# Warsaw Metro Line 2: TBM Excavation in difficult condition: The underpass of Vistula River

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ABSTRACT: the central stretch of the Warsaw Metro Line 2 has an extension of 6.3 km with 7 stations. The paper describes the underpass of Vistula River, by a minimum overburden of 7 m and a total excavation length of 300 m, which has been the most critical of several significant underpasses dealt during the TBM excavation. The collected geotechnical data and bathymetric surveys have been necessary to check the absence of tunnels buoyancy. The soil masses behaviour and the TBM parameters have been defined according to the A.DE.CO-RS design approach. During the excavation the soil masses behaviour has been checked by back analyses carried out on the results collected in real time in the tunnel sections close to the river. Continuous check of the machinery excavation parameters showed the respect of the design expectations. The TBM's excavation has been realized in safety conditions, without technical problems and respecting high rate of production.

## 1 INTRODUCTION

Warsaw, the capital of Poland with approximately two million inhabitants, has two underground metro lines. The second line (Line 2), which will have 22 stations distributed along a route of approximately 30 km, is under completion. The central stretch of this line (see Figure 1) has been executed with a turnkey contract awarded to the international AGP Consortium, formed by contractors ASTALDI (Italian), GÜLERMAK (Turkish) and PBDIM (Polish).

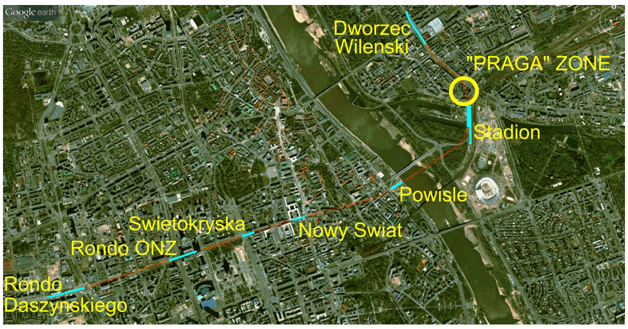



Figure 1. Warsaw Metro – Central section of Line 2

The scope of work for ROCKSOIL S.p.A. in the project was to carry out the design and to provide technical assistance on site during tunnel construction and performance of all the related works, including those reported in this paper.

The central section of Line 2 is 6.3 km long and has seven stations between the two terminus stations at Rondo Daszynskiego and Dworzec Wilenski. These twin bore tunnels were driven by using four TBM-EPB (tunnel boring machine-earth pressure balance) with a diameter of 6.3 m.

The metro Line 2 underpasses several buildings, structures and infrastructures. A number of difficult underground passages have been realized along the route e.g. the passage below the historical zone of Praga buildings (see Figure 1) and the passage below the existent Line 1 of the Metro, located between the Rondo ONZ and Swietokrzyska stations, with less than 3 m of soil between the existing tunnels and the new tunnels

The paper discusses the underpass of Vistula River, which has been the most critical issue of the Line 2.

The Vistula River underpass is placed between the station Stadion and the station Powisle (see Figure 1).

The first tunnel excavated has been the left (L) tunnel, on the right side of the excavation's direction, from Stadion to Powisle station.

The length of the left track under exam is 919.90 m, while the right track is 934.54 m long.

The minimum overburden is 7 m, while the bed of the Vistula River has here a width of 300 m (see Figure 2).

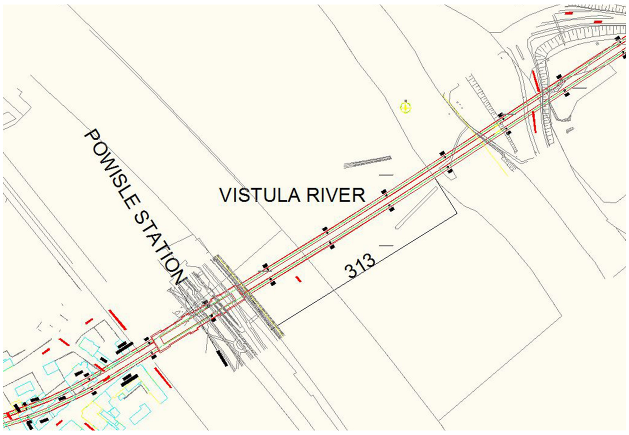


Figure 2. Warsaw Metro – Vistula River width

Preliminary to the excavation, a campaign of geological, geotechnical and bathymetric surveys of the Vistula River bed has been executed. The collected geotechnical data and bathymetric surveys have been necessary to check the absence of tunnels buoyancy during and after the TBM excavation.

During the underpass of the river it was not possible to measure the settlements of the natural ground surface. Therefore, before the river underpass, it was essential to conduct an accurate analysis of the TBM parameters and of the ground surface monitoring data collected in the previous tunnel stretches, and particularly in those most close to the river.

This back analysis confirmed the validity of the design assumptions and permitted to fine tune the TBM excavation parameters for the under passage of the river.

The design prevision has been respected with a high tunnel excavation production rate. The mechanized excavation has been performed in safety conditions without any technical problem. The long-term behaviour of the tunnel has been evaluated with respect to low overburden. The analysis of the historical data of the Vistula bed erosion has been used in the numerical model prepared to evaluate the tunnel's possible buoyancy.

