besides the design it is necessary to monitor real settlements and compare them with predicted ones. A further step is to optimize construction methods in regards to the limiting factors and to deal with differences and inaccuracies of any model. A lot of theoretical literature with regard to these topics exist. An evaluation of monitored settlement data in comparison to the predicted settlements during design at the construction site S21 - Main Station[MS] South does not exist. Therefore I decided to pick this specific topic as my master's thesis.

The state of the art and an empirical prediction method are explained first. The huge amount of collected data is sorted and important and prior data is shown. Afterwards the settlement behaviours at the construction site due to the excavation of the emergency access tunnel [EAT] are evaluated. Trends, values and assumptions of settlements predicted during design are compared with the collected data. The final conclusions are presented in regard to the main causes of settlements.

2 Theoretical Fundamentals

2.1 Site Investigation

Site investigations are performed at underground projects to determine properties of soil or rock and the behaviour of the ground due to planned constructions. The outcomes are ground behaviour, substructure and infrastructure design parameters which are used to determine geotechnical, geoenvironmental, geological and hydrological risks to humans, property or the environment [BS EN 1997-1 (2004)]. It is a phased exercise.

1. Preliminary investigation
2. Design investigation
3. Accompanying investigation

It is essential to collect all data obtained from each phase and to check them carefully. It is necessary to ensure original objectives are satisfied and assumptions are verified. If the collected data contains incorrect or implausible values, it may cause an immense additional work later and could lead to high additional costs.

2.1.1 Preliminary Investigation

Preliminary investigation defines the collection of existing data and information on the one side and a decision process on the other. It is necessary to check feasibility of projects in this stage. Just a few tests are performed to roughly determine the main parameters. The determined parameters and decisions are the basis to study different versions and variations.

2.1.2 Design Investigation

Design investigation defines the main investigation parts. Laboratory and penetration tests, pilot holes are performed and drill cores selected. Besides the process of determining the ground behaviour in-situ, samples are taken and tested in the laboratory too. This is necessary because most of the properties can't be determined in-situ.

2.1.3 Accompanying Investigation

The last phase of the investigation process is to observe, monitor and control during the construction process. It is necessary to confirm the assumed behaviour and constitutive laws or to show differences in monitored and assumed behaviour. If confirmation is not possible, assumptions and models need to be updated to describe the correct behaviour. Methods therefore are for example face mapping, testing of excavated material, water inflow measurement and water testing.

2.1.3.1 Water Inflow and Water Properties

Water inflow into the tunnel need to be considered in the design stage of planned constructions. The inflow volume in l/s is simply measured with a bucket and a stopwatch. Further information as for example water inflow from distinct layers, joints or faults are documented as part of face mapping. Samples are taken according to the standards of sample extraction and tested in the laboratory to determine physical and chemical properties. This is done for example to determine the possibility of contamination or to assign specific water inflow to a specific groundwater body. Parameters which are measured in-situ are:

- temperature
- ph
- electrical conductivity
- out-gassing of CO_2

2.1.3.2 Determination of Rock Strength - Method after WBI[1]

The method after WBI is used at the construction site S21 to determine the rock strength of excavated material and compare the results with design assumptions. This method uses an empirical relation between water content of samples and the rock strength. A sample of the excavated material is weighed and dried for twenty-four hours with a constant temperature of 50°C. Afterwards the sample is weighed again and the original water content is calculated as difference of the weighing results. The result is compared to a data base and provides an estimation of the rock strength.

2.2 Conventional Excavation

Conventional excavation is divided in two main excavation methods. These two are Drill & Blast and mechanical excavation. The first one uses explosives to fragment and loosen the rock. The second uses road-headers, excavators or other machines to excavate the material. The usability depends on conditions in-situ, the environment, logistics and cost factors. Figure 2.1 shows the usability of different excavation methods in reference to the strength of the rock mass.

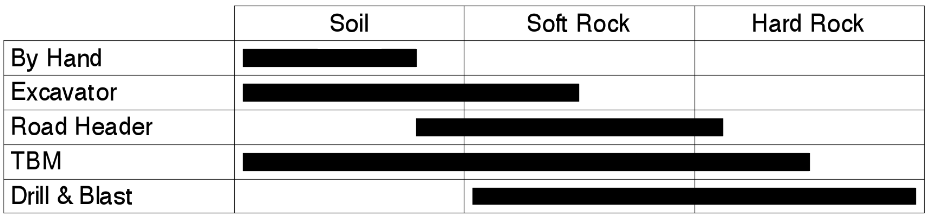

Figure 2.1: Usability of Different Excavation Methods in Reference to the Rock Mass Strength

Different excavation methods have more or less impact on displacements and settlements. A main parameter to keep displacements and settlements small is the time till support is installed after excavation. The largest displacement rates occur close to the face. Therefore the time between excavation and newly installed support should be as short as possible.

[1]WBI GmbH established by Prof. Dr.-Ing. Walter Wittke

2.3 Settlements

Dominating movements at construction sites are displacements in the tunnel and surface settlements. Settlements need to be observed, monitored, controlled and evaluated to prevent humans and infrastructure from risks or damage. Infrastructure is not really affected by uniform settlements in general. It is affected by non-uniform distribution of foundation settlements resulting in tilting. In general uniform settlements of 20 to 50 mm and relative settlement differences up to fifty percent are acceptable at construction sites and the vicinity. Depending on the type of structure limit relative rotation values between 1:1000 and 1:300 are used. The given values apply to sagging mode. For hogging mode these values need to be divided by two. The following figure defines parameters of foundation movement.

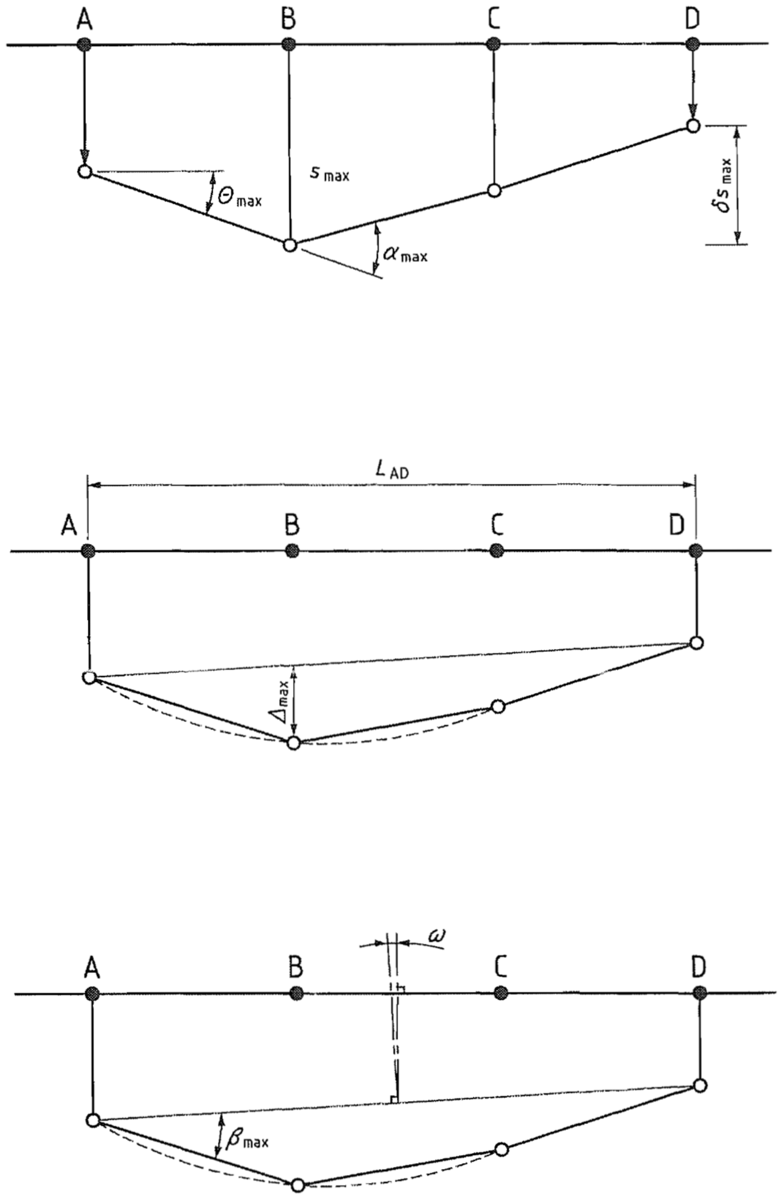

Figure 2.2: Parameters of Foundation Movement [BS EN 1997-1 (2004)]

s ..... settlement

δs ..... differential settlement

Θ ..... rotation

α ..... angular strain

δ ..... relative deflection

ω ..... tilt

β ..... relative rotation

L ..... foundation length

2.3.1 Reasons for Settlements

Settlement reasons can be divided in five main groups.

1. Loading
2. Pit excavation, mining, tunnelling and cavity collapse
3. Change of groundwater level [GWL]
4. Erosion and suffosion
5. Ground freezing or melting

Groups number two to four are very significant for tunnelling projects and therefore are explained. Ground freezing or melting are due to commonly stable temperature conditions in a tunnel only relevant if freezing or melting are artificially created for construction purpose. Loading is not significant for tunnelling projects because in general settlements due to loading had already occurred.

2.3.1.1 Pit Excavation, Mining, Tunnelling and Cavity Collapse

Settlements due to excavations are unavoidable. The construction of open space in any rock or soil causes stress concentrations and stress rearrangements which themselves cause displacements, settlements and deformations. The analytical solution after Feder & Arwanitakis (1976) can be used to calculate secondary stresses at a circular tunnel. These secondary stresses are needed to calculate displacements and in a further step settlements.

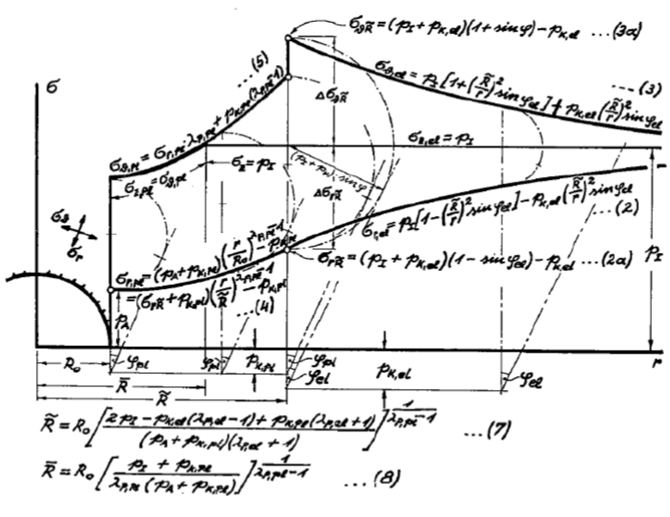

Figure 2.3: Analytical Method after Feder [Feder & Arwanitakis (1976)]

2.3.1.2 Change of Groundwater Level

The GWL can naturally but also artificially due to pumping or drainage due to excavation increase or decrease. A GWL increase in cohesive soil reduces the strength and stability of the system. This causes settlements and an increase of potential dangers to existing infrastructures. A GWL decrease causes an increase of effective stresses in the affected layers and therefore settlements depending on the thickness and the constrained modulus of each affected layer. An additional load of approximately 10 kN/m^2 per meter of groundwater level lowering matches the increase of effective stresses. This load is equivalent to the uplift force of water. Layers below a new GWL are unloaded due to water load reduction which causes and uplift in the affected layers. Uplift and artesian pressure in lower layers may lead to ground water inflow in the invert of excavations.

2.3.1.3 Erosion and Suffusion

Groundwater flows in sandy-silty or lightly cohesive sandy-stony soil can form fine-grained deposits along the underground water body or flush these materials out and create voids (called piping). The loss of fine-grained material in specific sections and deposits of it in others result in partial strength and stability reduction, which again increases settlements.

2.3.2 Mathematical Definition of a Settlement Trough

A settlement trough can be defined in the longitudinal and transversal section. These two sections are defined independently from each other and referenced to one specific excavation. If there are more excavations than one, the same amount of mathematical definitions as tunnel excavations is necessary to define the settlement troughs. These separately defined settlement troughs for longitudinal and transversal parts are super-imposed later on. Figure 2.4 shows the different parameters which are used in the mathematical definitions.

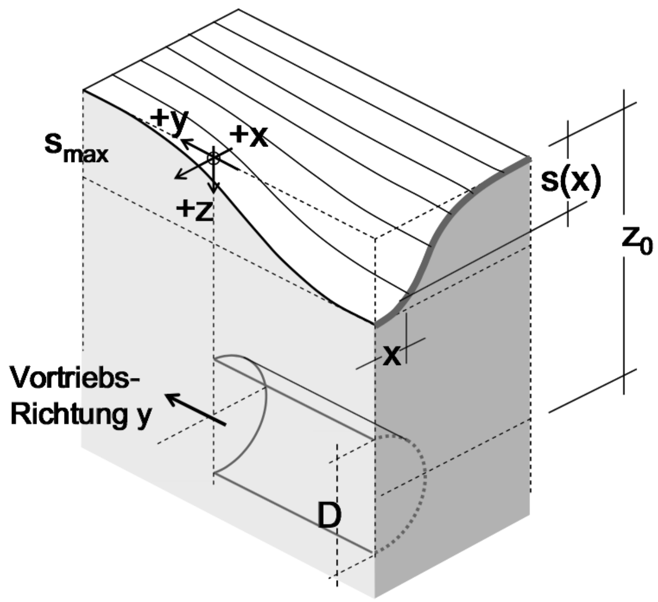

Figure 2.4: Settlement Trough Parameters [Fillibeck (2012)]

$s_{(x)}$	.....	settlements depending on distance x
s_{max}	.....	maximum settlements (above crown)
x	.....	variable distance referenced to tunnel axis
D	.....	tunnel diameter
z_0	.....	vertical distance between surface and tunnel axis

2.3.2.1 Transverse Settlement Trough

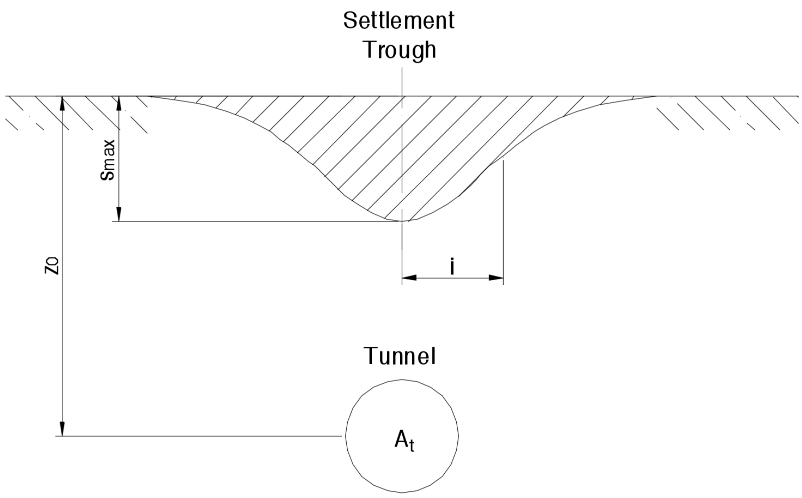

Figure 2.5: Transverse Settlement Trough [Fillibeck (2012)]

s_{max} ..... maximum settlement (above crown)

z_0 ..... vertical distance between surface and tunnel axis

A_t ..... tunnel cross-section area

i ..... distance between tunnel axis and point of inflection

It is necessary to define a function which describes the form as realistically as possible to describe the settlement trough in a mathematical model. Mathematical functions like 4th polynomial, sinus functions, composed functions or Gaussian distribution curves are used. According to Peck (1969b) the Gaussian distribution curve [Equation 2.1] fits best.

$$s_{(x)} = s_{max} * e^{\frac{-x^2}{2*i^2}} \tag{2.1}$$

$s_{(x)}$ ..... settlements depending on distance x

s_{max} ..... maximum settlements (above crown)

x ..... variable distance referenced to tunnel axis

i ..... distance between tunnel axis and point of inflection

The point of inflection i is equivalent to the standard deviation after Gauß. It defines the width of the settlement trough. The maximum settlements above the crown s_{max} are calculated after Equation 2.2 and depend on volume loss VL_s, tunnel cross-section area A_t and point of inflection i.

$$s_{max} = VL_s * \frac{A_t}{\sqrt{2 * \pi * i}} \tag{2.2}$$

The volume loss VL_s defines the size of the settlement trough. It should not be mixed up with volume loss VL_t. Figure 2.6 shows the difference of these these two parameters.

$$VL_s = \frac{V_s}{A_t} \tag{2.3}$$

$$VL_t = \frac{V_t}{A_t} \tag{2.4}$$

V_s ..... settlement trough cross-section area

V_t ..... tunnel cross-section area after excavation

A_t ..... tunnel cross-section area after displacements

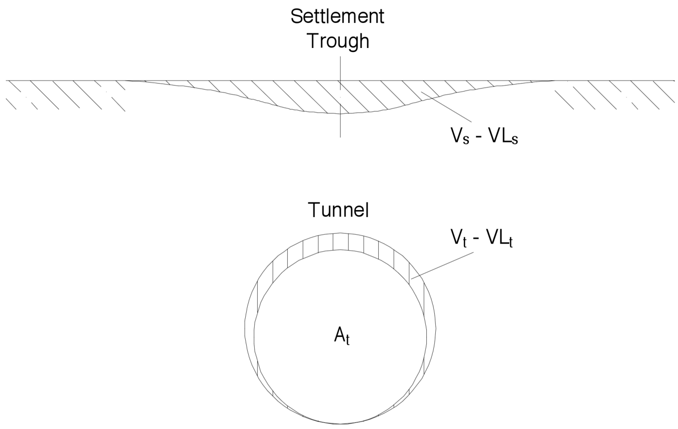

Figure 2.6: Definition of Volume Loss VL_s and VL_t [Fillibeck (2012)]

The settlement trough at construction sites with more than one planned excavation is calculated as super-imposition of each part of each settlement trough due to each excavation.

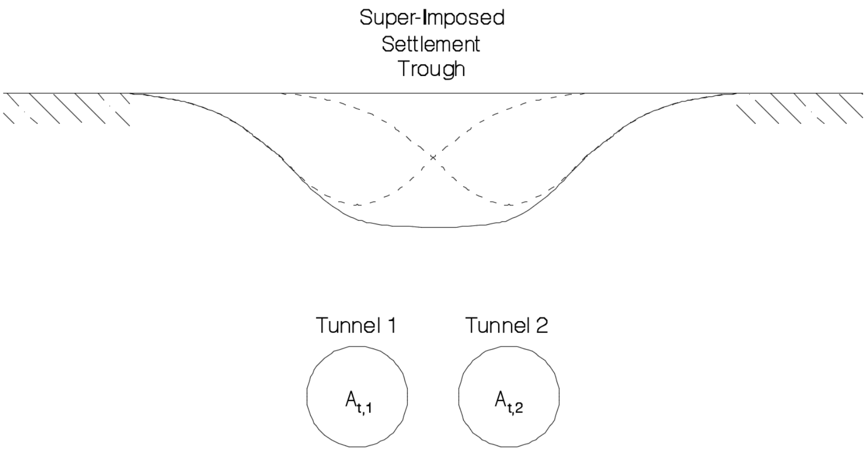

Figure 2.7: Imposed Settlement Trough [Fillibeck (2012)]

$A_{t,1}, A_{t,2}$ ..... tunnel cross-section area

2.3.2.2 Longitudinal Settlement Trough

According to Fillibeck (2012) the integrated normal distribution curve [Equation 2.5] fits best to describe the longitudinal settlement trough in a mathematical approximation.

$$s_{(y)} = \int_{-\infty}^{y} t_y * e^{\frac{(-y+i_y)^2}{2*(\frac{s_{x,max}}{t_y*\sqrt{2*\pi}})^2}} \, \mathrm{d}y - s_{x,max} \tag{2.5}$$

$$\text{with } t_y = \frac{1}{n_y} \tag{2.6}$$

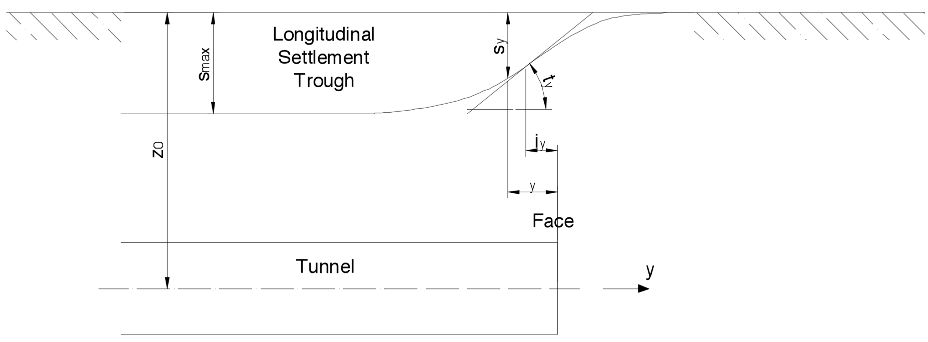

Figure 2.8: Longitudinal Settlement Trough [Fillibeck (2012)]

s_y	.....	settlements depending on distance y
y	.....	distance to the face
t_y	.....	gradient
i_y	.....	distance between point of inflection and face
s_{max}	.....	maximum settlements (above crown)
n_y	.....	reciprocal value of the gradient

2.3.3 Empirical Prediction Method after Fillibeck (2012)

This method was developed by the use of monitoring data of several tunnelling projects in Munich. As for all empirical methods a huge amount of data, in this case tunnelling and settlement parameters, form the basis. The developing process splits up in following steps:

1. Gathering of information
2. Suitability check
3. Evaluation of geodetic data
4. Comparison of calculated and real data
5. Establishment of mathematical model
6. Determination of characteristic parameters
7. Finite element method [FEM] simulation
8. Development of prediction method for geological situation in Munich

9. Comparison of results to other geological regions

10. Development of prediction method in soil or loose rock

The empirical relations of volume loss VL_s and point of inflection i to overburden and constrained modulus are shown in Table 2.1, Table 2.2, Table 2.3 and Table 2.4.

Table 2.1: VL_s for Conventional Excavation Part I[Fillibeck (2012)]

	Atmospheric Excavation non-cohesive ground	Atmospheric Excavation cohesive ground
$VL_{s,50\%}$	$(0.037 * z_0 - 0.10) * \frac{120}{E_{100,ref}}$	$(0.016 * z_0 + 0.31) * \frac{100}{E_{100,ref}}$
$VL_{s,90\%}$	$0.037 * z_0 * \frac{120}{E_{100,ref}}$	$(0.016 * z_0 + 0,47) * \frac{100}{E_{100,ref}}$
$VL_{s,99}\%$	$(0.037 * z_0 + 0.09) * \frac{120}{E_{100,ref}}$	$(0.016 * z_0 + 0.61) * \frac{100}{E_{100,ref}}$

Table 2.2: VL_s for Conventional Excavation Part II [Fillibeck (2012)]

	Compressed Air Excavation
$VL_{s,50\%}$	$(0.005 * z_0 + 0.26) * \frac{100}{E_{100,ref}}$
$VL_{s,90\%}$	$(0.005 * z_0 + 0.43) * \frac{100}{E_{100,ref}}$
$VL_{s,99}\%$	$(0.005 * z_0 + 0,57) * \frac{100}{E_{100,ref}}$

$VL_{s,50\%}, VL_{s,90\%}, VL_{s,99\%}$ ..... volume loss with specific significance level

z_0 ..... overburden

$E_{100,ref}$ ..... constrained modulus

Table 2.3: VL_s for Continuous Excavation [Fillibeck (2012)]

	Hydro Shield Earth Pressured Balance Shield Compressed Air Balance Shield
$VL_{s,50\%}$	$0.0033 * \frac{A_t}{z_0} - 0.82$
$VL_{s,90\%}$	$0.0064 * \frac{A_t}{z_0} - 0.75$
$VL_{s,99}\%$	$0.0093 * \frac{A_t}{z_0} - 0.70$

$VL_{s,50\%}, VL_{s,90\%}, VL_{s,99\%}$	.....	volume loss with specific significance level
A_t	.....	tunnel cross-section area
z_0	.....	overburden

Table 2.4: Point of Inflection [Fillibeck (2012)]

Non-Cohesive Ground	$K_{G/S}$ [-]	Cohesive Ground	$K_{T/U}$ [-]
loose / moderate dense	0.25 / 0.50	soft / stiff	0.30 / 0.60
moderate dense / dense	0.40 / 0.60	stiff / rigid	0.50 / 0.90

$K_{G/S}$	.....	variable for gravel and sand
$K_{T/U}$	.....	variable for clay and silt

Two suggestions are given by Fillibeck (2012) to avoid unrealistic assumptions.

