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Prediction of tunnel-induced settlements in soft ground

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ABSTRACT: Tunneling in soft ground always leads to stress redistribution and displacements in the surrounding soil. Therefore, when planning shallow tunnels in areas sensitive to settlements, the prediction of the deformations above the tunnel is very important. In order to describe tunnel-induced settlement troughs, the Gaussian distribution function is most commonly used. Its input parameters are the point of inflection i and the volume of the settlement trough, described by the so called volume loss VL_s . These two parameters have to be defined in order to be able to predict a settlement trough which is close to that in situ. In the literature, the results of settlement measurements are frequently presented, however the results are mostly not comparable due to different boundary conditions. Therefore, the range of published settlement troughs vary widely and there are very few as well as inaccurate data information concerning the size of the parameters i und VL_s . In this paper the empirical prediction-method of Fillibeck is introduced to determine the parameters i und VL_s which was deduced from more than 350 carefully selected and proved measurement results as well as accompanying 2D- and 3D- FE-calculations using advanced constitutive models.

1 INTRODUCTION

Nowadays the realistic prediction of tunnel-induced settlements in urban areas is very important, in order to gain the acceptance of the population for a tunneling project. In this case numerical analyses as well as empirical procedures are suitable. The inputs of such empirical procedures are derived from earlier tunneling projects. The empirical methods mostly use the function of Gaussian distribution function to describe the settlement trough. The input parameters are the point of inflection i and the volume of the settlement trough, described by the so called volume loss VL_s or otherwise abbreviated VL in the literature.

In this paper an empirical method is introduced to predict i and VL_s for shotcrete or shield excavation which was deduced from a large number of carefully selected and proved measurement results. For shotcrete excavations more than 200 measured sections of the Munich subway as well as 50 further measured sections under other geological conditions as well as more than 100 measuring sections for shield excavation could be considered. With the help of these measurements as well as 2D- and 3D- FE-back analysis using advanced constitutive models, the influence of the following parameters was determined:

- geological situation
- tunnel depth
- size of the excavation
- soil stiffness

Because of the large number of comparable measurements, it was possible to describe the size of the settlement trough in relation to the probability of occurrence. So the probability of falling below or exceeding the maximum settlement, tangent inclination or deflection ratio can be predicted.

The evaluations shown are the results of research performed in the context of a Habilitation at the Zentrum Geotechnik of the TU München (Fillibeck (2010)). There the evaluations are described and proven in detail.

2 DESCRIBING THE SETTLEMENT TROUGH

2.1 *Mathematical description of the settlement trough*

The settlement trough across the heading is considered. As comparisons showed, the settlement trough can be described suitably by the Gaussian distribution (Schmidt (1969); Peck (1969)). The settlement $s(x)$ at the point x and the volume of the total settlement trough V_s at the ground level per meter heading are defined by 2 parameters being; the maximum settlement s_{\max} as well as the distance of the point of inflection of the Gauss function from the tunnel axis I (Equation 1 and Fig. 1).

$$s(x) = s_{\max} \cdot e^{-\frac{x^2}{2 \cdot i^2}} \quad (1)$$

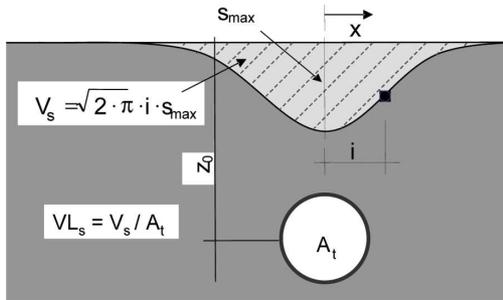


Figure 1. Definitions pertaining to the settlement trough.

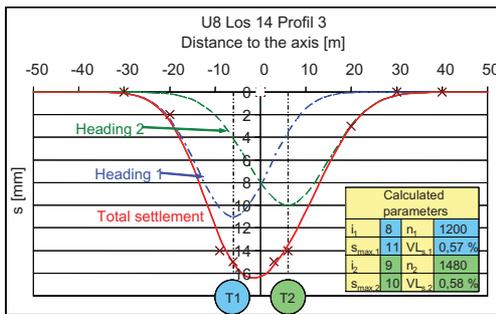


Figure 2. Analysis of a total settlement trough.

