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Research paper

Parameters used for prediction of settlement trough due to TBM tunnelling

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Abstract: One of major design problems associated with shallow tunnelling in urbanized areas is the prediction of ground displacements caused by the construction process. Advanced tunnelling techniques such as shield tunnelling using Earth Pressure Balance Tunnel Boring Machines (EPB-TBMs) allow for significant reductions of settlements observed at the ground surface in comparison to tunnelling methods used in the past. The predictions of these displacements are often based on semi-empirical methods and prior experience. In addition to relative simplicity of such methods, their robustness and decades of validation in many tunnelling projects make them attractive for practical use. The tunnelling-induced settlement trough at the ground surface can be described by inversed Gaussian distribution function. It requires only the assumption of two parameters, namely: expected volume loss (V_L) and the distance to the point of inflection (i_y), which is dependent on the empirical trough width parameter (K) and the tunnelling depth (z_0). The values of those parameters have a strongly empirical nature; they should be established based on comparable experience obtained from full scale tunnelling projects with similar technique and at similar ground conditions. The paper presents the problem of variability of those parameters and discusses the need for its assessment. As volume loss is strongly related to the tunnelling technique, the study focuses on EPB-TBM tunnelling as the most commonly implemented one in recent years. Variability of parameters observed for different ground conditions in different countries is summarized. Finally, preliminary assessment of variability of settlements observed in Warsaw region is presented.

Keywords: tunnelling, EPB TBM, settlement trough, volume loss

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1. Introduction

Tunnels, among other geotechnical structures, have a unique character [1]. They rely on the ground around to provide the support, as much as on the structural lining itself. Although tunnel design problems can range from as far as the mutual interaction of underground structures [2] or the use of a tunnel as a source of renewable energy [3], the problem of tunnel construction itself still remains the most common concern. Tunnelling using Tunnel Boring Machines (TBMs) is associated with three main design considerations and requirements [4–6]: 1) maintaining face stability during TBM drive; 2) structural design and performance of the segmental lining; and 3) reducing tunnelling-induced settlements and impact on ground surface and adjacent structures. Tunnelling, especially at shallow depths, usually leads to surface deformations even in the case of well controlled tunnelling operations [7], and the settlements observed above the tunnels are strongly related to the implemented tunnelling technique [8]. Because the potential damage to properties of third parties is considered as one of the major risks associated with underground construction works [9], the choice of tunnelling method and ground deformation prediction is of major concern for successful construction. Effectiveness and safety of tunnel design is as much dependent on the design assumptions as on the skill and care of execution, which makes tunnel design still a highly empirical field of geotechnical engineering, despite significant advances in simulations and predictions of tunnelling processes [10].

Nowadays, Earth Pressure Balance (EPB) is the type of TBM, which is the most commonly used in practice [11]. EPB-TBM is a full-face shield, applied primarily in soft grounds, in which case, the face stability is provided by the excavated material, as its pressure is maintained through controlling shield thrust and the speed of removing spoil from the working chamber at the face of the shield. Originally, this type of shield was intended for use in fine-grained soils. Modern EPB TBMs, however, together with available range of conditioning agents, allow for tunnelling in a very wide range of ground conditions.

The paper gives an overview of ground deformation assessment methodology used in practice and focused on the variability of some of its main design parameters: maximum settlements, tunnelling-induced volume losses, and parameters characterizing the width of the settlement trough. An overview of those parameters in various ground conditions in different countries is presented, based on worldwide database of documented tunnelling projects. Finally, an overview of already summarized variability of EPB-TBM tunnelling-induced settlements observed in Warsaw area is provided. Even though the tunnelling activities in Warsaw were mostly focused on development of the metro [12–14] and sewage systems [15], sufficient experience base exists to provide region-specific design guidance for future tunnelling projects. Moreover, as further projects are under consideration also in other cities in Poland [16], documenting, cataloguing and analyzing empirical results from local tunnelling case studies is of utmost importance for fostering the use of underground space in Polish cities.

2. Prediction of ground displacements

Tunnelling process causes transversal and longitudinal displacements in a form of a settlement trough. Majority of tunnelling-induced ground movements occur at minimal longitudinal strain, approximately representing plane strain conditions (2D) [8]. Therefore, use of transversal settlement profile in impact assessment is justified and most commonly used for standard design applications.