1. A high volume loss significance level corresponds to a moderate distance of the point of inflection i

2. A moderate volume loss significance level corresponds to a low distance of the point of inflection i

3 The Stuttgart - Ulm DB Railway Project

The Stuttgart - Ulm DB railway project implies the two sub-projects "Restructuring of Stuttgart Railway Node - S21" and "Construction of New Line Wendlingen - Ulm". It is one of the largest public railway transportation projects in Baden-Württemberg ever. The project is mainly developed by Gerhard Heimerl, a former professor for railway engineering. It is part of "Magistrale für Europa", which is a Trans-European transportation network project for the construction of a high-speed railway line between Paris and Bratislava, with a branch-off to Budapest. It was published in 1994 and started 2010. The main project parameters are:

- 116.6 km railway line
- 63.4 km tunnel
- 9.3 billion Euro total costs [estimation in 2013]

3.1 Restructuring of Stuttgart Railway Node - S21

The project S21 is divided into several planning permission sections (PPS), which are namely PPS 1.1, PPS 1.2, PPS 1.3a/b, PPS 1.4, PPS 1.5, PPS 1.6a and PPS 1.6b. The main construction purpose of each PPS is listed below:

- PPS 1.1 - City centre valley crossing and main station
- PPS 1.2 - Filder tunnel
- PPS 1.3 - Filder region and airport connection
- PPS 1.4 - Filder region to Wendlingen
- PPS 1.5 - Feuerbach and Bad Cannstatt link

- PPS 1.6a - Obertürkheim and Untertürkheim link
- PPS 1.6b - Untertürkheim sidings

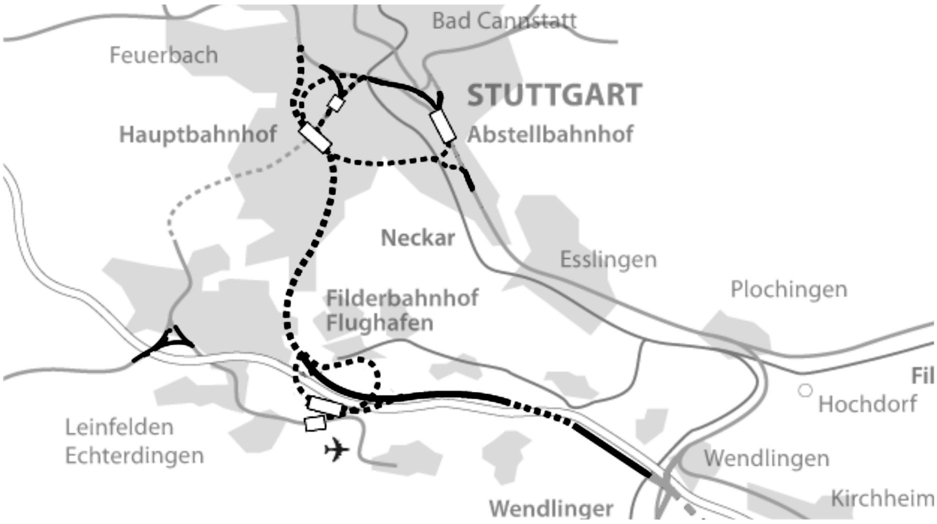

Figure 3.1: Overview Stuttgart 21 [DB Projekt Stuttgart–Ulm GmbH (I) (2018)]

The project covers nearly 60 km of new railway line, three new stations, one new rapid transit station and one new railway yard. 33 km out of 60 km new railway line are built underground by tunnelling or other methods like cut-and-cover. A major construction part of S21 is the main station in the city centre. It will be constructed underground as through-passing railway station. The historical railway station on the surface stays the same. It will be integrated in the new infrastructures. The construction works will restructure approximately 100 ha urban space in the centre of Stuttgart. This is equal to forty percent of the inner city‘s current space.

3.2 Construction of New Railway Line Wendlingen - Ulm

The new line Wendlingen - Ulm will be constructed as highspeed railway connection next to the highway A8. The project is dived in several PPSs, which are namely PPS 2.1a/b, PPS 2.1c, PPS 2.2, PPS 2.3, PPS 2.4, PPS 2.5a1, PPS 2.5a2 and PPS 2.5b. The main construction purpose of each PPS is listed below:

- PPS 2.1a/b - Albvorland: Wendlingen-Kirchheim (S21 connection)
- PPS 2.1c - Albvorland: Kirchheim-Aichelberg
- PPS 2.2 - Albaufstieg

- PPS 2.3 - Albhochfläche
- PPS 2.4 - Albabstieg
- PPS 2.5a1 - Mainstation Ulm
- PPS 2.5a2 - Donaubridge Ulm
- PPS 2.5b - New Ulm 21

The project covers nealy 60 km of new railway line. Five tunnels with a total length of 30 km in one direction are planned for the new line. A major construction part is the Schwäbische Alb crossing. PPS 1.1 - PPS 1.4 handle the constructions at foreland, ascent, plateau and descent at the region Schwäbisch Alb.

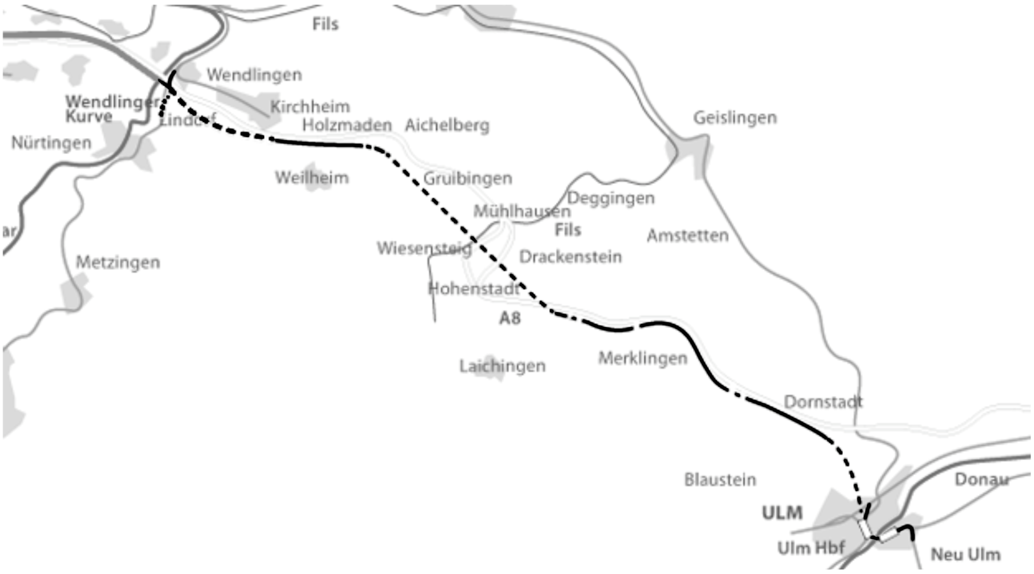

Figure 3.2: Overview New Railway Line Wendlingen - Ulm [DB Projekt Stuttgart–Ulm GmbH (I) (2018)]

4 Construction Site - Main Station [MS] South

4.1 Construction Site

The construction site MS South is located at the Gebhardt-Müller-Platz. In Figure 4.1 the construction site of the new MS [left upper corner], the construction site MS South [center] and tunnels 801, 802, 901 and 902 [right bottom corner] are shown.

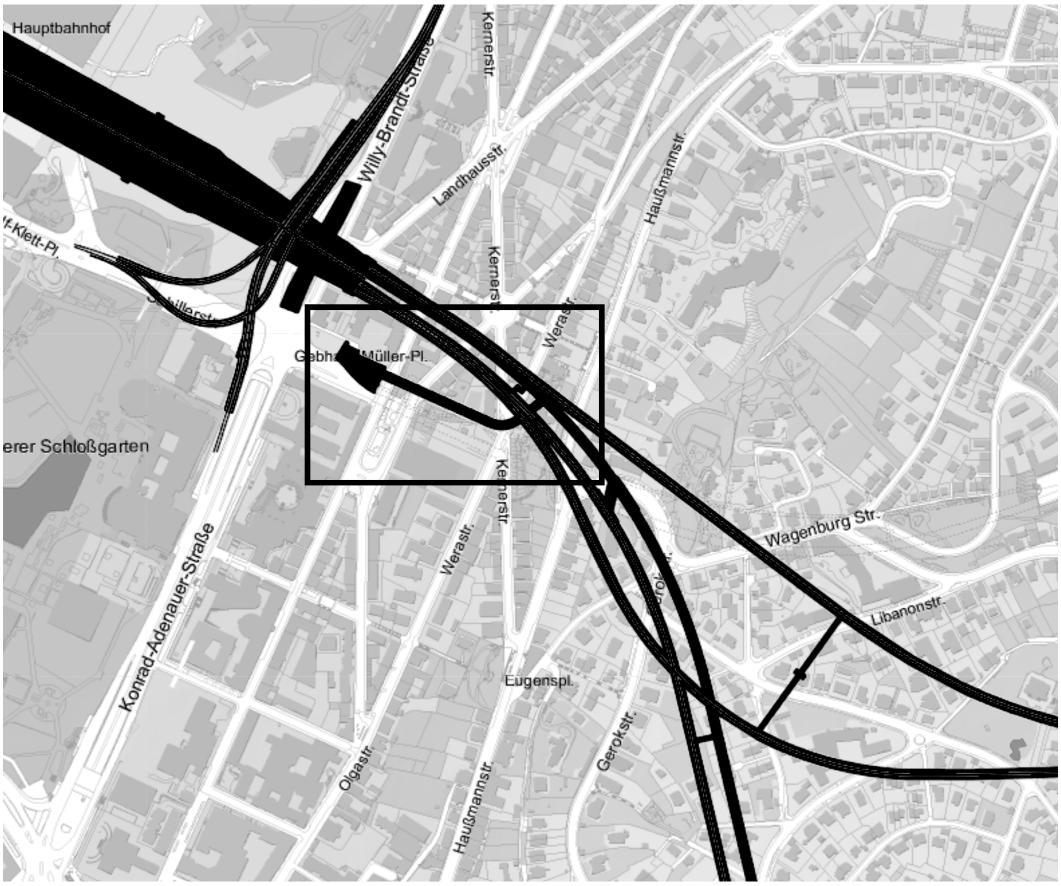

Figure 4.1: Overview of the Construction Site MS South [DB Projekt Stuttgart–Ulm GmbH (II) (2018)]

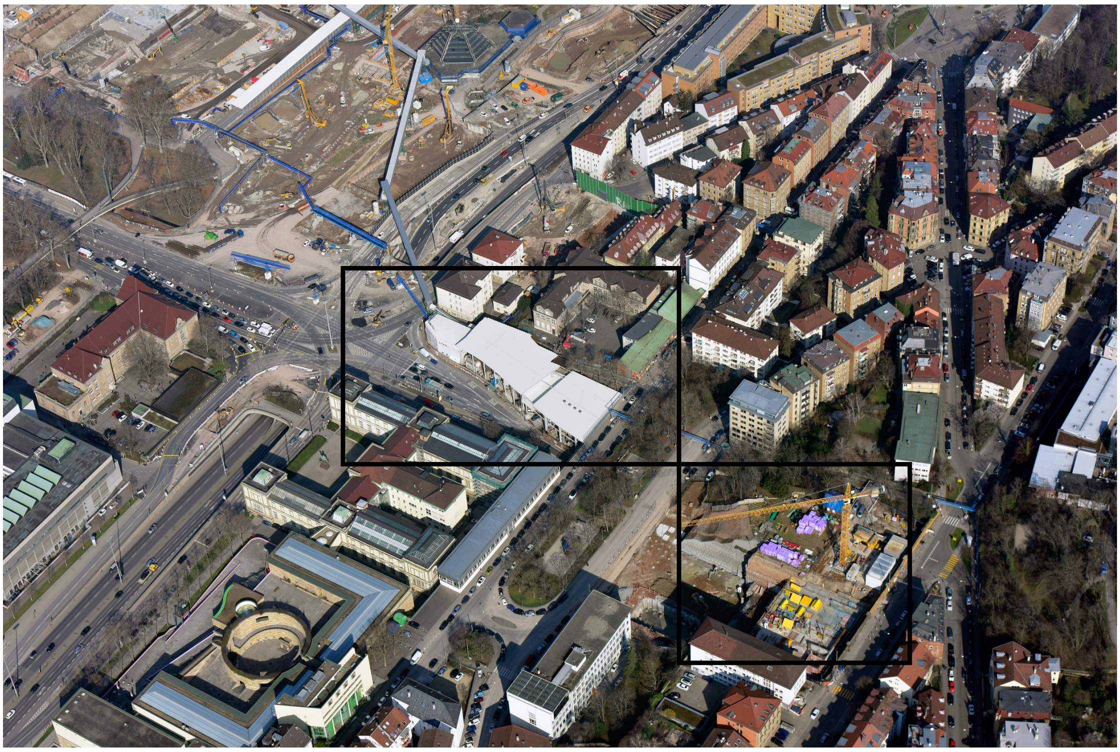

Figure 4.2: Construction Site MS South [center] and Construction Site of John Cranko School [DB Projekt Stuttgart–Ulm GmbH (I) (2018)]

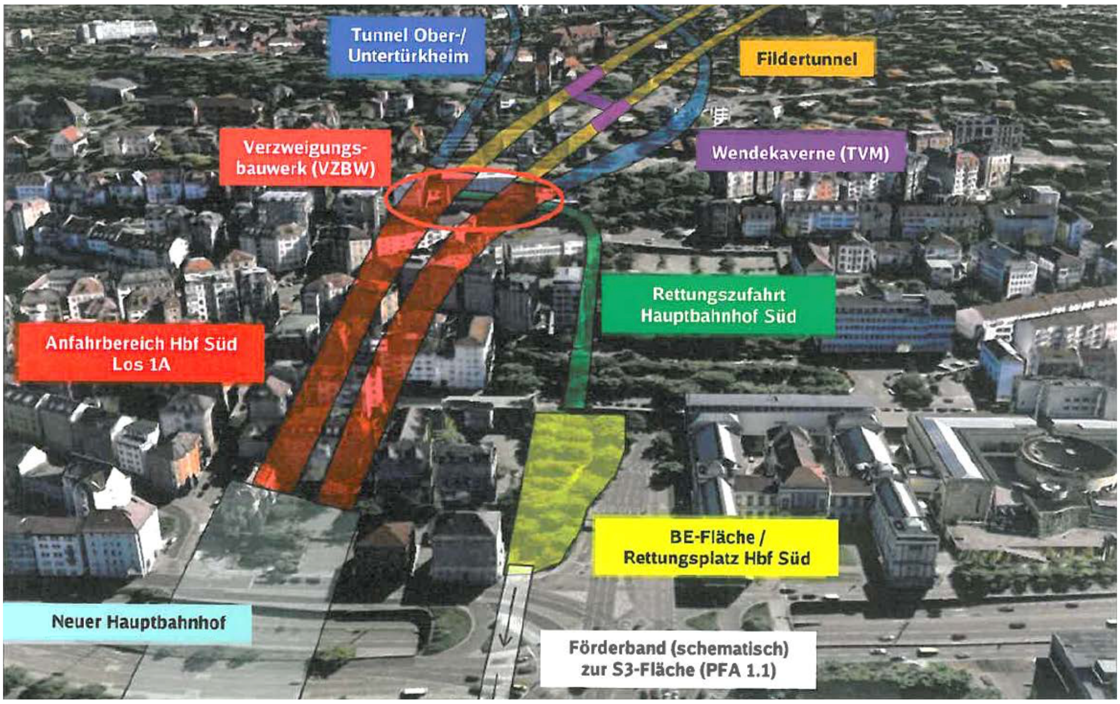

Figure 4.3: Overview of Restructuring of Stuttgart Railway Node in the City Centre [DB Projekt Stuttgart–Ulm GmbH (I) (2018)]

At the construction site MS South the emergency access tunnel [EAT], tunnels 801, 802, 901 and 902 and four pillar galleries are part of the construction work. Tunnels 801 and 802 are excavated, starting from the EAT, in direction MS and in direction Filder portal [FP]. Tunnels 901 and 902 are excavated, starting from the EAT, in direction MS and in direction intermediate heading Ulmer street [IH US]. From the pillar galleries tunnels 801/901 and 802/902 are excavated to the MS as one combined tunnel each. The pillar galleries, shown as black blocks in Figure 4.4, are necessary to stabilise the system because the remaining thickness between the tunnels is to small that the ground could carry the load.

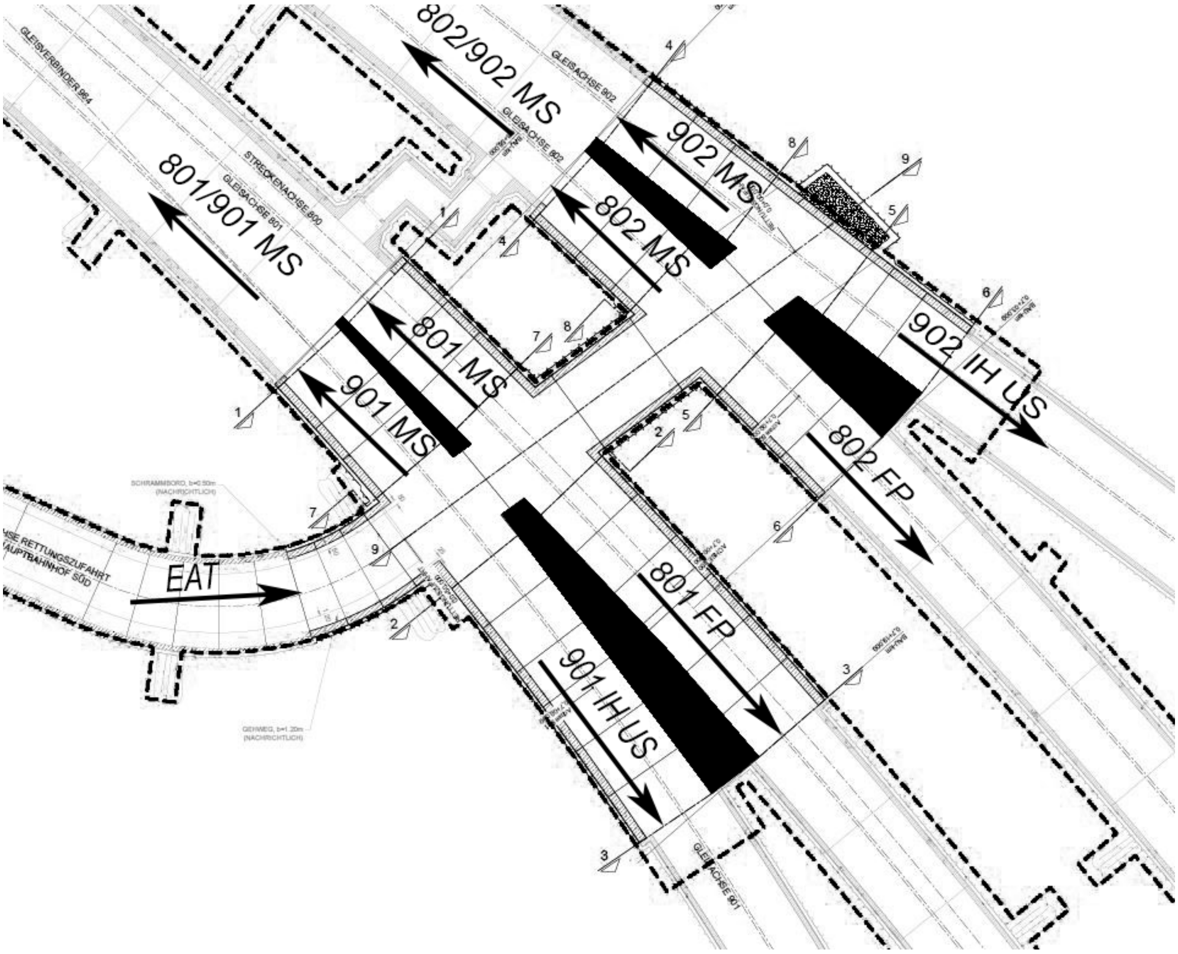

Figure 4.4: Layout Plan of MS South

Figure 4.4 shows the layout plan of the construction site MS South. Tunnel 902 IH US is the only tunnel which is at the moment under construction. The tunnel excavations of 801 FP and 802 FP are stopped due to approval issues. The excavation of tunnels 801 MS, 802 MS, 901 MS, 902 MS and 901 IH US has not started yet. Three out of four pillar galleries are finished.

Table 4.1: Excavation Status - Construction Site MS South

Tunnel	Status
EAT	finished
801 FP	TM 106
801 MS	not excavated
802 FP	TM 108.8
802 MS	not excavated
901 MS	not excavated
901 IH US	not excavated
902 IH US	TM 1090.8
902 MS	not excavated
pillar gallerie east - FP	finished
pillar gallerie east - MS	finished
pillar gallerie west - FP	finished
pillar gallerie west - MS	not excavated

Table 4.1 is referenced to the construction processing status of 1st January 2017.

4.2 Excavation of Emergency Access Tunnel [EAT]

The EAT has a total length of 235 m. From TM 0 to TM 160 it underpasses the existing Wagenburgtunnel North with a ten degree slope and afterwards is constructed horizontal. In Figure 4.5 the intersection of Wagenburgtunnel North and EAT over the first 45 m is shown.

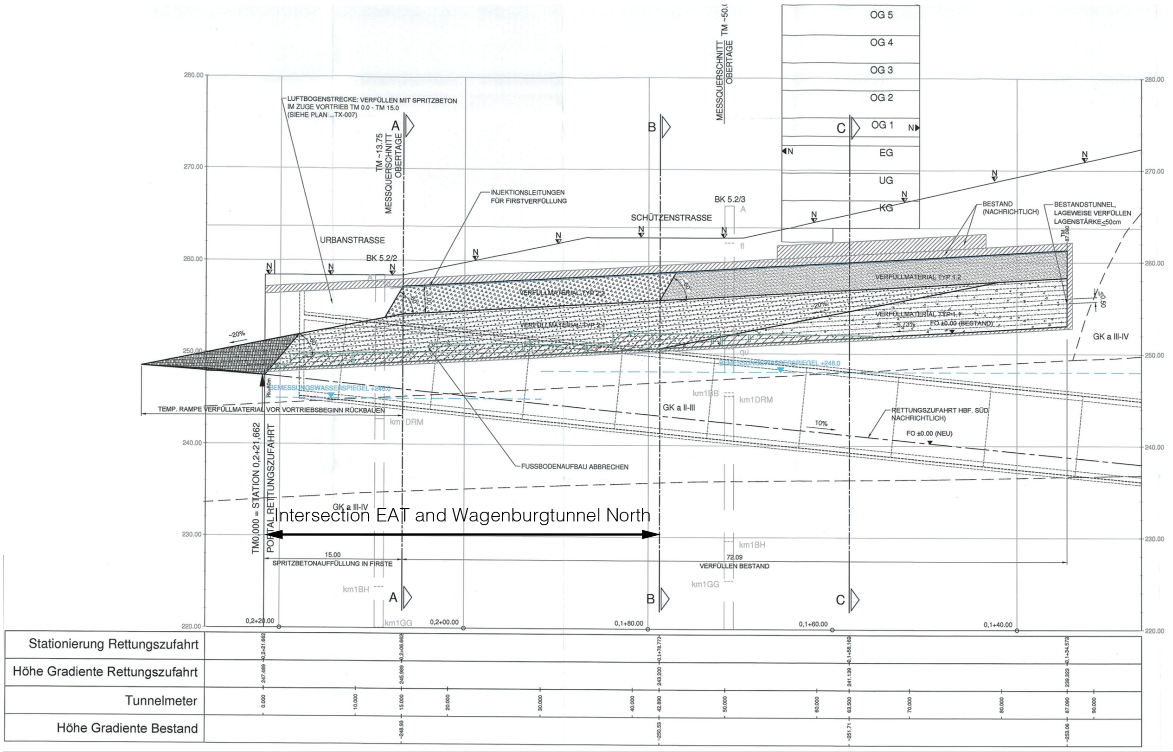

Figure 4.5: Intersection of EAT and Wagenburgtunnel North

From TM 0 to TM 63.5 the EAT was excavated by cut and cover, which means after the installation of the inner lining the remaining open space was filled with lean concrete. From TM 63.5 to TM 235 the partly full-face but mostly two-step excavation was done by excavators. After constructions at MS South are finished the EAT will be used as access to the tunnel system for emergency forces.

From TM 0 to TM 63.5 no support is needed because the EAT is built more or less inside the existing Wagenburgtunnel North. Pipe umbrellas are used as pre-support from TM 63.5 to TM 180 to reduce displacements and settlements and to be able to safely excavate soil-like highly leached gypsum keuper. From TM 180 to TM 235 tube spiles are installed as pre-support.

Figure 4.6: Emergency Access Tunnel TM 0 to TM 160

Figure 4.7: Emergency Access Tunnel TM 160 to TM 235

4.3 Geological Conditions[1]

Six different layers define the geological model of MS South. The EAT intersects with four of them. At the intersection of Wagenburgtunnel North and EAT filling material of the Wagenburgtunnel North dominates. This material is assigned to the first layer. Therefore this talus deposit layer and filling material dominates the ground behaviour at the first 65 m.

The second layer is middle gypsum horizon with leached gypsum keuper of mainly classes II and III. The third blue-lead layer also consists of leached gypsum keuper of classes II and III. The fourth layer, called dark red merl consists mainly of leached gypsum keuper of classes III and IV. The layers blue-lead und dark red merl therefore are treated as part of the layer middle gypsum horizon. Deeper layers of minor interests are Bochinger horizon, deep gysum layers and clayey keuper.