According to the Gauss function the point of inflection corresponds to the standard deviation.

The calculation of the volume of the settlement trough per meter heading V_s can be calculated by means of Figure 1.

If one defines the volume loss VL_s as the ratio between the cross-section of the settlement trough V_s and that of the tunnel A_t , then the unknown s_{max} can be expressed alternatively in terms of VL_s (Equation 2). Subsequently, VL_s as it is commonly termed in the literature, is used instead of s_{max} next to i , as a second parameter for the description of the settlement trough.

$$s_{max} = VL_s \cdot \frac{A_t}{\sqrt{2 \cdot \pi} \cdot i} \quad (2)$$

2.2 Determination of VL_s and i from the results of measurement

In the case of parallel tunnel headings, the total settlement trough results from the sum of the two settlement troughs of the single tunnels. Thus, the total settlement trough is described by 4 parameters $VL_{s,1}$, $VL_{s,2}$, i_1 and i_2 .

Figure 2 exemplarily shows the analysis of the two shotcrete-headings of the U8 subway, Lot 14 in

Munich, with the two single settlement troughs, the total settlement trough (addition of the single settlement troughs) and the corresponding measured values (crosses). According to the principle of the least squared smallest error squares, the total settlement trough was determined by mathematical variation of the 4 first unknown parameters, so that the total settlement trough matched the measured values (mathematical optimization) the best. Hence the parameters $VL_{s,1}$, $VL_{s,2}$, i_1 and i_2 were derived.

3 GENERAL DATA FOR THE ANALYSIS OF THE SHOTCRETE HEADINGS

The majority of the analyses of the shotcrete headings are based on measurements done in Munich. According to the heading methods performed on the Munich subsoil, a distinction has to be made between headings in the quarternary gravel and those of the predominantly fine-grained, tertiary soils.

Figure 3 shows exemplarily a quarternary heading above groundwater. The top heading is initially excavated with a total cross section of about 35 m² to 42 m². The large supporting core for the working face stabilization as well as the use of spiling or lagging sheets in the crown, are salient. Very coarse grained gravels are injected in advance as required.

The tertiary heading is affected by the adjacent water pressure of the tertiary sand layers within the tertiary clays and silts. If the sand layers can be dewatered, an atmospheric heading is possible. In Munich top headings with short reaching heads predominate (Fig. 4).

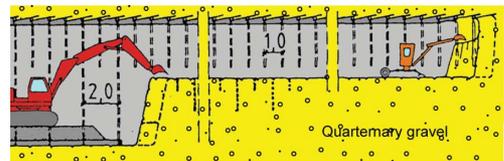


Figure 3. Top heading in the quarternary gravel.

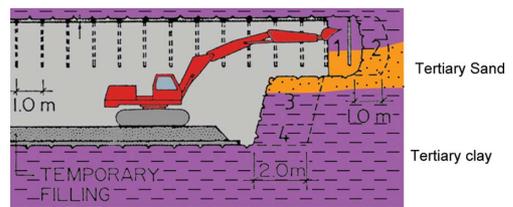


Figure 4. Heading in the tertiary.

If the sand layers cannot be dewatered, the possibility of a heading with the help of compressed-air support exists, where the air pressure must be maintained at a higher level than the maximum water pressure. To make sure that the excess air pressure does not become too high, it can be expedient to decrease the water pressure in the relevant sand layers by means of dewatering.

The analyses usually include two parallel tunnel tubes, whereby it was differentiated between synchronous and offset synchronous headings (4). Offset synchronous headings are characterized by a minimum distance of 25 m between the working faces of the respective tunnel tubes in heading direction.

The distance between both tunnel tubes (soil pillar) is an important parameter governing the settlements in parallel headings. The relationship between width of the soil pillar and tunnel diameter d is named ratio (a/d). By analyzing the shotcrete headings, the results show that the tunnel tubes influence each other and the settlement troughs increase if:

- $a/d < 0,3$ for the offset synchronous and synchronous headings or
- $1 < a/d < 0,3$ and tunnel depth $z_0 > 12$ m for synchronous headings.