The prediction models used in geotechnical engineering, also in the case of ground deformation predictions, can be divided into three broad categories [17, 18]: analytical (closed-form solutions), semi-empirical, and numerical. Each of these approaches has different characteristics and accounts for design uncertainties in a different way [19]. Comprehensive tunnel-ground-structure interaction analysis is considered as too complex for the use of analytical closed-form solutions [20]. Clarke and Laefer [21], in the context of staged analysis in impact assessment, stated that the level of detail of the analysis can vary from semi-empirical prediction models under greenfield conditions, with relatively simple deformation criteria, up to 3D numerical modelling; this range of methods is usually encountered in practice. However, Guglielmetti et al. [22] pointed out that application of numerical methods to assess all the cases along the tunnel can be very time consuming and might lack flexibility to give quick feedback. Similarly, Fu et al. [23] pointed out that numerical methods are more suitable for detailed analysis in particularly complex cases. As a result, advanced calculation methods are often not practical for the use by the engineers when less complex, but reliable models are available. In addition, Pickles and Henderson [24] discussed the over-reliance on the numerical modelling over the more conventional design calculation models and engineering judgment; they assessed that there is still a place for calculation methods based on proven experience. On the other hand, very simple semi-empirical models should be considered only for preliminary evaluation, and the analysis with the use of more advanced models should follow at later stages of the design [25, 26]. The practical implications are that the industry gives preference to well calibrated, reliable semi-empirical models, even if they tend to be more conservative than the more advanced numerical methods. The need of the industry for such models has been emphasized by Lambe [27] and still exists today [28, 29]. This results in the ongoing use of established empirical models and simpler numerical methods (2D) for ground deformation assessment; those usually are based on expected volume loss as one of the most important design parameters.

According to Peck [4], the maximum settlement above a tunnel ($S_{\max}$) can vary between tunnel cross-sections, usually, over a predictable range. Three possible types of settlements above a tunnel were distinguished, which can be related to the limit state design philosophy [17, 28] as follows:

- Normal settlements – the most probable, which should be considered as the baseline for impact assessment and to which the observed settlements are compared during construction.
- The greatest settlements that may occur at some cross sections – for verification of functionality and ultimate limit states of adjacent structures, even under adverse, unlikely conditions, ensuring that their probability of occurrence is sufficiently limited when potential consequences are considerable.
- Settlements caused by non-routine events (e.g. local collapse) – such values are difficult to predict and can be considered as accidental situations; they can be avoided by proper risk management procedures and increasing the robustness of design.

Each of those settlement types would be associated with different magnitude of volume loss that leads to its occurrence. This emphasized the need for assessment of variability range of such parameters.

3. Semi-empirical prediction model

Usually, the settlements above a tunnel are more or less symmetrical about its vertical axis, forming a trough with a shape roughly-resembling a Gauss distribution function [4,30], (Fig. 1), also known as "error curve", mentioned earlier in research done by Litviniszyn [31] and Martos [32]. This pattern of deformations is considered as well characterizing the actual settlement profiles observed in practice [33]. Empirical models based on Gaussian approximation have a considerable advantage of relative simplicity, ease of use, and proved validation in many sources of reference, in various ground conditions, and for various tunnelling techniques [34].

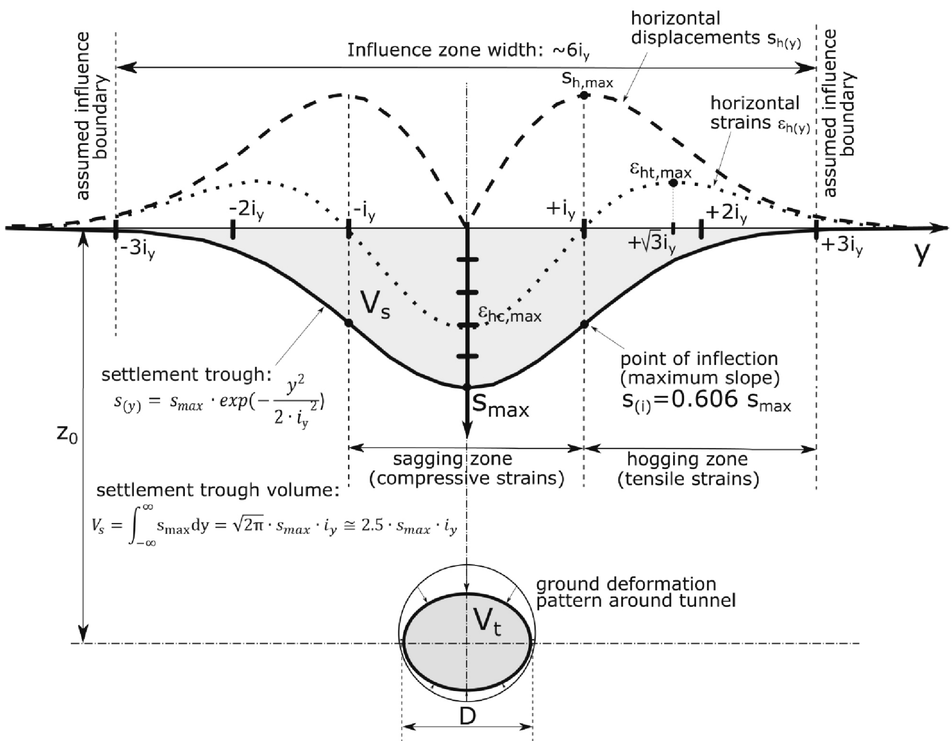

Fig. 1. Idealized transverse settlement trough approximated by normal distribution curve