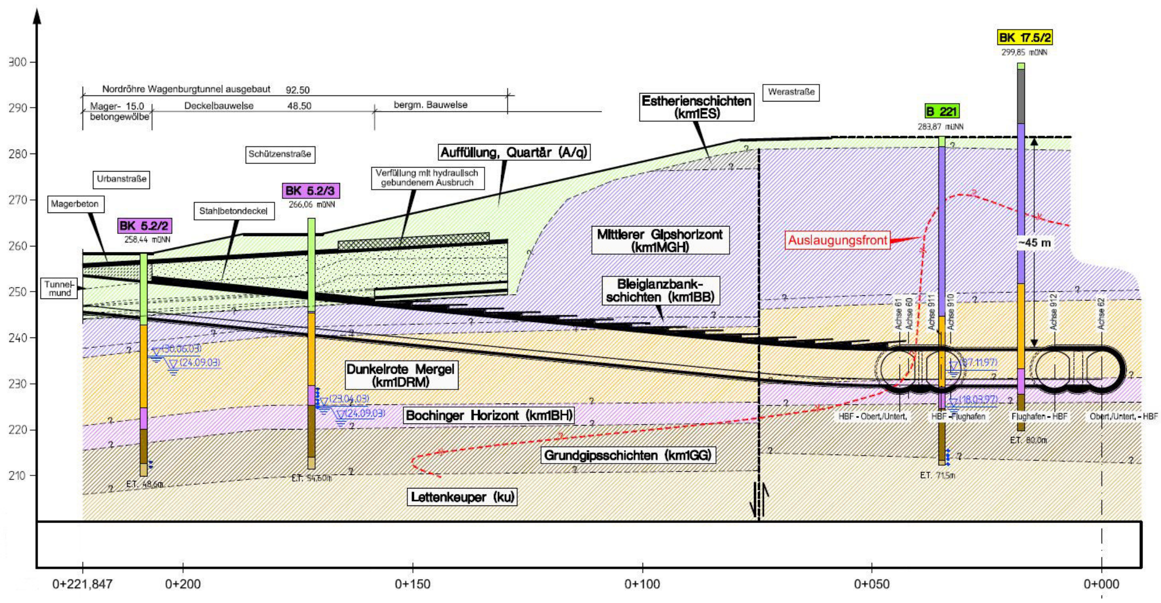

Figure 4.8: Geological Longitudinal Section of the EAT [WBI GmbH (2010)]

Leached gypsum keuper is the result of dissolved gypsum due to chemical reactions with groundwater. The EAT is mainly excavated in geological units with leached gypsum keuper of classes II to IV. The different classes of leached gypsum keuper are listed in Table 4.2.

[1]The geological conditions are taken from "Tunnelbautechnisches Gutachten Fildertunnel Streckenachse 910: km 0+432 - 9+900" by WBI GmbH.

Table 4.2: Classification of Leached Gypsum Keuper

Class	Description
class I	rock-like silt stone
class II	silt stone (moderate solid to friable, highly fractured)
class III	silt stone (friable to very friable, highly fractured)
class IV	leaching residuals

In general classes II, III and IV are comparable to soil classes 4 and 5 after DIN 18300 (2015). At the design sections class III dominates. This class has quite bad rock mass properties [RMP] and the samples of class III show mostly mixed to fine-grained soil with light to moderate plasticity and semi-solid to solid consistency. The characteristic RMP for class III are given in Table 4.3.

Table 4.3: Characteristic Values of Leached Gypsum Keuper Class II - IV [WBI GmbH (2010)]

Parameter	Range	Char. Value
UCS	3 - 15 MPa	6 MPa
friction angle	-	25°
cohesion	0 - 0.04 MN/m^2	0.02 MN/m^2
E-modulus	80 - 200 MPa	150 MPa

Non-leached gypsum keuper dominates the deeper layers Bochinger horizon and deep gysum layers. It is comparable to soil classes 6 and 7 after DIN 18300 (2015). The characteristic RMP of non-leached gypsum keuper are given in Table 4.4.

Table 4.4: Characteristic Values of Non-Leached Gypsum Keuper without Anhydrite [WBI GmbH (2010)]

Parameter	Range	char. Value
middle gypsum horizon:		
UCS	5 - 25 MPa	15 MPa
friction angle	-	35°
cohesion	0 - 0.1 MN/m^2	0.04 MN/m^2
E-modulus	2000 - 4000 MPa	3000 MPa
deep gysum layers:		
UCS	5 - 60 MPa	20 MPa
friction angle	-	35°
cohesion	0 - 0.1 MN/m^2	0.04 MN/m^2
E-modulus	4000 - 6000 MPa	5000 MPa

The RMP of non-leached gypsum keuper are far better than the RMP of leached gypsum keuper. The UCS is around four times higher and the E-modulus is around thirtyfive times higher.

4.4 Overburden

The construction site is located at a hillside and the tunnel is constructed with a slope. Therefore the overburden varies from approximately 5 to 60 m. The correlation of surface elevation, tunnel crown and overburden are shown in Figure 4.9. For the first 50 m overburden is less than 10 m. Low overburden generally generates higher settlements but the EAT intersects for the first 63.5 m with the Wagenburgtunnel North and therefore the low overburden is not a big deal. The EAT is built under protection of the Wagenburgtunnel North.

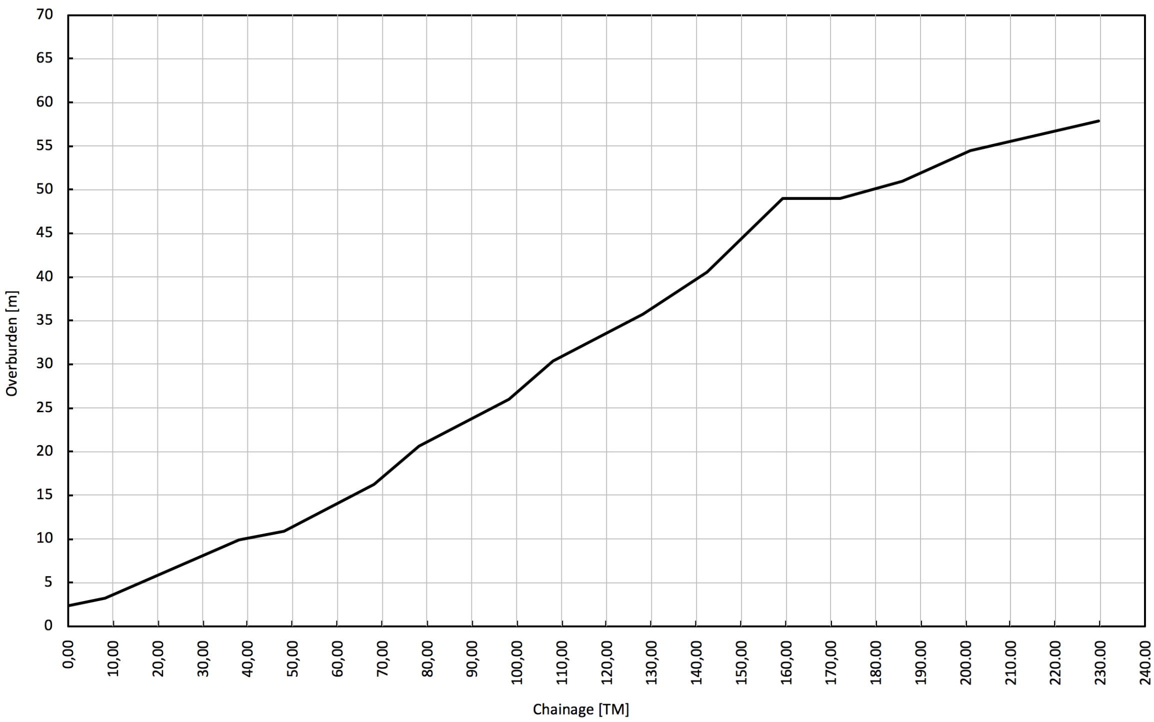

Figure 4.9: Overburden along EAT

4.5 Surface Measurement Points

The monitoring program splits up in two periods. The first one[2] handles the monitoring of surface measurement sections TM 0 to TM 130 [Figure 4.10] and the second one[3] handles the monitoring of surface measurement sections TM 130 to TM 235 [Figure 4.11]. The first monitoring period started on 13.06.2013 and ended on 24.04.2015. The second monitoring period started on 31.10.2014 and did not end yet. Measurement sections TM 130 to TM 235 have rarely been monitored during the first monitoring period and measurement sections TM 0 to TM 100 have rarely been monitored during the second monitoring period. The measurement sections TM 110 and TM 120 of the first period were the only ones which have been monitored for a short time at the beginning of the second monitoring period. The overlap of the two monitoring programs therefore is not documented well.

[2]Plan Package 602 - Plan Nr.: A-01-20-22002-04-TX-041e
[3]Plan Package 605 - Plan Nr.: A-01-20-22002-04-TX-141b

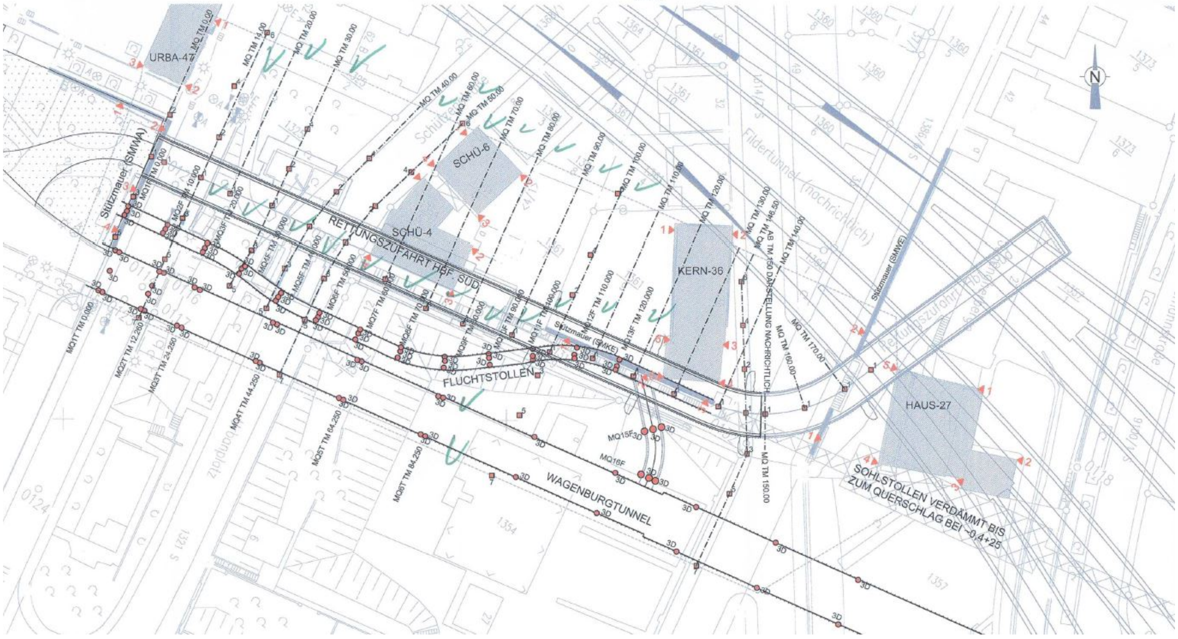

Figure 4.10: Surface Measurement Points/Sections TM 0 to TM 130

Figure 4.11: Surface Measurement Points/Sections TM 130 to TM 235

Measurement points are installed at the surface as levelling points for geodetic monitoring of surface settlements. According to the two monitoring programs measurement points are installed at least 50 m ahead of the progressing face. The monitoring interval of each measurement section depends on the distance of the section to the face. Daily measurement is done for measurement sections which are ahead of the face and as long as the measurement section is less than 30 m behind the face. An interval of three times a week is set for mea-

surement sections which are less than 100 m behind the face. Measurement sections with larger distance to the face than 100 m are monitored once every second week till the inner lining is installed or settlements stop.
Measurement sections and points are located, beginning at the portal [TM 0], along the tunnel axis of the EAT with a maximum distance of 14 m. The regular distance between sections is 10 m. In total twenty-five measurement sections, which are shown in the figures above are installed. The underground construction affects the ground behaviour of a huge area with high building density in the city centre of Stuttgart. Therefore fifty-two additional measurement points are installed at seven houses. The addresses of them are listed below.

- Schützenstraße 4
- Schützenstraße 6
- Kernerstraße 36
- Haußmannstraße 27
- Werastraße 33
- Werastraße 42
- Werastraße 44

4.6 Groundwater Level Lowering [GWLL]

Stuttgart is known for several spa water springs and groundwater bodies with high water quality. The investigation programs explored different groundwater bodies at different depths at the construction site MS South. Therefore a specific water treatment plan was set up to ensure the safety of important groundwater bodies. It defines daily tests to determine physical and chemical properties of the water. The testing results are compared to properties of known groundwater bodies to elaborate if the inflow can be assigned to one of them.
GWLL systems have already been considered necessary for tunnel construction during the design phase. The main task was therefore not to effect important water bodies and to monitor the water inflow into the tunnel as described in the waste-water concept. Figure 4.12 shows the monitoring key points. Tunnels 801 FP, 802 FP and 902 IH US are equipped with water reservoirs to collect the inflowing water. The water in these reservoirs is pumped into the sedimentation tanks at the EAT. These waste-water pipes and water-supply pipes

are equipped with water gauges for each tunnel. A central gauge is placed at the pipe from the sedimentation tank to the water treatment system. Unfortunately the documentation of water flow started later than the installation of GWLL systems.

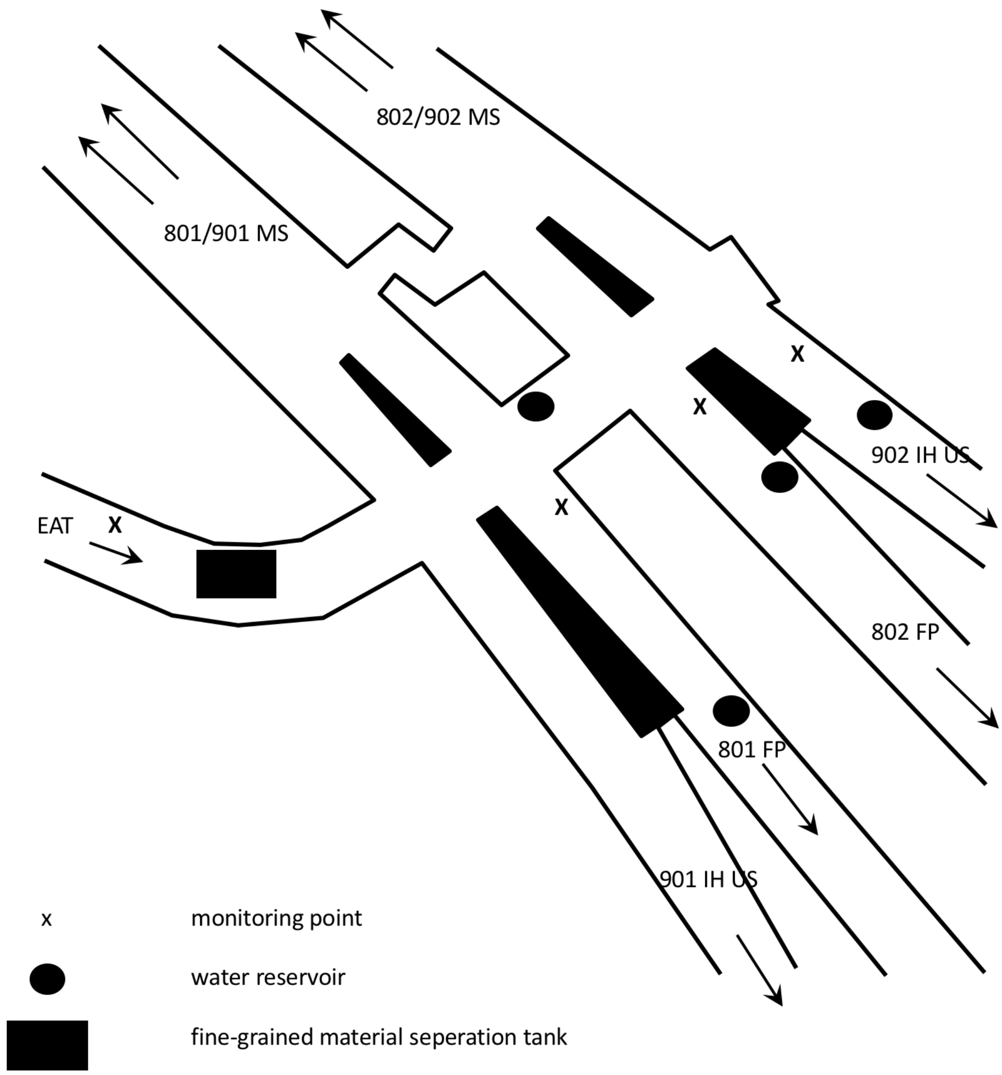

Figure 4.12: Waste-Water Measurement Concept

The readings of waste-water and water-supply gauges at each tunnel are recorded daily. The differences of the readings of waste-water gauge and supply-water gauge are the water inflow at each tunnel. The recorded water inflows of each tunnel are shown in Figure 4.13 for EAT, Figure 4.14 for 902 IH US, Figure 4.15 for 801 FP and Figure 4.16 for 802 FP.

The excavation of EAT started on 25.10.2013 and the excavation of 902 IH US on 21.11.2015. The water inflow monitoring started on 24.11.2015 at the EAT and therefore after the excavation start of 902 IH US. The average water inflow can be seen in Figure 4.13 as approximately 80 m^3/h.

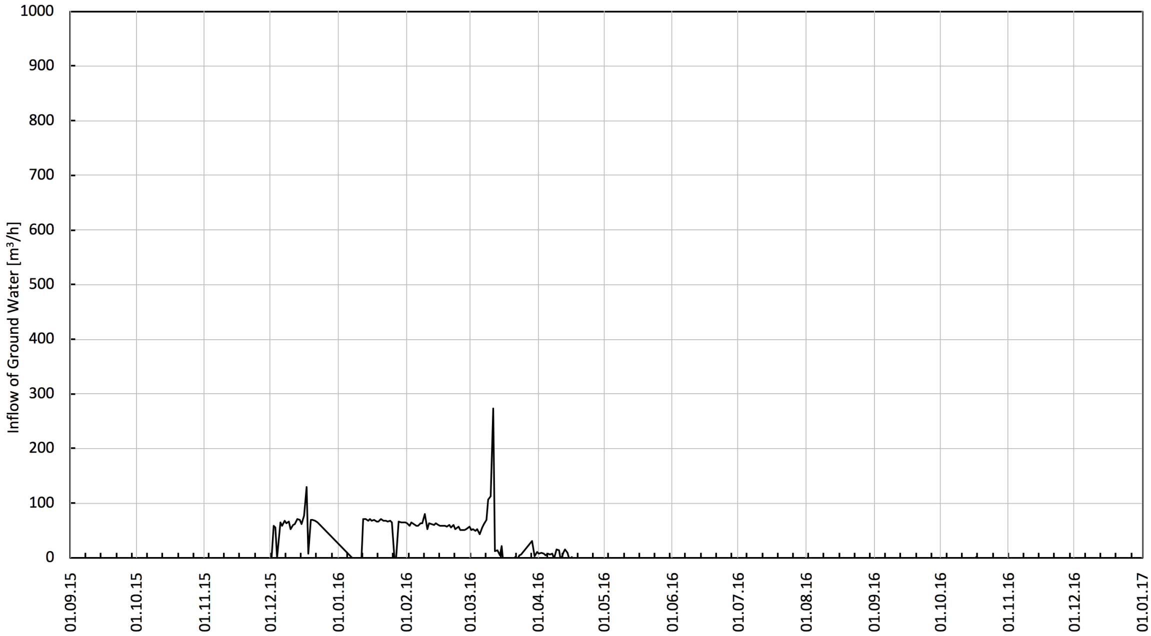

Figure 4.13: Water Inflow at EAT

Water inflow monitoring at tunnel 902 IH US started on 12.03.2016 (TM 313.8). In Figure 4.14 it can be seen that water flows in only two months. Initially the water inflow was around 300 m^3/h and decreased very quickly to zero. The water pumping due to GWLL at tunnel 802 FP was around 300 m^3/h after GWLL started. It is most likely that tunnel 902 IH US and 802 FP intersect with the same ground water body and GWLL at 802 FP stopped the inflow at 902 IH US. Anyway there was water inflow only along the first meters of tunnel 902 IH US and afterwards the excavation does not intersect with ground water bodies at all.

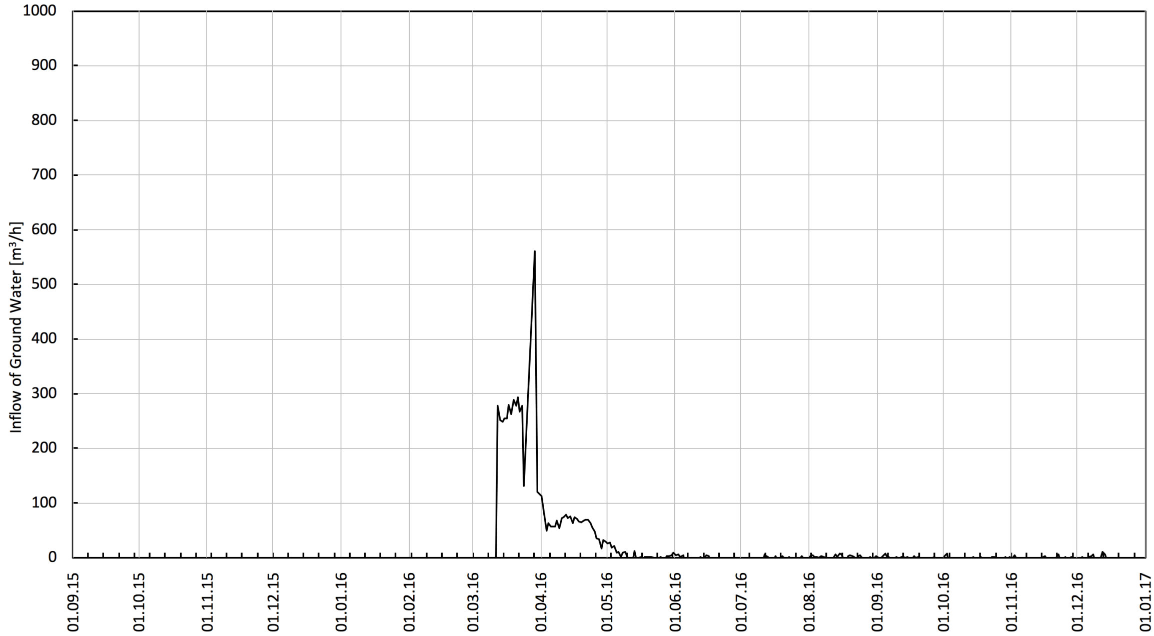

Figure 4.14: Water Inflow at Tunnel 902 IH US

The late start of monitoring at tunnel 801 FP after the installation of the GWLL system makes the interpretation of Figure 4.15 difficult. The closest assumption would be to assume a constant water pumping of 150 m^3/h.

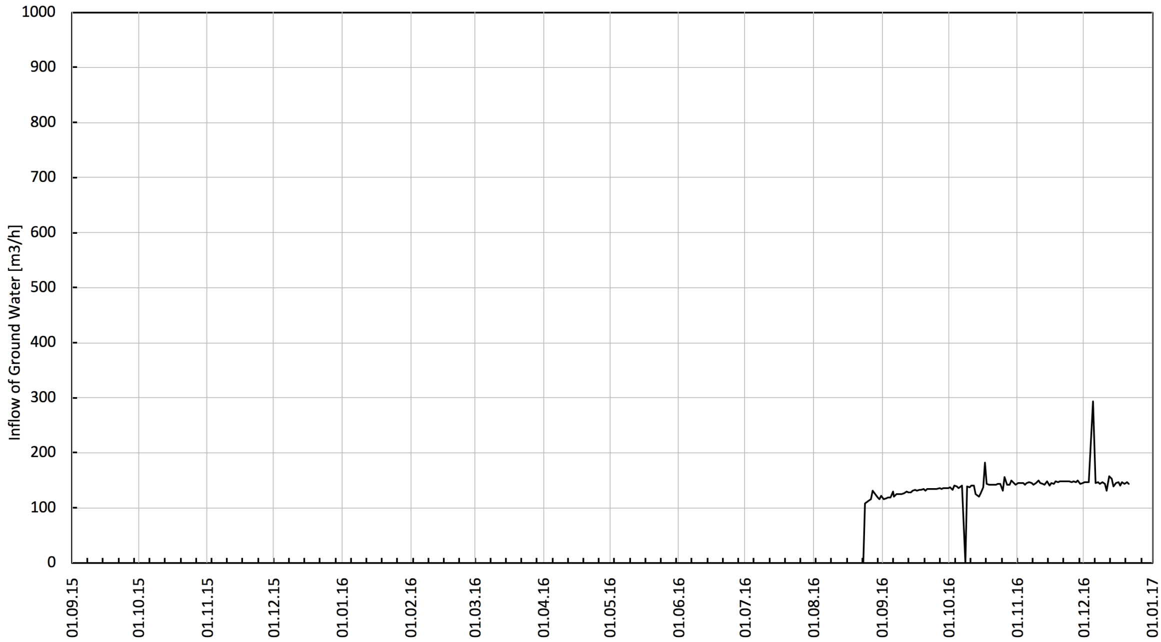

Figure 4.15: Water Inflow at Tunnel 801 FP

In Figure 4.16 an average water pumping due to GWLL of 300 m^3/h is shown at tunnel 802

FP for the first months of monitoring. It is most likely that 902 IH US, 802 FP and 801 FP intersect with the same ground water body. Therefore GWLL at tunnel face 801 FP and 802 FP most likely balanced after the first month and the water flow at tunnel face 802 FP is constant with approximately 100 m^3/h since then.

