Investigation and Planning for Rehabilitation Works on Bucharest Metro Line 4

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1. INTRODUCTION

In 1975, the Bucharest Metro Company was established and began the construction of a metro network. Nowadays it encompasses over 70 km of underground lines, 49 stations and 4 depots (Fig. 1). The Bucharest Metro lines develop basically on the North-South and East-West directions, with some rings in the central part of the city. The Bucharest Metro carries more than 27\% of the total mass transit volume [2].

![Bucharest Metro Network](image)

Fig. 1. Bucharest Metro Network.

The Bucharest Metro is under expansion and modernization. Now there are 3.2 km and three stations under construction in Line 4 Northwest (Section 2: 1 Mai Station – Laminorului Station). Line 4 Southeast (4.8 km), Section 1: Nicolae Grigorescu Station - Anghel Saligny Station (former Linia de Centura Station), was commissioned on 19\textsuperscript{th} of November 2008 after a long rehabilitation working program and now, from operating point of view, became a part of Line 3 - Line Red (see section in circle, Fig. 1). This paper is referring about this successful work.
The construction works began in 1989: tunnels (open face shield method and cut-and-cover method) and stations (cut-and-cover method). Due to the lack of funds, the construction works on this section were stopped in 1996, leading to flooding of tunnels and stations. At that time, all the tunnels were 100% completed and stations were 40-60% completed. No works have been carried out in the past.

For the completion of construction works and rehabilitation of the remaining structures, the Bucharest Metro Company (Metrorex SA) had signed loan agreements with the European Investment Bank (EIB). One of the requirements of these agreements was a geotechnical and tunneling evaluation on the completion of construction works to safeguard the existing infrastructure of Line 4 Southeast. This objective was assumed by Prof. André P. Assis (at that time President of ITA), by elaborating a complex report [1], which strongly supported the Romanian specialist team in front of EIB.

The case of Bucharest is paradigmatic in the sense of the successful building of a continuous and fruitful collaboration between the Romanian authorities and the EIB over last two decades. Indeed, EIB has been active in financing projects in Romania since the early 1990s. The Bucharest Metropolitan Area has a population close to 2.5 million inhabitants and extends over a large surface area of 710 km². The city’s public transport consists of four major mass transit modes: metro, tram, trolleys and bus. In 1996, EIB made its first loan to the sector in Romania, with EUR 100 million for the completion of part of the metro network extension works. Since 2000, three further loans, totalling some EUR 550 million have been signed for the upgrading and extension of the network, the modernisation of the signalling system, the purchase of 30 high technology trains and the improvement of stations, as well as the construction of a new line. In addition, EIB lent EUR 70 million for the renewal of 11 tramway line sections in the city.

2. GEOTECHNICAL AND HYDROGEOLOGICAL CONDITIONS

Geomorphologically speaking, the underground structures of this line are located between two rivers (Dimbovita and Colentina – see Fig. 2), in regions dominated by quaternary formations, which have developed horizontally and vertically [2], [3].

Recent site investigations have confirmed the lithological sequence defined at the beginning of the works. A simplified geological profile is composed by fill, followed by layers of sand and clay. A typical lithological sequence is presented below, with some geotechnical comments based on characterization indexes (Fig. 3):

- Type 1 layer – non-uniform and heterogeneous fills, in different consolidation stages.
- **Type 2 (upper clay) layer** – upper clay complex, encompassing silty clays and sandy clays, with thickness ranging from 2 to 10 m. This stratum presents low clay activity, confirmed by moderate values of plasticity indexes (IP). It seems to be normally consolidated clay.

- **Type 3 (upper sand) layer** – sand and gravel complex (Colentina), composed by medium to large sands and small gravels, with thickness ranging from 5 to 9 m. Grain size analyses indicate to high uniformity.

- **Type 4 (lower clay) layer** – intermediate clay complex, composed by clays with limestone concretions, and silty clays. This stratum presents a high clay activity, confirmed by high values of plasticity indexes (IP). It is common the presence of sand lenses in this layer, in the order of meters. It also seems to be a normally consolidated clay, slightly more consistent and stronger than the upper clay layer (Type 2 layer) due to its greater depth.