Therefore, these headings were not taken into consideration in the following analysis.

Likewise, settlements resulting from dewatering during the tertiary heading were not considered, since the dewatering caused very wide and flat settlement troughs (widths > 200 m). Thus, no damage potential results from the dewatering because of the very small settlement differences (Fillibeck & Zaunseder (2008)).

4 EQUATIONS FOR THE DETERMINATION OF THE VOLUME LOSS

4.1 Volume loss for shotcrete excavations

In the following analysis it was distinguished between:

- atmospheric shotcrete excavations in fine grained soils,
- atmospheric shotcrete excavations in coarse grained soils and
- shotcrete excavations with compressed air support.

Initially, the various results of measurements for the Munich subway ($\sim 35\text{--}42$ m²) were evaluated. Thereby the volume loss was determined, according to the geological situation and the tunnel

depth z_0 . Figure 5 shows exemplarily the volume loss for atmospheric shotcrete excavations in the fine-grained tertiary soil.

The average volume loss can be described well by the regression lines shown in Figure 5. However, in practice it is also of interest to determine the settlement trough with higher confidence intervals or with lower probability of occurrence. For this the confidence interval of the measured values has to be determined. In Figure 5 the results for the upper confidence intervals of 90% and 99% are shown. They can also be described by a linear regression. A confidence interval of 99% means for example, that the probability of occurrence of a volume loss lying above the linear regression value amounts to 1%. Thus, the required volume can be determined as a function of the selected confidence interval.

In Figure 6 the measured volume loss with the regression lines (mean values) of the 3 shotcrete excavation methods are summarized.

The volume loss of the atmospheric shotcrete excavation is about twice as large as the shotcrete excavation with compressed air support. The compressed air supports the working face and the tunnel wall, so that altogether smaller settlements arise.

Subsequently, the evaluations were supplemented by measurements conducted in 50 further

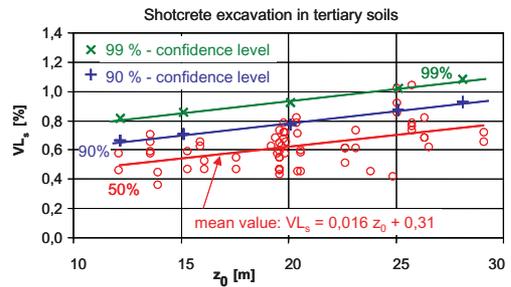


Figure 5. V_{L_s} at atmospheric shotcrete excavations in the tertiary soils.

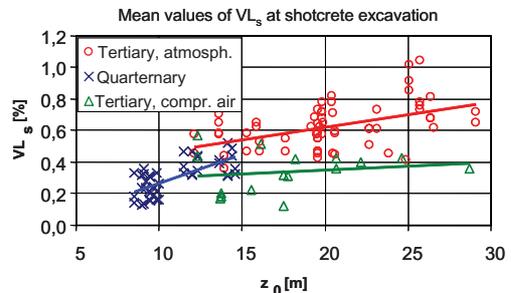


Figure 6. Volume loss of the different shotcrete excavation methods.

shotcrete excavations under other ground conditions. As it is shown on the basis of the evaluations and also on the comparison with FE-analyses, it can be assumed in good approximation that the size of the settlement trough is inversely proportional to the soil stiffness. This was considered in the equations to determine the volume loss. They are presented in Table 1 with confidence intervals of 50% (average value) and 99%.

As a result, the modulus $E_{100,ref}$ corresponds to the stiffness of the adjacent soil in MN/m² at a reference normal stress of 100 kN/m².

4.2 Volume loss of very large cross sections driven in several shotcrete-sections

Very large cross sections cannot be driven in one heading, as the working face is liable to fail. Therefore, they are subdivided into several headings. Hence, the question arises as to whether the given equations can also be used for these very large cross sections. This was examined with the help of 4 very large shotcrete excavations, to which extensive measurement results were available.

The following results are presented using the example of the “Theresienwiese” Station (Fig. 7) of the subway U5/9 in Munich. This atmospheric shotcrete excavation with a break out section of about 180 m² was carried out in 1979 in the tertiary clays. The whole cross section was divided into 5 headings, divided into top heading, bench and invert. For the empirical calculations each top heading, bench and invert were regarded as individual headings.

