Panel discussion: Tunnelling for Turin Railway Link
Débat de spécialistes: Réalisation du tunnel pour le raccord ferroviaire de Turin

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ABSTRACT: This paper presents the underground works carried out in connection with the construction of the Turin Railway Link, in North Italy, where specialized techniques of ground treatment and pre-support have been adopted over a total tunnel length of 352 m. Following a brief description of the ground conditions, observations and monitoring for a typical section during excavation are discussed.

RESUME: Ce texte présente les travaux souterrains concernant la construction du Raccord Ferroviaire de Turin, en Italie du nord, ou des techniques particulières de traitement de terrain et de pre-support ont été adoptées pour la longueur total d'un tunnel de 352 m. Suite à une brève description des conditions du terrain, les résultats des observations et du monitorage sur une section typique pendant l'excavation, sont expliquées.

1. INTRODUCTION

Turin is in the process of upgrading the city railway system. This will allow to improve links with the main Italian cities and to create a regional network that offers passengers connections with urban transport via six stations. As shown in the general layout of Project Phase 2, between the railway stations of Lingotto and Porta Susa (Fig. 1), the lines and stations will be mostly underground, in order to link parts of the city formerly separated by the old lines which crossed the city in trenches. This will promote a remodelling of the urban environment with construction of a tree-lined central avenue ("Spina Centrale"). According to the new General City Planning, a 10 km long complex will be created with landscaped areas, services, residential buildings, technological and business sections.

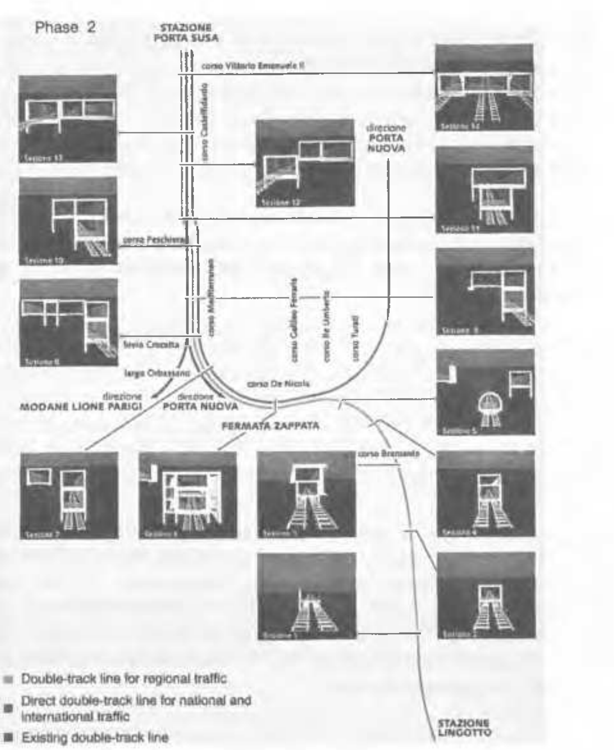

Fig. 1 - Layout of Project Phase 2 (Lingotto-Porta Susa) of the Turin Railway Link. The cross-sections indicate the complexity of the works.

Following construction of the North Section of the railway link between 1986 and 1989 (Phase 1), the Phase 2 is near to completion. This is providing four new completely underground tracks which are added to the two existing ones. With the covering of the railway trenches completed, the construction of the tree-lined avenue and green areas, as mentioned above, is to start in spring 1998. The Phase 3 of the Project which will link the North and South Section is at present in the design stage.

Most of the construction work for Phase 2 took place by using the cut and cover method. Due to the difficulty experienced with the usual clamshell bucket, boring of the diaphragm walls involved the use of "hydromill" units. However, a total tunnel length of 352 m was excavated by conventional methods of underground construction and by using specialized techniques of ground treatment and pre-support. This presentation will be devoted to this latter section of the project.

2. GEOLOGICAL CONDITIONS AND GEOTECHNICAL CHARACTERIZATION

The subsoil conditions in Turin are characterized by the presence of a sand and gravel deposit. Typically this comprises a layer of sand and gravel, ranging from medium to highly dense, down to a depth of 8 to 10 m; below this depth lenses of conglomerate are often present. This condition is well represented in Fig. 2, where a typical cross section of the soil at the face is

Fig. 2 - Typical cross section of the soil at the face during excavation of the cut and cover tunnel: ch 3155 m.

shown during excavation of the cut and cover tunnel. The gray less dense upper layer is gradually changing into the brown one, which is locally intercepted by cemented lenses of conglomerate.