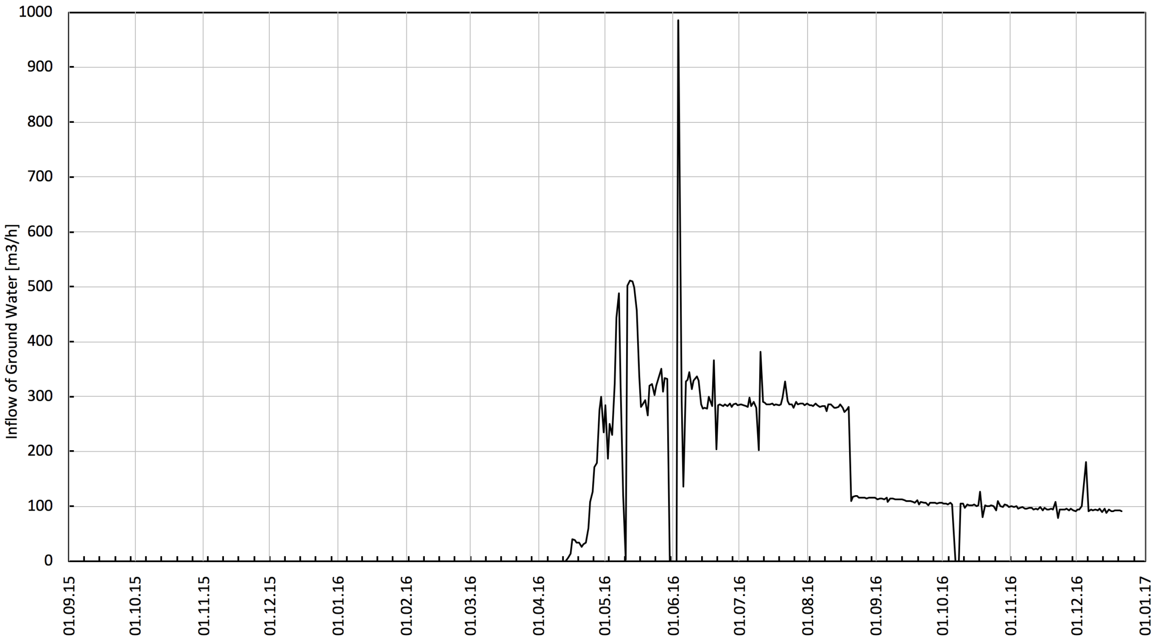

Figure 4.16: Water Inflow at Tunnel 802 FP

5 Evaluation at Design Sections

During the project design phase of S21 2D numerical analysis of settlements were carried out with the software SOFISTIK[1] and the use of FEM. According to report PGS 21 (I) (2013) and report PGS 21 (II) (2014) the design settlements at the construction site are in general comparable to settlements at similar underground construction projects. The FEM analysis of surface settlements were carried out at five design sections from TM 0 to TM 166 and additional ones from TM 166 to TM 235 at the EAT. All five sections which are part of this evaluation, are shown in Figure 5.1 and Figure 5.2.

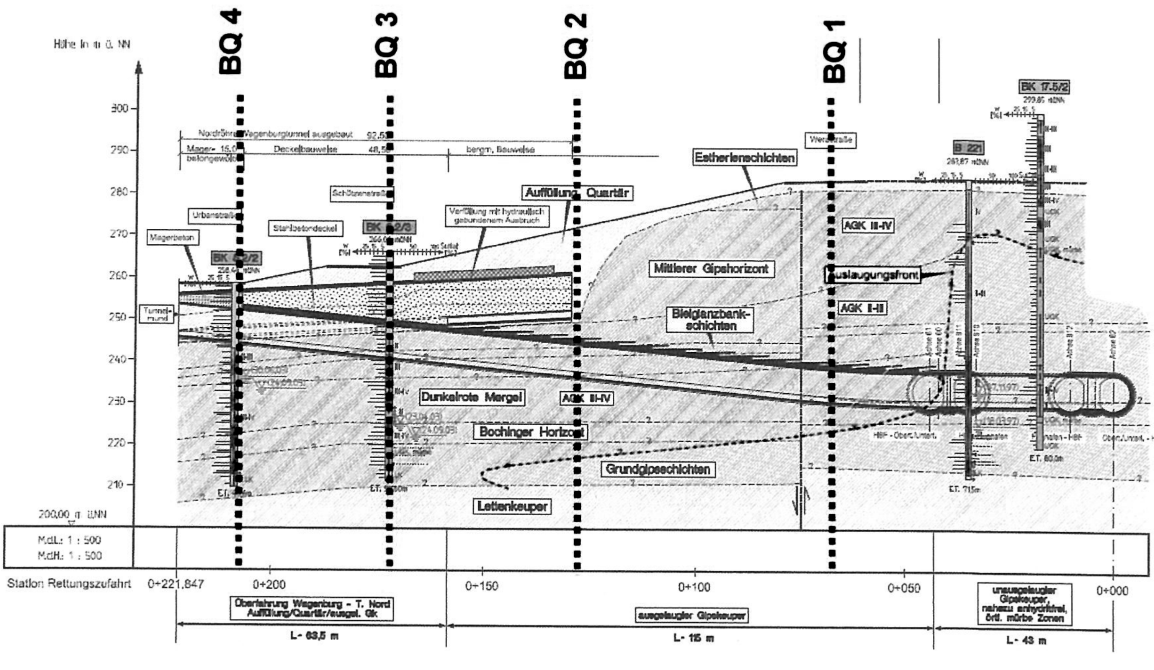

Figure 5.1: Position of Design Sections DS 01, DS 02, DS 03 and DS 04 [PGS 21 (I) (2013)]

[1]SOFISTIK is a numerical modelling software from SOFISTIK AG.

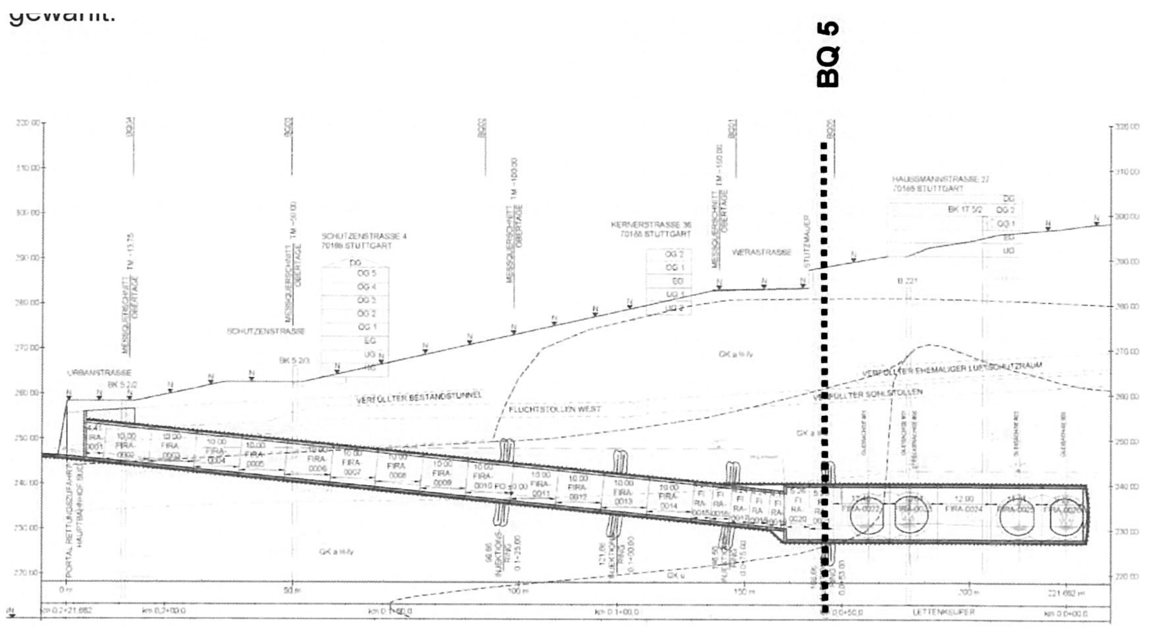

Figure 5.2: Position of Design Section DS 05 [PGS 21 (II) (2014)]

This chapter provides information about all known conditions, measurement data, calculated values and comparative values of each design section. The information is structured for each of these sections as following:

- Geological Conditions
- Design settlements [SOFISTIK] [PGS 21 (I) (2013); PGS 21 (II) (2014)]
- Predicted settlements after Fillibeck (2012)
- Warning and alarm levels
- Measured settlements
- Comparative settlements due to GWLL
- Measured displacements

All tables, evaluations and conclusions are referenced to the construction processing status of 1st January 2017.

5.1 Design Section 04 [DS 04]

Design simulations are performed at cross-section TM 15. The results at DS 04 are valid from TM 0 to TM 15. The EAT intersects over the whole length of DS 04 with the Wagenburgtunnel North and it is constructed with a ten degree slope in this section.The existing foundation of the Wagenburgtunnel North is partly demolished and partly additional excavations were necessary at the invert to build the new tunnel. Girders with low spacing are used as support.

5.1.1 Geological Conditions

DS 04 is characterized by the geological layers talus deposit and filling material and middle gypsum horizon. The gypsum keuper of the middle gypsum horizon is highly leached and assigned to class II to III. Layer thicknesses at cross-section TM 15 are listed in Table 5.1.

Table 5.1: Layer Thickness at Design Cross-Section TM 15

Layer	Thickness [m]
talus deposit and filling material	8.9
middle gypsum horizon	17.3

The tunnel cross-section is mainly-situated in the the cross-section of Wagenburgtunnel North and the layer middle gypsum horizon. Nearly no excavation is necessary to construct the new tunnel inside the existing Wagenburgtunnel North. The geological conditions therefore doesn't really influence settlements. The GWL at DS 04 is 245 m above NN. The tunnel position is shown in Figure 5.3.

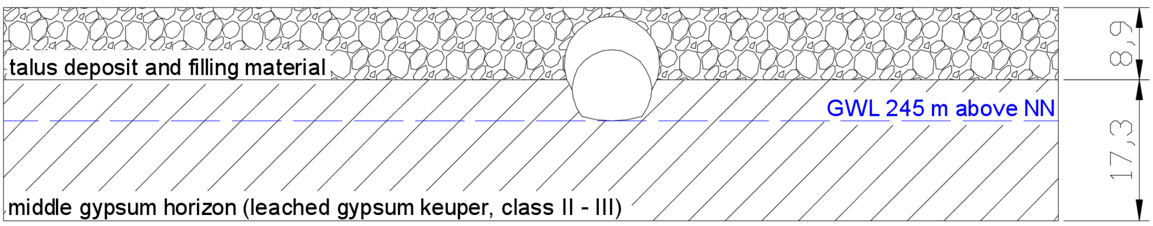

Figure 5.3: Design Cross-Section TM 15

5.1.2 Design Settlements

Due to the intersection of the Wagenburgtunnel North and the EAT no excavation step is modelled in SOFISTIK. At this design section the refill steps with lean concrete at 50 %

and at 100 % are modelled. A possible decaying process of the Wagenburgtunnel North is modelled too to determine the importance of this case.

Table 5.2: Design Settlements according to SOFISTIK at DS 04 [TM 15] [PGS 21 (I) (2013)]

FEM Modelling Step	Maximum Design Settlements [mm]
refill Wagenburgtunnel North 50 %	1.58
refill Wagenburgtunnel North 100 %	3.83
rotting Wagenburgtunnel North	3.38

5.1.3 Predicted Settlements after Fillibeck (2012)

The empirical prediction method after Fillibeck calculates settlements due to underground excavation. The intersection of the EAT with the Wagenburgtunnel North requires nearly no excavation and therefore the method is not usable at DS 04.

5.1.4 Warning and Alarm Level

According to report PGS 21 (III) (2013) warning levels are defined as the maximum settlement values of the numerical analysis with conservative RMP. The observational method is used to reduce risks to humans, property and the environment. The method is based on the comparison of periodical measurement values and defined warning or alarm values.

Table 5.3: Warning and Alarm Levels at DS 04 [PGS 21 (III) (2013)]

	TM 0 - TM 15
FEM result	<5 mm
pre-warning level	not defined
warning level	5 mm
alarm level	10 mm

5.1.5 Measured Settlements

At DS 04 measurement section TM 14 shows the highest settlements. The section is defined by seven measurement points 13-000014-01 to 13-000014-07. The settlement development is shown in Figure 5.4.

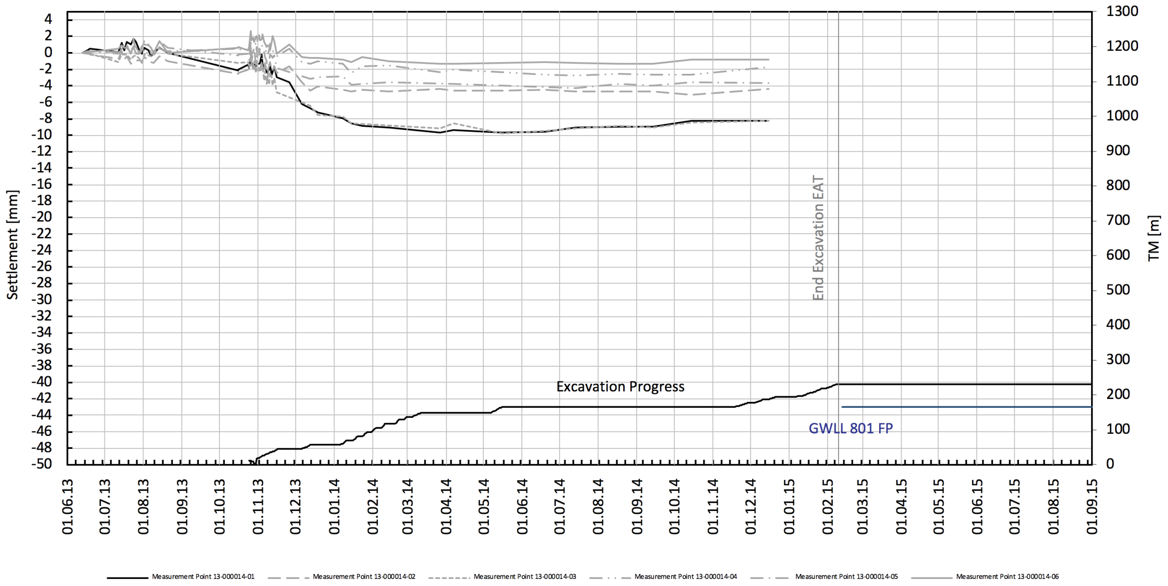

Figure 5.4: Settlements and Excavation Progress at Measurement Section TM 14

The monitoring period started on 13.06.2013 and the excavation of the EAT on 25.10.2013. After the construction started settlements developed continuously till the excavation reached TM 150 (13.03.2014). Further excavations didn't influence the settlements anymore and they remained constant. At TM 14 settlements reached 10 mm in total. GWLL started after the first monitoring period stopped.

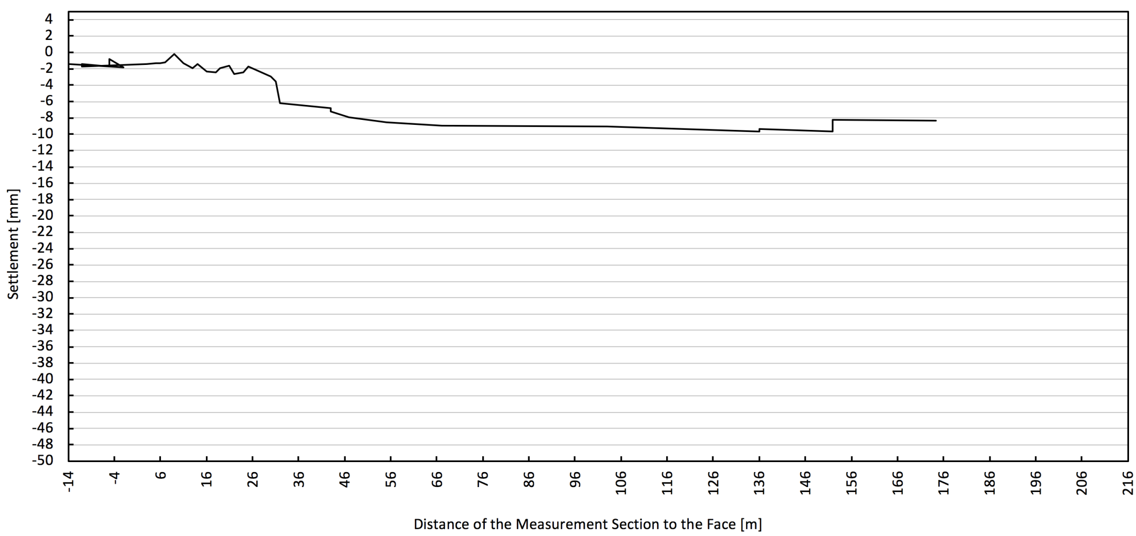

Figure 5.5: Settlements at Measurement Section TM 14

The influence of the construction stops from 17.11.2013 to 05.12.2013 and from 13.12.2013 to

06.01.2014 can't be evaluated because too few measurements took place during these stops. The slight uplift at the end of the monitoring is most likely measurement noise and therefore is not considered. In Table 5.4 measured settlements are compared to design settlements and to the defined alarm level. Measured settlements are more than twice the design settlements and nearly reached the alarm level. SOFISTIK results underestimate settlements but total settlements are in a proper range anyway.

Table 5.4: Design and Measured Settlements and Alarm Level at TM 15

Settlements [mm]		Difference [mm]	Alarm Level [mm]
design	measured		
3.83	10	6.17	10

The settlement trough is defined by longitudinal and transversal settlement profiles. The transversal one is defined by measurement points at the measurement section. Figure 5.6 shows the transversal settlement trough at TM 14. The trough itself is very shallow and flat. The maximum settlements of 10 mm occur above the tunnel crown at point 13-000014-01.

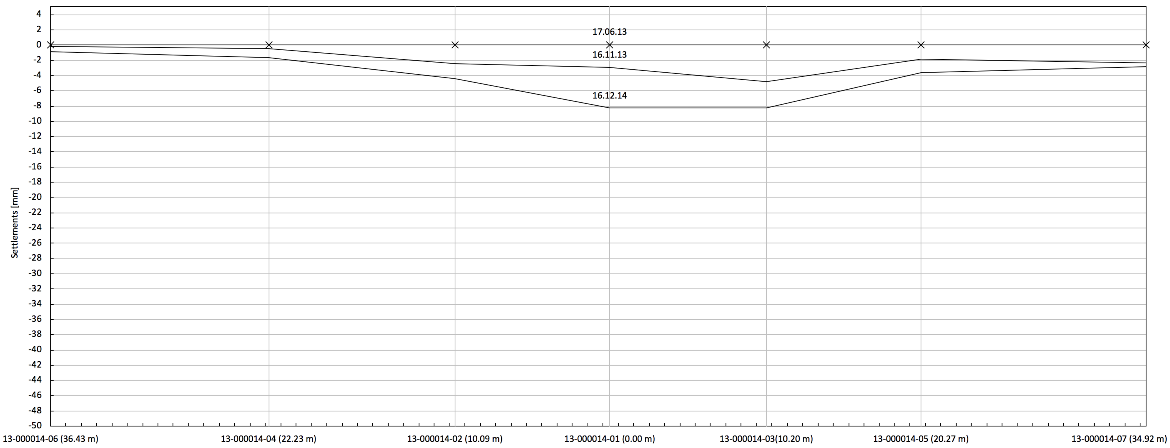

Figure 5.6: Transversal Settlement Trough at Measurement Section TM 14

5.1.6 Comparative Settlements due to GWLL

The groundwater level is 14 m below the surface and below the invert of the EAT at the middle gypsum horizon. GWLL started after the first monitoring period stopped. The influence of GWLL can't be evaluated at measurement section TM 14 and therefore can't be evaluated at DS 04 at all. The influence of local GWLL at tunnel 801 FP and 802 FP would be neglected at DS 04 anyway. This assumption is based on the distance between DS 04 and the GWLL pumps at the tunnel faces of 801 and 802 with a minimum of 150 m and uncertainties about groundwater bodies.

5.1.7 Measured Displacements

At DS 04 a direct relation between displacements at the tunnel crown and settlements is not possible because the EAT is built inside the existing Wagenburgtunnel North. The measured displacements most likely took place due to the refill of the open space between the new lining of the EAT and the Wagenburgtunnel North.

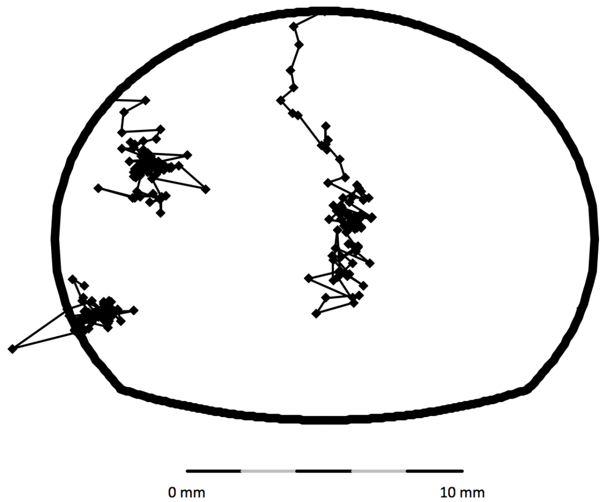

Figure 5.7: Displacements at Measurement Section TM 13

Table 5.5: Measured Settlements at TM 14 and Displacements at TM 13

Settlements [mm]	Displacements [mm]	Difference [mm]
10	13	-3

5.2 Design Section 03 [DS 03]

Design simulations are performed at cross-section TM 49.5. The results at DS 03 are valid from TM 15 to TM 45. The EAT is underpassing the Wagenburgtunnel North along this section. Since the Wagenburgtunnel North has no invert, just foundations, the tunnels don't intersect any longer. The EAT is constructed with a ten degree slope in this section and full-face excavated. Tube spiles are used as pre-support over the whole length of this section.

5.2.1 Geological Conditions

DS 03 is characterized by the geological layers talus deposit and filling material and middle gypsum horizon. The gypsum keuper of the middle gypsum horizon is highly leached and assigned to class II to III. Deeper layers of middle gypsum horizon are also leached but reach higher classes III to IV. Layer thicknesses at cross-section TM 50 are listed in Table 5.6.

Table 5.6: Layer Thickness at Design Cross-Section TM 50

Layer	Thickness [m]
talus deposit and filling material	18
middle gypsum horizon	19.6 (12 + 7.6)

According to the geological conditions the tunnel below the Wagenburgtunnel North is partly-situated in the layer talus deposit and filling material and mainly in middle gypsum horizon. The GWL is 245 m above NN from TM 15 to TM 30 and 248 m above NN from TM 30 to TM 45. The tunnel position is shown in Figure 5.8.

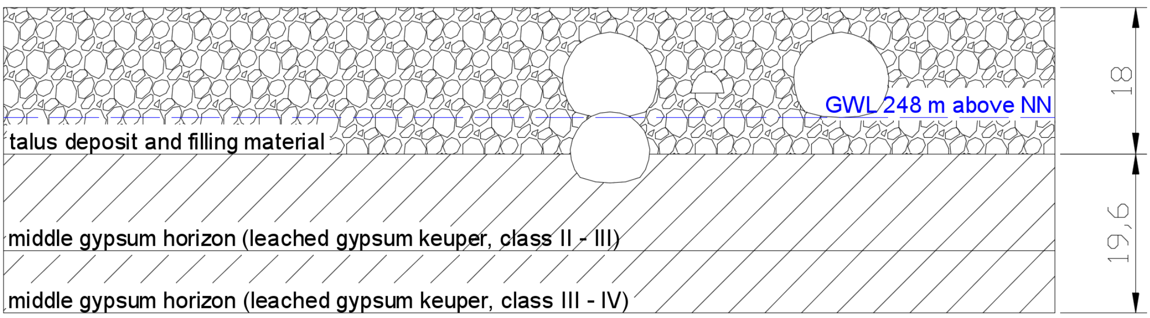

Figure 5.8: Design Cross-Section TM 50

5.2.2 Design Settlements

The excavation is modelled in SOFISTIK as full-face excavation only. Pre-Support (tube spiles) is not taken into account. The numerical analysis were carried out without distinguishing between young or hardened shotcrete material parameters. The used young's modulus is the conservative design value of 5000 MPa.

Table 5.7: Design Settlements according to SOFISTIK at DS 03 [TM 49.5] [PGS 21 (I) (2013)]

FEM Modelling Step	Maximum Design Settlements [mm]	
	characteristic RMP	conservative RMP
	DS 3-1	DS 3-2
softening (α = 0.25)	0.00	0.00
full-face excavation (E = 5000 MPa)	1.46	5.77

5.2.3 Predicted Settlements after Fillibeck (2012)

Like at DS 04 the empirical prediction method after Fillibeck is not usable at DS 03 due to intersection of the EAT with the Wagenburgtunnel North.

5.2.4 Warning and Alarm Level

According to report PGS 21 (III) (2013) warning levels are defined as the maximum settlement values of the numerical analysis with conservative RMP.

Table 5.8: Warning and Alarm Levels at DS 03 [PGS 21 (III) (2013)]

	TM 15 - TM 45
FEM result	<15 mm
pre-warning level	15 mm
warning level	20 mm
alarm level	not defined

5.2.5 Measured Settlements

At DS 03 measurement section TM 30 shows the highest settlements. Measurement section TM 50 shows a similar settlement behaviour with very similar values. The design section is at TM 49.5 and therefore measurement section TM 50 with measurement points 13-000050-01 to 13-000050-06 is evaluated. The settlement development is shown in Figure 5.9.

