Copenhagen Cityringen Metro: EPB-TBM head pressure definition

Author: Giuseppe LUNARDI, Rocksoil S.p.A., Milan (IT), giuseppe.lunardi@fastwebnet.it
Co-author: Luca MANCINELLI, Rocksoil S.p.A., Milan (IT), luca.mancinelli@rocksoil.com
Andrea ZIMBALDI, Rocksoil S.p.A., Milan (IT), andrea.zimbaldi@rocksoil.com

Topic: Planning and Designing Tunnels and Underground Structures

Keywords: *TBM, support pressure, settlement, design revision*

The paper describes a relevant design aspect for the Copenhagen Cityringen Metro Line, which consists of two single-track tunnels, each approximately 16 km in length, 17 underground stations, 3 construction and ventilation shafts and an access ramp.
The tunnel of 5.8m of diameter is excavated with an EPB TBM and the tunnel ring is composed by 5+1 RC segments, 1.4m long and 0.3m thick.
Technological aspects and general problem solving are discussed and described. In detail, the effect of the design choices about the definition of the correct TBM support pressure (driving strategy) is analysed, by comparing settlement previsions and satisfactory monitoring results.
The analysis starts from the examination of the second excavated tunnel stretch, which is taken as an example of the typical geological/geotechnical conditions found in the Cityringen: the overburden varies from a minimum of 15-16m close to stations to a maximum of 25m and the excavation is both in Limestone and in Quaternary soils.

1. Introduction

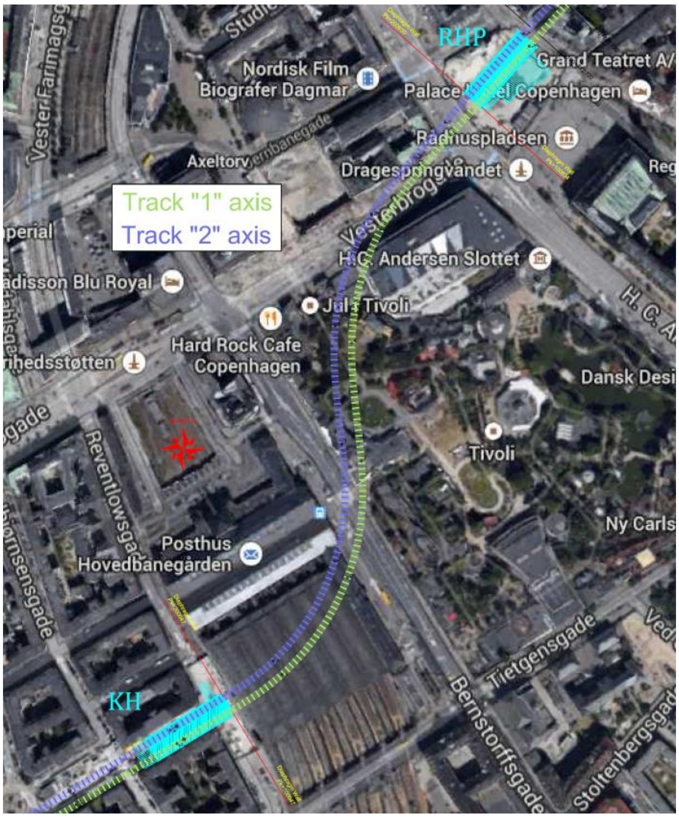

Fig. 1 *– Typical Cityringen tunnel stretch*

Copenhagen Cityringen Metro Line consists of two single-track tunnels, each of them approximately 16km long, 17 underground stations, 3 construction and ventilation shafts and an access ramp.
The overburden of the tunnel varies from 10-15m to 25-35m passing from highly overconsolidated Quaternary soil to the competent Copenhagen Limestone bedrock.

The selected Tunnel Boring Machine (TBM) is an Earth Pressure Balanced (EPB) Kawasaki, with a head diameter of 5.8m, and the tunnel ring is composed by 5+1 segments, 1.4m long and 0.3m thick, made of concrete C40/50 and steel bars B500b.

In the area affected by the excavation there are lots of buildings worthy of preservation, some of them made on piles, existing railway and metro lines and other underground utilities, such as a heating tunnel.
Fig. 1 shows the tunnel stretch in which TBMs will excavate below Tivoli Park and Copenhagen Central Station.

2. General aspects of the design of a mechanized tunnel excavation

The realization of a tunnel in an urbanized area is a challenging construction, with geological/ geotechnical conditions continuously varying with the TBM advance.

In general the design of the permanent structure does not present particular problems. In facts, thanks to its circular shape, the lining is mainly subjected to axial load and normally the minimum normative requirement for reinforcement is enough to bear the small bending moment and, for this, in some projects fiber-reinforced concrete is preferred to typical reinforcement bars.
Most demanding conditions for the lining segments are the transportation phase from the manufacturer site to the construction area and the TBM advance phase, with the ring loaded laterally by the pushing rams. When the designer analyses adequately these load conditions and a good level of precision is assured during the whole segments cycle, from production to installation, the tunnel structure can bear easily most of the live loads of its life cycle.