Initially, the settlement troughs of the partial headings were determined according to Equations 1 and 2, considering a volume loss with a 50% probability of occurrence according to

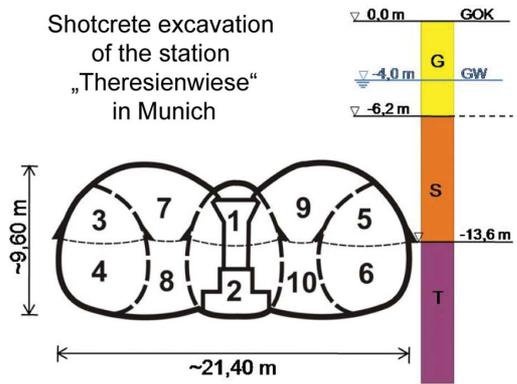


Figure 7. Excavation work at the station Theresienwiese of the U5/9 in Munich.

Table 1 and the point of inflection i corresponding to Section 5.

These settlement troughs were referred to the tunnel axis and subsequently superimposed, resulting in the total settlement trough shown in black in Figure 10.

The measured results agree very well with the settlement trough determined by the empirical method. This was also the case with the 3 other cross sections examined. Thus, the total settlement trough of a cross section, which is driven in several partial headings, can be determined by superimposing the single settlement troughs of the individual headings using the volume loss for the corresponding partial cross sections. The same applies to parallel tunnel headings (see Section 3). This is however not valid if the partial cross sections affect each other during heading, for example at synchronous headings with a pillar ratio a/d less than 0,3, see Section 3.

4.3 Volume loss for shield tunneling

In the following only shield tunneling methods with face support (hydraulic, earth pressure and compressed air shield tunneling) were considered, since these apply to settlement-endangered areas in granular soil.

For the investigations of the volume loss with shield tunneling, over 100 measured cross-sections of current construction measures could be evaluated. Excavations prior to 1995 were not considered, since in the past few years the mechanical engineering has substantially improved, so that fewer settlements have been reported in the past few years (see above).

The following results are valid in granular soil, however not in soils with less than a soft consistency, very loose density or structure-sensitivity. This classification is necessary, since for exam-

Table 1. VL_s at a confidence interval of 50% und 99%.

$VL_{s,50\%}$	$VL_{s,99\%}$
<i>Atmospheric shotcrete excavation in gravel</i>	
$(0,037 \cdot z_0 - 0,10) \cdot \frac{120}{E_{100,ref}}$	$(0,037 \cdot z_0 + 0,09) \cdot \frac{120}{E_{100,ref}}$
<i>Atmospheric shotcrete excavation in fine-grained soil</i>	
$(0,016 \cdot z_0 + 0,31) \cdot \frac{100}{E_{100,ref}}$	$(0,016 \cdot z_0 + 0,61) \cdot \frac{100}{E_{100,ref}}$
<i>Shotcrete excavation with compressed air support</i>	
$(0,005 \cdot z_0 + 0,26) \cdot \frac{100}{E_{100,ref}}$	$(0,005 \cdot z_0 + 0,57) \cdot \frac{100}{E_{100,ref}}$

ple the dynamic demands of the tunnel boring machines can lead to relevant additional settlements. This cannot be documented with the procedure described herein.

According to evaluations the volume loss tends to be smallest with hydraulic shield driving, however the difference to those with compressed air or earth pressure support is very small. Beyond that, no clear differences were observed with different subsoil and supporting pressures. Therefore, all the different tunnel driving methods were regarded as being independent of the subsoil and the supporting pressures.

Figure 9 illustrates the volume loss as a function of A_t/z_0 . As the regression (power function) shows, the volume loss decreases with the A_t/z_0 .

In order to once again be able to indicate the volume loss as a function of the confidence interval, the examined range was divided into 3 sections. For each section the confidence interval was determined and then approximated by means of a balance function for the total range. In Figure 9 the curves and equations for the determination of the

volume loss for a confidence interval of 50% and 99% are shown.