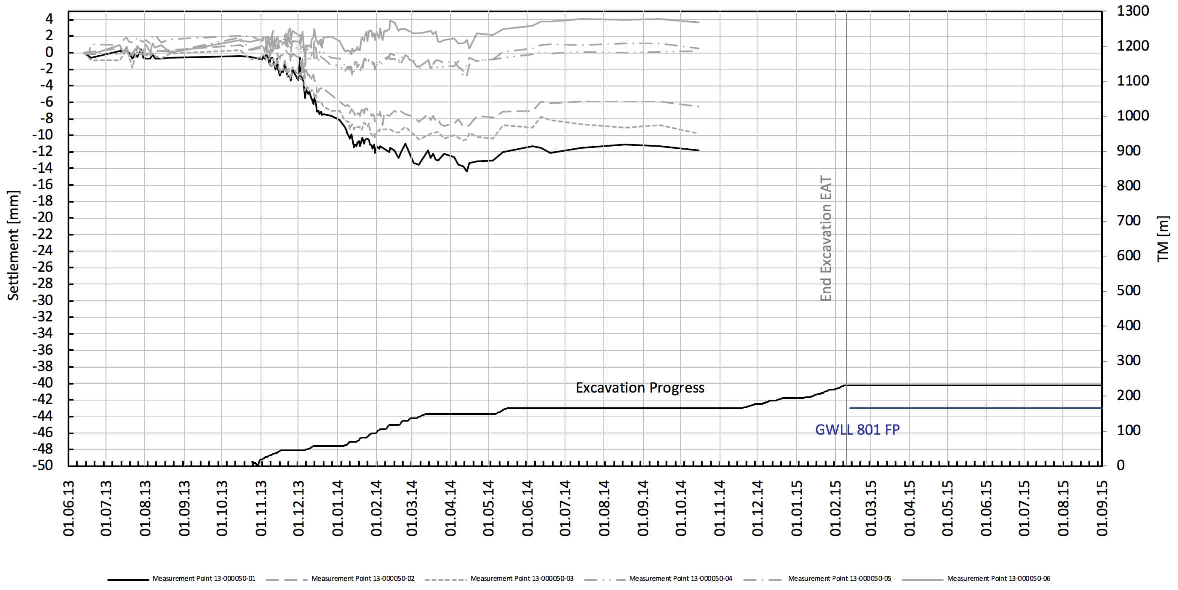

Figure 5.9: Settlements and Excavation Progress at Measurement Section TM 50

Measurement section TM 50 is part of the first monitoring period which started on 13.06.2013. After the construction started on 25.10.2013, settlements developed continuously till the excavation reached TM 150 (13.03.2014). Further excavation didn't influence the settlements anymore and they remained constant around 12 mm. GWLL started after the first monitoring period stopped.

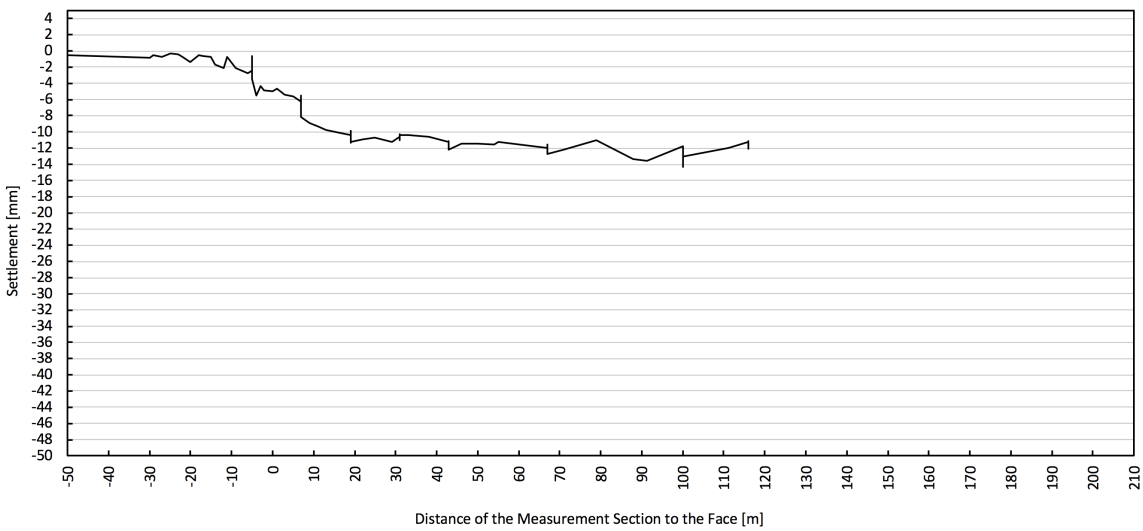

Figure 5.10: Settlements at Measurement Section TM 50

Even at the construction stops from 17.11.2013 to 05.12.2013 and from 13.12.2013 to 06.01.2014 settlement developed continuously (vertical lines). In Figure 5.10 the first stop can be seen at -5 m distance to the face and the second at 7 m distance to the face. Some more shorter stops with an settlement increase during the construction stop can be seen in Figure 5.10. In Table 5.9 measured settlements are compared to design settlements and to the defined warning level. The measured settlements are more than twice the design settlements.

Table 5.9: Design and Measured Settlements and Warning Level at TM 50

Settlements [mm]		Difference [mm]	Warning Level [mm]
design	measured		
5.77	14	8.23	20

The transversal settlement trough is defined by measurement points at the measurement section. Figure 5.11 shows the transversal settlement trough at TM 50. The maximum settlements of 14 mm occur above the tunnel crown at point 13-000050-01.

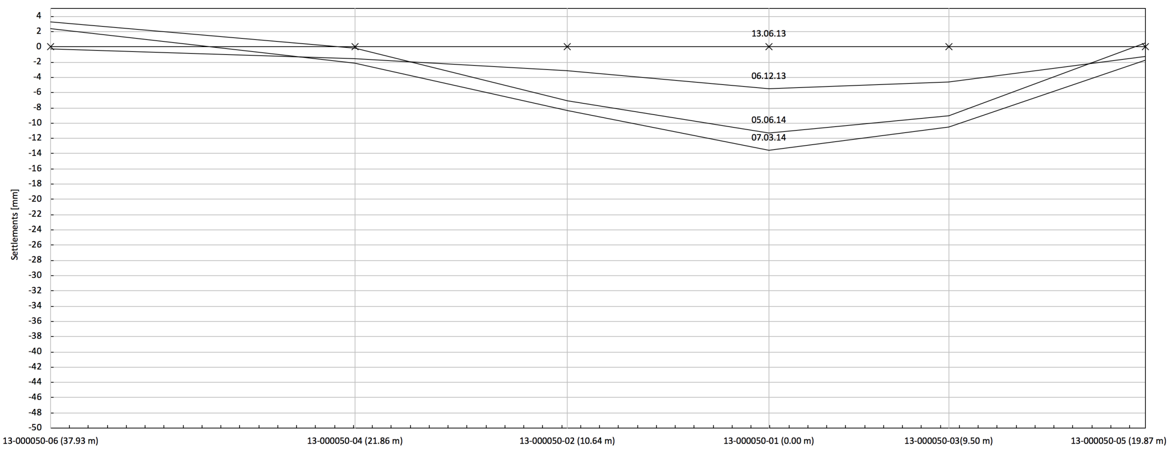

Figure 5.11: Transversal Settlement Trough at Measurement Section TM 50

5.2.6 Comparative Settlements due to GWLL

The groundwater level is 13.5 m below the surface and at the talus deposit layer and filling material. GWLL started after the first monitoring period stopped. The influence of GWLL can't be evaluated for measurement section TM 50 and also can't be evaluated for DS 03 at all. Perhaps settlements at DS 03 increased after the start of the GWLL.

5.2.7 Measured Displacements

Displacements at the tunnel crown are a good indicator for settlements if the overburden is very low. At DS 03 the maximum overburden is approximately 10 m. At measurement section TM 50 displacements are 9 mm at the crown and settlements are 14 mm. Around 30 % of the displacements already take place ahead of the face and are therefore not measurable. In addition the zero measurement takes place delayed after the excavation. Therefore total displacements of the ground will be higher than measured. If we assume the measured displacements are around 65 % total displacements will be around 14 mm. The estimated total displacements indicate that the surface settlements are approximately equal the vertical crown displacements.

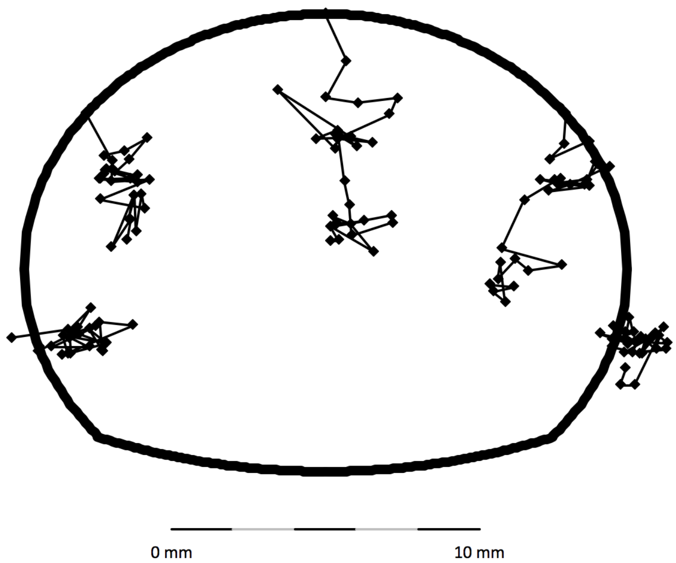

Figure 5.12: Displacements at Measurement Section TM 50

Table 5.10: Measured Settlements and Displacements at TM 50

Settlements [mm]	Displacements [mm]	Difference [mm]
14	9	5

5.3 Design Section 02 [DS 02]

Design simulations are performed at cross-section TM 95. The results at DS 02 are valid from TM 45 to TM 90. The EAT doesn't intersect with the Wagenburgtunnel North. The excavation is done as two-step excavation and the EAT is constructed with a ten degree slope in this section. Pipe umbrellas are used as pre-support over the whole length of DS 02.

5.3.1 Geological Conditions

DS 02 is characterized by the geological layers talus deposit and filling material, middle gypsum horizon and deep gysum layers. The gypsum keuper of the middle gypsum horizon is highly leached and assigned to class II to III. Deeper layers of middle gypsum horizon are also leached but reaches higher classes III to IV. Layer thicknesses at cross-section TM 95 are listed in Table 5.11.

Table 5.11: Layer Thickness at Design Cross-Section TM 95

Layer	Thickness [m]
talus deposit and filling material	23.5
middle gypsum horizon	33.5 (17 + 16.5)
deep gysum layers	6

According to the geological conditions the tunnel cross-section is situated in the layer middle gypsum horizon over the whole length of the this section. The GWL is 248 m above NN. The tunnel position is shown in Figure 5.13.

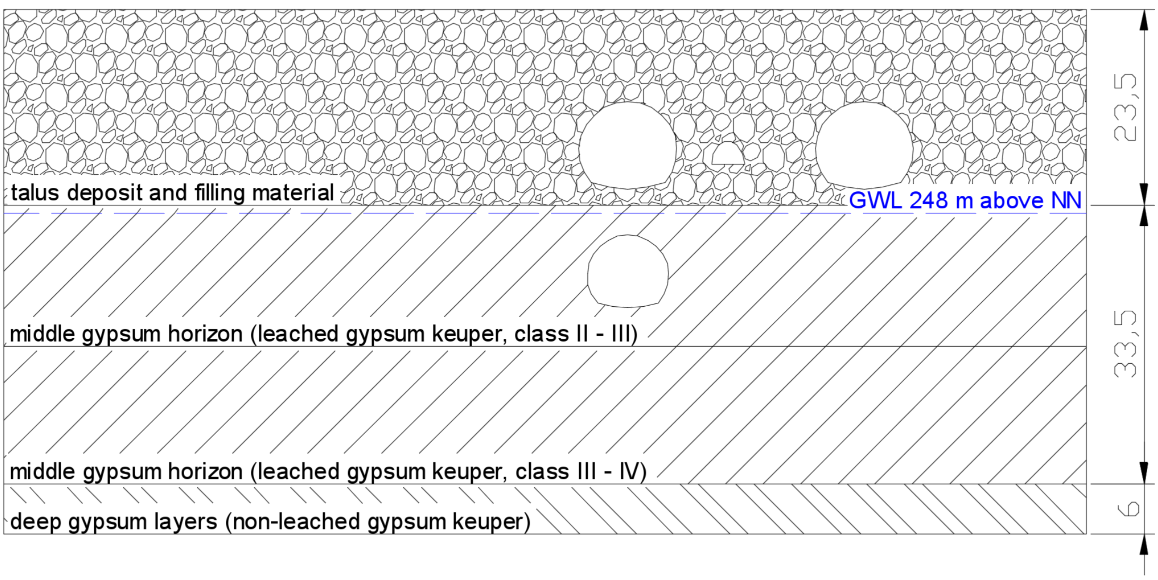

Figure 5.13: Design Cross-Section TM 95

5.3.2 Design Settlements

The excavation is modelled in SOFISTIK twice. First as full-face excavation [DS 02-1 and DS 02-2] and second as two-step excavation [DS 02-3 and DS 02-4]. Pre-support (pipe umbrellas) is not taken into account. The numerical analyses were carried out without distinguishing between young or hardened shotcrete material parameters. The used young's modulus is the conservative design value of 5000 MPa.

Table 5.12: Surface Settlements according to SOFISTIK at DS 02 [TM 95] [PGS 21 (I) (2013)]

FEM Modelling Step	Maximum Surface Settlements [mm]	
	characteristic RMP	conservative RMP
	DS 02-1	DS 02-2
softening ($\alpha = 0.5$)	2.82	7.84
full-face excavation (E = 5000 MPa)	6.24	8.92
	DS 02-3	DS 02-4
softening top heading ($\alpha = 0.5$)	1.98	4.38
excavation top heading (E = 5000 MPa)	10.05	17.26
softening invert ($\alpha = 0.5$)	11.11	2.17
excavation invert (E = 5000 MPa)	12.15	0.64

5.3.3 Predicted Settlements after Fillibeck (2012)

Fundamental conditions for the empirical prediction method after Fillibeck are horizontal excavation of tunnels in soil or soil-like ground. Longitudinal and transversal settlement troughs are calculated independent of each other for each tunnel excavation and super-imposed afterwards. The method is not applicable for excavations of pillar galleries. The excavations at MS South do not fulfil all requirements but it is used for rough comparative reasons. The required parameters overburden, constrained modulus, cross-section area and assumed point of inflection are listed in Table 5.13.

Table 5.13: Input Parameters of Empirical Method after Fillibeck (2012)

section	overburden [m]	$E_{100,ref}$ [MPa]	A_t [m^2]	i	$VL_{s,50\%}$	$VL_{s,90\%}$	$VL_{s,99\%}$
DS 02	25	200	78.5	0.5	-	0.44	0.51

Table 5.14: Predicted Settlements after Fillibeck (2012)

$s_{max,50\%}$ [mm]	$s_{max,90\%}$ [mm]	$s_{max,99\%}$ [mm]
-	19	22

5.3.4 Warning and Alarm Level

According to report PGS 21 (III) (2013) warning levels are defined as the mean value of the maximum design settlements caused by top heading excavation with an immediate closure of the lining and a stepped excavation with conservative RMP.

Table 5.15: Warning and Alarm Levels at DS 02 [PGS 21 (III) (2013)]

	TM 45 - TM 90
FEM result	<15 mm
pre-warning level	15 mm
warning level	20 mm
alarm level	not defined

5.3.5 Measured Settlements

At DS 02 measurement section TM 80 shows the highest settlements. The section is defined by one measurement point 13-000080-01 and the settlement development is shown in Figure 5.14.

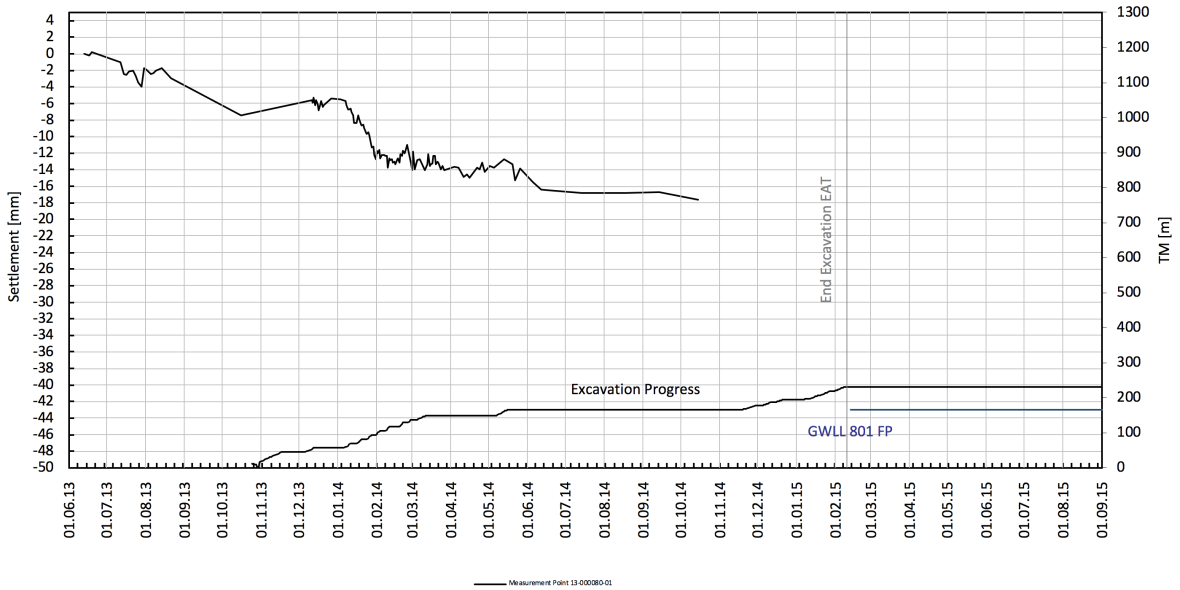

Figure 5.14: Settlements and Excavation Progress at Measurement Section TM 80

Measurement section TM 80 is part of the first monitoring period which started on 13.06.2013. After the construction started on 25.10.2013 settlements developed continuously till 01.07.2014. Further excavations didn't influenced the settlements anymore and they stagnated around 18 mm. GWLL started after the first monitoring period stopped.

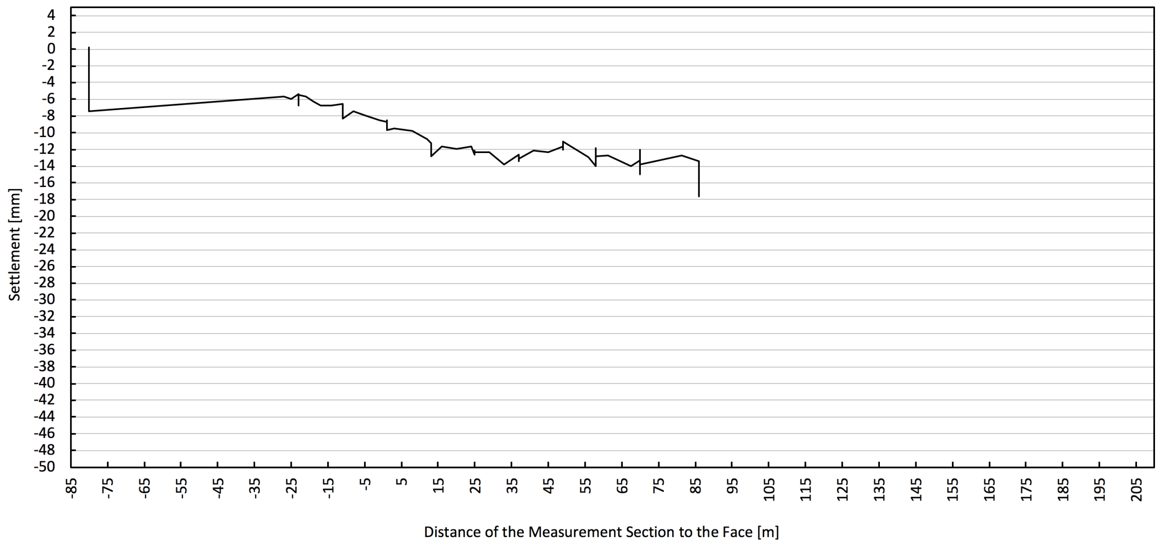

Figure 5.15: Settlements at Measurement Section TM 80

During all construction stops settlements developed continuously (vertical lines). Even during the long construction stop from 16.05.2014 to 18.11.2014 settlements increased from 13 mm to 18 mm. In Table 5.16 measured settlements are compared to design and predicted settlements and to the defined warning level. Design and predicted settlements are more or less the same as measured settlements. The expected behaviour matches the real settlement behaviour.

Table 5.16: Design, Predicted and Measured Settlements and Warning Level at TM 80

Settlements [mm]			Difference [mm]		Warning Level [mm]
design	predicted	measured	to design	to predicted	
17.26	19	18	0.74	-1	20

5.3.6 Comparative Settlements due to GWLL

The groundwater level is 24.5 m below the surface and at the middle gypsum horizon. GWLL started after the first monitoring period stopped. The influence of GWLL can't be evaluated for measurement section TM 80 and also can't be evaluated for DS 02 at all. Perhaps settlements at DS 02 increased after the start of the GWLL.

5.3.7 Measured Displacements

At DS 02 the maximum overburden is approximately 24 m. At measurement section TM 80 displacements are 3 mm at the crown and settlements are 18 mm. Even if pre-relaxation and delayed zero measurement are assumed, displacements do not correlate with settlements.

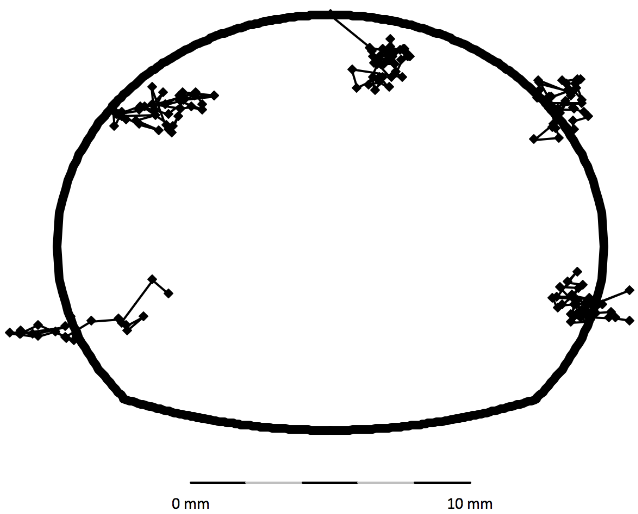

Figure 5.16: Displacements at Measurement Section TM 80

Table 5.17: Measured Settlements and Displacements at TM 80

Settlements [mm]	Displacements [mm]	Difference [mm]
18	3	15

5.4 Design Section 01 [DS 01]

Design simulations are performed at cross-section TM 150. The results at DS 01 are valid from TM 90 to TM 150. The excavation is done as two-step excavation and the EAT is partly constructed with a ten degree slope in this section. Pipe umbrellas are used as pre-support over the whole length of DS 01 to reduce displacements and settlements.

5.4.1 Geological Conditions

DS 01 is characterized by the geological layers talus deposit and filling material, middle gypsum horizon, blue-lead layers, dark red marl and deep gysum layers. As described earlier the layers blue-lead layers and dark red marl have similar geological and geotechnical properties as the middle gypsum horizon and therefore they are treated as part of middle gypsum horizon. The gypsum keuper of the middle gypsum horizon is highly leached and assigned to class II to III. Deeper layers of middle gypsum horizon are also leached gypsum keuper but reaches higher classes III to IV. Layer thicknesses at cross-section TM 150 are listed in Table 5.18.

Table 5.18: Layer Thickness at Design Cross-Section TM 150

Layer	Thickness [m]
talus deposit and filling material	23.5
middle gypsum horizon	33.5 (17 + 16.5)
deep gysum layers	6

The overburden at design cross-section TM 150 is approximately 45 m. According to the geological conditions the tunnel cross-section is fully-situated at the layer middle gypsum horizon. The tunnel position is shown in Figure 5.17.

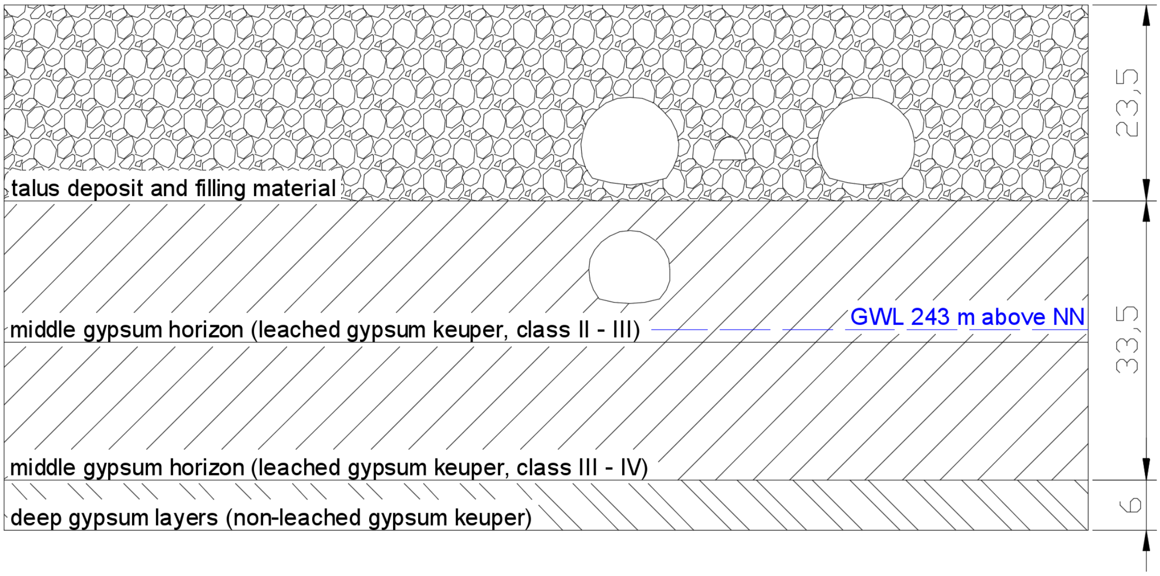

Figure 5.17: Design Cross-Section TM 150

5.4.2 Design Settlements

The excavation is modelled in SOFISTIK twice. First as full-face excavation [DS 01-1, DS 01-2, DS 01-5 and DS 01-6] and second as two-step excavation [DS 01-3 and DS 01-4]. Pre-support (pipe umbrellas) is not taken into account. The numerical analyses were carried out without distinguishing between young or hardened shotcrete material parameters. The used young's modulus is the conservative design value of 5000 MPa.