Underground transportation is a fast, green and easy way to connect different parts of a city, in new growing areas like in highly urbanized zones. Underground works allow the construction of these infrastructures and need to be developed reducing at maximum the impact on people and on what existing. The management of the works under buildings and structures in general represents then the most important aspect to be taken into account.
Neglecting subjects like vibration and noise, the main objective related to the realization of a new metro line is to limit the settlements of buildings, infrastructures and utilities in the zone affected by the excavations.

2.1 General elements on TBM induced settlements

In general, the subsidence profile related to tunnel excavation can be obtained with detailed 3D/2D FEM models or with empirical formulas, as described by Attewell and Woodman in [1] and O' Reilly and New in [2], assuming the value of some parameters from bibliography.
Normally, evaluations with empirical formulas are conservative and monitored settlements are lower than foreseen ones. This is an embraceable way to proceed with the design, considering the TBM is not excavating in a perfect homogeneous material and unexpected events, such as a support pressure loss, can always happen.
Talking about EPB TBM, every study is oriented in the direction of the definition of the parameters that can reduce settlement impact, said head and backfilling injection pressures [7]. Considering that the second one is a function of the first (0.5 to 1.0 additional bars) the problem is focused on the definition of the right value of the TBM support pressure S_{TBM}, an "equilibrium" between settlement control and working performance.
Once defined the adequate S_{TBM}, the backfilling pressure is rapidly obtained, as for the volume to be injected, evaluated with a brief calculation of the space of the annular void to be filled. On the operative side, some tests are necessary on the bi-component injection mix, that must be able to give a confinement to the mounted ring in a short time, without loss of material.

3. Dual analysis of the deformative problem

The definition of the adequate S_{TBM} is needed to locally assure the excavation face stability and to globally limit the induced settlements up to the ground level.

A rapid approach to analyse this problem in its complexity is to use an axisymmetric model, that is appropriate for excavation at high depth. In the Cityringen project, the tunnel is mainly shallow, with an overburden equivalent to 2-3 tunnel diameters, and the ratio between horizontal and vertical stresses at excavation level does not allow the adoption of any axisymmetric model.

3D models would represent the best solution to analyse completely the TBM advance, with the possibility to introduce buildings and infrastructures loads spatially but this approach is time consuming and it's suggested for critical passages and not for the study of a whole metro line of more than 10km. In addition, 3D models depend on some hypotheses and on the geotechnical characterization: the validity of the results is related to how much the soil behaviour, monitored during the tunnel excavation, differs from its engineering representation with a schematic constitutive law.

The reduction of the geostatic load effectively acting on the lining after the TBM advance and the disposition of the ring, automatically calculated in a 3D model, can be well simulated in a 2D model considering specific curves, in accordance, for example, with Panet [5]. This method refers to deep tunnels and is not applicable to shallow tunnels where, in addition, soil and rock layers can be found at the tunnel face contemporaneously.

Therefore, for Cityringen project it was chosen to split the analyses between

- the definition of the TBM support pressure S_{TBM} to assure face stability and
- the evaluation of the settlements induced by the TBM excavation (deduced with the empirical formulation [1], considering the gaussian curve obtained with the parameters shown in Table 1).

Table 1 *– K parameter and Volume loss V_L*

Soil type	Design Parameters		
	K [-]	V_L [%]	
		Most Likely	Worst Credible
Quaternary soil	0.35	0.5	1.0
Mixed face condition	0.35	0.5	1.0
Limestone	0.60	0.1	0.3

Building Damage analysis has been carried out in accordance with Boscardin and Cording [3] and Mair, Taylor and Burland [4], considering the subsidence profile obtained in the Most Likely condition, while Worst Credible scenario has been used as a support for the evaluation of critical passages.

The calculation of S_{TBM} has been refined both in the design phase and during the construction phase.

4. The definition of EPB-TBM head working pressures

After a detailed analysis on geologic, geotechnical and hydraulic scenarios, TBM pressures design approach followed for Copenhagen Cityringen was based on equilibrium formulas. This choice was done as to fit at best local conditions, taking into account underground formations properties and hydrostatic load.

Initially, to limit perturbation and to proceed with the safest condition, S_{TBM} was assumed equal to the at-rest horizontal pressure S_{K0}, related to the vertical effective pressure S'_V by the K_0 coefficient and to the hydrostatic pressure S_w:

$$S_{TBM} = S_{K0} = S'_{K0} + S_w = K_0 \cdot S'_V + S_w$$

After investigations, two main aspects had to be considered like weak points in the definition of S_{TBM}:

- the high overconsolidation of soil with OCR up to 8-10 brought to a wide range in the definition of K_0, as shown in Table 2;
- at design phase different levels of water table were defined for the analysis, as shown in Fig. 2, and only at the time of the excavation it was possible to check the water table level with piezometers.

The second tunnel stretch, excavated from Nørrebros Runddel Station to Nuuks Plads Station, Fig. 2, gives evidence of the limit of the method: the overburden varies from a minimum of 15-16m close to stations to a maximum of 25m and the excavation is driven both in Limestone and in Quaternary soils.