For future practice it is of substantial importance that in the cross sections examined here from the recent past the volume loss consistently lies below 0.5% and thus clearly under the value of 2.0%, indicated frequently in publications on shield driving.

Recently, the technique of the tunnel boring machines has improved in important settlement-relevant sectors (for example by grouting the gap behind the shield tail and by keeping the supporting pressure constant). Therefore, with newer shield headings, smaller settlements result and thus smaller maximum volume losses can be assumed. Due to inevitable fluctuations, it however is suggested that no volume loss smaller 0.25% be considered for the settlement prediction, as would result from large A_t/z_0 -values.

5 DETERMINATION OF THE POINT OF INFLECTION

In the following, the results of the Munich headings are initially presented again. As already recognized in other publications, the point of inflection i depends particularly on the geological conditions (grain size distribution, density and consistency) and on the tunnel depth.

Therefore, the quaternary and tertiary layers of the Munich subsoil are regarded separately, depending on the tunnel depth.

Figure 10 shows the point of inflection in the quaternary gravel i_G in relation to the tunnel depth z_0 . As to be expected, i_G increases with z_0 . It can be described by means of the regression line given in Equation 3.

$$i_G = 0,57 \cdot z_0 \quad (3)$$

The determination of the point of inflection in the tertiary layers beneath the quaternary layers

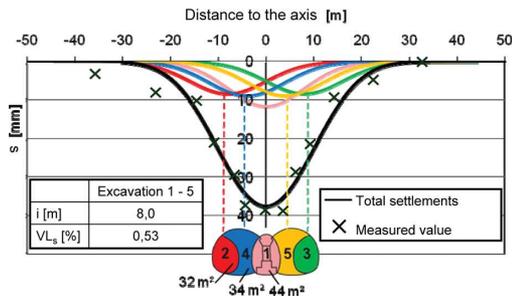


Figure 8. Comparison of measured results and empirical correlation.

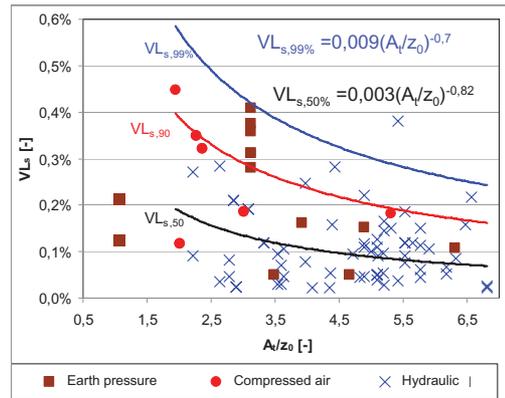


Figure 9. Volume loss with shield tunneling.

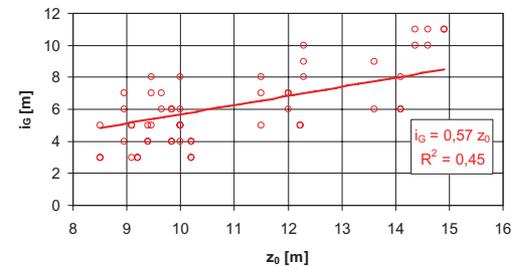


Figure 10. Point of inflection i_G in the quaternary gravel.

is somewhat more complicated, since it concerns a multi-layer system (see also New & O'Reilly (1991)). The point of inflection of the total settlement trough i_{ges} consists of individual contributions of the respective layers. This is shown in Figure 11, where i_G , i_s , i_{CL} and i_s are the points of inflection of the respective layers and $z_0 - d_{CL}$ is the thickness of the gravel and sand layer.

Assuming that i_G and i_s of the quaternary gravels and tertiary sands are approximately alike with equivalent layer thicknesses (they possess similar stiffness and strength) and that i_G is determined by Equation 3, the point of inflection of the fine grained tertiary soil layers i_{CL} can be determined. This is illustrated in Figure 12, a plot of i_{CL} against the layer thickness d_{CL} .

The expected linear dependence on z_0 is also obtained here. The point of inflection i_{ges} of the multi-layer system can be determined by means of the following equation:

$$i_{ges} = 0,82 \cdot d_{CL} + 0,57 \cdot (z_0 - d_{CL}) \quad (4)$$

Equations (3) and (4) are valid for underground conditions comparable to those in Munich. These are medium dense to dense gravels or sands and clays and silts of firm to stiff consistency respectively.

