WARSAW METRO'S IMPROVEMENTS

Poland's capital Warsaw has two million inhabitants, but only one underground metro line. A second line, with 28 stations over 30km, is under construction. The central section of this line is being built under a turnkey contract awarded to the international AGP Consortium, made up of contractors Astaldi of Italy, Gulermak of Turkey and PBDIM of Poland. Italy's Rocksoil is providing technical assistance to the project, and here describes some of the challenges involved

ROCKSOIL'S ROLE on this project has been design and ongoing technical assistance on site during tunnel construction, driving the line tunnels and carrying out related works.

The central section of Line 2 is 6.3km long. It has seven stations between two terminus stations at Rondo Daszynskiego and Dworzec Wilenski. These twin bore tunnels are to be driven using four 6.3m diameter EPBMs.

A number of difficult underground passages have been identified along the route: between the Powisle and Stadion stations; under the Vistola River; and, under the two tunnels of Line One of the metro between the Rondo Onz and Swietokrzyska stations. Between Rondo Onz and Swietokrzyska, less than 3m separates the existing tunnels from those to be driven.

Last, but not least, in terms of complexity, is the passage under the new monumental complex of buildings in the "Prague" district between the Stadion and Dworzec Wilenski stations, discussed in this paper.

In this area remain Warsaw's only buildings undamaged by the bombings of the Second World War. Today, they are in a critical condition, due to poor construction and maintenance.

The constraints of tunnel alignment meant that the tunnel needed to be designed to pass only of 6-8m beneath these historically important buildings' foundations.

Poland's Instutut Techniki Budowlanej (ITB) was called in to conduct detailed structural analysis of the buildings. The ITB classified the buildings as at potential risk of collapse, even under normal conditions.

In order to reduce the risk connected with excavation to a minimum, the AGP consortium first designed and implemented structural intervention to protect some of these buildings, in preparation for the passage of TBMs. It also asked Rocksoil to develop a ground improvement plan that would take into account the fragile and historic nature of these buildings.

The geometry of the historic buildings and numerous other existing structure is extremely complex. The area includes extensive underground utilities, and only offers small surface spaces. This made it impossible to carry out the ground improvement treatment required using conventional methods. Recourse had to be made to complex construction technology in order to improve the

ground, forming an arch around the upper profile of the crown of the tunnels to be driven.

Given the large extent of the zone to be treated with ground injections, Rocksoil designed an innovative approach using "horizontal directional drilling" (HDD). This started from an

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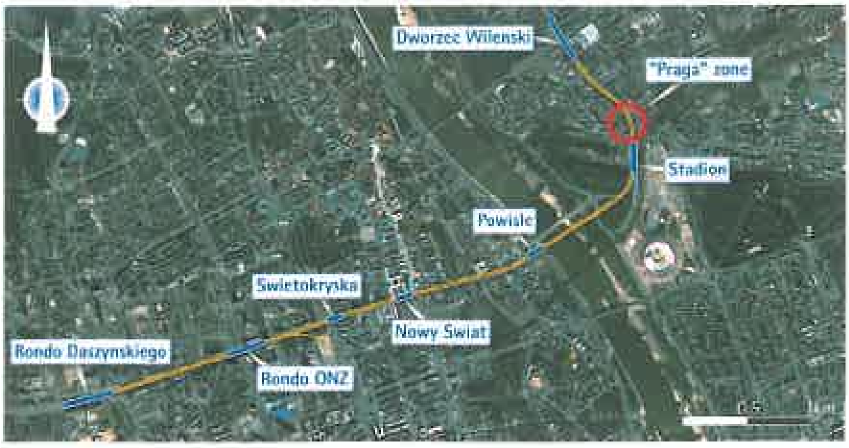
area relatively distant from the buildings, and gradually reached the correct depth around the tunnels to be excavated. These drillings were arranged parallel to the profiles of the future tunnels. They were designed to follow the tunnels for the whole 250m length of this section of the project.