## 2 SURVEY PHASE

A campaign of geological, geotechnical and bathymetric surveys on the bed of the river has been executed before starting the excavation, in order to collect the necessary information for the design.

The survey phase was essential for many reasons. First of all in order to understand the behaviour of the river water table, in term of maximum and minimum level, and then in order to check the seasonal fluctuation of the water level of the Vistula River. In some seasons, usually in May, the level can increase up to 10.00 m. The second study regarded sediment/debris movement in the Vistula River bed in order to prepare the "Geological-engineering" documentation for the II underground Line – passage under the Vistula River. This study was also important in order to verify relevant changes in the depth of the River's bed.

Both these issues can be relevant in term of stress on the tunnel final lining, and in term of buoyancy of the tunnel along the entire Vistula River underpass. All the collected data have been used to define the TBM parameters according to the A.DE.CO-RS design approach (Lunardi, P., 2006) in order to get a safe underpass of the river.

### 2.1 *Geology and Geotechnics*

Along the tunnel alignment the soils interfered by the excavation were mainly soils of alluvial origin (sands and clays). The overburdens were included between 4.2 and 16.9 meters. The water head varied from 2.8 up to 11.6 meters over the crown of the tunnel. The Figures 3 and 4 show a description of detected different soils and lithology, which interfered with TBM excavation.

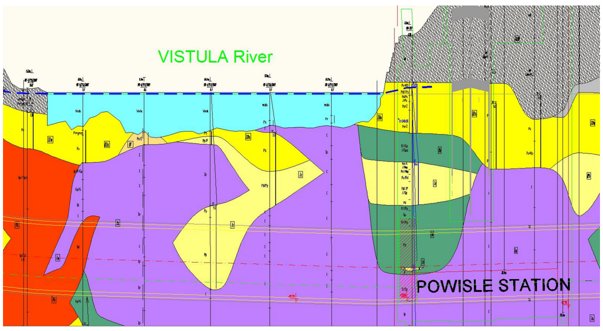
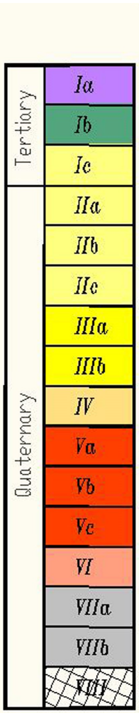


Figure 3. Geology




| Number of layer | Prevailing kind of soil (acc. PN-86/B-02480) |
|---|---|
| Ia | High plasticity clays, high plasticity silty clays and more plastic clays |
| Ib | Clays, silty clays and silts |
| Ic | Medium, fine and silty sands |
| IIa | Fine and silty sands |
| IIb | |
| IIc | |
| IIIa | Medium and coarse sands |
| IIIb | |
| IV | Gravels and sand-gravel mixtures |
| Va | Clays and Sandy clays |
| Vb | |
| Vc | More plastic clays, more plastic silty clays, silty clays |
| VI | Sandy clays, silts |
| VIIa | Sandy muds |
| VIIb | Clayey muds |
| VIII | Anthropogenic fills |

Figure 4. Lithology – Legend

The analysis of the geological and geotechnical data showed heterogeneous soil conditions along the whole tunnel alignment.

## 2.2 *Bathymetric survey*

On June 2010 and August 2012, measurements of the bed of the Vistula River, in a strip of 74 meters symmetrical to the course of the Metro Line 2, have been executed. The measurements were carried out with the use of electronic equipment: probes, GPS, and direct measurement techniques. Interval measurement probe was set at 2.5 m in order to obtain a more complete picture of the bottom (see Figure 5).

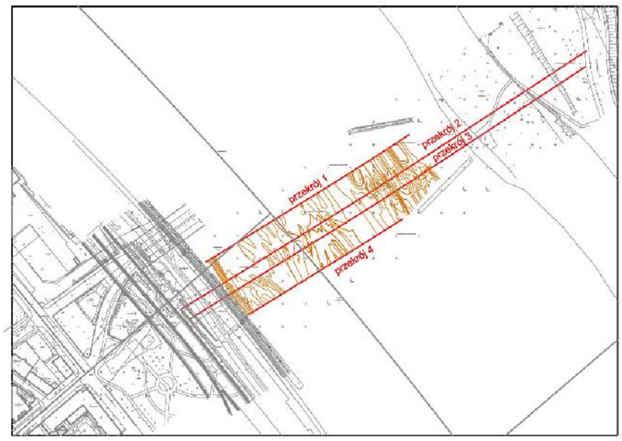



Figure 5. Bathymetric survey – Plan view

The level of the Vistula River on June 2010, checked by a water gauge in the Port of Prague (*Port Praski*), was +1.85 m with respect to the "0" of the Vistula River as a benchmark adopted in the study. On the other hand in 2012 the level of water was very low and amounted to 0.91 m below the “0” Vistula River.

Four longitudinal sections were prepared, two on the axis of the tunnels and two at given distances from them. Figure 5 shows a plan view of the 4 control cross sections realized. Figures 6 and 7 show the two main cross sections with the bathymetric measurements of 2010 (green line) and 2012 (red line).