- **Type 5 (lower sand) layer** – sand complex (Mostistea), composed by medium to fine sands and dusty sands, with great thickness, normally deeper than borehole depths. This stratum also presents high uniformity of grain sizes.

Analyzing some borehole logs from penetrometer tests and the longitudinal hydrogeological profile of this section, it was noticed that the clay layers seem to be interrupted in several locations (Fig. 4). This fact justifies the water pressure levels for the two sand formations be approximately the same. In other words, the links between the upper and the lower sand layers establish the water pressure level for these layers. This coincidence of water levels had been reported numerous times in the analyzed past documents. A geotechnical testing programme was carried out at the beginning of the works in order to obtain design parameters.

![Fig. 4. Simplified geological profile.](image)

After stopping construction works and consequent tunnel flooding, some leaching of sand sediments has occurred towards tunnels, probably caused by high hydraulic gradients in the regions close to lining segment joints. This phenomenon, allied to construction disturbances, may have caused loosening of the ground surrounding tunnels and consequently changes in their geotechnical properties. This suspicion had led to a new site investigation programme, based on penetrometer and laboratory tests. New transversal sections were investigated by penetrometer tests, spaced from 5 to 80 m, depending on present requirements of the underground structures, based on field displacements. Their results were compared to controlled values obtained in areas not affected by underground works (in average 50 to 70 m away from them), representative of the local soil conditions. As a general conclusion, there was a decrease in the penetrometer strength in areas where the water table was raised. A broad analysis showed a strength reduction up to 50% in the upper clay (Type 2) and upper sand (Type 3) layers. The most affected areas also coincided with zones where excessive displacements were measured within the tunnels [1], [4].

A complementary penetrometer tests for Zone 2 was completed, encompassing more than 100 tests. Its results confirmed the same trend observed on Zone 1, but slighter less affected (the number and dimensions of these disturbed areas are less than those found in Zone 1). In conclusion, regarding the results for Zone 2, the decrease in the penetrometer strength was between 35 to 50%, mostly in some zones located in the upper sand (Colentina) layer, up to a depth of 5 m, and more pronounced in areas surrounding tunnel portals of shafts and stations.

Investigations included a complementary geotechnical testing programme, mainly focused on deformability and strength parameters for all sand and clay layers around the underground
structures. The main objective of this testing programme was to evaluate the properties of the soil mass after loosening, what is indicated by the penetrometer results. Especially for non-cohesive materials (sand layers), it was observed the tremendous effect of loosening on the main geotechnical parameters (deformability and strength). Along this section, the upper sand layer (Type 3) was the one more affected by loosening.

3. EVALUATION OF CONSTRUCTION PHASES

Design for implementing Line 4 started in 1988, including site investigations, hydrogeological studies and urban constraint surveys of public utilities, building etc. These investigations and studies indicated to a water table quite high and a geological profile composed by intercalated sand and clay layers. Construction works began in 1989, as well as a monitoring programme, which included surface marks, building settlement pins, tunnel convergence pins, piezometers and water level wells. Considering the total length of 4,8 km, 1,2 km of the underground structures were excavated by cut-and-cover methods (galleries), 2,1 km by tunnelling methods (tunnels) using semi-mechanised open-face shields and the remaining length is relative to stations and shafts. Construction works continued till 1996 in different phases, including some stoppages, when they were completely halted. The main characteristics of the works when stopped were: a) Pre-cast concrete lining segments of the tunnels already installed; b) Filling grouting between ground and tunnel support mostly done; c) Water tightness grouting and segment joint stuffing barely done; d) Track foundation on tunnel inverts mostly done; e) Dewatering system shut off and consequent flooding of underground structures.