The shape of the deformation profile is characterized by the maximum value (maximum settlement $S_{\max}$ above the centreline of a tunnel) and the value of standard deviation (distance from the centreline to the point of inflection i_y). The location of the points of inflection separates the zone of compressive (at a distance from $-i_y$ to $+i_y$ from tunnel centreline) and tensile stresses, distinguishing the areas where sagging and hogging occur, respectively (Fig. 1). At that distance, also maximum horizontal deformations can occur. It is commonly assumed that, for a single tunnel, the total zone of influence delimited by the detectable settlements (over the entire trough width) approximately equals 5÷6i_y [4, 35, 36]. Rankine [35] estimated the extent of the zone of influence at 6i_y or 3z_0. In general, this value can be considered as total width for most practical purposes [34]. The distribution of settlement in a transversal direction ($s_{(y)}$), caused by a single tunnel, is expressed by the following function, in its basic form [4]:

$$s_{(y)} = s_{\max} \cdot \exp\left(-\frac{y^2}{2 \cdot i_y^2}\right)$$

where: $S_{\max}$ – maximum settlement above the centreline of the tunnel, y – distance in the transversal direction from the tunnel centreline, i_y – distance to the point of inflection in the transversal direction

For prediction purposes, O'Reilly and New [37] proposed a linear dependence between the trough width parameter K and the tunnel depth z_0:

$$i = K \cdot z_0$$

where: K – settlement trough parameter, z_0 – depth to the tunnel axis.

Furthermore, the volume loss parameter (V_L) is commonly used as a useful index describing the influence of tunnel construction on ground deformations. It is defined as a percentage ratio of the total volume of the settlement trough (V_S) with respect to theoretical volume of tunnel excavation (V_t) [4, 34, 38]. The volume of settlement trough V_S and the volume loss V_L are calculated as:

$$V_S = \int_{-\infty}^{\infty} s_{\max}\, \mathrm{d}y = \sqrt{2\pi} \cdot s_{\max} \cdot i_y \cong 2.5 \cdot s_{\max} \cdot i_y$$

$$V_L = \frac{4 \cdot V_S}{\pi \cdot D^2}$$

where: D – diameter of the tunnel.

Considering those relationships, the function describing the settlement profile, for a single tunnel, can be expressed as:

$$s_{(y)} = \frac{V_S}{\sqrt{2\pi} \cdot i_y} \cdot \exp\left(-\frac{y^2}{2 \cdot i_y^2}\right) = \sqrt{\frac{\pi}{2}} \cdot \frac{V_L \cdot D^2}{4 \cdot K \cdot z_0} \cdot \exp\left(-\frac{y^2}{2 \cdot (K \cdot z_0)^2}\right)$$

Despite simplifications of this semi-empirical approach, it provides an expedient and reasonably representative prediction of ground surface deformation profile. The main relative advantage of this approximation is its simplicity and dependence of the deformation profile only on two uncertain parameters, assumed as potentially variable in a design, namely, the ground volume loss V_L and the trough width parameter K (related to the distance to the point of inflection). Franzius [39] described them as two crucial design parameters. Different values of those parameters are proposed in the literature based on large-scale observations from case studies documenting tunnelling experiences. As volume loss is considered as dependant on both the tunnelling technique and ground conditions [40], the trough width parameter K is generally considered as independent of the tunnelling method and dependant only on the type of the ground.

4. Variability of input parameters

Although calculation model assuming Gaussian-type displacement profile is quite convenient for calculation purposes, the main design problem lies in the assumption of representative values of parameters affecting the design; especially, when considering their highly empirical nature. The variability of those parameters cannot be ignored and caution should be exhibited when selecting the values in deterministic design framework. Some idea about variability of the parameters can be derived from scientific literature and published case studies, especially, those reported by the industry.

4.1. Observed variabilities in published case studies

Table 1 presents summary of trough width parameters and ranges of their variability, as reported in literature by various authors. In order to provide clear overview and allow a comparison, the values were divided into different ground types; provided division is an extension of four principal ground categories distinguished by Peck [4]. Such qualitative distinction, despite generalization and simplification, is useful for comparison of tunnelling experiences from different regions.