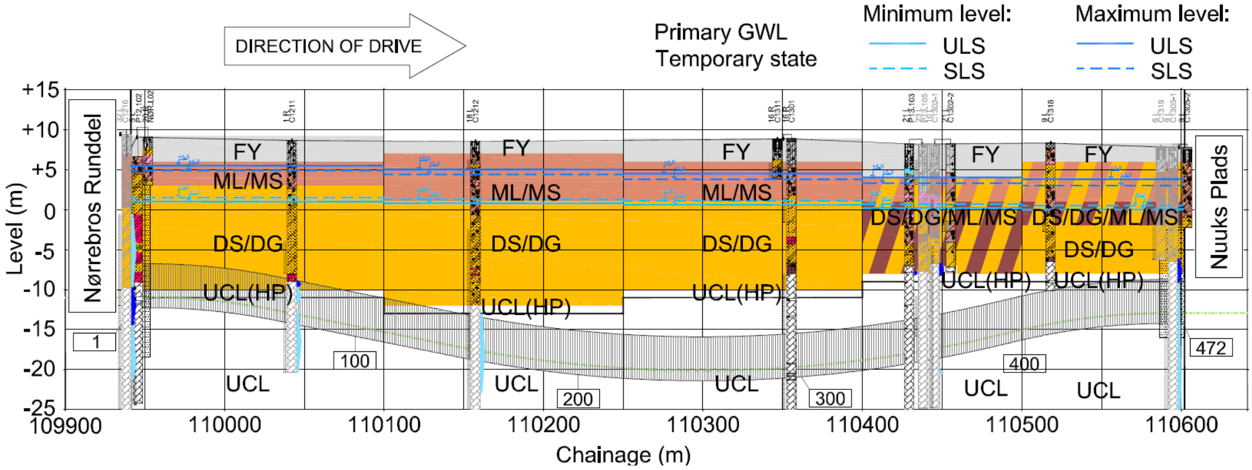

***Fig. 2** – NOR-NUP tunnel stretch*

***Table 2** - Geotechnical parameters – SLS characteristic values*

Geotechnical unit		FY	Quaternary soil		Limestone		
			ML/MS	DS/DG	UCL(HP)	UCL	MCL
Soil weight	γ_{sat} *[kN/m³]*	19.0	22.0	21.0	21.0	21.0	21.0
Friction angle	*ø [°]*	25	32	38	45	45	45
Effective cohesion	*c' [kPa]*	0	10	0	100	100	100
At-rest coefficient	K_0 *[-]*	0.55	0.71 1.20	0.52 1.20	0.70 1.70	0.60 1.70	0.60 1.60
Elastic modulus	*E [MPa]*	2	230	300	800	1500	1500
Poisson's ratio	*ν [-]*	0.33	0.30	0.25	0.30	0.30	0.30

The chosen method brought to an overestimation of S_{TBM} in the zones at higher depth, with the excavation fully in limestone, made by UCL stiff rock, where no problems related to hole stability and negligible tunnel convergence were expected. Even without considering the upper limit of the K_0, a useless support pressure up to 3.5/4bar for excavation in limestone was obtained and this would have brought to an untenable high wear of the TBM cutting tools.
Therefore it was decided to follow a different approach: Kovari's one [6], already applied in other projects with satisfactory results. Even if based on a simplified formula and graphs for the determination of some coefficients, this method considers the 3D arch effect around the excavation. The effective TBM pressure S'_H is given by the formula:

$$S'_H = F_0\,\gamma'\,D - F_1\,c' + F_2\,\gamma'\,\Delta h - F_3\,c'\,\Delta h/D$$

with: $\Delta h = h_0 - h_{EPB}$
h_0 = elevation of water table
h_{EPB} = piezometric head of the chamber
c' = soil cohesion
F_0, F_1, F_2 and F_3 = dimensionless coefficients that depend on the friction angle ø, on the geometric parameters H (overburden height) and D (tunnel diameter), on the ratio (h_0-D)/D and on the ratio between dry and submerged unit weights.

Imposing no filtration forces (Δh null), with high c' and ø for Limestone, F_0 is less than 0.2 and S_{TBM} is slightly higher than S_w:

$$S_{TBM} = S'_H + S_w = F_0\,\gamma'\,D - F_1\,c' + S_w \approx S_w$$

Kovari's formula defines the lower boundary for the equilibrium and a coefficient has to be applied to increase the safety factor, in particular where excavation is in cohesionless soil and the tunnel overburden is low.

Verification of the face stability during tunnel excavation has to be carried out in accordance with Eurocode in A2-M2 combination (reducing geotechnical parameters by the factor 1.32, in accordance with DS EN 1997-1 DK NA).