From 1996 on, only a controlled monitoring programme has continued, including piezometric and water level measurements using dewatering wells. These measurements pointed to water level rising and later stabilization at its hydrostatic equilibrium position (in average 6 to 7 m below surface), what means that most of the underground structures were flooded and some damages may have occurred to them. Starting in summer 2000, a new geological and geotechnical investigation, based mainly on penetrometer tests, was elaborated. A geotechnical laboratory testing programme was carried out and a more intense monitoring programme was implemented, in order to evaluate changes in geotechnical properties, water level, ground and building displacements [1], [3].

Before dewatering the flooded structures, the findings from investigations, construction records and personal communications pointed to: cracks or even crushes of tunnel lining segments; severe geometry changes from the original circular shape of the support rings, eventually leading to loss of ring stability; presence of sand sediments inside tunnels, indicating leaching through segment joints, and consequently ground loosening around the tunnel; ground settlements and some failure zones observed at the surface.; variations in the hydrostatical level; some damages and gaps in shaft and station structures due to poor construction quality; some misalignments and gaps in diaphragm walls due to poor construction quality.

Fig. 5. View on construction methods according to the geological profile.
Taking into account all findings and probable consequences, it was decided to establish technical solutions for dewatering and construction, aiming to minimize structure and ground displacements and ensure stability. As the tunnel longitudinal axes present an anticline shape, it was established to work independently, in terms of dewatering, in two zones. Then, for the sake of technical reasons and simplicity during construction works, the total length (4.8 km) of Line 4 Southeast was divided into two zones [1] (see Fig. 5):

- Zone 1: from Nicolae Grigorescu Station to PSS IOR 2 Shaft, with 2.2 km of length.
- Zone 2: from PSS IOR 2 Shaft to former Linia de Centura Station, with 2.6 km of length.

4. STRUCTURAL EVALUATION AND SAFETY ANALYSIS

The infrastructure included at that time: tunnels, cut-and-cover structures (galleries and stations), buildings and public utilities (water supply, sewage, power, telephone and communication cables etc.). In order to perform the structural evaluation and safety analysis of these structures, complete reviews of monitoring data and visual inspections were conducted. Initially, for obvious reasons (flooding), visual inspections only focused on shafts and stations. Tunnel excavation was executed using a semi-mechanized open-face shield, with an external diameter of 6.4 m and supported by concrete lining segments. These segments were 0.35 m thick, yielding an internal diameter (initial clearance) of 5.7 m. The tunnel has an overburden varying from 5 to 12 m, crossing all major geological strata. However, layers around tunnels were predominantly composed by sands and gravels and rarely by clays. The distance between tunnels varies from approximately 0.25 to 1.0 tunnel diameter (1.8 to 6.4 m). In general, they were closer in the proximities of stations and shafts and then they separate apart [3], [4].

During tunneling, the water table was lowered and maintained approximately 2 m below the tunnel invert by dewatering wells, which were still in operation at least 50 m ahead and 150 m behind the tunnel face. Tunneling was done by the open face shield. Soil excavation at the shield face was done by shovel when dominated by sands and by pneumatic hammer and shovel when clays. The tunnel face excavation was supported by steel plates. When arriving to or departing from stations and shafts, shield breakthrough was facilitated by concrete bulkheads. This procedure has caused some overbreak in the excavated section, leading to loss of ground and consequently excessive surface settlements, or even some failure zones close to shafts and stations (see Fig. 6.).

![Fig. 6. Shield breaking through. Failure zone.](image)

Another important characteristic of the tunnel excavation that also caused some ground displacements was related to shield advances, which were done with the face fully or partially opened. In some occasions, the shield was not immediately jacked towards the tunnel face in order to increase the excavation rate, leaving some span unsupported and consequently leading to loss of ground (ground movements towards the tunnel excavation perimeter). As the layer around the tunnel was composed predominantly by non-cohesive materials (sands and gravels), these ground movements easily propagated to ground surface, leading to settlements and soil mass loosening (see Fig. 7.).

![Fig. 7. Loss of ground mechanism.](image)
This mechanism may propagate throughout the sand layer if this construction procedure remains, leading to disturbances of the whole layer. On the other hand, sandy ground movements tend to close any empty space between the ground excavation perimeter and the support (lining), which is very positive for the adequate behavior of the tunnel support system [1].