In the case of cohesionless soils (ground type 1), including sands and gravels, values of trough width parameter K can range from as low as 0.20 up to 0.60, usually falling between 0.25 and 0.45, and commonly averaging around 0.35. This can result in relatively narrow settlement trough, which in turn may lead to a small zone of significant displacements. In comparison, the cohesive soils (type 2) and hard to stiff clays (type 3) are characterized by K values from 0.35 to 0.90, commonly averaging around 0.50. In the case of hard to soft clays (type 4), the values are between 0.30 and 0.70. All those values are generally in line with K values reported in or back-analysed from specific tunnelling projects, as summarized in the database of worldwide case-studies compiled at Polish Building Research Institute (Fig. 2). The most significant variation is observed for cohesionless grounds (type 1), with K value averaging at approx. 0.31, where most cases fall within the range of 0.20 to 0.60. For cohesive granular soils (type 2), most cases resulted in K between 0.30÷0.60, averaging at approx. 0.44. In the case of clays, the values average at 0.49 and 0.47 for stiff and soft clays, respectively, in both cases falling primarily between 0.40 and 0.70. Table 1 additionally includes values reported for other, less common ground types, although the number of such case studies is limited and they are not further discussed in this paper. However, reported values can be used as initial source of reference for tunnelling projects at similar ground conditions.

The other important design parameter, i.e. volume loss, also may present significant variability, not only in various ground conditions, but also for different technologies of tunnel construction. EPB-TBM tunnelling, considered in this paper, can achieve low volume losses of less than 1% in variety of soils [38]. According to Mair and Taylor [6], in sands and gravels, more scatter is observed in data from various case studies than in the case of clays; in general, volume loss tends to be greater in cohesionless soils than in cohesive ones [40]. This is due to the fact that, in the case of EPB-TBM tunnelling, higher volume losses can be expected in grounds further from the range of its standard application, with insufficient conditioning, or under unfavourable conditions (e.g. occurrence of boulders). Table 2 reports some worldwide

Table 1. Examples of values of trough width parameters reported in the literature

Ground Type	Ground type classification	Specific ground type (if given)	K (at the surface)	Proposed by	Region (if given)
GT 0	Highly mixed	Fills	0.30	Mahdi et al. [41]	Paris, France
		Alluvium	0.30	Mahdi et al. [41]	Paris, France
GT 1	Cohesionless granular	Sands above water table	0.20÷0.30	O'Reilly and New [37]	UK
		–	0.25	Rankine [35]	–
		Sand	0.25	Sinclair et al. [42]	Singapore
		–	0.25÷0.35	Anagnostou and Kovári [43]	–
		–	0.30÷0.50	Simic and Gittoes [44]	Lisbon, Portugal
		Sands and gravels	0.35 (0.25÷0.45)	Mair and Taylor [6]	–
		Gravel/sand Loose – medium dense	0.25÷0.50	Fillibeck and Vogt [45]	–
		Gravel/sand Medium dense – dense	0.40÷0.60		
GT 2	Cohesive granular (general)	–	0.50	Glossop [46], Rankine [35]	–
		Glacial deposits	0.50÷0.60	O'Reilly and New [37]	UK
		–	0.40÷0.50	Anagnostou and Kovári [43]	–
GT 3	Clays (hard to stiff)	Stiff fissured clays	0.40÷0.50	O'Reilly and New [37]	UK
		Medium stiff to hard soils	0.40	Sinclair et al. [42]	Singapore
		London Clay	0.35÷0.50	New and Bowers [47]	London, UK
		Clay/silt Semisolid – solid	0.50÷0.90	Fillibeck and Vogt [45]	–

Table 1 [cont.]

Ground Type	Ground type classification	Specific ground type (if given)	K (at the surface)	Proposed by	Region (if given)
GT 3/4	Clays (general)	–	0.50 (0.40÷0.60)	Mair et al. [48], Mair and Taylor [6]	–
GT 4	Clays (hard to soft)	Soft silty clays	0.60÷0.70	O'Reilly and New [37]	UK
		Soft soils	0.70	Sinclair et al. [42]	Singapore
		Clay/silt Soft – Semisolid	0.30÷0.60	Fillibeck and Vogt [45]	–
GT 5	Organic soils	No case studies identified			
GT 6	Weathered rocks	Moderately weathered rocks	0.20	Sinclair et al. [42]	Singapore
		Residual soil (Granite)	0.50	Shirlaw et al. [49]	Singapore
GT 7	Soft rocks	Anhydrite and gypsum bearing claystones	0.12÷0.14	Egger [50]	Stuttgart, Germany
		Claystones	0.36	Egger [50]	Stuttgart, Germany
		Marls	0.50	Mahdi et al. [41]	Paris, France
		Limestone	0.55	Mahdi et al. [41]	Paris, France
GT 8	Hard rocks	–	No volume loss	Sinclair et al. [42]	Singapore
		Granite	0.30	Shirlaw et al. [49]	Singapore