The remaining part of this paper examines the design predictions generated by this new innovative technology and compares them with what actually happened during construction following TBM tunnel advance.

Design and construction was carried out on the basis of the principles of the ADECO-RS approach (Analysis of Controlled Deformation in Rocks and Soils) (Lunardi, 2006).

Which, as is known, employs the following stages:

- The survey phase: with the acquisition of information on the pre-existing equilibriums in the places in question (characteristics of the buildings affected by excavation, the geology and geotechnical characteristics of the ground, the natural stress-strain states, etc.);
- The diagnosis phase: numerical analyses are conducted in this phase, in the absence of ground improvement and stabilisation intervention, in order to assess the behaviour category of the ground's response to the action of excavation and the level of risk associated with each building;
- The therapy phase: this is based on the results produced during the diagnosis phase. The zones to be treated are identified and the ground improvement treatment is designed, by numerically assessing the beneficial effects that are sought, which would result in the behaviour of the ground's response to excavation being in category A (stable core-face) and place all the buildings in a damage class associated with a level of risk that can be considered acceptable;
- Operational phase and monitoring during construction: excavation and the intervention planned is carried out in this phase. The adequacy of the design is verified continuously and appropriate adjustments are introduced, which are nevertheless already allowed for in the design itself. The adjustments are based on the results of monitoring and then on the consequent back-analysis of the data.



Above: Figure 1, The central section of Warsaw Metro Line Two

SURVEY PHASE

Geology of the places

The tunnel alignment of Line 2 runs through considerably heterogeneous ground. The ground directly affected by excavation consists mainly of alluvial deposits (sands and clays). The overburdens above the extrados of the crowns of the tunnels are between 10m and 14m, while the water table lies at between 4.5m and 8m. A description of the lithologies present in the zone in question is given in Table 1 and Figure 3.

Description of the monumental complex of the Prague area

Building in the monumental complex of the "Prague" district, affected by the passage of TBM's beneath it, dates back to the period between the end of the 1800s and the beginning of the 1900s. The main load-bearing structures of those buildings consist predominantly of brick masonry. Reinforced concrete structural components are present in some cases. The floors consist mainly of structural components in wood and steel, but rarely in reinforced concrete. The depth of the foundations varies between 3m and 4m beneath street level. Most of these buildings were affected by the devastation of the Second World War and were not rebuilt, except in sporadic cases, but above all they have not been maintained over the years. They have suffered the severe effects of the deterioration of the materials in the course of time. Past development of cracking, together with the deterioration in the materials has resulted in most of these buildings being classified as having non–ordinary structural behaviour. Furthermore, recent demolition of some bordering buildings and a fire, which affected part of these buildings, has further aggravated an already critical structural situation. In this context, in the more critical situations, it was

Table 1 Description of geological levels

Geological level	Description
VIII	Backfill soils
VI	Sands and clayey silts
Vh	Sandy clays and clays
IV	Gravelly sands and gravels
IIIa - IIIb	Medium and coarse sands
IIb	Fine and silty sands
Ib	Clays, silty clays and silts
Ia	Highly plastic clays

Source: Authors

Table 2: Structural intervention carried out before the passage of the TBMs

Building – number	*Year of construction*	*Type of load-bearing structure*	*Initial condition*	*Structural Intervention Carried Out*
Targowa 21 - 152	1890	Masonry	Critical Building uninhabited	Openings walled-up
Targowa 19 - 151	1910	Masonry	Critical Building uninhabited	Openings walled-up
Targowa 19 - 148				-
Targowa 17a - 149	1910-1913	Masonry and some reinforced concrete components	Cracking, deteriorated materials and lack of maintenance	-
Targowa 17a - 150				-
Targowa 17d - 146				Steel chains
Targowa 17d - 144				Wooden beams around windows
Targowa 15 - 142	1928	Reinforced concrete	Cracking, deteriorated materials and lack of maintenance	-
Zamoyskiego 27 - 137	1939	Masonry and some reinforced concrete components	Cracking, deteriorated materials and lack of maintenance	-
Zamoyskiego 27 - 129				
Zamoyskiego 29 - 130	1910-1913	Masonry	Cracking, deteriorated materials and lack of maintenance	Steel chains and wooden beams around windows
Zamoyskiego 31 - 131	After 1945	Masonry	Cracking, deteriorated materials and lack of maintenance	-

Source: Authors

considered best to carry out structural intervention to protect some of the buildings (Table 2) in preparation for the passage of the TBM's.