Table 5.19: Surface Settlements according to SOFISTIK at DS 01 [TM 150] [PGS 21 (I) (2013)]

FEM Modelling Step	Maximum Surface Settlements [mm]	
	characteristic RMP	conservative RMP
	DS 01-1	DS 01-2
softening (α = 0.5)	3.00	6.00
full-face excavation (E = 5000 MPa)	8.00	14.00
	DS 01-3	DS 01-4
softening top heading (α = 0.5)	3.00	6.00
excavation top heading (E = 5000 MPa)	9.00	15.00
softening invert (α = 0.5)	10.00	17.00
excavation invert (E = 5000 MPa)	11.00	18.00
	DS 01-5	DS 01-6
softening (α = 0.25)	1.00	3.00
full-face excavation (E = 5000 MPa)	3.00	5.00

5.4.3 Predicted Settlements after Fillibeck (2012)

Like at DS 02 the method is used at DS 01 for rough comparative reasons. The required parameters overburden, constrained modulus, cross-section area and assumed point of inflection are listed in Table 5.20.

Table 5.20: Input Parameters of Empirical Method after Fillibeck (2012)

section	overburden [m]	$E_{100,ref}$ [MPa]	A_t [m^2]	i	$VL_{s,50\%}$	$VL_{s,90\%}$	$VL_{s,99\%}$
DS 01	45	200	78.5	0.5	-	0.60	0.67

Table 5.21: Predicted Settlements after Fillibeck (2012)

$s_{max,50\%}$ [mm]	$s_{max,90\%}$ [mm]	$s_{max,99\%}$ [mm]
-	26	29

5.4.4 Warning and Alarm Level

According to report PGS 21 (III) (2013) warning levels are defined as the mean value of the maximum settlements caused by top heading excavation with an immediate closure of the lining and a stepped excavation with conservative RMP.

Table 5.22: Warning and Alarm Levels at DS 01 [PGS 21 (III) (2013)]

	TM 90 - TM 150
FEM result	<15 mm
pre-warning level	15 mm
warning level	20 mm
alarm level	not defined

5.4.5 Measured Settlements

At DS 01 measurement section TM 150 shows the highest settlements. The section is defined by one measurement point 13-000150-01 and the settlement development is shown in Figure 5.18.

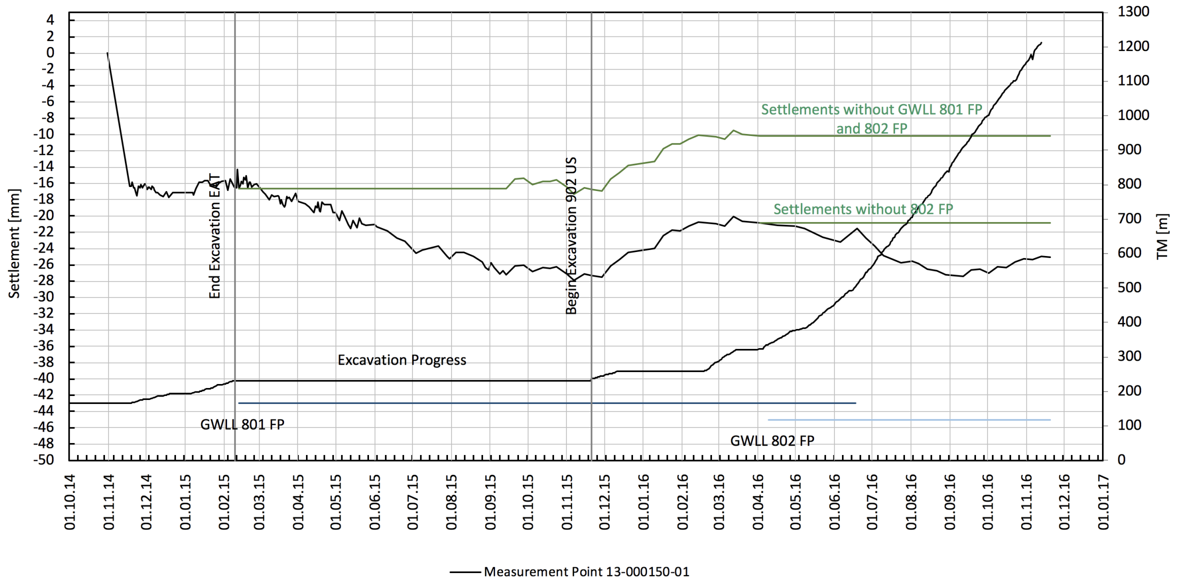

Figure 5.18: Settlements and Excavation Progress at Measurement Section TM 150

Measurement section TM 150 is part of the second monitoring period which started on 31.10.2014. At this date the excavation reached TM 166. The surface measurement section should have been installed 50 m ahead of the progressing face. The excavation at TM 100 was done on 02.02.2014. The second measurement period started 283 days too late. Therefore settlements are assumed to be much higher than the measured ones.

The first huge increase of 17 mm settlements took place in between the zero measurement on 31.10.2014 and the second measurement on 18.11.2014. The linear trend between the two measurement dates as shown in Figure 5.18 is definitely not linear in the reality. Afterwards settlements remained constant till GWLL started on 13.02.2015. Simultaneously to GWLL settlements increased in a very linear mode without ongoing excavation till early November 2015. At this time settlements reached the maximum of 28 mm.The GWLL continued, but after the beginning of November 2015 an uplift tendency can be seen in Figure 5.18. The GWLL at tunnel 801 FP was reduced, no excavation took place and GWLL at tunnel 802 FP was installed and started on 10.04.2016. After GWLL 802 FP started the uplift trend changed to a linear increase of settlements till approximately 25 mm. The settlement development without GWLL is shown as green line. Once without GWLL at all and once only without GWLL at tunnel face 802 FP. It is not for sure if the uplift would have also taken place without GWLL. Settlements due to GWLL would therefore be around 15 mm.

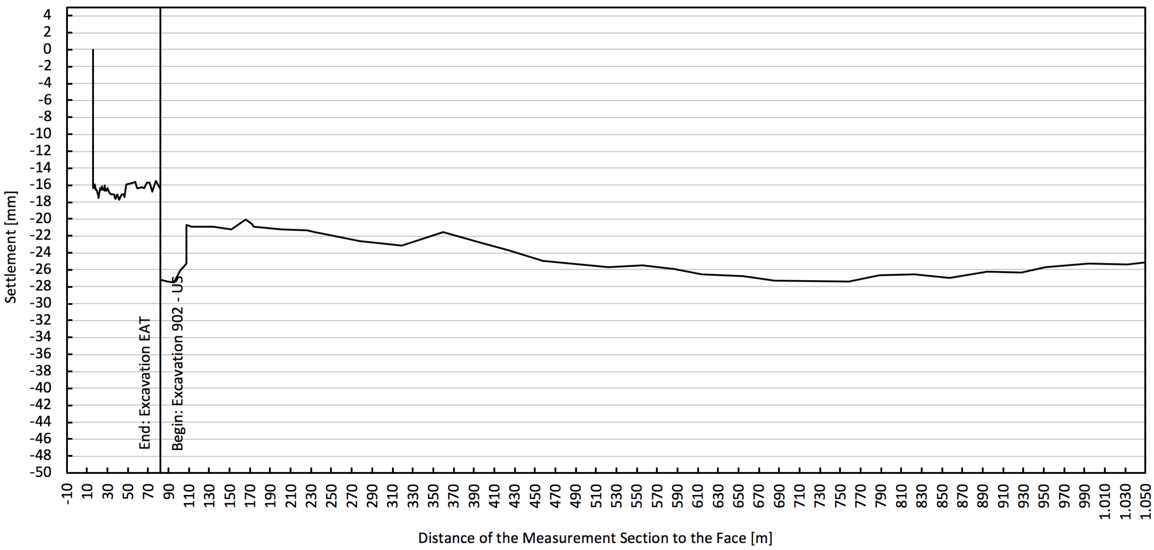

Figure 5.19: Settlements at Measurement Section TM 150

In Figure 5.19 the settlement increase during the construction stop from 09.02.2015 to 20.11.2015 can be clearly seen as vertical line at 81.6 m distance to the face. In Table 5.23 measured settlements are compared to design and predicted settlements and to the defined warning level. Predicted settlements are more or less the same as measured settlements. The design underestimated settlements because no GWLL was modelled with SOFISTIK. The predicted behaviour after Fillibeck matches the real settlement behaviour pretty well.

Table 5.23: Design, Predicted and Measured Settlements and Warning Level at TM 150

Settlements [mm]			Difference [mm]		Warning Level [mm]
design	predicted	measured	to design	to predicted	
18	26	28	10	2	20

Figure 5.20 shows the transversal settlement trough with maximum settlements of 28 mm above the crown at measurement point 13-000149-01 at TM 149.5.

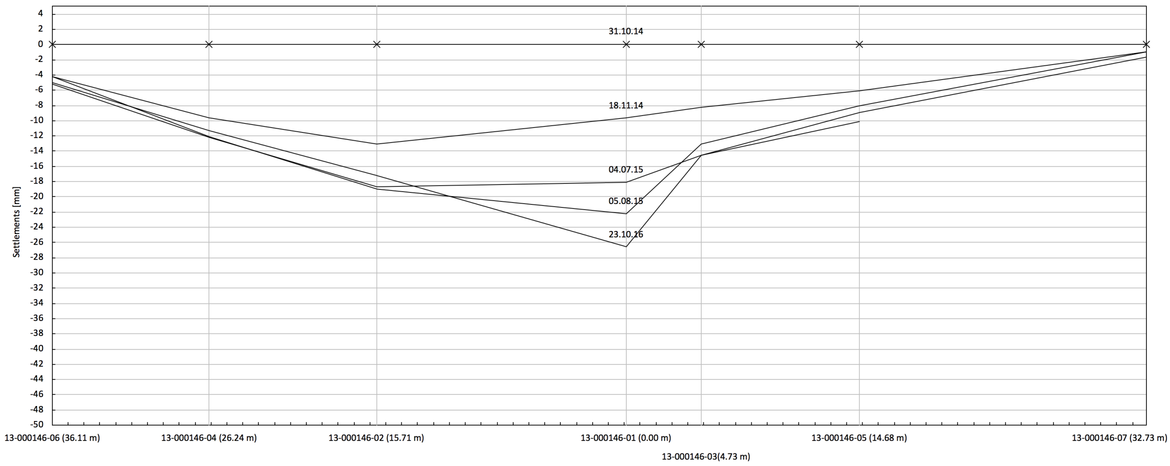

Figure 5.20: Transversal Settlement Trough at Measurement Section TM 149.5

5.4.6 Comparative Settlements

The numerical analysis in SOFISTIK do not take GWLL into account. Due to additional stresses in the ground, caused by GWLL at tunnel 801 FP and 802 FP the measured settlements are not directly comparable to the design settlements. The settlements caused by GWLL, named comparative settlements, are calculated after the theory of additional stresses in the ground caused by GWLL. The formula therefore is:

$$s = \sum_{i=1}^{i=depth} \delta\sigma_{mean,i}/E_S * (x_i - x_{i+1}) \tag{5.1}$$

s	.....	settlements due to GWLL
σ_{mean}	.....	additional stress at specific layer
E_S	.....	constrained modulus
x	.....	depth

The constrained modulus depends on the stress range at the specific depth and layer. It is assumed that initial stresses already took place and stagnate. Therefore the constrained modulus for reloading is used. The calculations are done for GWLL of 1 m, 2 m, 5 m and 10 m and listed in Table 5.24.

Table 5.24: Comparative Settlements due to GWLL at TM 150

Layer	Thickness [m]	max. eff. Stress [MPa]	Settlements [mm]
GWLL - 1 m			
talus deposit and filling material	23.5	235	0
middle gypsum horizon	33.5	467	6
deep gysum layers	6	563	1
		Total:	7
GWLL - 2 m			
talus deposit and filling material	23.5	235	0
middle gypsum horizon	33.5	477	11
deep gysum layers	6	573	2
		Total:	13
GWLL - 5 m			
talus deposit and filling material	23.5	235	0
middle gypsum horizon	33.5	507	26
deep gysum layers	6	603	5
		Total:	31
GWLL - 10 m			
talus deposit and filling material	23.5	235	0
middle gypsum horizon	33.5	557	43
deep gysum layers	6	653	10
		Total:	53

GWLL definitely caused settlements from 13.02.2015 to early November 2015 and from 10.04.2016 to mid of September 2016. In the first period settlements are approximately 11 mm and in the second approximately 4 mm. This are 14 mm in total which are caused by GWLL. This would correspond to GWLL of 2 m which is a plausible and verifiable value at the construction site. In Table 5.25 design, measured and comparative settlements are listed.

Table 5.25: Design, Measured and Comparative Settlements at TM 150

Settlements [mm]			Difference [mm]
design	measured (max.)	comparative (2 m GWLL)	
18	28	13	15

5.4.7 Measured Displacements

At DS 01 the maximum overburden is approximately 45 m. At measurement section TM 150 displacements are 1 mm at the crown and settlements are 28 mm. Even if pre-relaxation and delayed zero measurement are assumed displacements do not correlate with settlements.

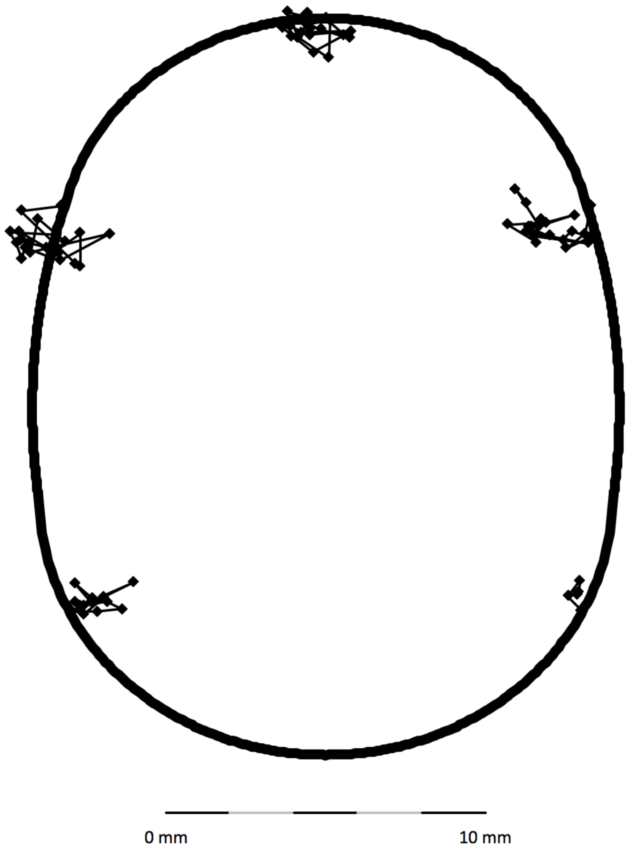

Figure 5.21: Displacements at Measurement Section TM 150

Table 5.26: Measured Settlements and Displacements at TM 150

Settlements [mm]	Displacements [mm]	Difference [mm]
28	1	27

5.5 Design Section 05 [DS 05]

Design simulations are performed at cross-section TM 170. The results at DS 05 are valid from TM 150 to TM 166. This part of the EAT is horizontal. The intersections with pillar gallery east MS and FP, tunnels 901 MS and IH US and tunnels 801 MS and FP take place in between TM 150 and TM 166. The excavation is done by excavators and pipe umbrellas are used as pre-support to reduce displacements and settlements.

5.5.1 Geological Conditions

DS 05 is characterized by the geological layers talus deposit and filling material, middle gypsum horizon, blue-lead layers, dark red marl and deep gysum layers. Blue-lead layers and dark red marl have similar geological and geotechnical properties as middle gypsum horizon and therefore they are treated as part of middle gypsum horizon. The gypsum keuper of the middle gypsum horizon is highly leached and assigned to class II to III. Deeper layers of middle gypsum horizon are also leached but reaches higher classes III to IV. Layer thicknesses at cross-section TM 170 are listed in Table 5.27.

Table 5.27: Layer Thickness at Design Cross-Section TM 170

Layer	Thickness [m]
talus deposit and filling material	7.4
middle gypsum horizon	61.9
deep gysum layers	17.3

Design cross-section TM 170 is the section of the EAT from TM 0 to TM 166 with maximum overburden of approximately 50 m. According to the geological conditions the tunnel cross-section is fully-situated at the middle gypsum horizon with highly leached gypsum keuper. The tunnel position is shown in Figure 5.22.

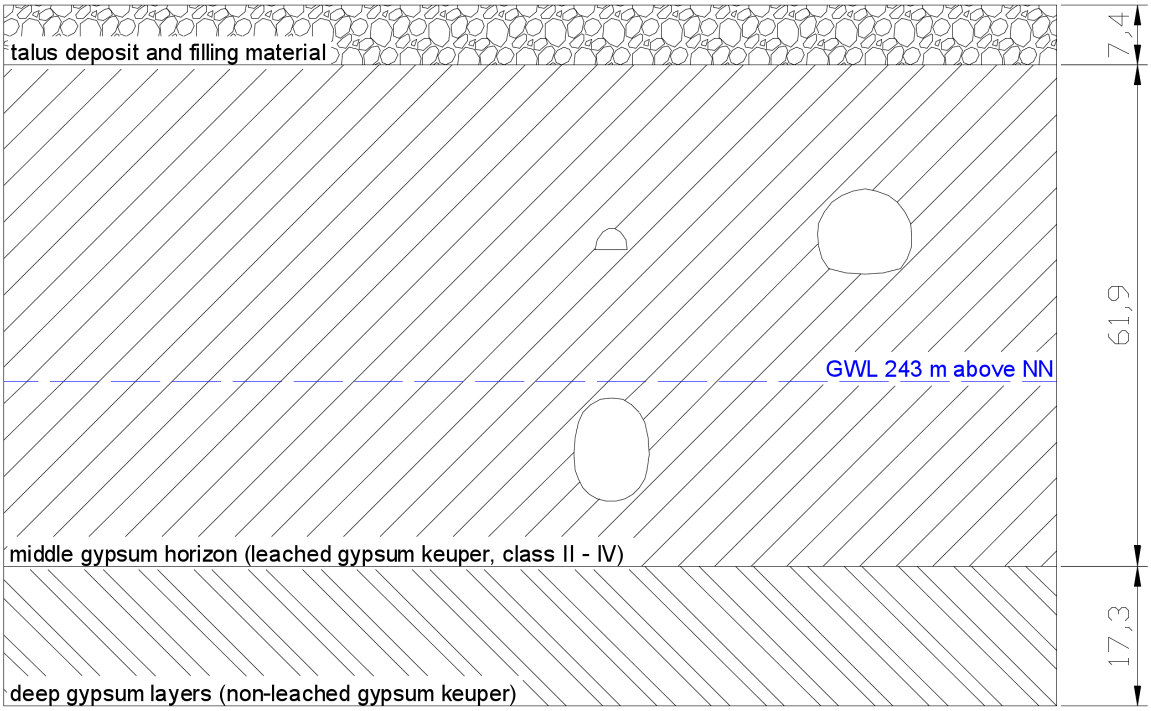

Figure 5.22: Design Cross-Section TM 170

5.5.2 Design Settlements

The excavation is modelled in SOFISTIK as two-step excavation only. Pre-support (pipe umbrellas) is not taken into account. The numerical analyses were carried out without distinguishing between young or hardened shotcrete material parameters. The used young's modulus is the conservative design value of 5000 MPa.

Table 5.28: Surface Settlements according to SOFISTIK at DS 05 [TM 170] [PGS 21 (II) (2014)]

FEM Modelling Step	Maximum Surface Settlements [mm] characteristic RMP	conservative RMP
	DS 05-1	DS 05-2
softening top heading ($\alpha = 0.5$)	3.00	6.00
excavation top heading (E = 5000 MPa)	8.00	7.00
softening invert ($\alpha = 0.5$)	9.00	10.00
excavation invert (E = 5000 MPa)	13.00	13.00
	DS 05-3	DS 05-4
excavation top heading (E = 5000 MPa)	6.00	5.00
softening invert ($\alpha = 0.5$)	7.00	8.00
excavation invert (E = 5000 MPa)	9.00	8.00

5.5.3 Predicted Settlements after Fillibeck (2012)

The method is used at DS 05 for rough comparative reasons. The required parameters overburden, constrained modulus, cross-section area and assumed point of inflection are listed in Table 5.29.

Table 5.29: Input Parameters of Empirical Method after Fillibeck (2012)

section	overburden [m]	$E_{100,ref}$ [MPa]	A_t [m^2]	i	$VL_{s,50\%}$	$VL_{s,90\%}$	$VL_{s,99\%}$
DS 05	50	150	161.5	0.35	-	1.48	1.55

Table 5.30: Predicted Settlements after Fillibeck (2012)

$s_{max,50\%}$ [mm]	$s_{max,90\%}$ [mm]	$s_{max,99\%}$ [mm]
-	92	102

5.5.4 Warning and Alarm Level

According to report PGS 21 (III) (2013) warning levels are defined as the mean value of the maximum settlements caused by top heading excavation with an immediate closure of the lining and a stepped excavation with conservative RMP.

Table 5.31: Warning and Alarm Levels at DS 05 [PGS 21 (III) (2013)]

	TM 150 - TM 166
FEM result	<10 mm
pre-warning level	10 mm
warning level	15 mm
alarm level	not defined

5.5.5 Measured Settlements

At DS 05 measurement section TM 165 shows the highest settlements. Measurement section TM 170 shows a very similar settlement behaviour with very similar values. The design section is TM 170. Although measurement section TM 170 with the measurement points 13-000170-01 to 13-000170-06 is not part of DS 05 it is evaluated in regards to comparability of design and measured settlements. The settlement development is shown in Figure 5.23.

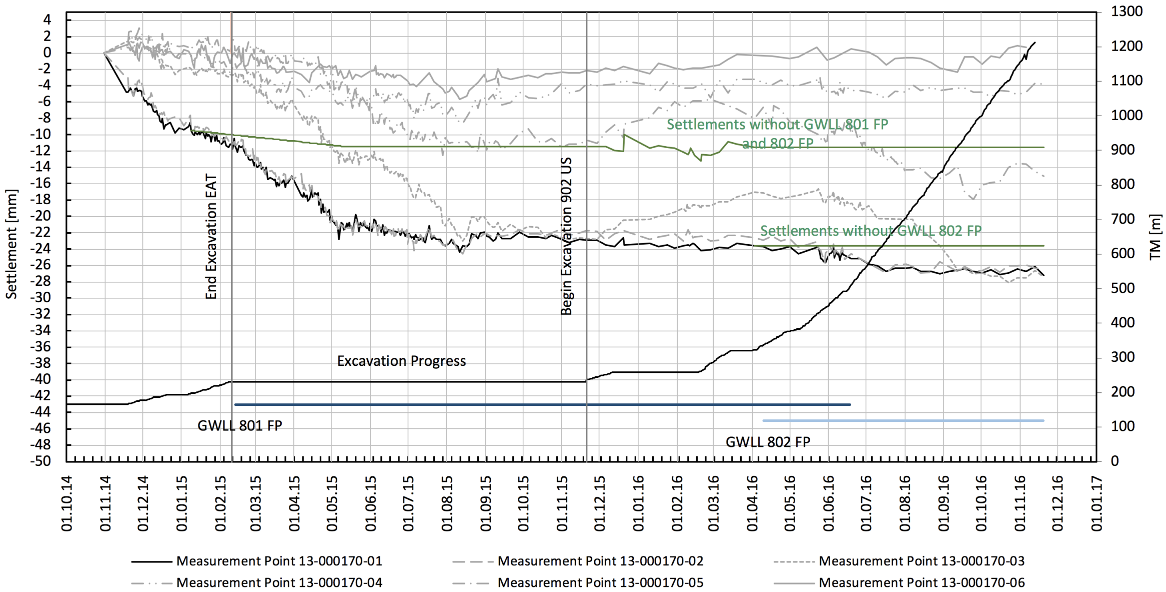

Figure 5.23: Settlements and Excavation Progress at Measurement Section TM 170

Measurement section TM 170 is part of the second monitoring period which started on 31.10.2014. As measurement section TM 150, the measurement section TM 170 was installed too late. The second measurement period started 253 days too late. Therefore settlements are assumed to be much higher than the measured ones.

The settlement developments at the monitoring points at the section are pairwise similar. Measurement points 13-000170-01 and 13-000470-02 show the highest settlements with similar behaviour. Settlements at measurement points 13-000170-03 and 13-000470-05 do the same and 13-000170-04 and 13-000170-06 too.

At measurement points 13-000170-01 and 13-000170-02 settlements linearly increased to 10 mm, shortly remained constant and linearly increased again afterwards, also during the construction stop, to approximately 23 mm. Afterwards settlements stagnated till the end of the construction stop on 21.11.2015 and increased very slowly to 27 mm after excavations started at tunnel 902 IH US till 10.04.2016. The impact of GWLL at the tunnel face 801 FP can be seen from the beginning on 13.02.2015 till mid of August 2015. During this period no excavations were done but settlements steadily increased. Out of 23 mm, 10 mm are related to GWLL. GWLL at tunnel face 802 FP had a far smaller impact on the behaviour at these two measurement points. This impact can be seen as increase from 24 mm to 28 mm from 10.04.2016 till mid of July 2016. The settlement development without GWLL is shown as green lines. After the excavation of EAT ended settlements stagnated at around 10 mm and additional settlements are related to GWLL as seen in Figure 5.23.