***Table 3** – Reduced geotechnical parameters*

Geotechnical unit		DS/DG	UCL(HP)	UCL
Characteristic value	*c' [kPa]*	0	100	100
	Ø [°]	38	45	45
A2-M2	*c' [kPa]*	0.0	75.8	75.8
	Ø [°]	30.6	37.1	37.1

S_{TBM} values obtained with this last approach were applied during the excavation of the first tunnel stretch. At the beginning, with the tunnel at a depth of 20-30m fully excavated in limestone, settlements less than 1-2mm where registered at ground level, in accordance with [1]. In the final part of the tunnel stretch, with the upper portion of the tunnel being progressively in quaternary soil, settlements higher than 1cm were recorded.
Some settlements were related to a TBM stoppage but the general trend of the settlements in the area with regular TBM advance was close to the foreseen settlements in most likely condition.
Even if a ground settlement up to 1cm is typical for a TBM excavation at low overburden with 0.5% of V_L, it was decided to increase S_{TBM} in the zones excavated in quaternary soil and mixed face condition, to avoid damages to the buildings in the influence zone.

For the second tunnel of the first stretch and for the following stretches, the application of Kovari's approach was limited to the zone excavated fully in limestone and in the other zones the S'_H was taken between the S'_{K0} and the S'_{KA}, the effective active pressure, equal to:

$$S'_{KA} = K_A \cdot S'_V - 2\,c' \cdot (K_A)^{0.5}$$

with: $K_A = (1\text{-sen } Ø) / (1\text{+sen } Ø)$

In mixed face condition, S'_{K0} and S'_{KA} were calculated for all the strata encountered at the tunnel depth separately, without introducing an average pressure: this fluctuation can be seen in the first part of the graph in Fig. 3.

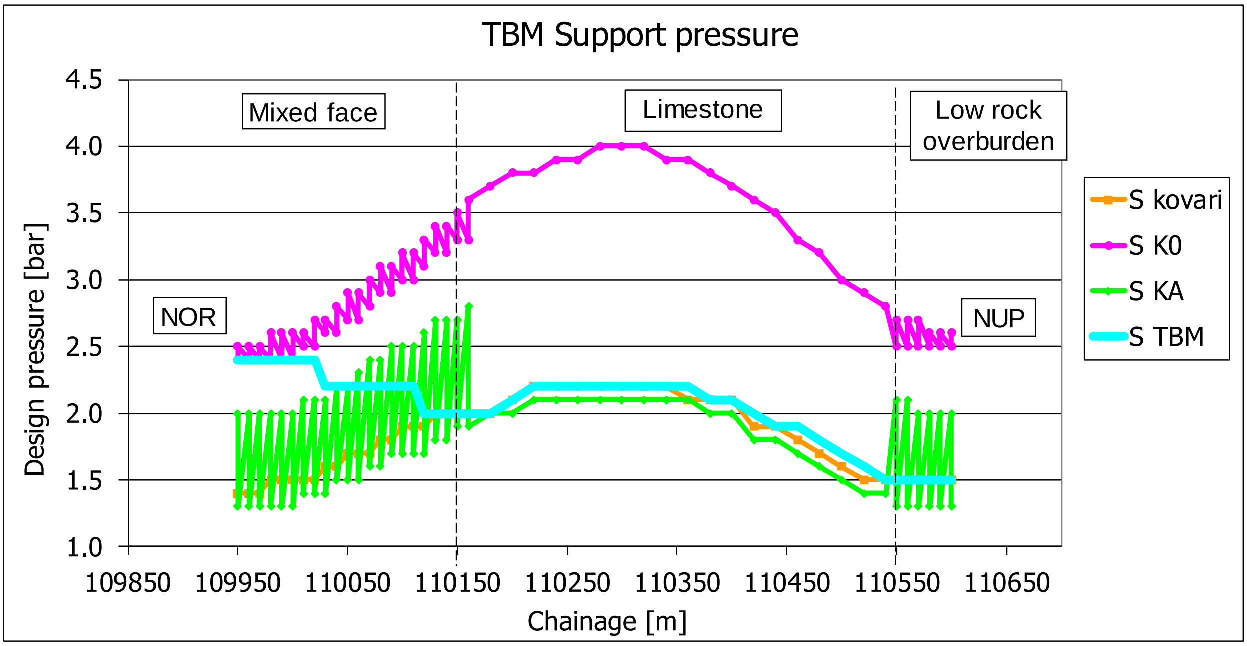

***Fig. 3** – Approaches comparison for the definition of the Total Support Pressure – NOR-NUP – Track1*

In Fig. 3, being S_w the same for the different approaches, Kovari's S'_H is similar to S'_{KA} for the excavation in limestone and unsafely lower than S'_{KA} in mixed face condition.
In the first part of the stretch, considering the operations for the break-out from the station, S_{TBM} has been taken equal to at-rest pressure to limit perturbation in the cohesionless DS/DG unit. S_{TBM} has been reduced progressively to the active pressure boundary with the tunnel deepening in the limestone, while the at-rest pressure was increasing to values two times higher than the adopted ones. In the last part of the stretch is has been verified that Kovari's value was adequate also where rock overburden above the tunnel was about 1-2m.

It's a common practise to calculate S'_H as an average value between S'_{KA} and S'_{K0}, but there are some variables which can affect significantly the result:

- the evaluation of S'_{KA} and S'_{K0} depends on the geotechnical characterization, in particular K_A and K_0;
- for shallow tunnels, the geostatic load can differ from free field condition, due to high building loads or foundations close to the excavation;
- S_{TBM} is compared to the initial geostatic load, but at the TBM head part of the geostatic load has been already lost [5];
- S'_{KA}, S'_{K0} and S_w are linked directly to the definition of the watertable.