This intervention made it possible to increase the coefficient of stability of the buildings considered most sensitive to excavation operations.

DIAGNOSIS PHASE

It is well-known that the excavation of a tunnel induces a change in the stress-strain state of the ground around the cavity, with consequent deformation which develops in the ground both longitudinally towards the tunnel face and transversely around the cavity. The consequent subsidence, which manifests on the surface, above all when the overburden is shallow, depends on the characteristics of the ground tunnelled, the position of the water table, the dimensions and depth of the tunnel, the excavation method employed and the speed of advance of the face (Anagnostou, 2008).

Both numerical and empirical methods were used to calculate the subsidence profile. Numerous experimental observations (Peck and Schmidt, 1969; Attewell and Farmer, 1974) show that surface subsidence curves can be represented well by a Gaussian normal probability function with two variables: the maximum subsidence w_{max} (along the centre line of the tunnel) and the distance i_0 between the two inflection points of the curve, on which the width of the depression depends (Figure 4, page 36). The subsidence function can therefore be expressed by the following equation:

$$(1)\ w(x) = w_{\max} \cdot e^{\left[-\frac{x^2}{2i_0^2}\right]}$$

Where:
w(x) represents the subsidence at the abscissa x. The value of w_{max} can be obtained by establishing the expected volume of the subsidence depression, equal to the volume VP lost during excavation, i.e. volume of ground excavated in excess of the theoretical dimensions of the cavity. The volume lost is generally expressed as a percentage of the theoretical volume of the excavated tunnel, V_{tun}. In the case in question, a design assumption was made on the basis of values found in the literature for similar contexts, that VP would be equal to 0.60 per cent, which was confirmed as reliable by numerous back-analysis calculations carried out during construction.

In order to calculate the distance i_0, O'Reilly and New (1982) showed that when the size of the overburden above the crown of the tunnel is greater than

Above: Figure 2, Soil treatment around the tunnel the diameter of the tunnel, a direct relationship can be assumed between the parameter i_0 and the centre line of the tunnel z_0 as follows:

(2) $i_0 = K \cdot z_0$

Where:
the coefficient K is basically a function of the nature of the ground to be excavated.

In the case in question the coefficient K was assumed on the basis of values found in the literature to vary between 0.30 and 0.50 (respectively for sands and clays).

Once the depression and subsidence values were known, Burland & Wroth's theory (1974) was used to assess the resulting damage on the buildings by taking account of the following fundamental factors:

- The overall rigidity of the structure with the definition of the type of structure;
- The resistance to cracking of the materials of which the building is composed.

On the basis of experimental data, Boscardin & Cording (1989) correlated the expected damage to buildings with the values for the resistance to cracking of the materials on a scale of between 0 (negligible damage) to 5 (severe damage).

In terms of that classification, the damage class that was considered acceptable for the buildings in the Prague district affected by excavation operations for Line 2 of the Warsaw Metro was class 2, corresponding to slight damage with modest and easily repairable cracking being identified.

Risk analyses developed on the basis of the classic theories just described produced the results reported in Table 3.