At measurement points 13-000170-03 and 13-000470-05 settlements increased slowly to 5 mm first. After GWLL began at tunnel face 801 FP a fast and steady increase of settlements to 22 mm at 13-000170-003 and 11 mm at 13-000170-05 due to GWLL took place. Till mid of Dezember 2015 settlements stagnated and afterwards an uplift trend developed at both measurement points. The uplift started after the construction stop ended and the excavation of tunnel 902 IH US began. Settlements decreased by around 5 mm during the uplift. When GWLL started at 802 FP the ground behaviour changed and settlements increased again. At both measurement points the impact of GWLL can be seen from 10.04.2016 to mid of September 2016 with approximately 10 mm.

Measurement points 13-000170-04 and 13-000170-06 are not influenced by GWLL at all. These two measurement points are located on the opposite side of the EAT which was not under excavation now and where GWLL was not directly installed. The influence is very strong at measurement points 13-000170-03 and 13-000470-05 and strong at 13-000170-01 and 13-000470-02 as explained before.

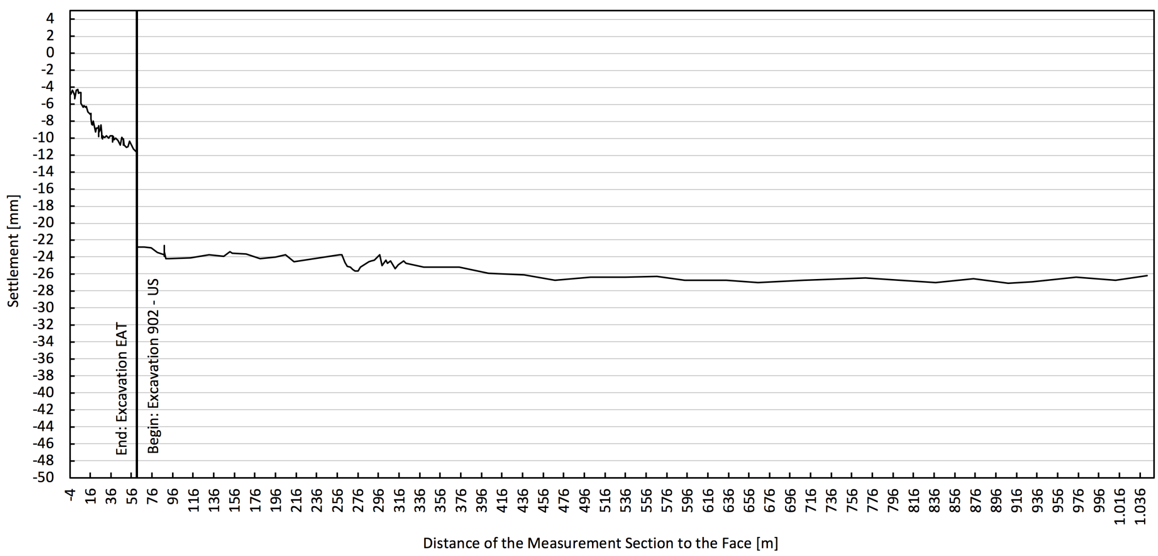

Figure 5.24: Settlements at Measurement Point 13-000170-01

In Figure 5.24 the increase of settlements during the construction stop from 09.02.2015 to 20.11.2015 can be clearly seen as vertical line at 61.6 m distance to the tunnel face. Settlements of 12 mm developed before, 11 mm during and 4 mm after the construction stop. In Table 5.32 design, predicted and measured settlements and the defined warning level.

Table 5.32: Design, Predicted and Measured Settlements and Warning Level at TM 170

Settlements [mm]			Difference [mm]		Warning Level [mm]
design	predicted	measured	to design	to predicted	
13	102	28	15	-74	15

The predicted behaviour after Fillibeck highly overestimates the real settlement behaviour and is not usable at DS 05. Design settlements match the real behaviour without GWLL pretty well.

The transversal settlement trough at TM 170 is shown in Figure 5.25. In comparison to the transversal settlement trough at the design sections DS 01 to DS 04 this one shows two measurement points with nearly equal settlements of around 30 mm. The impact area is bigger than the measurement section itself.

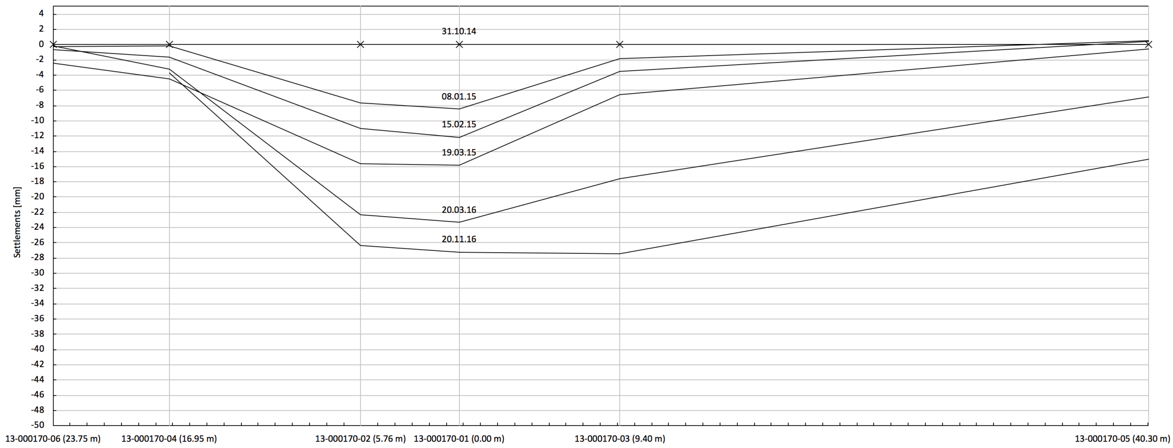

Figure 5.25: Transversal Settlement Trough at Measurement Section TM 170

5.5.6 Comparative Settlements

Settlements due to GWLL are calculated for GWLL of 1 m, 2 m, 5 m and 10 m with the theory of additional stresses in the ground caused by GWLL.

Table 5.33: Comparative Settlements due to GWLL at TM 170

Layer	Thickness [m]	max. eff. Stress [MPa]	Settlements [mm]
GWLL - 1 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	807	7
deep gysum layers	17.3	1084	3
		Total:	10
GWLL - 2 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	817	15
deep gysum layers	17.3	1094	6
		Total:	21
GWLL - 5 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	847	33
deep gysum layers	17.3	1124	14
		Total:	47
GWLL - 10 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	897	59
deep gysum layers	17.3	1174	29
		Total:	88

The design underestimates settlements because no GWLL is modelled with SOFISTIK. If we assume 10 to 15 mm settlements caused by GWLL the comparative settlements match this assumption pretty well. Settlements due to GWLL are in the range of 10 to 15 mm at measurement points 13-000170-01, 13-000170-02, 13-000170-03 and 13-000170-05. This would match with 1 to 1.5 m of GWLL in Table 5.33 and this is a reasonable value. GWLL has a high influence of around 35 to 55 % on the settlement behaviour.

Table 5.34: Design, Measured and Comparative Settlements at TM 170

Settlements [mm]			Difference [mm]
design	measured (max.)	comparative (1 m GWLL)	
13	28	10	18

5.5.7 Measured Displacements

At DS 05 the maximum overburden is approximately 50 m. At measurement section TM 170 displacements are 3 mm at the crown and settlements are 28 mm. Even if pre-relaxation and delayed zero measurement are assumed displacements do not correlate with settlements.

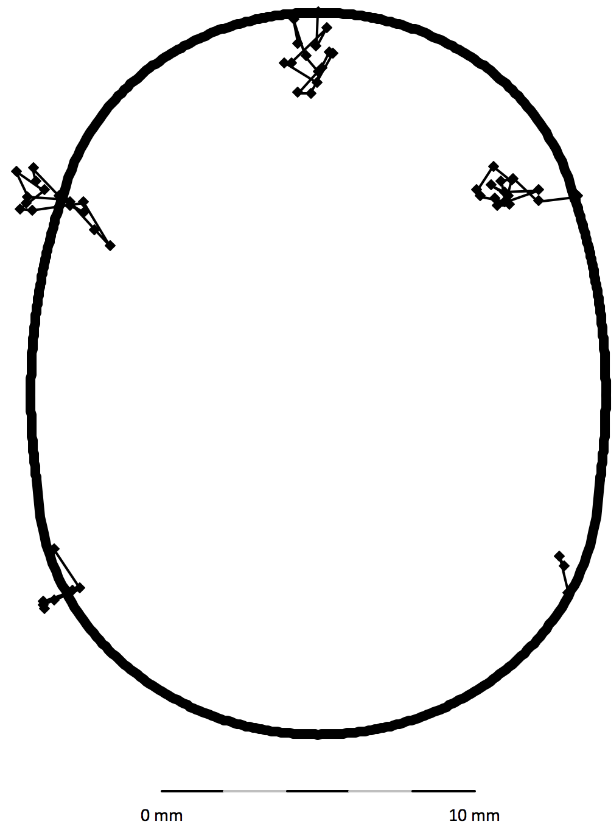

Figure 5.26: Displacements at Measurement Section TM 170

Table 5.35: Measured Settlements and Displacements at TM 170

Settlements [mm]	Displacements [mm]	Difference [mm]
28	3	25

5.6 Design Sections in between TM 166 and TM 235

Design simulations at design sections in between TM 166 and TM 235 are not accessible for the purpose of this master thesis. Therefore Johannes Heller (Vice-Chief Construction Supervision) and Johannes Bauer (Construction Supervisor) proposed to apply the same design, predicted and comparative settlements and warning and alarm values as at DS 05 at TM 166 to TM 235. The representative design section is named DS 06. The EAT intersects with pillar galleries East and West, tunnel 801, 802, 901 and 902.

5.6.1 Measured Settlements

The measurement section TM 175 shows the highest settlements and is shown in Figure 5.27.

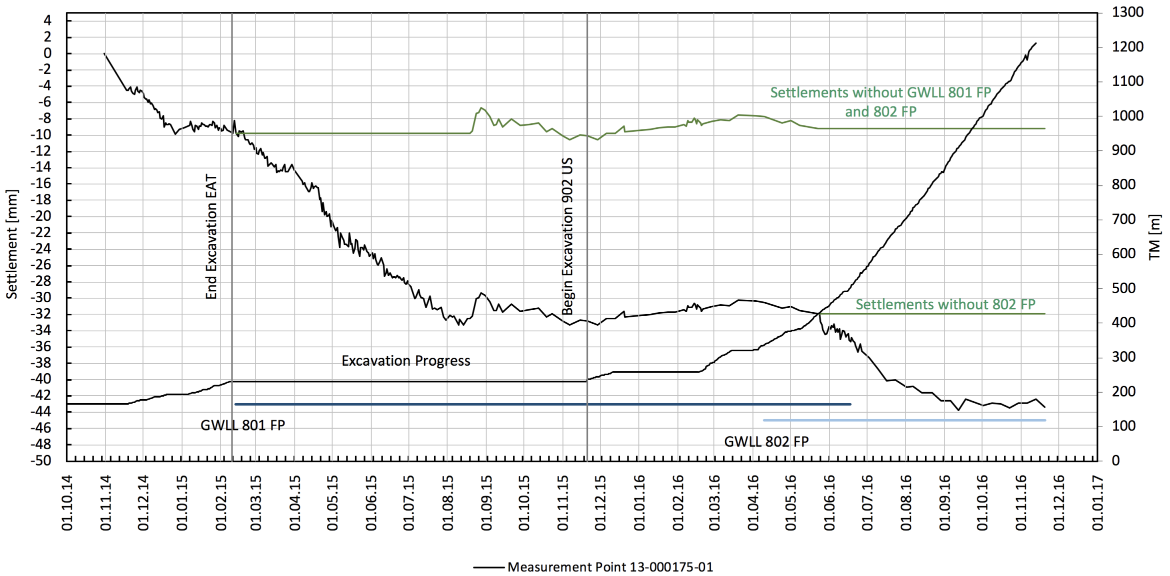

Figure 5.27: Settlements and Excavation Progress at Measurements Section TM 175

Measurement section TM 175 is part of the second monitoring period which started on 31.10.2014. This section shows the highest settlements of 44 mm along the EAT. At first settlements were monitored nearly linear till approximately 9 mm. Afterwards excavation was finished and settlements stagnate. The settlement development without GWLL is shown as green lines. Therefore settlements would be 9 mm in total for this section if GWLL did not take place. GWLL at tunnel face 801 FP started on 13.02.2015 and settlements started to increase in a linear behaviour till 33 mm. The short uplift on late August is related to the pit excavation at construction site John Cranko School. Till the start of GWLL at tunnel face 802 FP settlements are stable. GWLL at tunnel face 802 FP increase settlements till a maximum of 44 mm and stagnation afterwards.

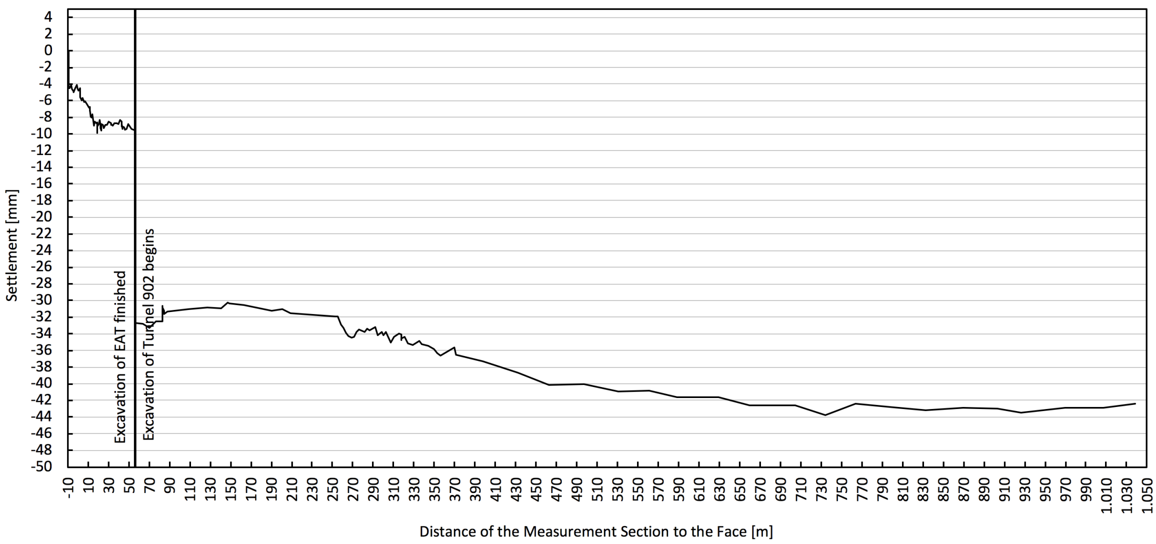

Figure 5.28: Settlements at Measurement Section TM 175

In Table 5.36 design, predicted and measured settlements and the defined warning level are shown. The predicted behaviour after Fillibeck highly overestimates the real settlement behaviour and is not usable at the section. Design settlements match the real behaviour without GWLL pretty well.

Table 5.36: Design, Predicted and Measured Settlements and Warning Level at TM 175

Settlements [mm]			Difference [mm]		Warning Level [mm]
design	predicted	measured	to design	to predicted	
13	102	44	31	-58	15

The transversal settlement trough at measurement section TM 175 shows the maximum settlements of 44 mm along the EAT. The impact area of the trough on the surface is bigger than the measurement section. The whole influence can not be seen.

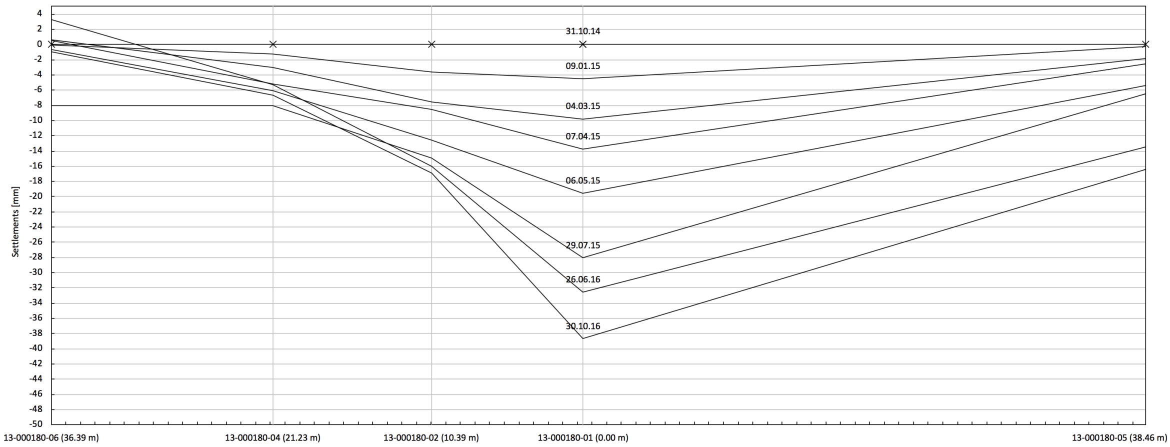

Figure 5.29: Transversal Settlement Trough at Measurement Section TM 180

The measurement sections in between TM 166 and TM 235 show very different settlement developments. Therefore a second measurement section is evaluated. The measurement section TM 225 represents in contrast to TM 175 the average settlement behaviour. The settlement development is shown in Figure 5.30.

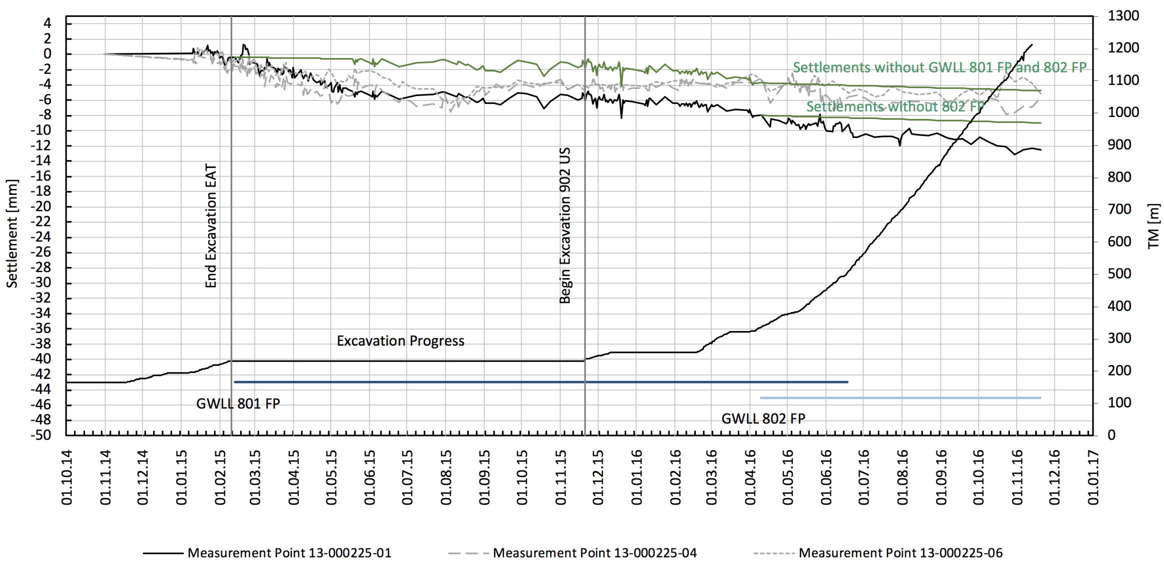

Figure 5.30: Settlements and Excavation Progress at Measurements Section TM 225

Measurement section TM 225 is part of the second monitoring period. In comparison to measurement section TM 175 less settlements occur and the impact of GWLL is high. Nearly all settlements are related to GWLL. Settlements around 8 mm are related to GWLL. The green lines in Figure 5.30 show the settlement developments without GWLL. Settlements around 5 mm are not related to GWLL. Better RMP at the end of EAT and at 902 IH US are the reason for less settlements.

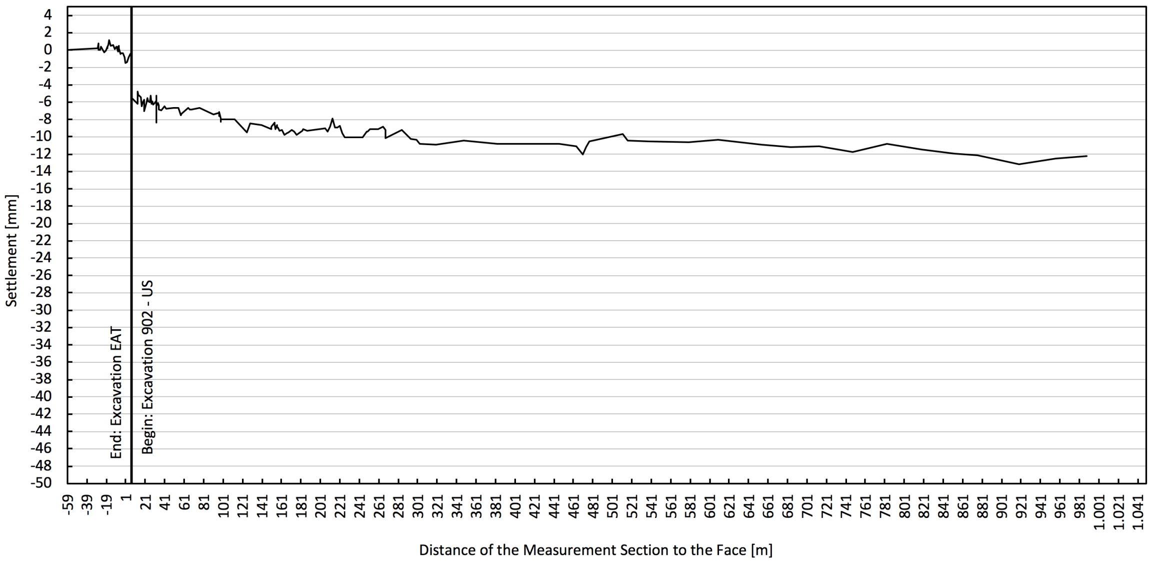

Figure 5.31: Settlements at Measurement Section TM 225

Settlements due to GWLL at the face of tunnel 801 FP can be seen in Figure 5.31 as vertical line at 6.6 m distance to the face. The ground behaviour is very stable afterwards.
In Table 5.37 design, predicted and measured settlements and the defined warning level are shown. The predicted behaviour after Fillibeck highly overestimates the real settlement behaviour and is not usable at the section. Design settlements match the real behaviour very well.

Table 5.37: Design, Predicted and Measured Settlements and Warning Level at TM 225

Settlements [mm]			Difference [mm]		Warning Level [mm]
design	predicted	measured	to design	to predicted	
13	102	13	0	-89	15

5.6.2 Comparative Settlements

Settlements due to GWLL are calculated for GWLL of 1 m, 2 m, 5 m and 10 m with the theory of additional stresses in the ground caused by GWLL. The same settlements due to GWLL as at DS 05 are applied at TM 166 to TM 235. The results of GWLL calculations are shown in Table 5.38.

Table 5.38: Comparative Settlements due to GWLL at TM 166 to TM 235

Layer	Thickness [m]	max. eff. Stress [MPa]	Settlement [mm]
GWLL - 1 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	807	7
deep gysum layers	17.3	1084	3
		Total:	10
GWLL - 2 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	817	15
deep gysum layers	17.3	1094	6
		Total:	21
GWLL - 5 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	847	33
deep gysum layers	17.3	1124	14
		Total:	47
GWLL - 10 m			
talus deposit and filling material	7.4	74	0
middle gypsum horizon	61.9	897	59
deep gysum layers	17.3	1174	29
		Total:	88

The design underestimate settlements at TM 175 because no GWLL is modelled with SOFISTIK. GWLL definitely caused settlements from 13.02.2015 to mid of August 2015 and from 10.04.2016 to mid of September 2016. In the first period settlements are approximately 22 mm and in the second approximately 12 mm. This are 34 mm in total which are caused by GWLL. This would correspond to GWLL of approximately 3 m which is a plausible and verifiable value at the construction site. In Table 5.39 design, measured and comparative settlements are listed.

Table 5.39: Design, Measured and Comparative Settlements at TM 175

Settlements [mm]			Difference [mm]
design	measured (max.)	comparative (3 m GWLL)	
13	44	34	10

Although no GWLL is modelled with SOFISTIK at TM 225 the design matches the reality very well. GWLL definitely caused settlements from 13.02.2015 to beginning of June 2015 and from begin of February 2016. In the first period settlements are approximately 5 mm. In Table 5.40 design, measured and comparative settlements are listed.

Table 5.40: Design, Measured and Comparative Settlements at TM 225

Settlements [mm]			Difference [mm]
design	measured (max.)	comparative (1 m GWLL)	
13	13	10	3

5.6.3 Measured Displacements

At TM 175 the maximum overburden is approximately 50 m. Displacements are 3 mm at the crown and settlements are 44 mm. Displacements do not correlate with settlements at TM 175.