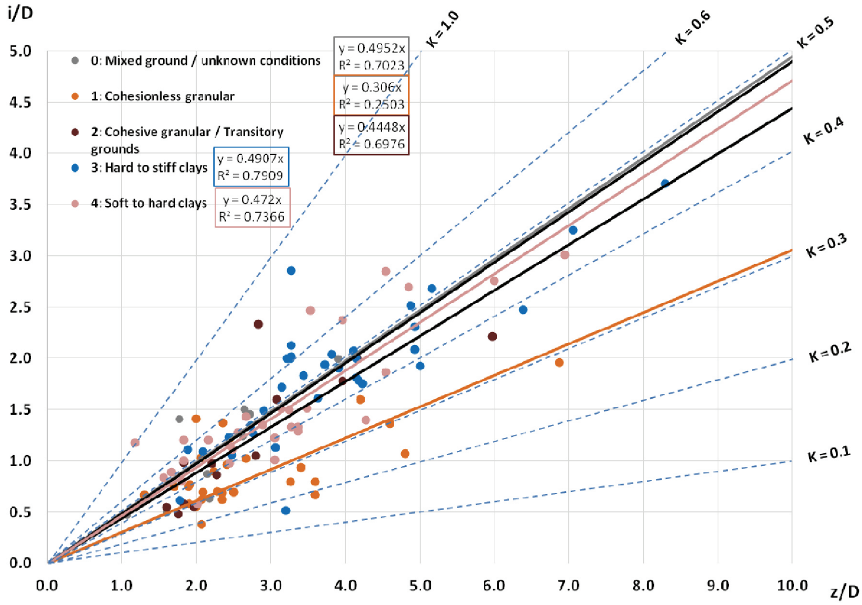

Fig. 2. Variability of trough width parameters K based on worldwide case studies (based on a database archived at Polish Building Research Institute)

case studies from tunnelling projects implementing EPB-TBMs. Even though trough width parameters (K) for different ground conditions show a lot of similarity, high variability in volume losses can be observed even for the same ground types. This might be caused by factors ranging from the specific characteristics of local ground conditions to differences in workmanship and tunnelling experience in different countries. In this context, the aspect of local experience should be emphasized and the publication of the reference case studies should be promoted.

As many published case studies are reporting only back-analysed values of the considered parameters, or show displacements at few specific cross-sections along the tunnel alignment, it is difficult to obtain a true view of the inherent variability. For that reason, data from two well-documented case studies were re-evaluated, where either displacements or parameter variations along the line were presented with statistically representative number of data points.

First case study is the Channel Tunnel Rail Link (CTRL) project from London reported by Bowers and Moss [58]. Based on the presented data and description, the variability of volume losses in the case of sands, clays, and at mixed conditions was derived and presented in form of histograms and cumulative probability functions in Fig. 3. Although derived volume losses for all types of grounds are of the same order of magnitude, some differences in the scatter of values are noticeable. However, what is the most important is the fact that the distributions seem to be skewed towards higher values, with occasional higher than usual volume losses occurring.

Similarly, a case study from Paris, reported by Mahdi et al. [41], despite the concentration of majority of values over a specific range, also shows some skew in the direction of higher values of all considered parameters.

Presentation of such detailed results from local case studies is necessary for providing rational guidance on selection of design parameters used in ground deformation predictions for future projects.

Table 2. Examples of worldwide case studies of EPB tunnelling with back-analysed parameters