2.1 *Investigation methods*

A comprehensive investigation has been carried out for the purpose of geological and geotechnical characterization and evaluation of design parameters. As usual, some difficulties were met in the characterization process of granular soils, due to the heterogeneities and the presence of cemented material. In all cases, it should be noted that, starting with Project Phase 2 (from 1990 to 1992) and moving into Phase 3 (with the tender design in progress), we have developed a progressively better understanding of soil behaviour, gaining confidence in design assumptions.

The following main investigations were carried out for design purpose:

- in situ investigations
  - borehole logging and sampling
  - standard penetration tests
  - recording of drilling parameters
  - seismic refraction, seismic reflection, and down-hole tomography
  - measurement of groundwater level
- laboratory testing
  - grain size distribution
  - mechanical tests on conglomerate samples.

2.2 *Results*

- NSPT Tests

The results of standard penetration tests in a number of boreholes are shown in the following table. On the basis of 170 tests (48 tests - 23% - with no penetration), the mean value for penetration resistance is 62. No significant change in penetration resistance in noted with depth, whereas a tendency to increase is clear in moving from zones 1-2 (Lingotto) to zones 3-4 (Porta Susa).

NSPT Results

(a) Phase 2 - Lingotto to Porta Susa

Penetration Resistance

depth b.s. (m)	n. data	mean	st. dev.
0.00-50.00	170	62.24	14.01
0.00-10.00	51	60.27	17.55
10.00-20.00	66	64.16	12.01
20.00-50.00	53	61.73	12.35

(b) Distribution

Penetration Resistance

zone	Chainage (m)	n. data	mean	st.dev.
1	1470-3000	44	61.70	17.24
2	3000-4300	78	59.11	12.17
3	4300-4810	21	67.23	11.92
4	4810-6119	27	68.25	12.18

- Observations made during drilling

Maximum use was made during drilling of systematic recording of the following main boring parameters:

- penetration rate
- rate of rotation
- bit pressure
- torsional moment
- flush pressure.

A typical plot of these parameters is shown in Fig. 3.

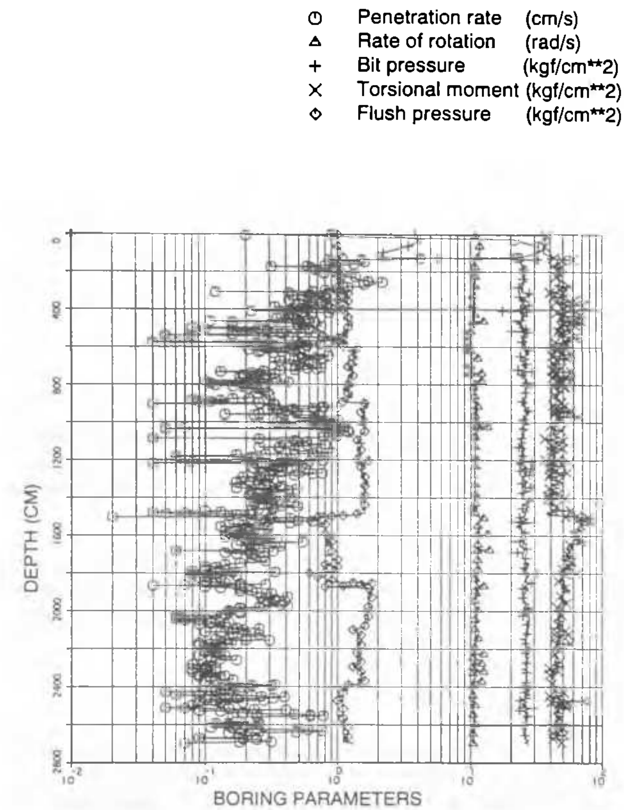

Fig. 3 - Typical plot of boring parameters used to compute specific energy vs. depth.

Based on the numerical values of the boring parameters which were subjected to statistical interpretation, the "ground specific energy" E (z) was computed as follows:

$$E(z) = Kz + E_o \ (KJ/m^3)$$

where:

z = depth below ground surface

$K = 61 \cdot MJ/m^3/m$

$E_o = 229\ MJ/m^3$.

- Seismic refraction survey

A number of seismic regraction surveys were carried out. This allowed the measurement of the longitudinal wave velocity V_p for three seismic units, from the ground surface to lower depth, as follows:

	V_p (m/s)
1	380-450
2	720-880
3	1376-1833

- Ground water level

The monitoring of the ground water level along the tunnel from 1979 to the present, the use of hystorical data available on the same ground water level, and the computation of flow nets allowed the development of a predictive model through time for the distribution of the piezometric heads up to the year 2010. The results obtained are depicted in Fig. 4, which shows a gradual increase of the piezometric head.

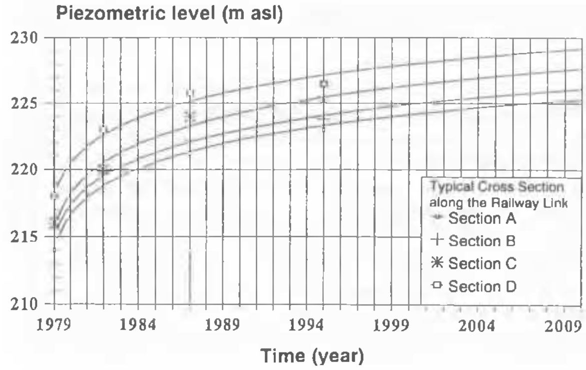

Fig. 4 - Groundwater level along the tunnel up to the year 2010. Actual data and modelling prediction.

- Grain size distribution

The following grain size distribution has been determined on a large number of samples obtained during borehole drilling:
- gravel and pebbles 51%
- sand 31%
- silt and clay 18%.

According to depth below surface the results are as follows:

Percent values	0.0 -50.0 m (115 samples)	0.0 -10.0 m (32 samples)	10.0 - 20.0 m (40 samples)	20 - 50.0 m (43 samples)
pebbles	0.86±3.10	1.22±4.05	0.72±2.66	0.70±2.68
gravel	50.07±17.26	54.78±15.15	54.05±14.99	42.86±18.58
sand	31.38±11.82	29.28±7.07	30.80±13.27	33.49±13.07
silt	16.00±10.4	13.00±10.96	13.00±5.75	20.00±11.40
clay	1.48±2.62	0.75±2.13	0.67±1.53	2.77±3.22

- Discussion

Based upon the results of the observations made during drilling (systematic recording of the boring parameters) and seismic refraction survey a significant improvement of the ground mechanical properties (i.e. deformability and strength) versus depth has been anticipated. In order to gain in the assessment of the geotechnical parameters to be used for design purposes, more recently, with Project Phase 3, deep test pits, 2 m in diameter and down to 15 to 20 m maximum depth, have been excavated. This allowed direct observation of subsurface conditions, collection of block samples of conglomerate, and performance of in situ plate tests, both vertically and horizontally, for the assessment of soil stiffness.

3. TUNNELLING METHODS

The underground work was carried out by using the cut and cover method for most of the tunnel length, according to a complex system of superposed and/or adjacent tunnels, always excavated without any interruption or interference with the existing double-track line (Fig. 1). Due to the constraints posed by the need to underpass this same line, a total tunnel length of 352 m needed be excavated by conventional methods as described in the following.

- Underground tunnel

The underground tunnel was excavated in 3 partial cross sections (crown, bench, invert) by using three different systems of ground treatment and pre-support as shown in Fig. 5:

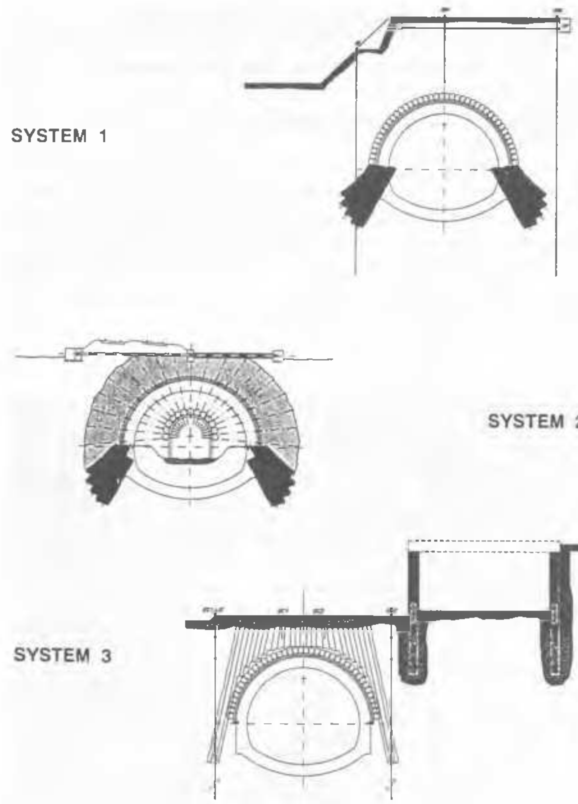

Fig. 5 - Ground treatment and pre-support methods adopted for the turmel excavated in 3 partial cross sections: crown, bench, invert.