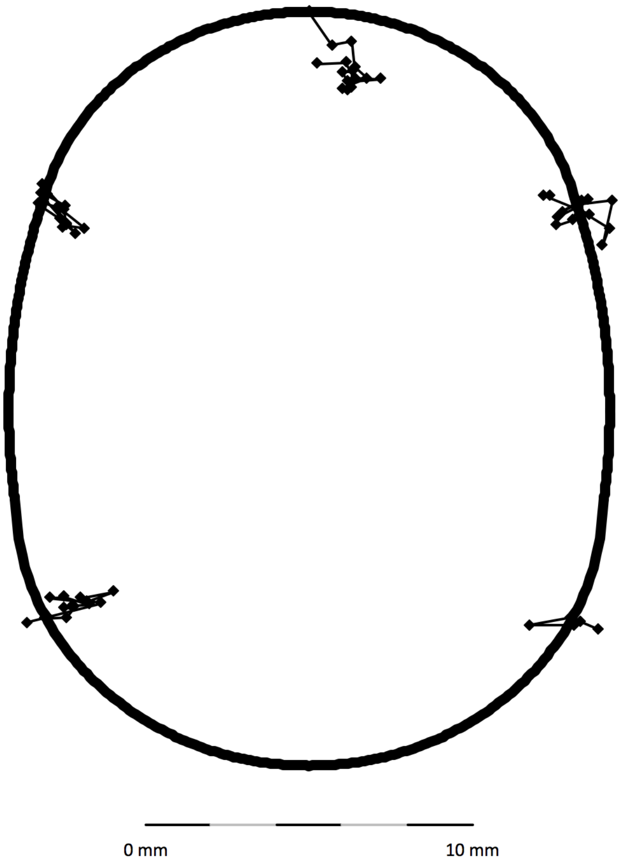

Figure 5.32: Displacements at Measurement Section TM 180

Table 5.41: Measured Settlements at TM 175 and Displacements at TM 180

Settlements [mm]	Displacements [mm]	Difference [mm]
44	3	41

At TM 225 the maximum overburden is approximately 58 m. Displacements are 1 mm at the crown and settlements are 13 mm. Like at TM 175 displacements do not correlate with settlements.

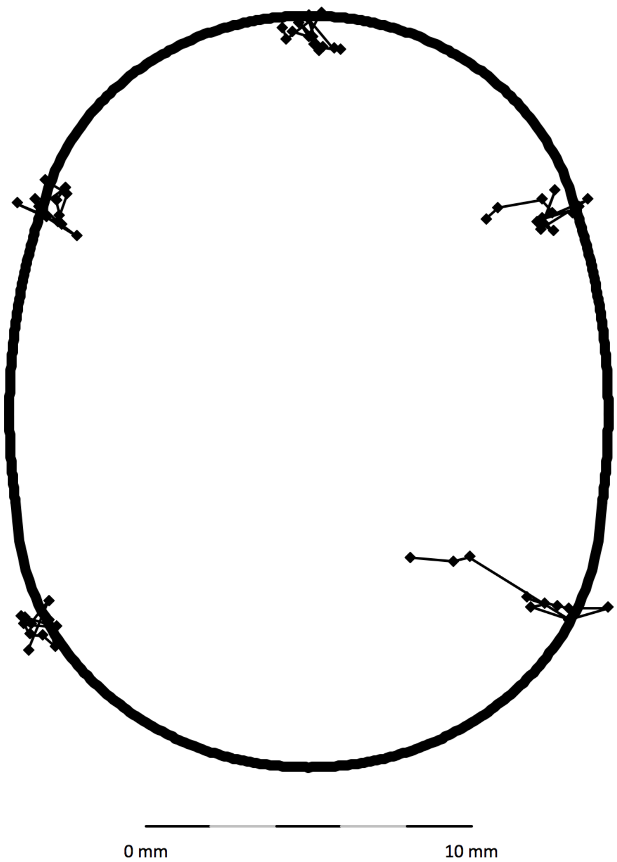

Figure 5.33: Displacements at Measurement Section TM 220

Table 5.42: Measured Settlements at TM 225 and Displacements at TM 220

Settlements [mm]	Displacements [mm]	Difference [mm]
13	1	12

6 Evaluation Over the Length of the EAT

The construction process at MS South is defined by a lot of different steps and actions. Therefore all of them are shown at the construction schedule at Figure 6.1 over time. The excavation of the EAT was finished first and afterwards pillar galleries East FP and West FP were excavated before the excavation of tunnel 902 IH US started. The excavation of pillar gallery East MS and approximately 100 m of tunnel 801 FP and 802 FP took place more or less simultaneously with the excavation of tunnel 902 IH US. GWLL at tunnel face 801 FP (TM 106) started on 13.02.2015 after the excavation of the EAT was finished. GWLL at tunnel face 802 FP (TM 108.8) started on 10.04.2016.

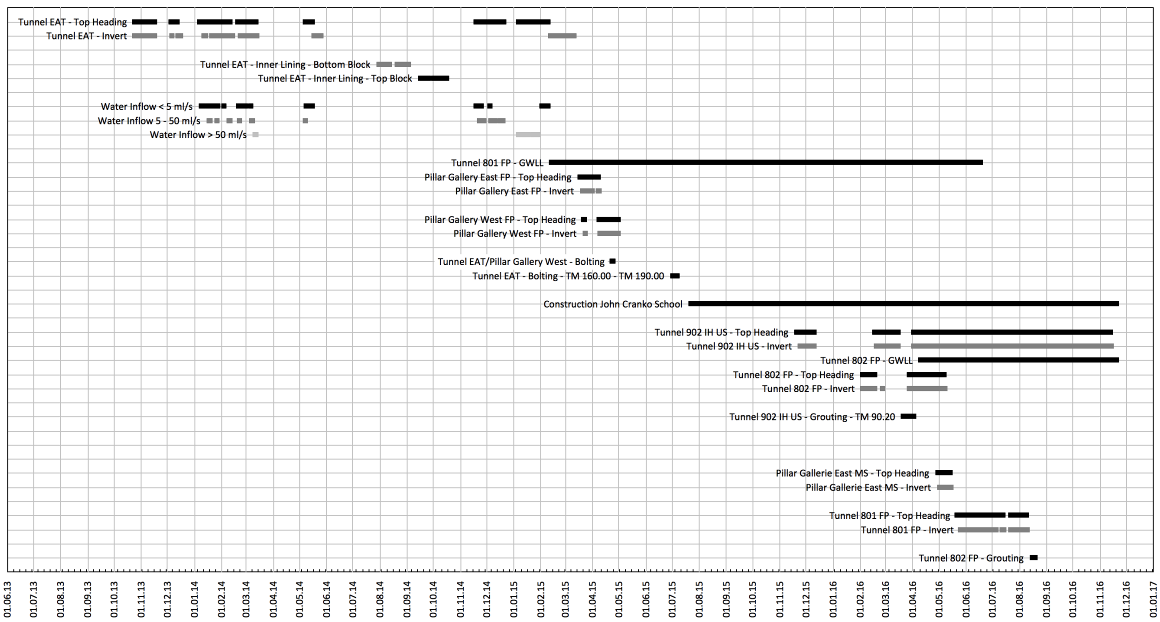

Figure 6.1: Construction Steps at MS South

6.1 Longitudinal Settlement Trough at EAT

The evaluations at design sections DS 01 to DS 06 show differences of design and measured settlements. These evaluations are performed independent at each section and putting them together provides an overview of differences along the EAT. A longitudinal settlements trough fits best to show measured settlements at measurement points above the tunnel crown. The longitudinal settlement trough in Figure 6.2 does not look like the longitudinal settlement trough defined by Fillibeck (2012). The inclination of EAT, therefore different overburden in combination with GWLL, highly local geotechnical deviations of the ground properties and the intersection of tunnels are the reason for this non common looking longitudinal settlement trough. Total settlements increase with the tunnel length more or less steadily till TM 170. A huge drop can be seen in Figure 6.2 from TM 170 to TM 175. At this measurement section the highest settlements of 43.75 mm occur. From TM 175 total settlements are decreasing in comparison to settlements at measurement section TM 175 till the last measurement section. The settlement trough is similar to a funnel or a cone with its peak of 43.75 mm at TM 175. The measured settlements in comparison to design settlements at each measurement section are shown in Table 6.1.

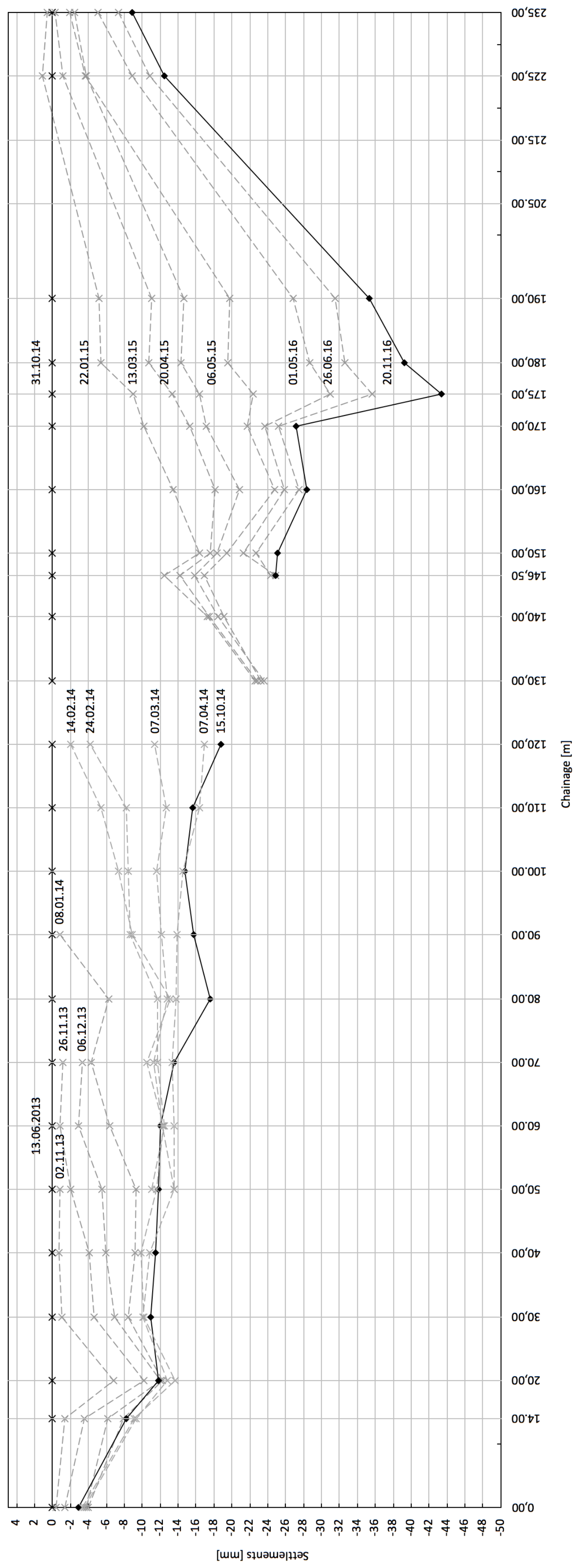

Figure 6.2: Longitudinal Settlement Trough along EAT

Table 6.1: Measured and Design Settlements at all Measurement Sections

Measurement-Section	Settlements [mm] measured	Settlements [mm] design	Difference [mm]
TM 0	5	3.83	1.17
TM 4	5	3.83	1.17
TM 14	10	3.83	6.17
TM 20	14	5.77	8.23
TM 30	15	5.77	9.23
TM 40	12	5.77	6.23
TM 50	14	17.26	-3.26
TM 60	14	17.26	-3.26
TM 70	14	17.26	-3.26
TM 80	18	17.26	0.74
TM 90	16	17.26	-1.26
TM 100	17	18	-1
TM 110	21	18	3
TM 120	25	18	7
TM 130	24	18	6
TM 140	20	18	2
TM 146.5	27	18	9
TM 150	28	18	10
TM 155	14	13	1
TM 160	28	13	15
TM 165	33	13	20
TM 170	28	13	15
TM 175	44	13	31
TM 180	40	13	27
TM 190	35	13	22
TM 205	22	13	9
TM 215	17	13	4
TM 225	13	13	0
TM 235	12	13	-1

As already explained at the specific design sections 01 to 06, surface settlements do not really correlate with displacements at the crown. This is also shown in Figure 6.3. Displacements at the crown (grey line) are not in the same range as settlements above the crown (black line).

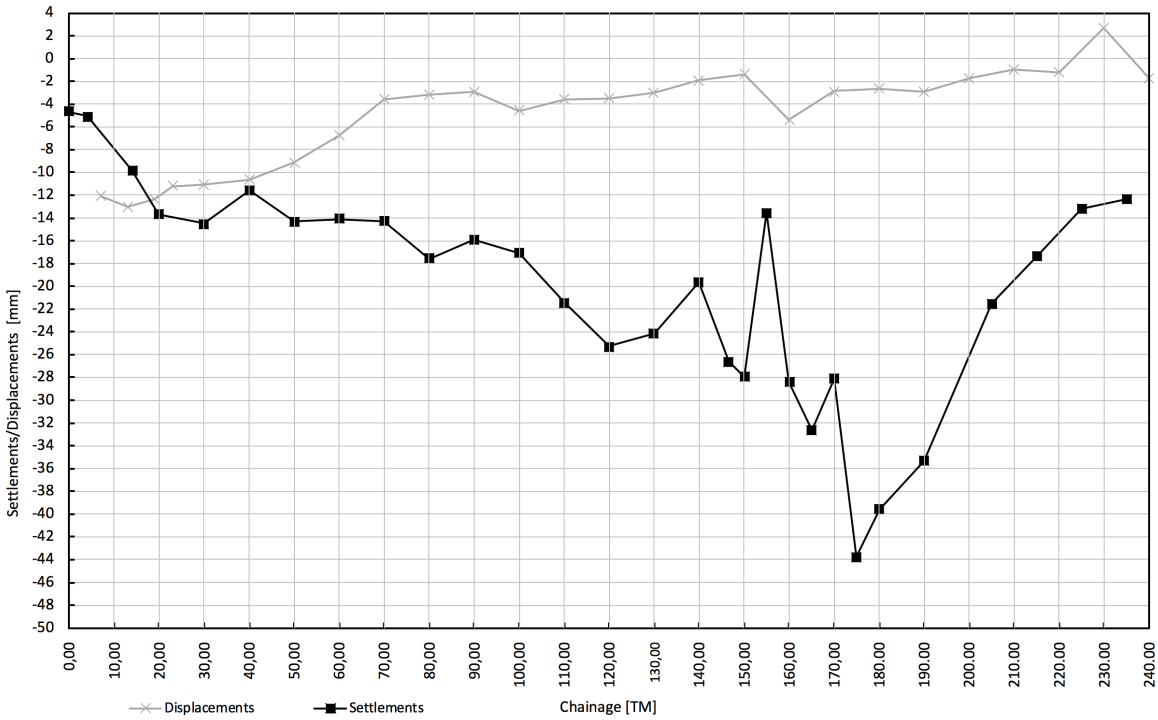

Figure 6.3: Settlements and Displacements along EAT

All above evaluations were made in 2D. A settlement trough is a 3D structure. Therefore the longitudinal and transversal settlement troughs, defined by the measurement points of each measurement section, are combined in one contour plot which shows settlements by a colour gradient. The contour plot in Figure 6.4 confirms the assumption of a settlement trough similar to a funnel.

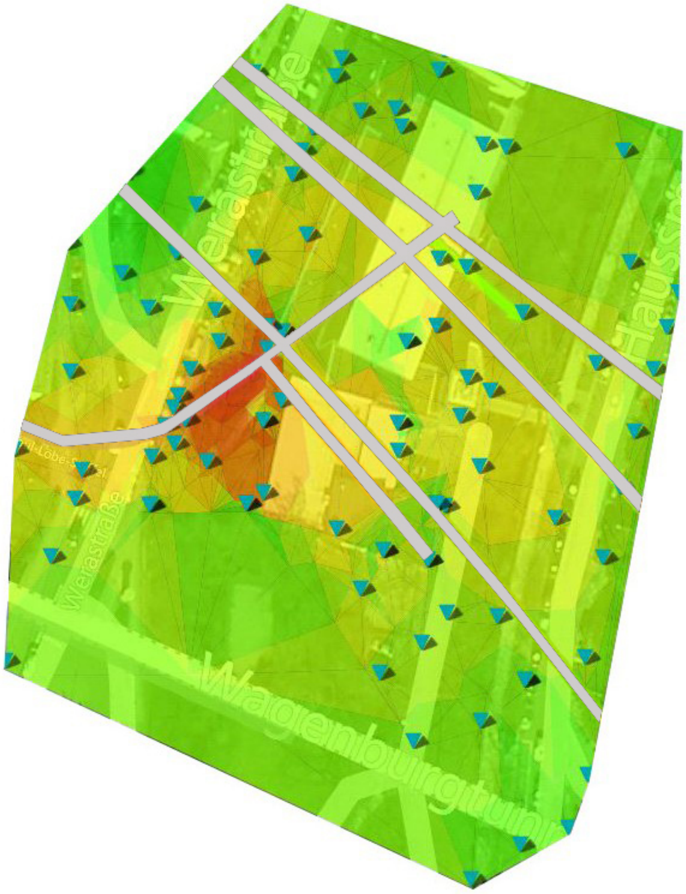

Figure 6.4: Settlement Contour Plot

6.2 Settlement Reasons along EAT

6.2.1 Construction Project: John Cranko School

Next to the construction site MS South the construction of John Cranko School started on 23.07.2015. As part of the construction work a pit was excavated. The removal of soil reduced the overburden at EAT and therefore unloaded the ground. The load reduction caused an natural uplift which can be seen at the affected measurement sections TM 170, TM 175 and TM 180 from 01.08.2015 to 01.10 2015. The maximum uplift is around 2 mm. This is not necessarily the maximum uplift caused by the pit excavation because a combination of settlements and uplift could have occurred at the same time. If the construction of John Cranko school influence settlements, it is a positive one.

Figure 6.5: Construction Site John Cranko School [Ralf Grolms (2016)]

6.2.2 Geological Model

The geological model has been continuously updated along the tunnel axis [Figure 6.6 and Figure 6.7] due to face mapping. The assumed geological layers and units fit pretty well to the geological situation in-situ. RMP of tested samples are not as good as assumed and RMP in the numerical models are therefore overestimated. The main difference to the assumed geological model are two filled carst cavities. The two cavities were not encountered or investigated during one of the investigation programs and highly influence the local ground behaviour. The first one is located between TM 63 and TM 66 and shown in Figure 6.6. The second one is located between TM 167 and TM 177 and shown in Figure 6.7.

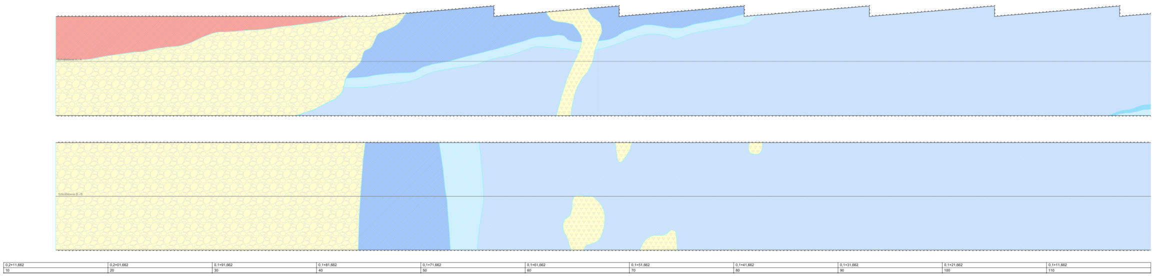

Figure 6.6: Geological Longitudinal Section TM 15 to TM 120

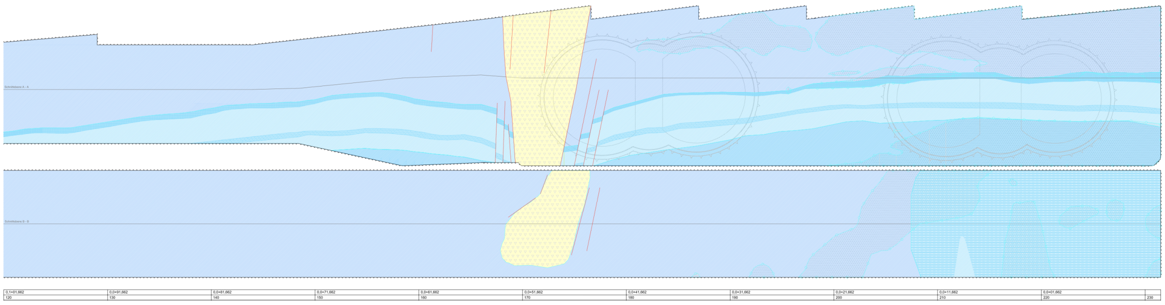

Figure 6.7: Geological Longitudinal Section TM 120 to TM 235

The filled carst cavities probably formed due to dissolved gypsum of the leached gypsum keuper. The poor RMP of the carst material at these sections highly increase settlements and displacements. The excavation from TM 63 to TM 66 was done from 10.01.2014 till 14.01.2014 and from TM 167 to TM 177 from 21.11.2014 till 17.02.2015. During this time and later on settlements increased significant at the measurement sections of this chainages. The highly unstable ground behaviour of the filled carst cavities and poor RMP increased settlements by around 15 mm maximum. Therefore the filled carst cavities are not the main reason of the settlement trough but support it.

6.2.3 Groundwater Level Lowering

GWLL started on 13.02.2015 at tunnel 801 FP and on 10.04.2016 at tunnel 802 FP. Vacuum pumps were installed at each face to get control over the water inflow at 902 IH US, 801 FP, 802 FP and pillar galleries because local grouting did not result in success. The biggest influence on settlements at the construction site MS South is GWLL. GWLL causes an increase of effective stresses in the affected layers and therefore settlements depending on the thickness and the constrained modulus of each affected layer. An additional load of approximately 10 kN/m^2 per meter of groundwater level lowering matches the increase of effective stresses. This load is equivalent to the uplift force of water. Layers below the new GWL are unloaded due to water load reduction which causes and uplift in the affected layers. The effect of GWLL to the ground behaviour can be seen at each measurement section of the second monitoring period. The first one ended before GWLL started. After GWLL started settlements increased. The inclination of the settlement development is the same for the impact of GWLL at tunnel face 801 FP and 802 FP as already shown at the evaluations at design sections. This ground behaviour can be explained. The vacuum pumps have a certain power and therefore a maximum lowering depth. The installation causes a specific lowering of the groundwater level and therefore specific settlements according to the ground behaviour at the section. The system balances itself to stable conditions which can be understood as stop of settlements. The largest impact of approximately 30 mm settlements due to GWLL takes place at measurement section TM 175. In Figure 6.8 comparative settlements due to GWLL and the longitudinal settlement trough are shown.

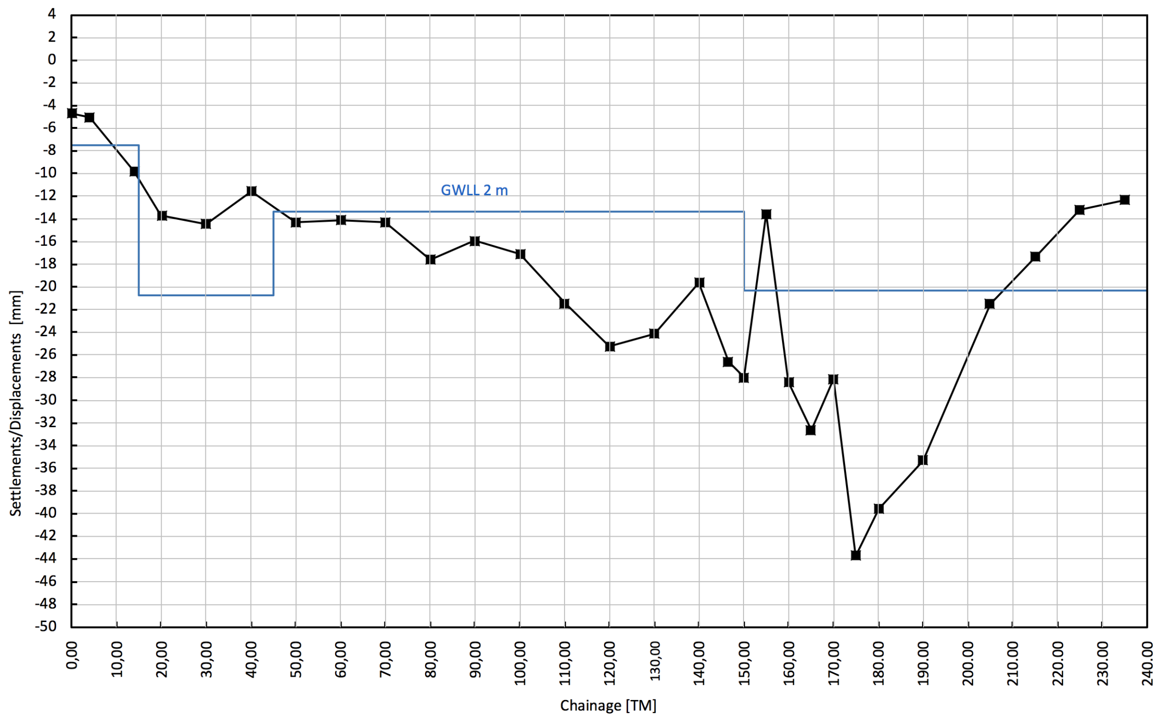

Figure 6.8: Maximum Settlements and Comparative Settlements due to 2 m GWLL along EAT

7 Conclusion

The evaluation at design sections and along EAT clearly shows the impact of GWLL on settlements and therefore the ground behaviour. Design settlements are defined as numerical results after SOFISTIK [PGS 21 (I) (2013); PGS 21 (II) (2014)] without taking groundwater conditions or GWLL into account. Therefore it is obvious that design and measured settlements differ a lot from each other. Warning and alarm levels are defined as numerical results after SOFISTIK with poor RMP. These levels were highly exceeded in quite a lot of monitoring sections. Additionally two filled carst cavities influence the ground behaviour strongly, especially from TM 150 to TM 190. At measurement section TM 175 the influence of GWLL and poor RMP of the cavity leads to the highest settlements of 44 mm of the settlement trough. Due to these reasons settlements at the construction site MS South were underestimated.

All tunnels direction MS are not excavated yet. At these tunnels the geotechnical and constructional challenges are to excavate with low overburden, large cross-section areas, high building density and to ensure the safety of infrastructure and not cause additional settlements or a growth of the settlement trough. The conclusion and the acquired knowledge from excavation and monitoring of the EAT shall be used to update and improve numerical models for the ongoing excavations at this construction site.

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