The more the soil has a good characterization, the more the hydrostatic load S_w is the main load: for the calculation of S_{TBM} it's advisable to consider the maximum level of the groundwater expected during the TBM passage, for this project equal to the maximum level at temporary phase in ULS state.
For a better definition of the hydrostatic load at the tunnel face, a filtration analysis could be useful, considering the real TBM advance and the permeability of the encountered strata.
For the variables listed above and the fact the TBM advance is continuously changing, with short stoppages during ring mounting phase and long stoppages for TBM maintenance, the filtration analysis has only a theoretical validity.

The chosen design approach has been sustained by:

- a real-time monitoring of the settlements at ground level and of the buildings in the influence zone, by the use of 3D pins and total station;
- the control of the level of the primary and secondary watertable, with twin piezometers;
- the control of the pressure sensors installed on the TBM head.

With an increased accuracy in the backfilling phase, the refined procedure for the calculation of S_{TBM} has allowed to monitor settlements normally lower than 5mm for excavation in mixed face condition/quaternary soil in the following 5+5Km of the Cityringen project already excavated at the time this paper is written.

5. Verification of the adopted approach in a critical passage

The TBM excavation of the fourth tunnel stretch, from Aksel Møllers Have Station to Frederiksberg Station, gave additional demonstration of the effectiveness of the adopted method.
The overburden varies from a minimum of 15-16m close to stations to a maximum of 22m [Fig. 4]. The initial and final parts of the tunnels are in mixed face condition and the TBM passes few meters below the Frederiksberg Center, a relevant shopping mall, and the existing Frederiksberg station of M1 and M2 metro line.

Approaching Frederiksberg station, S_{TBM} has been taken between the active pressure and the at-rest pressure, considering minimum K_0 [Fig. 5]. It was already demonstrated that an excessively high pressure does not reduce further the settlements and there is also the possibility of blow out, mainly when the tunnel overburden is low and excavation is in cohesionless soil.
In this TBM passage there where some critical aspects. The excavation passed below the basement level of the Frederiksberg Center and S_{TBM} has been evaluated considering two opposite boundary conditions: to limit the building settlements, giving an adequate support to the plinth's foundation, and to avoid heave and cracks in the weak concrete bottom slab of the basement.

After the passage of the TBM, settlements lower than 2mm have been registered on the ground, in the shopping mall and in the existing metro station. A volume loss V_L of 0.1% has been obtained for the part excavated in limestone and a V_L lower than 0.2% for the remaining parts in mixed face condition: these good results are better than the most likely V_L of 0.5% used in the design phase for the mixed face condition.

Table 4 *– Geotechnical parameters of the layers directly affected by the excavation – SLS characteristic values*

Geotechnical unit		Quaternary soil		Limestone	
		DL	ML/MS	UCL(HP)	UCL
Soil weight	γ_{sat} [kN/m^3]	21.0	22.0	21.5	21.5
Friction angle	ø [°]	27	34	45	45
Effective cohesion	c' [kPa]	15	20	100	100
At-rest coefficient	K_0 [-]	0.71 – 1.20	0.71 – 1.20	0.77 – 1.70	0.77 – 1.70
Elastic modulus	E [MPa]	130	320	800	1500
Poisson ratio	ν [-]	0.25	0.30	0.30	0.30