No.	Year	City	Case study	Norm. depth (z/D)	Dominant grounds (assigned ground type)	Surface settlement [mm]	Vloss [%]	K [–]	Reference
1	1981	San Francisco	San Francisco Clean Water Project	2.5	Recent Bay Mud (4)	30	3.1	0.45	Clough et al. [51]
2	1994	Lisbon	Lisbon metro extension	3.1	Miocen sands (1)	–	1.0	0.40	Simic and Gittoes [44]
3	1995	Tokyo	Tokyo Metropolitan Subway No. 12 line	2.1	Sand and gravel with boulders (1)	5.6	0.14	0.33	Kanayasu et al. [52]
4a	1995	Madrid	Madrid subway – Line 10 extension	2.0	Sandy clays (3)	2	0.52	–	Melis et al. [53]
4b					Alluvial sands, muds (0)	48	3.0	–	
5a	1996	Taipei	Taipei Rapid Transit System, Hsintien Line, Section B1, Contract 218	2.6	Silty sand, silty clay (0)	35	2.6	0.57	Ou et al. [54]
5b				3.1	Silty clay (4)	33	2.0	0.33	
5c				3.1	Silty clay (4)	20	1.3	0.40	Moh et al. [55]
6	1997	Singapore	North East Line of Mass Rapid Transit (Contract C703-C710)	2.6	Old Alluvium, fluvial sands and marine clays (2)	–	0.50	–	Shirlaw et al. [49]
7	1999	Rotterdam	Botlek Rail Tunnel	1.9	Holocene and Pleistocene sand (1)	37	1.0	0.40	Netzel [56]
8	1999	Singapore	Northeast Mass Rapid Transit Line – Contract 704	3.8	Granitic residual soil (6)	17.5	1.38	0.44	Lim et al. [57]
9a	2002	London	Channel Tunnel Rail Link (CTRL) contracts no. 220, 240, 250	–	Mixed clays and sands (0)	–	0.62	–	Bowers and Moss [58]
9b					Sands (1)		0.37		
9c					London Clay (3)		0.60		
9d					Alluvium and peat (5)		0.68		
9e					London Clay (3)		0.48		
9f					Sands (1)		0.53		
10	2010	Paris	Metro line 12 North Extension	2.2	Sand, Marl (7)	5.0	0.16	0.46	Mahdi et al. [41]
11a	2012	London	Crossrail – Contract C300 – Hyde Park	4.9	London Clay (3)	12	1.2	0.42	Ieronymaki [59]
11b						7.6	0.78	0.47	

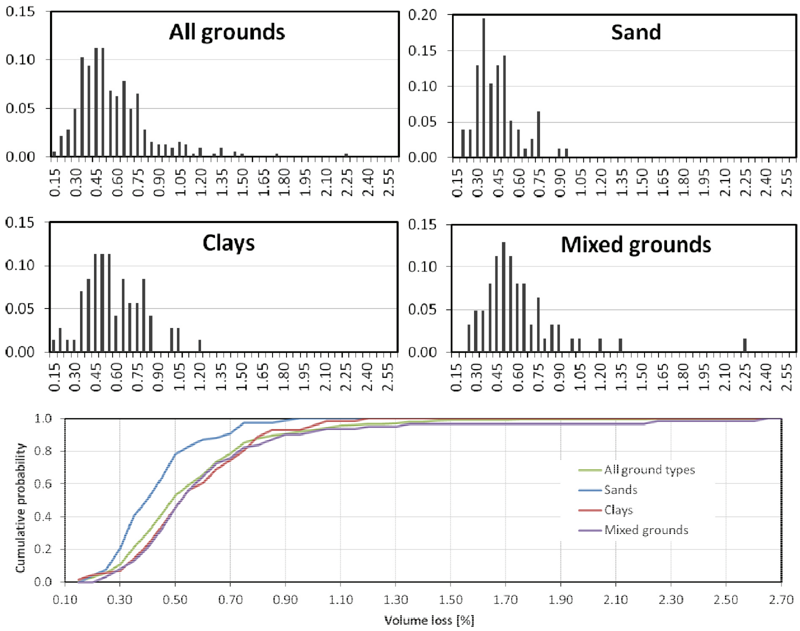

Fig. 3. Histograms and cumulative frequencies of occurrence of volume losses for different grounds for the EPB tunnelling case study from London [58]

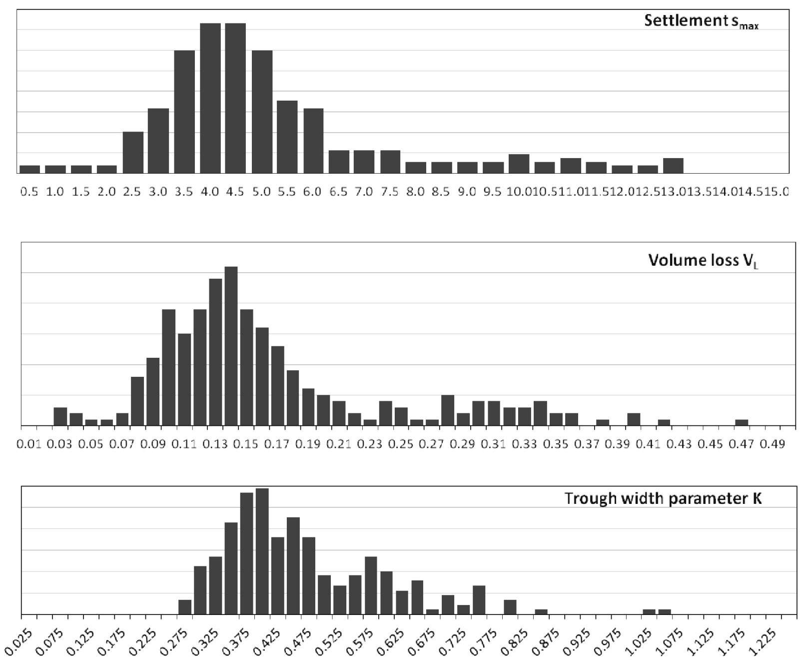

Fig. 4. Histograms of observed settlements and back-analysed parameters for an EPB TBM tunnelling case study from Paris [41]