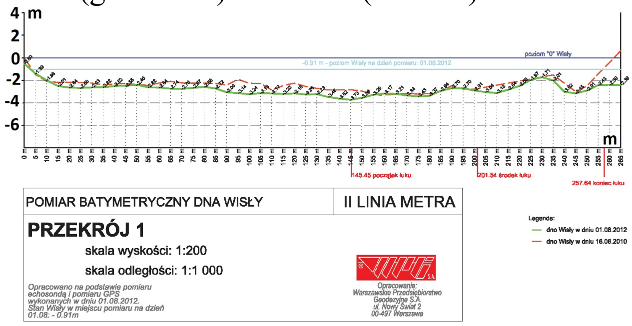



Figure 6. Bathymetric survey – Section 1

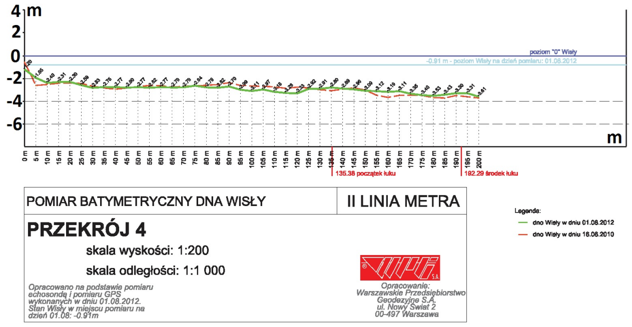



Figure 7. Bathymetric survey – Section 2

Small differences between the two bathymetric surveys were detected, with a magnitude from ±0.50 m to ±1.00 m. In every section, an increase of the erosion or an increase of bed level has been noted.

Figure 8. View of the Vistula River in the cross passage

# 3 CHECKS OF TUNNELS' BUOYANCY AND OF VISTULA BED STABILITY

In the first design stage the buoyancy of the tunnels and the stability of the Vistula bed after the TBM passage were checked.

## 3.1 *Buoyancy verification of the tunnel*

The verification of the tunnels' buoyancy has been successfully performed.

The procedure applied for the verification of the buoyancy is showed below for one of the sections examined along the TBM track under Vistula River. Several cases have been evaluated to reduce to a minimum level the risk of tunnels' buoyancy. Unlikely conditions have been simulated by varying the overburden and the unit weight of the soil.

According to the Archimedes' principle, the upward acting force on the tunnel structure per tunnel unit length is equal to:

$$S = \pi \cdot \frac{D^2}{4} \cdot \gamma_w = 3.14 \cdot \frac{6m^2}{4} \cdot 10 \frac{KN}{m^3} = 283 \frac{KN}{m}, \quad (1)$$

where $D$ is the external diameter of the tunnel lining and $\gamma_w$ is the water unit weight.

In the worst condition, corresponding to the minimum soil cover over the tunnels below the bottom of the River (about 7 m), this action is counterbalanced by the weight of the soil above the tunnel, that per tunnel unit length is equal to:

$$W = \gamma' \cdot h \cdot D = (20.6 - 10) \frac{KN}{m^3} \cdot 7m \cdot 6m \cdot = 445 \frac{KN}{m}, \quad (2)$$

where $D$ is the external diameter of the tunnel lining, $h$ is the minimum soil cover over the tunnels and $\gamma'$ is the soil effective unit weight.

A further favourable weight is the weight of the tunnel itself, which per tunnel unit length is equal to:

$$W_T = \pi \cdot D_m \cdot s \cdot \gamma_{cls} = 3.14 \cdot 5.7m \cdot 0.3m \cdot 25 \frac{KN}{m^3} = \quad , \quad (3)$$
$$= 134 \frac{KN}{m}$$

where $D_m$ is the average diameter of the tunnel lining, $s$ is the lining thickness, $\gamma_{cls}$ is the concrete unit weight. Therefore, the tunnels stability Safety Factor is equal to:

$$F_s = \frac{(W + W_T) * 0.9}{S * 1.1} = 1.71 > 1, \quad (4)$$

Table 1 shows the results of all the calculations developed, varying the overburden and the unit weight of the soil.

Table 1. Evaluation of the safety factors

| Overburden | Soil Weight [KN/m$^3$] | Safety factor |
|---|---|---|
| **7** | **20,6** | **1,71** |
| 6 | 20,6 | 1,52 |
| 5 | 20,6 | 1,34 |
| 4 | 20,6 | 1,15 |
| 7 | 20 | 1,60 |
| 6 | 20 | 1,43 |
| 5 | 20 | 1,26 |
| 4 | 20 | 1,08 |
| 7 | 19 | 1,48 |
| 6 | 19 | 1,33 |
| 5 | 19 | 1,17 |
| **4** | **19** | **1,01** |
| 7 | 18 | 1,36 |
| 6 | 18 | 1,22 |
| 5 | 18 | 1,08 |

The minimum Safety Factor calculated is equal to 1.01 with an overburden of 4.00 m, instead of 7.00 m (considering 3.00 m of erosion), and an unit weight of the soil of 19 kN/m$^3$ instead of 20.9 kN/m$^3$. According to the Eurocode, the checks have also been satisfied for really unlikely conditions.

## 3.2 *Stability verification of the Vistula River bed during Tunnels' excavation*

Due to the adopted excavation technology, the tunnels excavation, under the bed of the Vistula River, has not been negatively affected by the water head of the river and by the pore water pressure in the surrounding ground. This considering that the EPB TBM excavation technology is able to maintain constantly at the tunnel face a pressure that counter-balance both the geostatic and the hydrostatic pressure (see Figure 9).