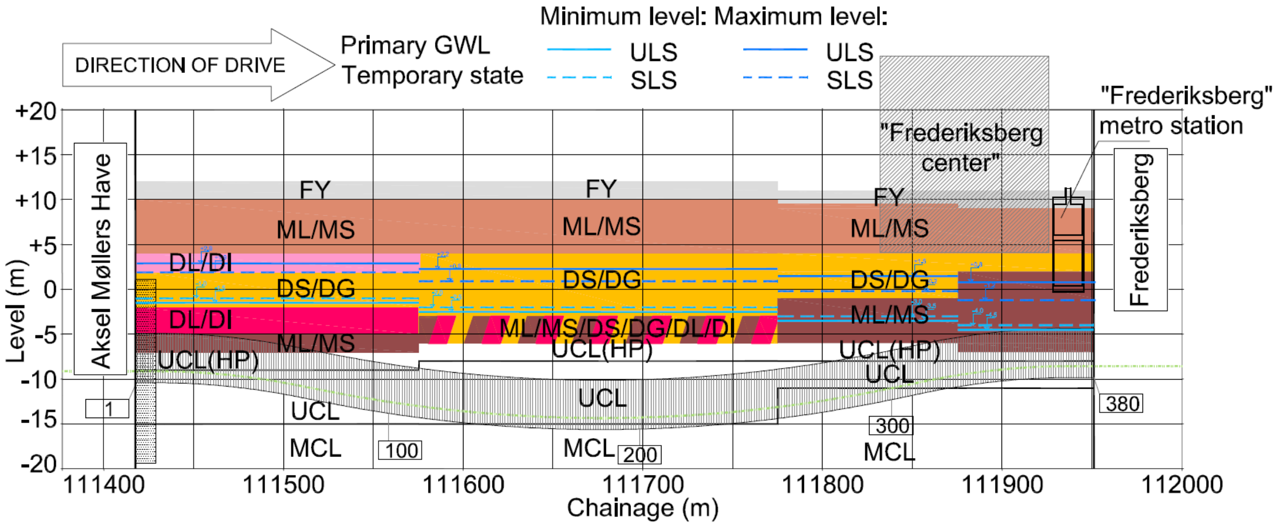

Fig. 4 *– AMH-FB tunnel stretch*

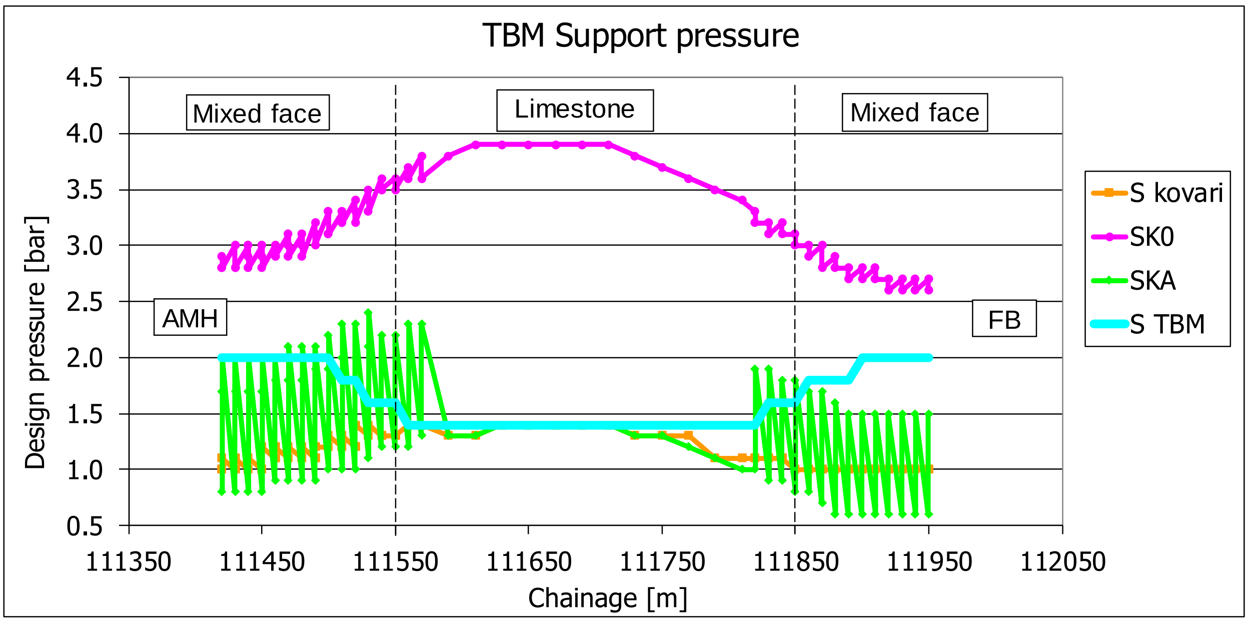

Fig. 5 *– Approaches comparison for the definition of the Total Support Pressure – AMH-FB – Track1*

5.1 Comparison theoretical and monitored settlement curves

Settlements of the ground and of the buildings affected by the TBM passage have been registered during the excavation of Tracks 1 and 2.
In addition to the automated 3D real-time monitoring, which shows a fluctuation of 1-2mm during a day cycle, a more precise daily reading is taken. With these last values, settlement curves are created and compared with theoretical ones [Fig. 6].

In both excavation phases, vertical displacements were below the theoretical values, [Fig. 6].