4.2. Observed variabilities in Warsaw, Poland

In the case of Poland, with the number of tunnelling projects rapidly growing, acquiring and analyzing data from case studies is especially important. So far, the majority of tunnelling experiences were obtained from the area of capital city of Warsaw. In this initial study, only short-term settlements (up to 2 weeks from TBM arrival at a given cross-section) for a single tunnel were considered and analysed. The settlements observed above the centrelines of metro line tunnels constructed in Warsaw, with the use of EPB-TBM, were summarized in the form of histograms (Fig. 5) based on the monitoring data related to ground displacements (data from ground pins only). As the ground conditions are the primary factor influencing the observed deformations, distinction was made into three generalized ground classes, in line with the

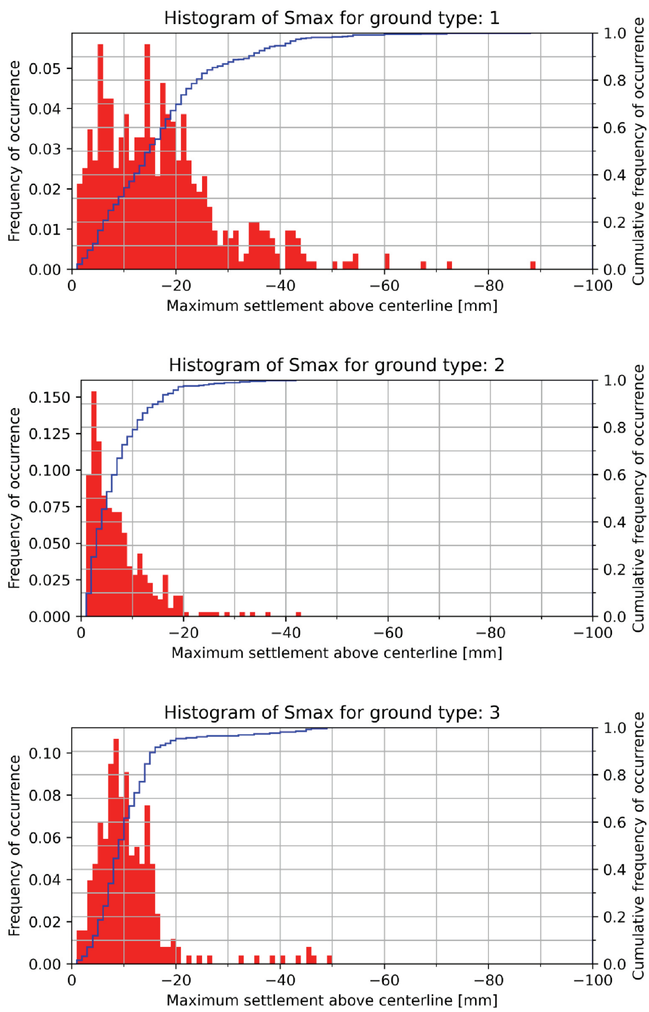

Fig. 5. Histograms of observed settlements due to EPB TBMs tunnelling in the region of Warsaw based on monitoring results from construction of M2 metro line tunnels

distinction made for the world-wide database before: type 1 – Quaternary cohesionless soils (sands, gravels); type 2 – Quaternary cohesive soils (glacial tills, silts, etc.); type 3 – Tertiary stiff Warsaw clays. The distinction was based on dominant ground conditions, primarily at the EPB-TBM face and in its vicinity above the tunnels. Presented settlements can be a basis for design predictions for future tunnelling projects implementing EPB-TBMs in the Warsaw area.

It can be observed (Fig. 5) that tunnelling in cohesionless soils resulted in significant variability of settlements at the ground surface. With approx. 80% in frequency of occurrence, settlements above centreline were up to 25 mm; this value of settlement can be considered as normal, expected settlement. With up to approx. 97% certainty, the settlements do not exceed 45 mm, which can be considered as the greatest expected settlements at some cross-sections. Higher settlements can occur occasionally, usually at the start of the TBM drive or due to non-routine events (e.g. encountering large boulders, etc.).

In the case of primarily cohesive Quaternary (glacial tills) and Tertiary (overconsolidated Warsaw clays) soils, the observed maximum settlements at the ground surface rarely exceeded 20 mm, with a limit of approx. 97% and 95% frequency of occurrence, respectively. However, occasional settlements, which can be considered as higher than expected, were also observed in those soils at some cross-sections. Despite that, the settlements due to EPB-TBM tunnelling in cohesive soils of Warsaw can be reliably predicted with high certainty. Further analysis is necessary to provide detailed assessment of variability of other design parameters, namely, volume loss and trough width parameter.