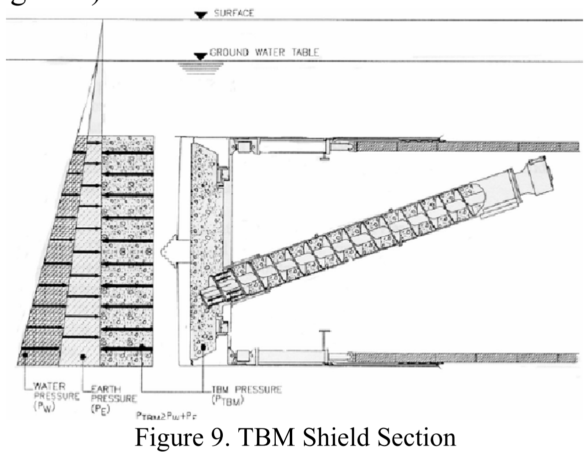


Figure 9. TBM Shield Section

For this reason, no instability of the bottom of the River was expected in connection with flow of soil pore water, from the bed of the river toward the TBM excavation chamber and with consequent fine soil particles transportation. This even in the heterogeneous soil conditions corresponding to the presence of permeable sandy lenses within the high plasticity clays.

This means that potential hydraulic connections between the Vistula River flow and the geotechnical layer passed by the TBM, if any, had no influence on the stability of the tunnels during excavation.

Therefore, no soil improvement of the sandy lenses within the soil below the bed of the Vistula River, by means of grouting or jet grouting, has been necessary to guarantee the safe proceeding of tunnels excavation.

Below the bed of the Vistula River tunnels excavation was carried out very cautiously, paying attention to the possible occurrence of sudden pressure changes at tunnel face, passing from prevailing clayey conditions to sandy soil condition, in order to face them by means of adequate actions in due time.

## 3.3 *Verification of the stability of the bed of the Vistula River during Metro service stage*

With reference to the available information (Żelaziński, J. et al., 2006) the erosion of the bed of the Vistula River during the period between 1950 and 1990, near the section where the Metro Line 2 underpasses the River, has been estimated major than 2 m.

Between 1990 and 2010, it has been observed a stabilization of the bed level, despite the intensive excavation of sand and gravel from the River bed itself (see Figure 10).

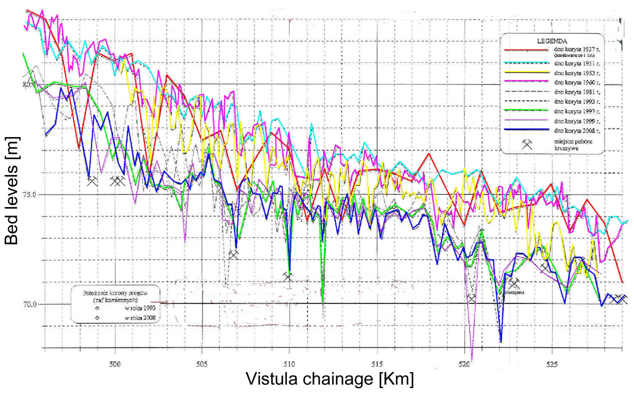



Figure 10. Vertical changes between 1937 - 2008 (Falacinski et al., 2009)

The stabilization of the bed level, despite small local differences, was confirmed by the bathymetric surveys executed during 2010 and 2012 (ref. paragraph 2.2).

According to the above considerations, it has been reasonably and conservatively, assumed that the erosion of the River's bed which may occur during the foreseen life of the Metro tunnels, will not exceed 3 m, providing that no increase of the exploitation of the bottom of the River, connected to the current stable conditions, will occur.

This hypothesis corresponds to a local residual minimum cover over the tunnels of 4 m. According to the verification procedure performed with reference to the current River bed condition, the stability verification of the tunnels against buoyancy have been still positively verified under erosion with a Safety Factor major than 1. The hypothesis of major erosion depth of the bottom of the River has been examined, in order to verify the worst scenario that may affect the tunnels' stability.

This verification is successfully complied with a tunnels stability Safety Factor major than 1 in case of tunnels overburden equal to 4 m (see paragraph 3.1). Anyway, in order to ensure the current stable condition of the bed of the River in the considered section:

- any activity implying the excavation of the bottom of the River above the tunnels, within the stretch above the tunnels of width equal to the zone measured between external sides of both tunnels, enlarged on both sides by the width equal to bottom depth H (m) of tunnels, will be forbidden;
- in the above mentioned stretch of the River, the foundation of boats mooring and of any other object will be forbidden too;
- along the course of the Vistula River, within the City of Warsaw, any modification of the River bed caused by regulation or of the exploitation activity of the bed of the River will be forbidden, if not subjected to a careful evaluation excluding any impact on the stability of the bed of the River in the considered section.

The hydraulic connections between the Vistula River flow and the geotechnical layer within the TBM passage will not influence tunnels behaviour during the service stage; this because the lining is not subject to deformation phenomena and is completely watertight by sealing gasket system around the tunnels.

### 3.4 *Monitoring of Vistula River bed*

The monitoring of the Vistula River bed should be performed periodically during the Metro Line 2 service life, in order to verify that any erosion of the Vistula River bed will occur. The monitoring of the Vistula River bed should be performed by direct measurements carried out along the Tunnel axes, by bathymetric survey performed from a boat equipped with a GPS device and an echosounder. In this way it will be possible to collect coupled measurements of position and depth.