Table 3: Results of risk analysis in the absence of ground improvement operations

Building – number	*Volume lost (%)*	*Maximum subsidence (mm)*	*Damage class*
Targowa 21 - 152	0.60	20	2
Targowa 19 - 151	0.60	20	2
Targowa 19 - 148	0.60	13	1
Targowa 17a - 149	0.80	21	2
Targowa 17a - 150	0.60	22	2
Targowa 17d - 146	0.80	23	3
Targowa 17d - 144	0.60	23	2
Targowa 15 - 142	0.60	25	3
Zamoyskiego 27 - 137	0.60	24	3
Zamoyskiego 27 - 129	0.60	23	2
Zamoyskiego 29 - 130	0.6	26	3
Zamoyskiego 31 - 131	0.6	7	2

Source: Authors

THERAPY PHASE

Design of operations with horizontal directional drilling

In order to reduce the risk level for the buildings in the "Prague" district to damage class 2, protective ground improvement operations were designed in the therapy phase, which not only improved the mechanical characteristics of the ground around the future excavation, but was also able to reduce the probability of foam leaking during TBM advance, with significant loss of volume and unpredictable subsidence on the surface as a consequence.

The intervention consisted of improving a circular section of ground above the crown of the excavation with a thickness of approximately 3m and a length equal to that of the passage beneath the buildings concerned.

The operations were particularly complex and unusual because of the length of the drilling required (250m) and the minimum radii of curvature of 100m and 300m for single or double curvatures respectively. In addition to the foundations of the buildings the ground treatment also involved a large sewer locally.

Valved PVC tubes were inserted in the drillings through which cement mixtures and appropriate chemicals were injected into the ground by permeation between the profile of excavation of the tunnels and the buildings above in order to produce the necessary increase in its mechanical properties.

On the basis of assessments carried out on similar ground treatments performed in grounds similar to those in question, it was considered reasonable from a design viewpoint to assume a Young's modulus of 2.5 times that of the natural ground for the treated ground.

In order to assess the effects of the treatment on the risk

Left: A horizontal directional drilling rig

Table 4: Results of risk analysis in the presence of ground improvement operations

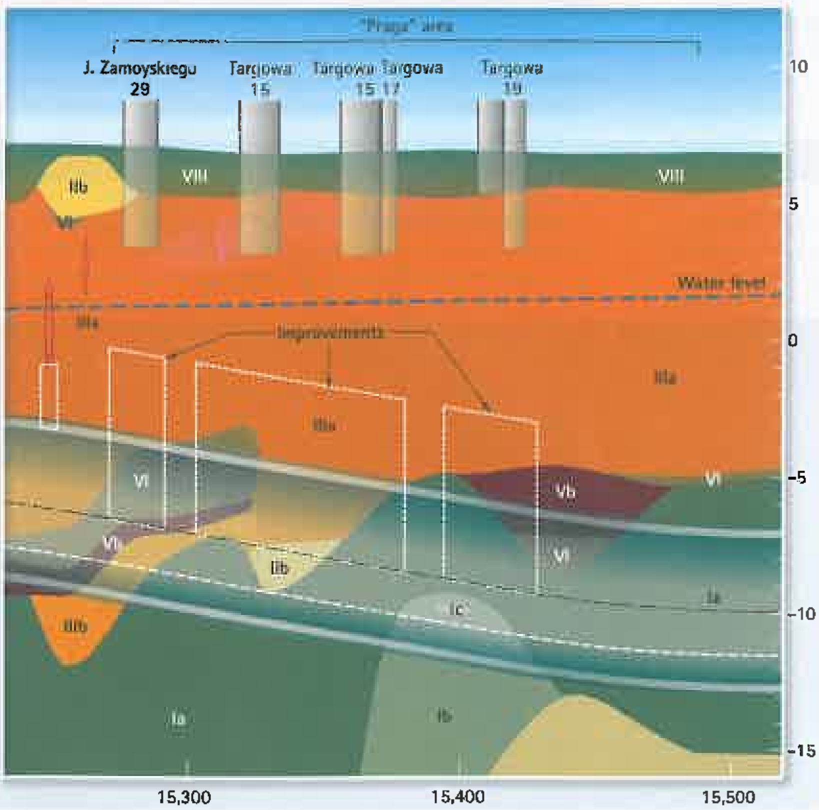
Building – number	*Volume lost (%)*	*Maximum subsidence (mm)*	*Damage class*
Targowa 21 – 152	0.44	15	2
Targowa 19 – 151	0.44	14	2
Targowa 19 – 148	0.44	10	0
Targowa 17a – 149	0.44	15	2
Targowa 17a – 150	0.44	16	2
Targowa 17d – 146	0.44	17	2
Targowa 17d – 144	0.44	17	2
Targowa 15 – 142	0.44	18	2
Zamoyskiego 27 – 137	0.44	17	2
Zamoyskiego 27 – 129	0.43	17	2
Zamoyskiego 29 – 130	0.43	18	2
Zamoyskiego 31 – 131	0.43	5	1