Stations and shafts were built by cut-and-cover methods. Construction phases included diaphragm walls, tiebacks or struts, bottom slab (invert) and the final structure (walls, slabs and pillars). Cut-and-cover structures also required dewatering during construction. But when construction works were halted in 1996, some stages of this standard procedure were not concluded to a satisfactory standard. Open space between ground and tunnel lining allowed water ingress through segment joints, leaching of sand sediments and attack on structures by polluted water. In order to ensure structure safety, it was crucial to point the main events during construction: a) tunnel excavation was performed in very difficult conditions, mainly due to random construction paces and stoppages caused by the lack of investment funds; b) large gaps between segments and some segment misalignments were noticed since the beginning. These misalignments caused some problems during shield pushing, leading to cracks and damages in lining segments. Filling grouting was completed, but the waterproof grouting was barely done. Despite all these problems, tunnels have not presented any major deformation trends or local instabilities, except in Zone 1 and close to tunnel portals; c) structures constructed by cut-and-cover methods (stations and shafts) presented some problems in the diaphragm wall panels. Some cracks occurred or some gaps were left by the concreting process, leading to leaching of sediments towards the structure. This loss of ground easily propagated till surface, due to the nature of the surrounding soils (non-cohesive materials), and consequently produced excessive settlement troughs, requiring later surface repairing [2], [3].

As part of the structural evaluation and safety analysis of the existing infrastructure, monitoring data play an important role. Monitoring started at the beginning of construction in 1989 and up to the stoppage of works was under responsibility of the contractors. After 1996, tunnel convergence monitoring was halted due to flooding, but buildings have been continuously monitored till now. In 1999, a complementary monitoring programme was implemented, focusing on settlement troughs. A plenty of data was provided, including: a) Convergence topographic measurements of tunnels; b) Surface marks for defining settlement troughs; c) Building pins. Monitoring data of building pins have not pointed to any concerns. Most readings float around few millimeters. Analyzing measurements of building pins and surface settlement trough, it was concluded that most buildings are quite safe because they are out of the settlement trough width. Unfortunately there had been no measurements inside the ground (extensometers) [4].

For completing the structural evaluation and safety analysis of the existing infrastructure, visual inspections were conducted, initially, before dewatering, only of stations and shafts, and later, after dewatering, of tunnels and cut-and-cover structures. Also, there were made: 1) Analysis of the water quality, especially in areas clearly identified as contaminated by urban waste and sewage; 2) Analysis of possible negative reaction caused by the contaminated water to the concrete and reinforcement bars 3) Mechanical destructive and electrical non-destructive tests to evaluate the degree of corrosion affecting exposed reinforcement bars. For the underground structures that have been flooded, it was established a plan of cleaning and disinfections to be carried out immediately after dewatering. It was included: the application of hot water under pressure, compressed air and high strength brush, in order to remove all fungus, bacteria, calcinations, oxides etc. It was recommended that the pressure of the water and compressed air mixture should be around 700 kPa [1], [2].

Numerical simulation of typical cross-sections (tunnels and galleries) pointed to:
- For tunnels surrounded by sand and clay layers, dewatering does not impose any relevant loading and displacement to tunnel structures and adjacent ground;
- For tunnels surrounded by a clay layer, dewatering imposes new loadings and displacements to tunnel structures and ground (clay layer consolidation was taken into account).
Despite some values are quite large in magnitude, acting stresses on tunnel lining and convergences were acceptable.

- For galleries surrounded by sand layers, dewatering will not impose any relevant loadings and displacements to gallery structures and adjacent ground, comparing the phases immediately before flooding and just after dewatering. However, dewatering will cause expected changes (magnitude and direction) on the shear force and bending moment diagrams of the diaphragm walls and bottom slab (invert).
- For galleries surrounded by sand and clay layers, dewatering will not impose any loadings and displacements to gallery structures and ground, comparing the phases immediately before flooding and just after dewatering.

In order to increase structural safety, there were performed two actions: a) lowering the ground (external) water level first and then, the inside water level, keeping the flow direction outwards the underground structure; b) performing the gallery dewatering only internally (inside level).