The stabilization of the Vistula River bed level has been observed, and the suggested frequency of this monitoring is every five years. The monitoring frequency can be adjusted, with reference to the results of the performed surveys.

Alert and alarm thresholds limit were fixed in function of the design buoyancy analysis. If alert threshold limit is reached, the frequency of monitoring of the river bed, should be increased and investigations should be undertaken in order to find out the cause of the observed erosion. If the phenomenon will not stop and alarm threshold limit is reached, specific studies should be carried out in order to define the intervention to stop and control the erosion of the river bed.

## 4 FDM ANALYSIS

The analyses performed for the passage under the Vistula River were two.

The first one is the normal scenario, with the water level positioned to the 0 m in reference to the 0 level of Vistula River.

The second one, related to the worst unlikely scenario, with the water level increase of 10 m above the 0 level of Vistula River. Both analysis have been performed for structural check of the TBM tunnel lining, and for a properly tuning of the TBM advance parameters, such as earth pressure, and grouting pressure during the Vistula River underpass.

The TBM tunnel lining consists of 30 cm thick pre-cast reinforced concrete segments.

Six segments, including the key, composed each ring. The rhomboid shape characterized four segments; the trapezoidal shape characterized one segment and the key.

The below list summarized main geometrical characteristics of the ring:

| | |
|---|---|
| external diameter: | 6.0 m |
| internal diameter: | 5.4 m |
| thickness: | 0.30 m |
| medium length: | 1.50 m |
| number of segments: | 5+1 (key) |

The F.D.M. analyses have been performed using FLAC 4.0 (Fast Lagrangian Analysis of Continua), a two dimensional explicit finite difference program, provided by ITASCA Consulting, for engineering mechanics computation.

### 4.1 *Constitutive model*

As stated above, the plasticization function F, defined by the failure criterion, allows identifying the soil zones in which the stress state has exceeded the limit of elastic behaviour of the material, and which are therefore subject to stresses involving irreversible deformations in the rock mass. The F.D.M. code makes it possible to adopt an elastic, perfectly plastic soil behaviour model. In the analyses carried out, an elastic-plastic behaviour has been assumed, adopting the Mohr-Coulomb failure criterion, applied in terms of effective stress, with associate flow rule and tensile strength $\sigma_t$=0.

### 4.2 *Initial stress*

During the first step of the numerical analyses, the state of stress, existing in situ before the start of execution of the tunnel, was imposed to the model by applying to it the geostatic load due to the weight of the materials and the water table. In all the analyses, the applied vertical load corresponds to the geostatic load at the different elevations of the model. For each calculation an at rest pressure coefficient $K_0$=0.5 was assumed, according to local soil conditions.

The Table 2 shows the mechanical characteristics of the soil, the position of the groundwater and the overburden relative to the calculation section.

Table 2. Geomechanical parameter of soils

| Description [m from Vistula River (V.R.)] | $\gamma$ [KN/m$^3$] | E [MPa] | c' [KPa] | $\phi$' [°] |
|---|---|---|---|---|
| 0 m ÷ -3 m | | | | |
| -3 m ÷ -9 m | 20.6 | 70+2.25z | 10 | 20 |
| -9 m ÷ -18.8 m | 20.6 | 20+4.5z | 0 | 41 |
| -18.8 m ÷ end model | 20.6 | 70+2.25z | 10 | 20 |

where γ is the weight, E is the Young modulus, c' is the drained cohesion, ϕ' is the drained friction angle and z is the quote from the bed of the Vistula River.

## 4.3 *Tunnel excavation and lining*

The tunnel excavation has been simulated removing the mesh elements contained inside the excavation boundary. The lining installation has been simulated by a ring made of beam elements, between gridpoints corresponding to the tunnel boundary, having an elastic constitutive law.

## 4.4 *Calculation phases*

The calculations have been performed in successive stages. The simulation of tunnel excavation and lining in a single calculation step allows considering the entire geostatic load bearing the tunnel lining.

<u>*Time 0*</u>: application of the geostatic loads and numerical model generation (grid, constraints, material properties). Geostatic situation (application of the geostatic loads, ground water 0 m V.R. for the first analysis and 10 m above 0 m V.R. for the second analysis).

<u>*Time 1*</u>: first tunnel excavation and lining;
<u>*Time 2*</u>: second tunnel excavation and lining.

The lining installation was simulated by a ring made of beam elements, between gridpoints corresponding to the tunnel boundary, having an elastic constitutive law. The verifications are always satisfied for both the analysis with no relevant plasticization. From the analysis with the level of water at 0 m in reference to the Vistula River, the horizontal pressure found, is compliant with the theoretical values given in the Rankine approach, and described in the next paragraph.

The Figures 11 and 12 show the horizontal stresses of the two analysis performed in the calculations.

# 5 DEFINITION OF TBM PARAMETERS

The correct definition of the "operational parameters" and of the "excavation data" in addition to their continuous monitoring, automatically recorded by EPB, were essential in order to properly performing the excavation.

The values of the TBM excavation chamber pressure for the tunnel section under Vistula River have been calculated assuming water table position at 0.0 m at Vistula River level, and checked in function of the real water level measured before the TBM's passage.