Source: Authors

levels, both numerical analyses were developed using the Itasca FLAC finite differences calculation programme, and empirical methods were derived from the classic theories described previously:

- The theoretical reduction in the volume of ground lost was assessed using the FLAC analyses and empirical methods;
- Once the expected values for lost volumes were known, the risk analyses were carried again out using the same criteria as those reported earlier

Below: Figure 3, Underground situation of the Prague district



Simulations of the different stages of tunnel excavation were carried out using two dimensional FLAC analysis, replacing the ground excavated with equivalent forces, which were gradually reduced as a percentage to reproduce the effect of tunnel advance. Ground improvement was simulated by increasing Young's modulus by 2.5 times the values for the natural ground.

The first numerical step was to calculate the percentage reduction of the excavation forces under free field conditions and in the absence of ground improvement, in order to obtain the congruence between the theoretical subsidence curve (with V_p=0.60%), obtained using classical methods and that calculated by using FLAC iteration analyses.

FLAC analyses were then carried out in the presence of improved ground, with the percentages of the excavation forces determined in this manner.

Finally, the percentages of the volume of ground lost were determined in the presence of ground improvement, comparing the subsidence depression calculated using those analyses with those calculated using empirical methods.

The percentages of volume lost determined in this manner were reduced by approximately 30 per cent compared to the value of V_p=0.60 per cent in the absence ground treatment. The risk analyses for the buildings under which the tunnels were to pass were calculated again with the new percentages for the volume of ground lost.

A significant reduction in expected subsidence and damage classes was seen, with the latter all within class two (see Table 4) following the ground improvement.

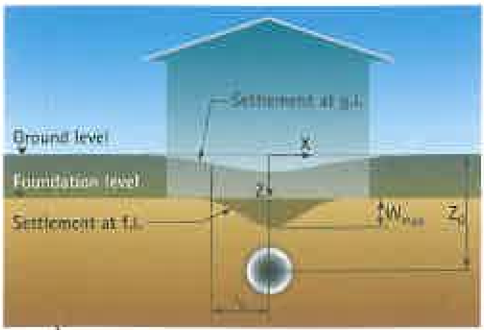
OPERATIONAL PHASE AND MONITORING DURING CONSTRUCTION

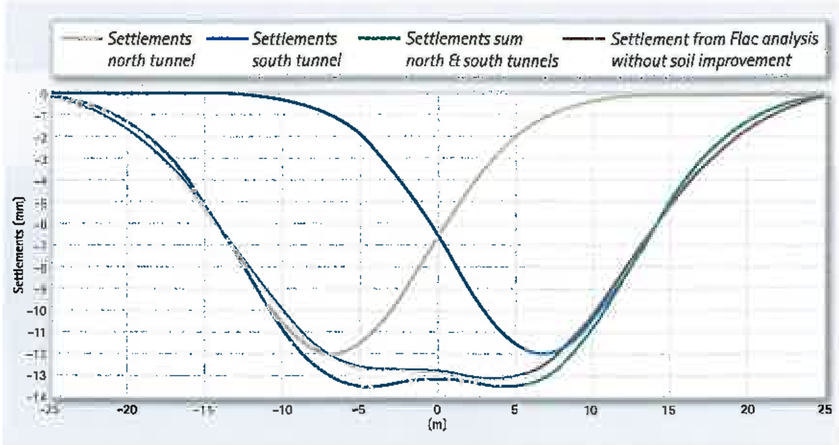
Operations using horizontal directional drilling (HDD)