5. SOLUTIONS FOR STRUCTURE REHABILITATION

Considering that most of underground structures were initially flooded, the main concerns were related to dewatering these structures safely and to rehabilitate them structurally. The proposed solutions and technologies were basic for: monitoring, dewatering, ground consolidation, structure water tightness and structure rehabilitation [2], [3].

However, considering the numerical simulation results, the complexity of the problem and economic matters, Prof. Assis suggested alternative solutions, procedures and technologies. These alternative solutions, procedures and technologies, which were applied with excellent results, included three main topics [1], [2]:

- Definition of a step-by-step approach (procedure and guidelines) for the works based on the Observational Method: solutions for all possible scenarios, in order to avoid stoppage of works for lacking of technical solutions, as well as to avoid cost increases during construction for adopting solutions not presented before; a complementary monitoring programme, including measurements of inside ground and tunnel crown settlements; a complementary laboratory-testing programme.
- Alternative solution and procedure for dewatering: external dewatering of the ground water table, using vertical wells, carried out slowly (same velocity as for flooding) up to the tunnel invert level; internal dewatering of the water inside the tunnel (natural drainage or by pontoons) also carried out slowly, to keep the internal (tunnel) water level always above the external (ground) one, in order to establish a flow outwards the tunnel, avoiding any chance of sediment leaching and further ground loosening.
- Ground consolidation technologies, when needed.

For Zone 1, the most affected part of the line, a complementary monitoring programme, including internal ground and tunnel crown settlements, was also implemented, adding two more instruments to the existing monitoring cross-sections (Fig. 8):

- Rod settlement gauges installed on tunnel crowns to measure displacements during dewatering.

Fig. 8. Typical monitoring cross-section.
Multi-point settlement gauges (extensometers) installed on the vertical axis from the tunnel crown to the surface. At least two points are recommended, one 2 m above the tunnel crown and the other in the middle distance from the first one and the surface.

Finally, special for Zone 1, it was performed a complementary geotechnical-testing programme and implemented a complementary monitoring programme. After dewatering, a visual inspection of the Zone 1 underground structures was performed and the consequent structural rehabilitation programme defined [2], [3]. Regarding these aspects, the main conclusions were:

- The geotechnical-testing programme was executed and preliminary parameters were providing reliable data to designers, contractors and client.
- The monitoring programme, including the extensometers, was implemented and used to evaluate structural and ground behavior during dewatering of Zone 1. Measurements were very small on the surface, but within the ground some large values were recorded, especially in the upper sand layer.
- Dewatering of Zone 1 underground structures occurred smoothly, without causing any disturbing loadings and displacements to structures and surroundings.
- In general, the visual aspects of the tunnels indicated to quite reasonable structural conditions. Strength tests of concrete and reinforcement bars provided results around the standard recommended limits. Local cracks and open joints were treated and grouted. A cleaning programme was applied to all flooded structures, since the water quality tests demonstrated some contaminations. This was also important for keeping health safety of the workers.
- Tunnel portals were severely damage and should be reinforced with a secondary lining, and in some cases, replaced by cut-and-cover structures where clearance was jeopardized. One tunnel section presented severe geometry changes, leading to potential stability problems. This section, approximately 180 m in length, required a secondary lining and a new track position.
- Also, due to the displacements measured by extensometers in some locations of Zone 1, a continuous monitoring programme was performed one year after completion and commissioning the line (November, 2009) and the measurements indicated safe and stability.

6. CONCLUSIONS

The main conclusions of rehabilitation works for Line 4 Southeast were:

- All the solutions, procedures and technologies were applied without any risks to the structures.
- The alternative solutions and procedures suggested and applied on the field, concerning dewatering and establishing a step-by-step approach based on the Observational Method (complementary monitoring programme), leaded to excellent results, in increasing ground and structural safety and preparing the infrastructure to receive installations and finishing works (also an EIB loan support) for commissioning the line on November, 2008, with a great success.

REFERENCES