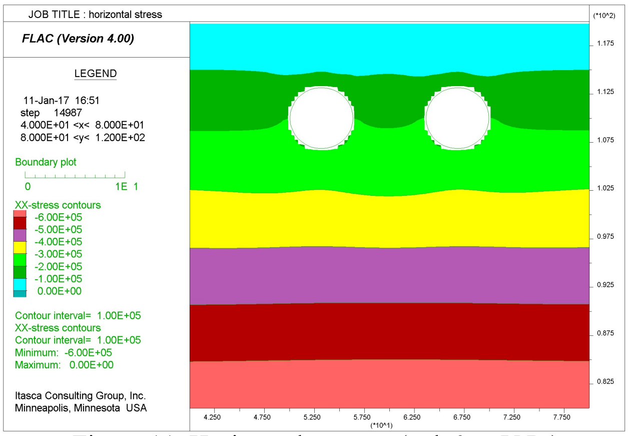



Figure 11. Horizontal stresses (w.l. 0 m V.R.)

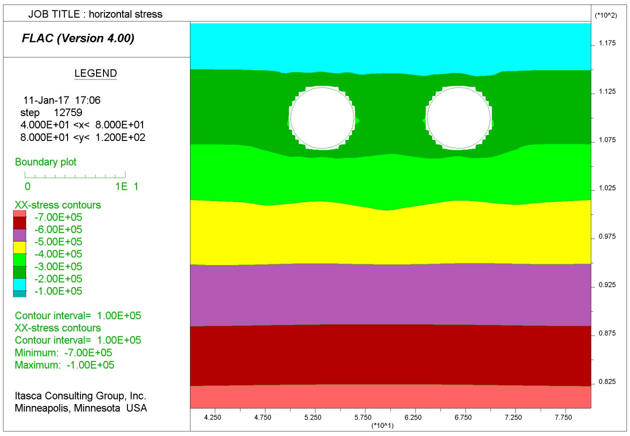



Figure 12. Horizontal stresses (w.l. +10 m above 0 V.R.)

During the excavation of TBM tunnels, pressures able to balance soil and water thrust at the front, must be ensured in the excavation chamber, in order to prevent the collapse and to contain the deformation framework. The evaluation of the containment pressures at the face starts from the examination of the geostatic stress state, in terms of effective stress and hydrostatic pressures related to the water level.

For the analysed stretch, soil's pressures have been determined every 100 meters, calculating $K_0$ and $K_a$ pressure, according to Rankine formulation. Pressures have been evaluated according to the "*Principle of Effective Stress*", by using a coefficient $K_0$ referred to soft soils in normal-consolidation conditions

$$K_0 = (1 - \sin \Phi), \tag{5}$$

where $\Phi$ represents the friction angle of the soil. The pressure values have been calculated according to the formula below:

$$\sigma'_h = \left[\gamma_{nat} \cdot \left(H + h_{(x,y)}\right) - \gamma_w \cdot H_f\right] \cdot K_0, \tag{6}$$

$$\sigma_h = \sigma'_h + u, \tag{7}$$

Where $\gamma_{nat}$ is the weight of the soil, $\gamma_w$ is the water unit weight, $h_{(x,y)}$ represents the distance between pressure's TBM sensor from the top of the tunnel, $H$ is the overburden at the top of the tunnel, $H_f$ is the water level with respect to the location of the pressure's sensors and $u$ is the water pressure at the sensors' level (see Figure 13).

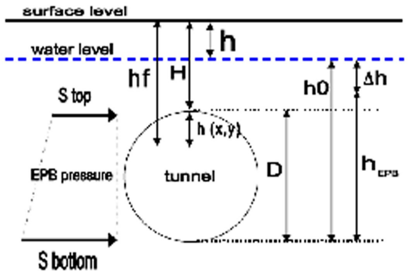


Figure 13. Input scheme

Corresponding pressures in active thrust conditions have been calculated, by using coefficient $K_a$ from Rankine theory:

$$K_a = tg^2\left(45° - \frac{\phi}{2}\right), \tag{8}$$

The analysis carried out led to a detailed estimation of the values of soil thrust at excavation level in geostatic and in active thrust conditions. Those values have been considered as the upper and the lower limit of the pressure that must be kept inside the excavation chamber, as indicative of a ground stress condition almost undisturbed. The estimated front pressure's values take into account the hydrostatic pressure, which has to be totally supported in order to avoid water flow.

In the examined case, the tunnels' overburden was included between 7.00 and 11.00 meters, and the water head, from the top of the tunnel, is about 11 m, considering the water level at 0 m V.R. The main soils interfered under the Vistula were Ia and IIc (see Figures 3, 4). In order to ensure the conditions of stability and to minimize the volume loss, it was necessary to provide "confinement" interventions around the cavity, which were able to exert enough pressure to keep the soil in undisturbed conditions, referring to tensional and deformative terms. Based on appropriate theoretical considerations and on previous experience in similar situations, the injection's pressure of bi-fluid mixture, was included between the value of the average lithostatic pressure and the pressure value registered in $h_1$ (sensor located at 1,4 m from the top of the tunnel) increased by 0.5 bar, in order to ensure the proper filling. In particular cases, the value of injection pressure of backfilling mixture can be equal to a pressure value registered in sensor $h_1$ increased by max 1 bar, in accordance to the monitoring data. The calculated pressures for the TBM passage below the Vistula River are between 1.5 bar to 2.1 bar. Backfilling volume has been estimated according to the geometrical characteristics of TBM. The theoretical volume of the injected mixture for each ring was equal to 4.35 m$^3$, considering the:

| | |
|---|---|
| Excavation diameter: | 6.30 m |
| External diameter of the ring: | 6.00 m |
| Ring length: | 1.50 m |

# 6 CHECKS DURING THE EXCAVATION

## 6.1 *Back analysis before the Vistula River underpass*

Due to the impossibility of measuring the settlements of the natural ground surface during the underpass of the Vistula River it has been necessary to conduct an accurate analysis in the previous tunnel stretches, and particularly in those closest to the river. Beside to the TBM parameters checks, described in next paragraphs for the data recorded during the river underpass, the ground surface monitoring data collected have been analysed. The Figures 19 and 20 compare the settlements recorded, before the Vistula River underpass, to the design values.

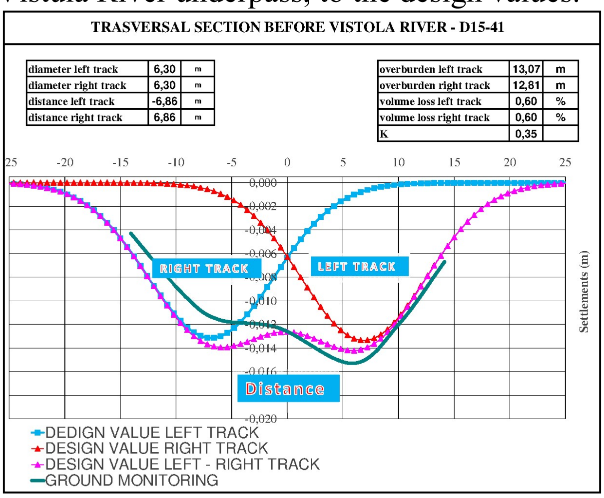


Figure 19. Settlements check before the Vistula River

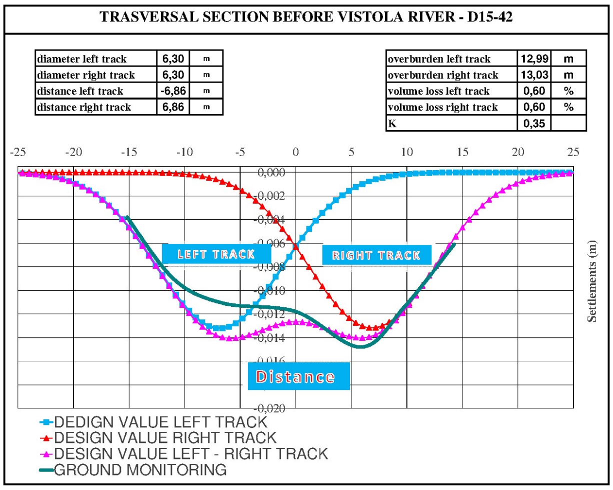



Figure 20. Settlements check before the Vistula River

The back analysis performed confirms the validity of the design assumptions.

## 6.2 *Check of TBM parameters*

Two different kind of data analysis were performed during the excavation of each tunnel stretch, and particularly during the excavation phase under Vistula River. The first analysis concerned the TBM's data process, and consisted in the registration every 10 seconds of all TBM's parameters and in their consequent summarization. The second analysis concerned the TBM's data calculate, and consisted in the registration of minimum, average and maximum values of all TBM's parameters for each entire ring advance and in their consequent summarization. Both the data above described have been summarized for each push and compared to the design value in term of:

1) earth pressure
2) grouting volume
3) grouting pressure
4) volume of excavated material

Furthermore, to have a proper control of the thrust, other parameters have been checked:

5) total foam and water
6) cutterhead penetration, speed and torque
7) force main thrust
8) advance speed
9) screw conveyor speed and torque

Every single change of the "standard" parameter was shared with the TBM manager in real time. The Figure 21 shows no relevant differences from the expected parameter and the registered parameters. The earth pressure value were always within the design limits.