The evaluation of excavations subject other underground conditions show that also the consistency and the density exert a certain influence. With increasing consistency and density the point of inflection exhibits a corresponding increase.

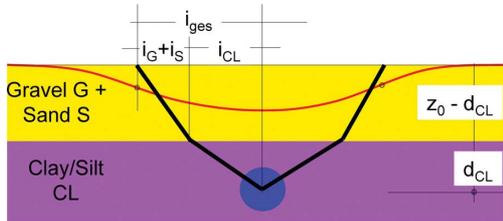


Figure 11. Point of inflection i_{ges} in layered soils.

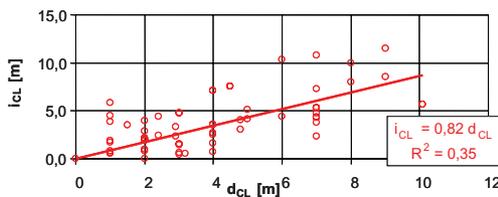


Figure 12. Point of inflection i_{CL} of the finegrained tertiary clays and silts.

Table 2. Points of inflection for different adjacent soils.

		i [m]
Gravel/sand	Loose—medium dense	0,25 to $0,5 \cdot z_0$
	Medium dense—dense	0,4 to $0,6 \cdot z_0$
Clay/silt	Soft—semisolid	0,3 to $0,6 \cdot z_0$
	Semisolid—solid	0,5 to $0,9 \cdot z_0$

By taking the available evaluations into consideration as well as published data, for example Mair & Taylor (1997) and Tan & Ranjith (2003), which are based again on different excavation evaluations, the following ranges of points of inflection may be proposed.

Since the results of the evaluations and the available published data were scattered, a bandwidth of points of inflection are indicated for the given ranges of the density or consistency. Thus, a small point of inflection can be assigned to a low density/consistency and a higher point of inflection to a higher density/consistency.

With the choice of a suitable point of inflection i it must be considered that with decreasing i the maximum tangent inclinations, strains and curvatures increase, with the result that—depending upon the location of the building with respect to the tunnel—the damage potential rises. However, the damage potential also increases with the volume loss. In order not to calculate an unrealistically high damage potential, two limit cases are suggested: Case 1 considers a high volume loss ($VL_{s,99\%}$) and a mean point of inflection and Case 2 a medium volume loss ($VL_{s,50\%}$) and a small point of inflection (according to a small density/consistency).

6 SUMMARY

With the empirical method presented herein, maximum settlement troughs resulting from shotcrete or shield tunneling can be predicted. The input parameters required to describe the transverse settlement trough by means of the Gauss function assumed here, are the point of inflection i as well as the volume loss VL_s .

The point of inflection i is a measure of the width of the settlement trough. The volume loss VL_s , which is defined as the ratio of the volume of the settlement trough per m heading to the tunnel cross section, corresponds to the magnitude of the settlement trough. Based on the results of measurements and taking the data of other publications into consideration, detailed proposals could be made for the choice of VL_s and i as a function of

the heading depth, the geology and the soil stiffness. Due to the multiplicity of the available measurements it was furthermore possible to specify the volume loss as a function of the probability of occurrence.

It is clear that not all possible boundary conditions can be considered with the empirical procedure presented here. But even if this succeeded, it is in the nature of the things that the settlements are subject to a certain scatter. Thus it is not possible to calculate a “true” settlement trough in advance but, at best a prediction depending on the probability of occurrence can be made. The described procedure, which is based on a very large number of comparable measurements and experiences, could be of great help and provide a possible solution. Alternative computations, which are usually implemented by means of finite element analyses, also require the estimation and/or variation of different inputs, if no regional experience with comparable tunnel headings is available. Experience shows, that this often leads to larger and frequently non realistic ranges of differently large settlement troughs, which can hardly be evaluated with accuracy. In this case, the determination of the settlement trough with the finite element method does not exhibit substantial advantages in relation to the simple empirical procedure presented here.

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