Geotechnical Aspects of Underground Construction in Soft Ground, Kastner, Emeriault, