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MONITORING THE LONG-TERM STRUCTURAL BEHAVIOUR OF THE TURDAȘ TUNNEL (SECTIONAL FORCES)

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Abstract. *This paper analyses the structural behaviour of the Turda Tunnel, constructed entirely from the surface using the cut-and-cover method, and determines the sectional forces based on in-situ measurements. The analysed system comprises a temporary structure consisting of diaphragm walls and steel struts, as well as a permanent structure formed by an inverted-arch base slab, straight walls, and an arch roof. The monitoring programme used inclinometers, vibrating-wire strain gauges, and pressure cells installed in two representative sections of the tunnel. The recorded data were converted into displacements, strains, unit stresses, bending moments, axial forces, and compression forces in the struts. The interpretation of the results took into account the variation of the effective modulus of elasticity of concrete and the effect of creep. The values obtained were analysed for the main construction stages and compared with the design results using stress diagrams, MM-NN interaction curves, and MM-NN-kk diagrams. In the diaphragm walls, the maximum monitored bending moment was 792 kN·m, while the yield moment of the section was 2,688 kN·m. The results indicate a safe structural behaviour, although the diaphragm walls operated predominantly in the cracked state during construction. The inner lining exhibited satisfactory behaviour, with adequate reserves of strength and stiffness. The differences from the design values highlight the influence of the actual construction stages, the applied construction technology, and the creep effect, confirming the need to calibrate the calculation model based on monitoring data.*

Keywords: *Turda Tunnel; tunnel constructed from the surface; cut-and-cover method; structural monitoring; sectional forces; diaphragm walls; inner lining; in-situ measurements; creep; MM-NN-kk diagram.*

INTRODUCTION

Tunnels constructed from the surface represent a solution used when the placement depth allows the execution of an open excavation. The cut and cover method involves constructing the structure inside the excavation, followed by restoring the ground above the tunnel. The behavior of such a structure is influenced by the characteristics of the soil, earth pressure, excavation stages, and the sequence of construction of the structural elements.

Structural monitoring is important both during construction and during the operation period, as it allows the tracking of displacements and the internal forces developed in the tunnel elements. The data obtained through in situ measurements can be used to verify the design assumptions, identify critical stages, and calibrate numerical calculation models.

The Turda Tunnel was constructed in a fully open excavation using the cut and cover method. The structural solution includes a temporary structure consisting of diaphragm walls and steel struts, as well as a permanent structure represented by the inner reinforced concrete lining. This consists of an inverted arch base slab, straight legs, and an arch. To monitor the behavior of the two structures, two monitoring stations were provided, located in ring 43 for the temporary structure and in ring 45 for the permanent structure.

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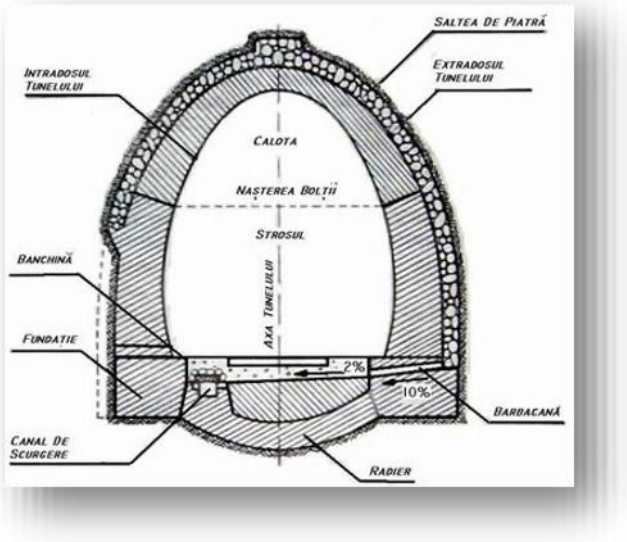


Figure 1. Cross-section through a tunnel

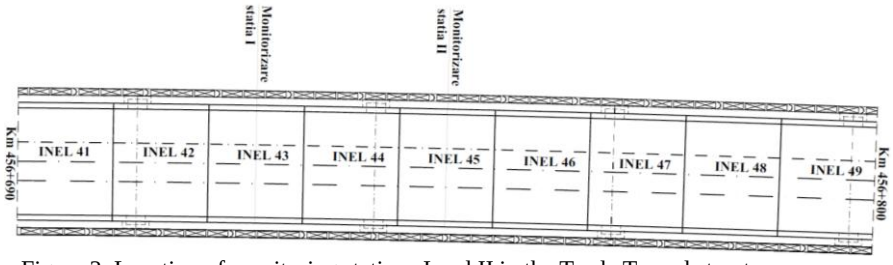


Figure 2. Location of monitoring stations I and II in the Turda Tunnel structure

The objective of this study is to evaluate the structural behavior of the Turda Tunnel during the main construction stages. The analysis follows the transformation of measurements performed with inclinometers, vibrating wire strain gauges, and pressure cells into displacements, strains, bending moments, axial forces, and compression forces in the struts. The monitored values are analyzed in relation to the results provided in the design and with the strength limits of the structural elements.

The study examines the temporary structure and the inner lining separately, as well as their interaction. The results allow the identification of the changes produced by the actual sequence of works, the long-term effects of concrete, and the adaptation of the construction technology to the conditions encountered at the site. The analysis contributes to the verification of structural safety and to the formulation of recommendations for the design and monitoring of tunnels constructed under similar conditions.

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MATERIAL AND METHODS

The research was carried out as a case study on the structural behavior of the Turda Tunnel during construction. The methodology included monitoring the temporary structure and the inner lining, converting the readings provided by the instruments into engineering quantities, and determining displacements, strains, unit stresses, bending moments, and axial forces. The results were correlated with the sequence of construction stages, in order to highlight the transfer of internal forces and the interaction between the two structures.

The temporary structure of the tunnel consists of diaphragm walls made of C25/30 reinforced concrete, with a thickness of 0.80 m, supported by two levels of steel struts. The permanent structure is made of C30/37 reinforced concrete and includes the inverted arch base slab, the straight legs, and the arch. For the sectional calculations, a structural strip with a width of 1.00 m was considered. The main geometric and mechanical characteristics of the analyzed elements are presented in Table 1.

Table 1

Characteristics of the tunnel structural elements

Structural element	**Concrete class**	**Considered section (cm)**	E_{cm} **(N/mm²)**	f_{ck} **(N/mm²)**	f_{ctm} **(N/mm²)**	f_{cd} **(N/mm²)**
Diaphragm walls	C25/30	80 × 100	31.000	25	17	2,60
Inverted arch base slab	C30/37	100 × 100	33.000	30	20	2,90
Straight legs	C30/37	60 × 100	33.000	30	20	2,90
Arch	C30/37	60 × 100	33.000	30	20	2,90

The reinforcement of the structural elements was made of S500 steel, for which the design resistance was considered. The struts were made of S235 steel, having a modulus of elasticity of 210,000 N/mm², characteristic resistance $f_{yk} = 235\ \text{N/mm}^2$, and a design resistence $f_{yd} = 204\ \text{N/mm}^2$. The upper strut SP I has a dimension of 609.6 × 14 mm, while the lower strut SP II has a dimension of 812 × 8 × 14 mm.

The monitoring system was organized in two stations located in the sectors with the maximum cover and internal forces. Station I, located in ring 43, was intended for monitoring the temporary structure, while Station II, located in ring 45, was used to monitor the inner lining. The location of the two stations allowed the separate analysis of the structures, as well as the evaluation of their interaction during construction.

Station I included vibrating wire strain gauges mounted on the reinforcement of both diaphragm walls, pressure cells installed at the level of the two struts, and inclinometer tubes placed in the walls. The strain gauges were arranged on both faces of the section, in the areas corresponding to the maximum values of the bending moments. Three levels were monitored: level I, located approximately 2.45 m below the capping beam; level II, at approximately 8.30 m; and level III, at approximately 12.70 m, at the level of the foundation of the inverted arch base slab.

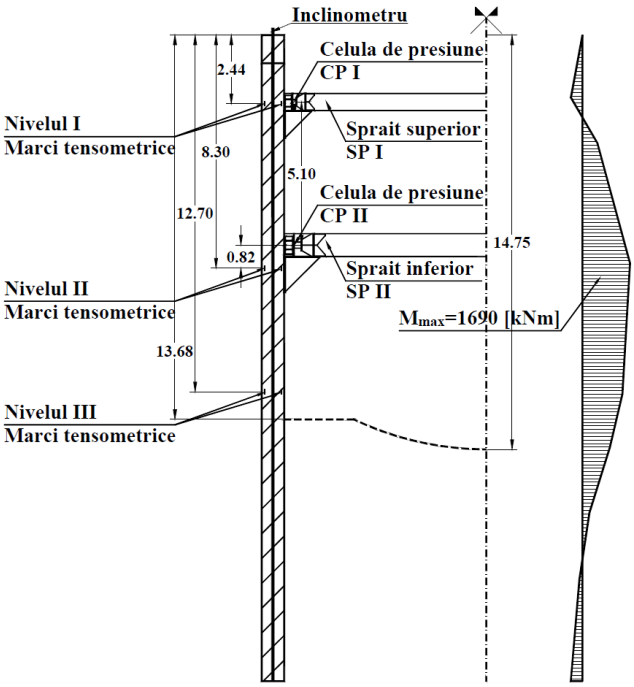


Figure 3. Placement of instruments at Monitoring Station I for the temporary structure.

Station II was intended for monitoring the permanent structure, consisting of the inverted arch base slab, straight legs, and arch. In the base slab, three strain gauges were installed in the section, at the inner face, at the middle, and at the outer face, in the central position and in the two lateral zones. In the straight legs and at the springings of the arch, two strain gauges were used in each section, while three strain gauges were installed at the crown of the arch. The positions of the instruments were correlated with the bending moment envelope obtained during the design stage.

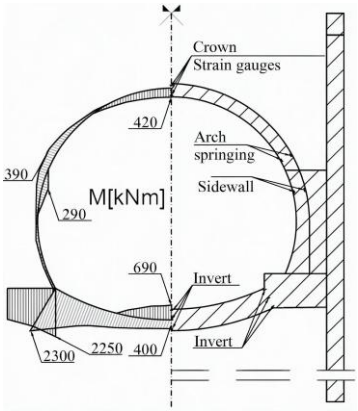


Figure 4. Location of instruments at Station II for monitoring the permanent structure

The construction of the inner lining was adapted to the technology available on site. The permanent structure was constructed in three stages: construction of the invert and placement of the infill concrete, construction of the sidewalls together with the backfill behind them, and construction of the arch. This sequence differed from the initial design solution, in which the inner structure was intended to be constructed in two stages, and influenced the distribution of internal forces and the interaction with the diaphragm walls.

The instrumentation used included C17 Digital Inclinometer vertical inclinometers, Sisgeo 0VK4200VC00 vibrating wire strain gauges, and pressure cells installed at the strut levels. Readings from the strain gauges and pressure cells were taken manually using the CRD400 Digital Reader. The inclinometers enabled the determination of lateral displacements of the diaphragm walls, the strain gauges provided strain measurements, and the pressure cells recorded the compressive forces developed in the struts.

Table 2

Instrumentation used for monitoring the Turda Tunnel

Instrument	**Type/model**	**Location**	**Measured quantity**
Vertical inclinometer	C17 Digital Inclinometer	Diaphragm walls of both tunnel tubes	Lateral displacement along the depth
Vibrating wire strain gauge	Sisgeo 0VK4200VC00	Diaphragm walls, invert, sidewalls, and arch	Strain
Pressure cell CP I	Maximum capacity: 4,000 kN	Upper strut SP I	Compressive force
Pressure cell CP II	Maximum capacity: 8,000 kN	Lower strut SP II	Compressive force
Manual data acquisition device	CRD400 Digital Reader Sisgeo	Connected to strain gauges and pressure cells	Signal reading and recording
Aparat	Tip/model	Amplasare	Mărime determinată
Înclinometru vertical	C17 Digital Inclinometer	Pereții mulați, pe ambele fire	Deplasarea laterală pe adâncime
Marcă tensometrică cu coardă vibrantă	Sisgeo 0VK4200VC00	Pereți mulați, radier, picioare drepte și botă	Deformația specifică
Celulă de presiune CP I	Capacitate maximă 4.000 kN	Șpraiț superior SP I	Forța de compresiune
Celulă de presiune CP II	Capacitate maximă 8.000 kN	Șpraiț inferior SP II	Forța de compresiune
Aparat de achiziție manuală	CRD400 Digital Reader Sisgeo	Conectat la mărci și celule	Citirea și înregistrarea semnalelor

The readings provided by the strain gauges in "Digit" units were converted into microstrain using the calibration factor specified by the manufacturer. For the 0VK4200VC00 gauge, the factor $G = 3{,}814$ was used, and the change in strain was determined relative to the initial reading using the following relation:

$$\Delta\varepsilon = (L_i - L_0) \cdot G,$$ where L_0 the initial reading, L_i the reading corresponding to the stage under analysis, and G the strain gauge calibration factor are represented, respectively. Positive values were associated with tension, and negative values with compression.

The pressure cell readings, expressed in mV/V, were converted into compressive forces based on the sensitivity and rated capacity of each instrument. A nominal sensitivity of 2 mV/V was considered for both cells, with a maximum capacity of 4,000 kN for CP I and 8,000 kN for CP II. The force in the strut was determined by relating the electrical reading to the sensitivity of the corresponding instrument. For the inclinometers, the displacement of each wall was obtained from the difference between the reading taken at a particular stage and the initial reading taken after the installation of the tubing.

The determination of the bending moment and axial force was based on the Navier–Bernoulli hypothesis, according to which plane sections remain plane after deformation. Based on the strains measured on opposite faces of the section, the curvature was determined using the following relation:

$$k = \frac{\varepsilon_{\text{sup}} - \varepsilon_{\text{inf}}}{h},$$ where ε_{sup}și ε_{inf} are the strains at the extremities of the section, and **h** represents the distance between them. The bending moment was calculated using the moment–curvature relationship

$$M = E_{c,\text{eff}} I k,$$ where $E_{c,\text{eff}}$ is the effective modulus of elasticity of concrete, **I** is the moment of inertia of the section, and k is the curvature determined from measurements.

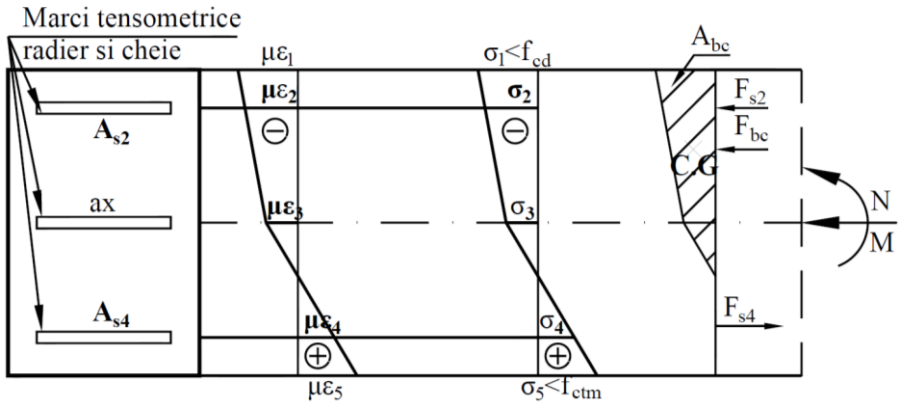


Figure 5. Theoretical model used to determine sectional internal forces from measured strains

The effect of concrete creep was included by using the effective modulus of elasticity, determined as a function of the concrete age and the duration of loading. The following relation was used: $E_{c,\text{eff}} = \frac{E_{cm}}{1 + \varphi(t, t_0)}$, where E_{cm} is the mean modulus of elasticity, and $\varphi(t, t_0)$ represents the creep coefficient. The effective modulus values were calculated separately for the diaphragm

walls, invert, sidewalls, and arch, as these elements were constructed and loaded at different times. The concrete stresses were obtained by multiplying the strains by the effective modulus of elasticity. These were compared with the mean tensile strength f_{ctm} and the design compressive strength. When the tensile strength of the concrete was exceeded, the section was considered cracked, and the determination of the axial force took into account the force of compression in the concrete and the forces developed in the reinforcement of the section. For the diaphragm walls, the absence of a strain gauge at the middle of the section required the use of correction factors when determining the axial force.

Table 3

Construction stages and corresponding works at the Turda Tunnel

Stage	Work performed
I	Excavation to approximately 1 m below the lower strut SP II
I	Installation of the lower strut SP II and continuation of excavation to the invert in tunnel tube II
II	Excavation to invert level in both tunnel tubes
V	Construction of the invert and placement of the infill concrete
V	Removal of the lower strut SP II
I	Construction of the sidewalls
II.I	Construction of the arch and placement of partial backfill
II.II	Removal of the upper strut SP I
III	Placement of backfill up to the level of the capping beam
X	Placement of backfill to near the original ground level

The analysis addressed the representative stages of the construction process, starting with excavation below the lower strut level and continuing with the construction of the invert, the successive removal of the struts, the construction of the sidewalls and arch, and the reinstatement of the backfill above the tunnel. The sequence of stages selected for interpreting the results is presented in Table 3. For each stage, the displacements of the diaphragm walls, the compressive forces in the struts, and the bending moments and axial forces in the temporary structure and inner lining were analyzed. The maximum values and the stages at which they occurred were compared with the design results. The safety of the representative sections was assessed using interaction curves $M-N$ and diagrams , $M-N-k$ by comparing the monitored internal forces with the cracking, yielding, and failure moments.

RESULTS AND DISCUSSIONS

The monitoring results were analyzed separately for the temporary structure, consisting of diaphragm walls and steel struts, and for the permanent structure, consisting of the invert, sidewalls, and arch. The interpretation focused on the evolution of bending moments and axial forces during the main construction stages, as well as the differences between the monitored values and those considered in the design.

Temporary structure

In the diaphragm walls, bending moments increased as the excavation deepened and the temporary supports were successively removed. At the second monitoring level, the maximum bending moment increased from 544 kN·m in stage I to 696 kN·m in stage III. After the construction of the invert and the removal of the lower strut, the values reached 723–755 kN·m, indicating the gradual transfer of lateral pressures to the inner structure.

Table 4

Evolution of bending moments in the diaphragm walls

Stage	***Construction condition***	***Tunnel tube I (kN·m)***	***Tunnel tube II (kN·m)***
Stage	***Construction condition***	***Tunnel tube I (kN·m)***	***Tunnel tube II (kN·m)***
Stage	***Construction condition***	***Tunnel tube I (kN·m)***	***Tunnel tube II (kN·m)***
Stage	***Construction condition***	***Tunnel tube I (kN·m)***	***Tunnel tube II (kN·m)***
Stage	***Construction condition***	***Tunnel tube I (kN·m)***	***Tunnel tube II (kN·m)***
I	***Excavation below the lower strut***	***480***	***544***
II	***Excavation to invert level***	***589***	***696***
V	***Removal of the lower strut***	***723***	***755***
I	***Construction of the sidewalls***	***769***	***792***
II	***Construction of the arch and removal of the upper strut***	***253***	***248***

The maximum monitored bending moment was 792 kN·m and was recorded in tunnel tube II after the construction of the sidewalls. Following the construction of the arch and the removal of the upper strut, the bending moment decreased to approximately 250 kN·m in both tunnel tubes. This decrease confirms the interaction between the diaphragm walls and the inner lining, which took up a significant proportion of the lateral earth pressure.

The maximum monitored bending moment remained well below the section's yielding moment of 2,688 kN·m. Analysis of the diagram $M - N - k$ shows that the diaphragm walls were in the cracked state during the main excavation stages, but the bending moments remained closer to the cracking moment than to the yielding moment. After closure of the permanent structure, the reduction in bending moments brought the section's behavior closer to the uncracked state.

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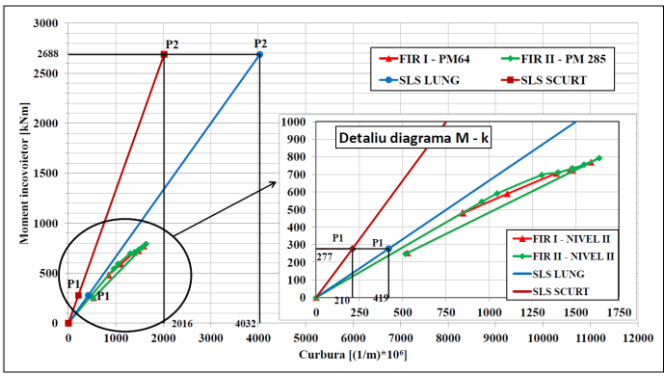


Figure 6. Position of the monitored values on the diagram $M - N - k$ for the diaphragm walls.

Permanent structure

In the permanent structure, the first internal forces were recorded after the construction of the invert. The bending moments in the lateral zones increased from 82–88 kN·m in stage IV.II to 144–148 kN·m after the construction of the sidewalls and to 282–289 kN·m after the construction of the arch and placement of the partial backfill. This evolution confirms that the invert progressively took up the lateral pressures transmitted by the diaphragm walls.

In the sidewalls, differences were observed between the two monitoring lines, caused by the non-uniform distribution of lateral pressures and by the actual sequence of construction works. In the arch, the highest stress was recorded at the crown after removal of the upper strut, when the bending moment temporarily reached 377 kN·m. After completion of the final backfill, the bending moment decreased to 163 kN·m, and the safety factor increased from 1.72 to 3.25. The analysis of the section at the arch crown shows behavior under eccentric compression, without reaching the strength limit. In the final stage, the section remained in the uncracked and elastic range. The results indicate that a sufficient strength reserve is available to withstand the operational loads. In general, the monitored values were lower than those estimated in the design. The differences can be explained by the actual sequence of construction works, the long-term effects of concrete, and the interaction between the diaphragm walls and the inner lining. Monitoring confirmed the satisfactory behavior of the tunnel and the importance of using in-situ measurements to verify and calibrate the calculation model.

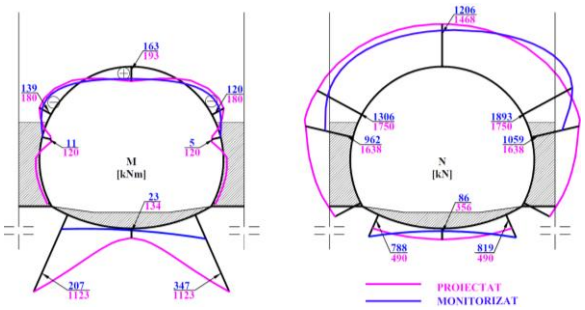


Figure 7. Comparison between the design and monitored internal forces in the inner lining.

CONCLUSIONS

Monitoring carried out during the main construction stages made it possible to assess the behavior of the temporary structure and the inner lining of the Turda Tunnel. Measurements taken with inclinometers, strain gauges, and pressure cells provided data for determining displacements, bending moments, axial forces, and the forces developed in the struts.

The results highlighted the evolution of internal forces according to the sequence of construction works. The maximum monitored bending moment in the diaphragm walls was 792 kN·m, significantly lower than the yielding moment of the section, 2,688 kN·m. After construction of the arch and removal of the upper strut, the bending moments in the diaphragm walls decreased considerably, confirming the transfer of internal forces and the interaction between the two structures.

The differences between the design and monitored results were influenced by the actual construction sequence, the creep effects of concrete, the placement of backfill, and the way the interaction between the ground and the structure was modeled. For this reason, in-situ measurements are useful for calibrating calculation models and for establishing design parameters that are closer to the actual behavior.

Overall, the sectional forces remained within the allowable limits, and the structure exhibited satisfactory behavior throughout the monitored stages. For a more accurate determination of axial forces, it is recommended to use three strain gauges in each analyzed section and to continue monitoring the inner lining during the operational period, particularly to observe any effects caused by soils with swelling potential.

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