- System 1: a jet-grouting arch created which consists of 60 cm diameter jet-grouting columns, driven ahead from the face to a length of 13 m, integrated with steel pipes inserted underneath; following crown excavation, the opening of the full section takes place under the protection of jet-grouting columns on the sidewalls.
- System 2: a supporting structure is placed below the railway tracks consisting of horizontal pipes driven orthogonally to the tunnel axis; a pilot tunnel is then excavated to allow for the creation of a 4 m thick arch consolidated by means of cement and non contaminant silicate injections; widening to the final cross section, by crown, bench and invert, then takes place under an umbrella of steel pipes.
- System 3: a reinforced arch around the tunnel is formed by means of jet-grouting columns driven down from the ground surface; crown, bench and invert excavation is then carried out, as usual under an umbrella of steel pipes driven ahead from the tunnel face.

- Performance monitoring

Performance monitoring was carried out during excavation, with the purpose to gain in the understanding of the deformational behaviour of both the ground surrounding the tunnel and the supporting structures. The most attention was devoted to measurement of surface movements and comparison between numerical model predictions (use was made of Finite Element and Finite Difference Modelling) and actual field behaviour. The purpose has always been to validate the assumed geotechnical parameters and the ground-support interaction schemes adopted for design.

The following measuring techniques have been used:
- surface levelling (this was extremely important for the actual monitoring of the railway line just above the excavation)
- convergence measurements
- multi-point borehole extensometers
- inclinometer tubes
- surface settlement systems.

In order to give a description of a typical cross section where monitoring was performed during excavation, reference is made in the following to the ground treatment and pre-support system 1 (Fig. 5 and 6). Displacements were measured by means of the multi-point borehole extensometers EB1, EB2; inclinometer tube IB; and surface settlement system SP.

Fig. 6 - Photograph of the ground conditions at the face in the underground tunnel. Details are shown of the jet-grouting columns and steel pipes as used according to ground treatment and pre-support system 1.

The tunnel construction process according to system 1 was carried out by advancing first the crown under the protection of the jet-grouting columns and steel pipes which form umbrella arches as a prelining. The excavation was incrementally advanced in steps of one metre to a fixed pattern. As shown in Fig. 6, steel sets are incorporated in shotcrete which is generally applied in two layers, each reinforced by steel mesh, to form a lining typically 250 mm thick.

The excavation of the lower heading (bench and invert) took place once the crown was excavated over the full tunnel length (352 m). In order to increase the bearing capacity of the ground along the lower edge of the shotcrete lining, and to improve stability of the sidewalls during bench excavation, a number of jet-grouting columns were driven down along the full length of the system 1 section, as shown in Fig. 5. A final lining of cast - in situ concrete was added inside the shotcrete lining at a later date to complete tunnel construction.

Monitoring during excavation was undertaken in order to obtain timely information about the performance of the tunnel. A typical plot of the ground displacements above the crown (EBI multi-point borehole extensometer) is given in Fig. 7 for the different ground treatment and excavation sequences. It shown that the ground just above the umbrella arch (monitored by the anchor 6 m below the surface) moves downward 8 mm with respect to the extensometer head as a consequence of the jet-grouting operation and crown excavation. Small relative movements occur in the meantime for the ground at 2 and 4 m below surface. With the tunnel excavation completed (July 1994) a stable condition is attained around the tunnel, with limited settlements of the surface resulting from the tunnelling work.

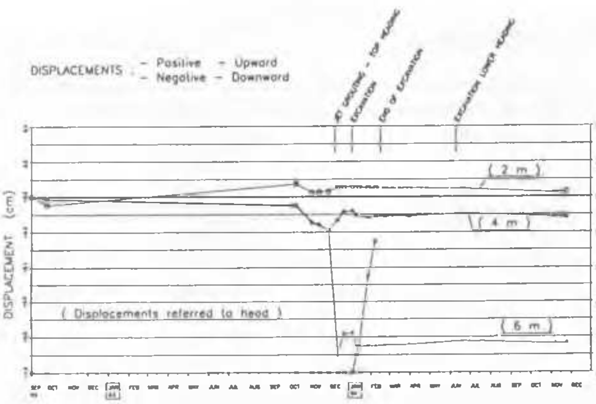

Fig. 7 - Ground displacements during tunnel excavation measured along the EB1 multi-point borehole extensometer above the crown.

4. CONCLUDING REMARKS

By placing the most emphasis on observations and monitoring during excavation, the tunnelling work for the Turin Railway Link could be completed successfully. It should be noted that the observations and monitoring were combined with numerical modelling. Therefore, the deformational behaviour and response of the ground around the tunnel, with different levels of ground treatment and pre-support could be assessed, as an increased level of confidence on the model behaviour and geotechnical characterization was being developed through time.

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