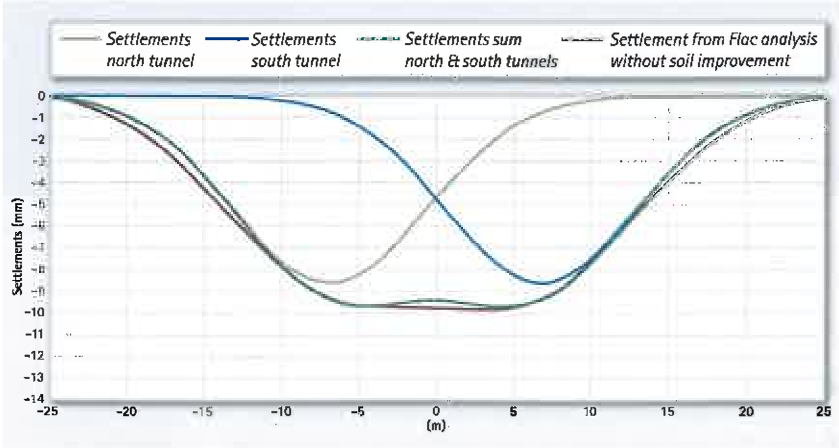
Ground improvement was carried out by the Italian contractor Icotekne using low pressure injections of cement and chemical mixtures, after first performing HDD.

This technology was used to achieve a controlled route by using a special asymmetrical drill bit with an angled surface which is piloted by an external magnetic localisation system by which the direction of the drilling could be guided as it advanced.

The tolerances were well within acceptable ranges (a few centimetres,







Top left: Figure 4, Subsidence depressions at street level and foundation level

despite the long lengths to be drilled and the short radii of curvature). In order to minimise the risk of local collapses, the bore was fitted with an appropriate steel lining.

The operation was carried out in the following stages:

1. Placing of electric cables on the surface to generate the magnetic field for the directional control of the drillings;
2. Drilling and subsequent installation of the preliminary steel lining with a diameter of 140 mm;
3. Insertion of a valved injection tube (two valves per metre) in PVC (diameter of 2in) inside the steel tube;
4. Execution of the primary sheathing injection using the cement mixtures and extraction of the steel lining at the same time. The injection of the sheathing at this stage replaces the preliminary steel lining;
5. Repeated injections of cement mixtures and chemicals at controlled pressures and volumes.

Type 52,R microfine cement grouting was used to facilitate the permeation of the cement mixtures.

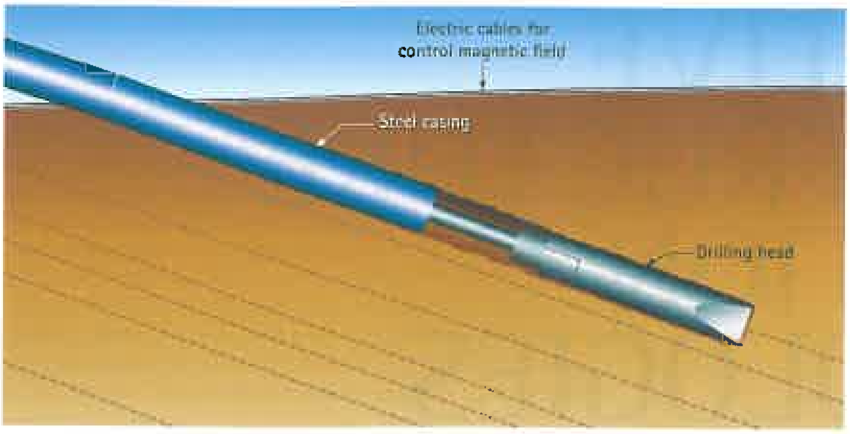
The quantities of the mixture injected were around 110-120 litres per valve, while the refusal pressures varied between 8 and 12 bar.