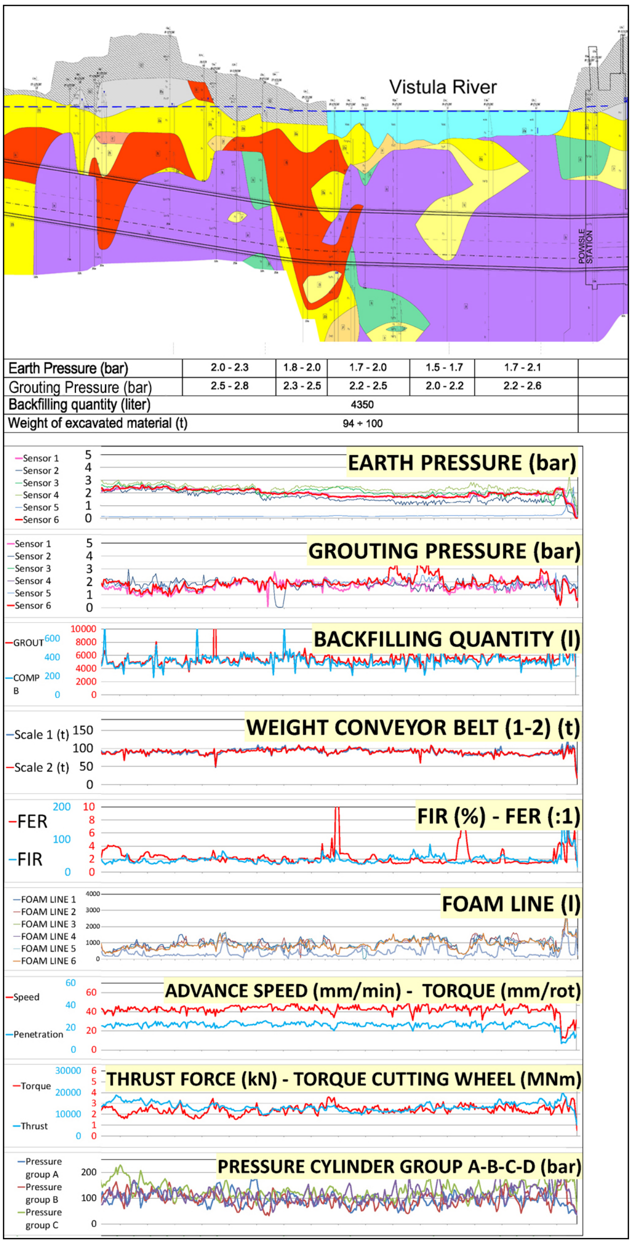



Figure 21. Resume of the tunnel stretch parameters

Figure 22 summarized, the face support pressure values recorded by the six sensors of the TBM's cutterhead, during tracks tunnel excavation. The TBM top sensors recorded pressure showed that the design value was consistently complying with the design value, which was between 1.5 and 2.1 bar (except for some fluctuations during excavation).

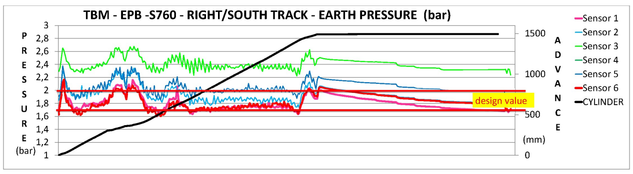



Figure 22. Earth pressure

Also the volume of the excavated material and the grouting volume were continuously recorded, in order to check the balance between the tunnel volume, the volume of excavated material and the grouting volume. The volume of the excavated material was controlled through the weighting of the muck extracted during the excavation. Figure 23 shows the design values together with the alarm and alert ones: no relevant inconsistencies have been recorded during the excavation under the Vistula River.

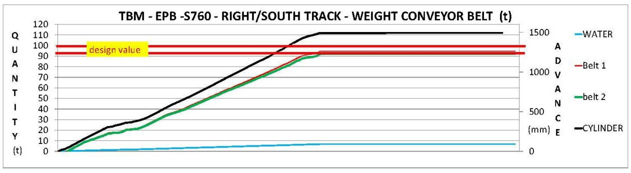



Figure 23. Volume of excavated material

The back filling quantity and pressure, for the most TBM advances had no relevant differences from design values. The Figures 24 and 25 report the TBM excavation's records (line from 1 to 4) related to injected backfilling pressure and volume respectively.

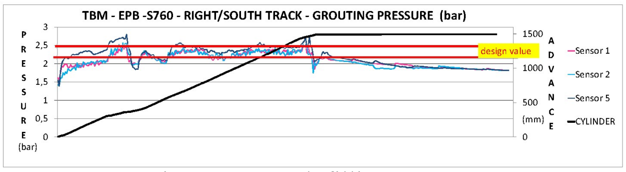



Figure 24. Back filling pressure

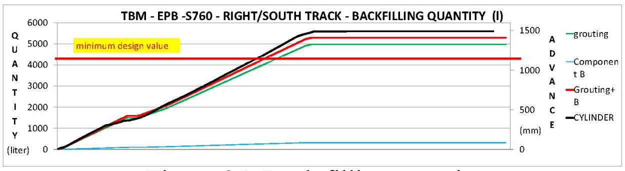



Figure 25. Back filling quantity

In reference to the theoretical amount of cement mortar no relevant inconsistencies have been recorded during excavation.

## 7 PRODUCTION RATE

The TBM's production during the underpass of the Vistula River highlighted high rate values. Referring to the left track, the underpass of the river comprises 176 lining rings, corresponding to 264 m. The tunnel section has been excavated from 22 July 2013 to 2 August 2013. Referring to the right track, the underpass of the river comprises 184 lining rings corresponding to 276 m. The tunnel section has been excavated from 4 September 2013 to 16 September 2013. The daily production was for the Left tunnel equal to 27 m/day and for the Right tunnel equal to 22 m/day. These values complied with the average values recorded during the excavation of the entire Metro Line 2.

## 8 CONCLUSIONS

The paper described the underpass of Vistula River, which has been the most critical issue of the central stretch of the Warsaw Metro Line 2. The underpass of Vistula River has been characterized by a minimum overburden of 7 m and a total excavation length of 300 m. The TBM's excavation has been performed in safety conditions, without technical problems and respecting high level of production.

The results of calculations and checks developed during the design stage have been confirmed during the excavation of the two tunnels. The real time analysis of the TBM parameters and the technical assistance on site guaranteed high production level.

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