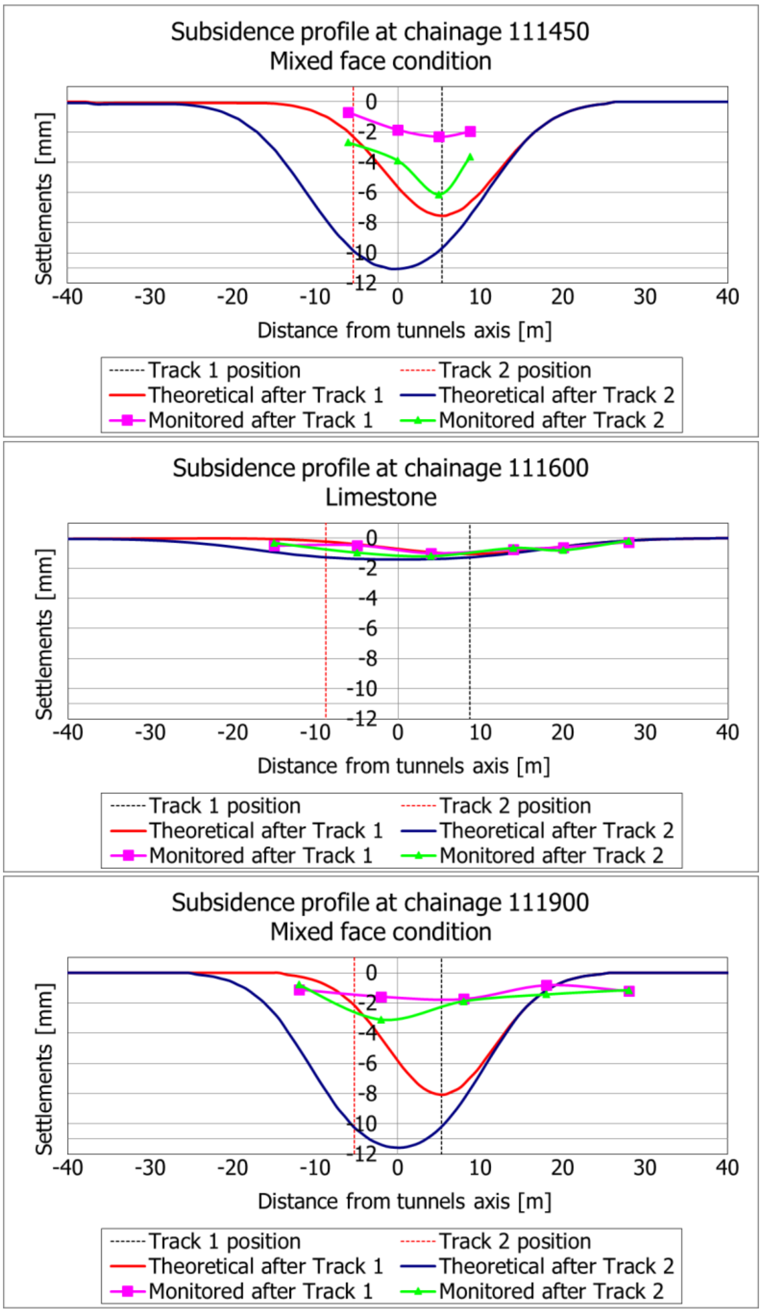

***Fig. 6** – Subsidence profiles comparison in AMH-FB tunnel stretch*

With the integration of the area defined by the interpolated subsidence curve on the monitored values, it's possible to calculate the effective V_L with reference to the tunnel area.
In Limestone, V_L is lower or about 0.1%, as for theoretical assumption: it has to be underlined registered values are less than 2mm, which is practically negligible for a tunnel excavation.
In mixed face condition, a V_L of about 0.2% has been obtained in the AMH-FB tunnel stretch, opposed to the expected theoretical 0.5%.

With reference to their evolution, settlements start with the TBM approaching and most part of them are already registered just 1 day after the passage of the TBM [1]: with an advance rate of about 15m/day, it is difficult to estimate at which phase of the tunnel realization the settlements refer to. Normally the backfilling injection stops the deformative process at the tunnel boundary, but the presence of cohesive soil strata delays the transmission of the displacements from above the tunnel crown to the ground surface.

As shown in Fig. 7 for a section in mixed face condition, after the ring erection in track 1 and the backfilling injection, the monitoring point "above track 1" and the lateral two ("between tracks" and "3m external to track 1") register an instantaneous vertical settlement, followed by an increase of about 50% in about two months until a new equilibrium condition.

It has to be underlined all the monitoring data have to be filtered adequately, taking the right zero measurement and excluding previous vertical settlements not related to TBM excavation. In Fig. 7, the monitoring point "above track1" registered 2mm from the installation to about 1 month prior to TBM arrival and this settlement has been taken out from the final value, which is about 6mm, as shown in first graph of Fig. 6.