5. Discussion and conclusions

The paper provided an overview of the problem of prediction of ground displacements caused by EPB-TBM tunnelling. As semi-empirical approach implementing inversed Gaussian function is still the common method used in design practice, selection of appropriate values of design parameters is still a concern, as their variability may significantly affect obtained predictions. For example, when considering the trough width, estimated based on parameter K, it is not straightforward whether upper- or lower-estimate value might be more unfavourable. Lower value of the parameter (e.g. for cohesionless soils: $K = 0.20$) will result in prediction of a much narrower settlement trough (smaller zone of influence) but with steeper slopes of the deformation profile (more negative impact on structures located within the zone). Conversely, higher value of the parameter (e.g. for cohesionless soils: $K = 0.60$) will lead to a much wider zone, but with smaller expected impact in terms of imposed deformations.

In the case of volume loss (V_L), higher values (e.g. $V_L \geq 1\%$) will certainly be more unfavourable. However, a conservative choice of too high value of V_L will lead to overprediction of the damage to adjacent structures; therefore, it may lead to unnecessary works associated with strengthening or underpinning them, significantly increasing the overall cost of a tunnel construction. What is important to emphasize, in the context of displacement prediction in tunnelling impact analysis, is that the minor changes, such as small adjustments in design methods, may be accepted relatively fast, compared to establishing completely new calculation models. Therefore, it is often better to calibrate existing models, based on local empirically derived data from real scale projects. For that purpose, establishing reference databases and documenting local experiences is necessary.

Based on the initial analysis of compiled worldwide database as well as data obtained from tunnelling projects in the region of Warsaw, high variability of impact observed in the case of cohesionless soils still remains an issue and deserves further studies. In sands, the range of settlements for EPB-TBMs can be expected to be up to 25 mm with approx. 80% certainty, and 45 mm with approx. 97%. Whereas, for cohesive soils (e.g. glacial tills, clays), settlements rarely exceed 20 mm (with 95÷97% certainty).

At this point, summarized experience of observed settlements above tunnels executed with the use of EPB-TBMs in Warsaw, despite complex geology of the region (e.g. presence of boulders, spatial variability of strata), proved that this technology can be successfully implemented under various adverse conditions. Observed variations in parameters can form a basis for more rational, statistically based selection of design parameters for future tunnelling activities.

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Parametry stosowane przy ocenie niecki osiadań wywołanej drążeniem tuneli tarczami TBM

Słowa kluczowe: tunelowanie, EPB TBM, niecka osiadania, utrata objętości

Streszczenie:

Jednym z głównych wyzwań związanych z projektowaniem płytko posadowionych tuneli na terenach zurbanizowanych jest predykcja deformacji podłoża wywołana procesem ich drążenia. Zaawansowane

techniki realizacje takie jak zastosowanie tarcz zmechanizowanych TBM typu EPB pozwala na znaczną redukcję osiadań obserwowanych na powierzchni terenu, w porównaniu do technik stosowanych w przeszłości. Przewidywanie tych przemieszczeń jest oparte głównie o stosowanie modeli pół-empirycznych oraz wcześniejsze doświadczenia porównywalne. Poza samą prostotą tych metod, dekady ich stosowania w projektowaniu tuneli stanowią wystarczającą walidację skłaniającą projektantów do ich wyboru w analizach projektowych. W podejściu pół-empirycznym, niecka osiadania na powierzchni terenu wywołana tunelowaniem jest opisywana odwróconą funkcją rozkładu normalnego Gaussa. Wymaga to jedynie przyjęcia założeń odnośnie dwóch parametrów, mianowicie: spodziewanej utraty objętości (V_L) oraz odległości do punktu przegięcia (i_y), która zależy od empirycznego parametru charakteryzującego szerokość niecki (K) oraz głębokości tunelu (z_0). Parametry te mają charakter silnie empiryczny i powinny być przyjmowane w oparciu o doświadczenia porównywalne uzyskane w skali rzeczywistej przy realizacji tuneli w danej technologii i podobnych warunkach gruntowych. Niniejszy artykuł przedstawia problem zmienności wyżej wymienionych parametrów oraz rozważa potrzebę ich dokładniejszej oceny. Ponieważ utrata objętości jest silnie związana z technologią realizacji tunelu, praca koncentruje się na technologii EPB-TBM, jako najpowszechniej stosowanej w ostatnich latach. Przedstawiono podsumowanie zmienności parametrów obserwowanych dla różnych warunków gruntowych w różnych krajach. Na koniec, przedstawiono również wstępną ocenę zmienności osiadań obserwowanych dla obszaru Warszawy.

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