Cross-hole tests were carried out to assess the success of the

Table 5: Comparison of volume lost and subsidence

Building - number	Diagnosis Phase		Therapy Phase		During Construction	
	V_p (%)	W_{max} (mm)	V_p (%)	W_{max} (mm)	V_p (%)	W_{max} (mm)
Targowa 21 - 152	0.60	20	0.44	15	0.41	8
Targowa 19 - 150	0.60	20	0.44	14	0.38	8
Targowa 19 - 148	0.60	13	0.44	10	0.38	7
Targowa 17a - 149	0.60	21	0.44	15	0.26	7
Targowa 17a - 150	0.60	22	0.44	16	0.26	6
Targowa 17d - 146	0.60	23	0.44	17	0.26	6
Targowa 17b - 144	0.60	23	0.44	17	0.26	6
Targowa 15 - 142	0.60	25	0.44	18	0.30	5
Zamoyskiego 27 - 137	0.60	24	0.44	17	0.31	5
Zamoyskiego 27 - 129	0.60	23	0.43	17	0.43	9
Zamoyskiego 29 - 130	0.60	20	0.43	18	0.43	9
Zamoyskiego 31 - 131	0.60	7	0.43	5	0.43	4

Source: Authors



Above: *Figure 7, Drilling stage and insertion of the preliminary lining*

Opposite (graphs): *Figures 5 and 6, Comparison of results using empirical methods and FLAC analysis in the absence of ground improvement*

treatment.

These confirmed the accuracy of the value that was adopted in the numerical analyses, with an increase in the Young's modulus of the improved ground, E_{impr}, compared to that for the natural ground, E_{nat}:

(2) $E_{impr} = 2.5 \cdot E_{nat}$

Stable tunnel advance with EPBMs

Two EPBMs were employed to drive the tunnels, capable of controlling subsidence of the ground by adjusting the operating parameters of the machine.

More specifically, the main parameters defined in the design and 'controlled' during construction were those relating to the stabilisation pressure in the excavation chamber and the backfill pressure of the grout injections behind the concrete segments.

To achieve this a complex system of topographical monitoring was designed and put in place in preparation for tunnel advance operations, based on readings of ground pins, mini prisms and levelling pins installed on the buildings above the tunnel.

The results of the monitoring were used to carry out numerical back analysis continuously, which made it possible to check the design machine parameters in real time and take appropriate corrective action promptly in order to conform to safety requirements.

Table 5 shows the percentages for volume lost and values for subsidence forecast at the design phase (diagnosis phase in the absence of intervention, therapy phase in the presence of ground treatment) and those measured during actual construction.

As can be seen the percentages of volume lost measured during construction were either in line with or below the theoretical values predicted by the design.

The result was the same for the maximum subsidence values observed. The width of the subsidence depressions recorded during construction were greater than those predicted theoretically.

This fact falls within the norm, because the numerical analyses conducted during the diagnosis and therapy phases were carried out under free field conditions, i.e. not taking account of interaction between the ground and the structure.

Therefore the real distribution of movements recorded during construction showed, as it was reasonable to expect, a better final configuration than that predicted by the design. The excellent results achieved confirmed the validity of the treatment employed and that it was properly executed.

Maintaining the percentages of lost volume recorded below those predicted by the design was the result of the proper and very efficient execution of the excavation operations.

In this respect, good levels of production were recorded, sufficient to ensure that the machines remained below the complex of monumental buildings for a limited period of time, while passing beneath them.

CONCLUSIONS

Rocskoil designed an innovative method to improve ground, with horizontal directional drilling of 250m in length, and with a short radii of curvature for the difficult task of driving a tunnel beneath the complex of monumental buildings in the Prague district of Warsaw, consisting of numerous historical buildings in precarious conditions of stability.

The operations were carried out well and together with the survey carried out by the AGP Consortium's TBM team, this made it possible to drive a tunnel beneath the district under safe conditions and in full compliance with the design specifications.

The success of the operation is borne out by the results of the monitoring performed during construction, which showed volumes of ground lost in line with or below those predicted by the design. The maximum subsidence recorded by the teams was always less than that predicted

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