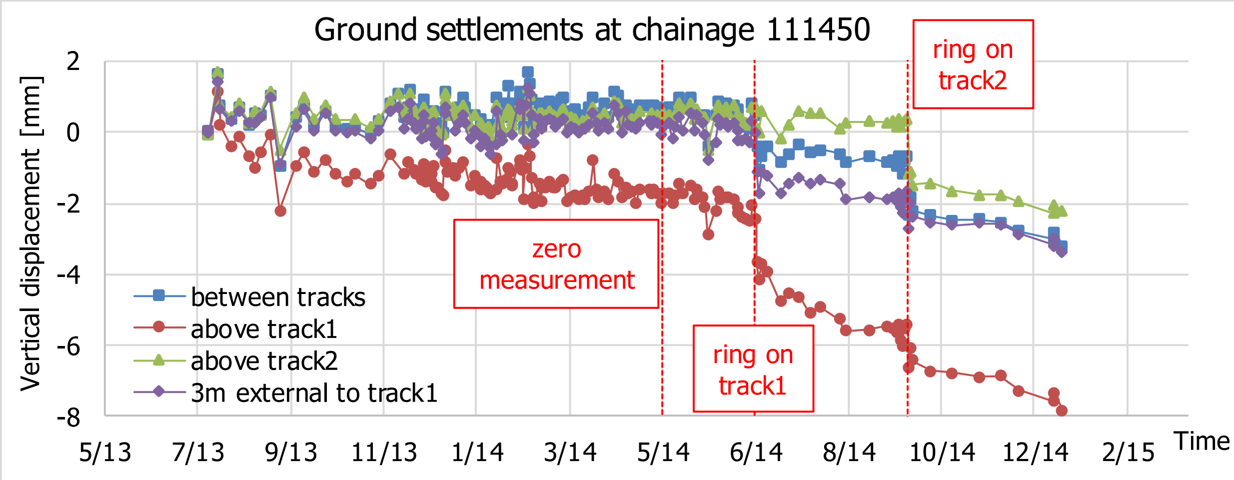

***Fig. 7** – Long term settlement evaluation*

6. Conclusion

What happens during a TBM tunnel excavation is a complex 3D problem with high variability and lots of uncertainties. To manage the amount of induced settlements, the designer has principally the possibility to define an adequate S_{TBM}. Many authors proposed different procedures to calculate it and every method give its best in some ground conditions. The designer must then make a choice and refine assumptions and results on the basis of the monitoring data, during the so called "learning phase".

As required by the Standards, the definition of S_{TBM} has to comply with the verification of the tunnel stability and be completed by upper and lower alert and alarm limits.
As in other aspects involving ground excavation and stability, there is a difference between the designed value for S_{TBM} and the registered ones; pressures at the TBM head are continuously varying during the working stages, as shown in Fig. 8 where also limits are reported (see Table 5). Even with this fluctuation in volume and in pressure at the front face and in the backfilling phase, an adequate S_{TBM} can make the TBM excavation proceed with limited induced settlements.

***Table 5** – S_{TBM} upper and lower alarm ratio*

Upper Alarm limit	Upper Alert limit	Lower Alert limit	Lower Alarm limit
+30%	+20%	-10%	-15%

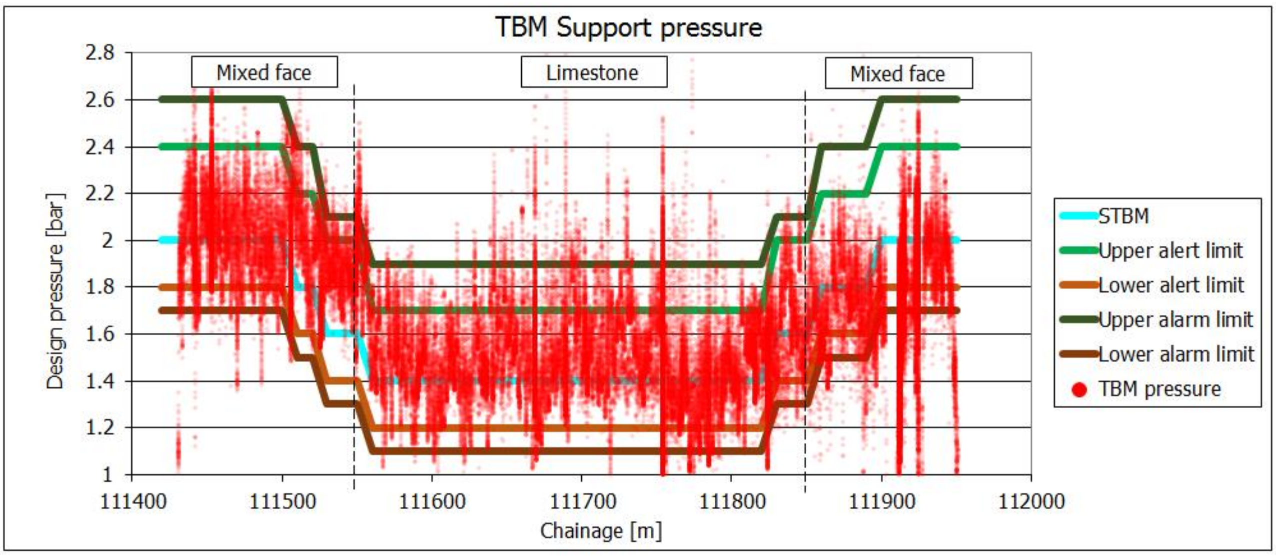

***Fig. 8** – S_{TBM} and TBM head sensor pressure comparison*

References

[1] Attewell, J. H. & Woodman, J. P. 1982. Predicting the dynamics of ground settlement and its derivatives caused by tunnelling in soil. Ground Engineering, 15, 13-26.

[2] O'Reilly, M. P. & New, B. M. 1982. Settlement above tunnels in the United Kingdom – their magnitude and prediction. Tunnelling 1982. IMM London 173-181.

[3] Boscardin, M. D. & Cording, E. G. 1989. Building response to excavation-induced settlement. Int. Geo. Egrg. Asce, 115, 1, 1-21.

[4] Mair, R. J., Taylor, R. N. & d, J. B. 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunnelling. Geotechnical aspects of underground construction in soft ground. Mair & Taylor, Rotterdam Balkema.

[5] Panet, M. & Guenot, A. 1982. Analysis of convergence behind the face of a tunnel. Tunnelling 1982. IMM London 197-204.

[6] Anagnostou, G. & Kovari K. 1996. Face stability condition with Earth-Pressure-Balance Shields. Tunnels and Underground space Technology, 11, 2, 165-173.

[7] Gatti, M. C., and G. Cassani. Ground loss control in EPB TBM tunnel excavation. Underground space - The 4th dimension of metropolises (2007).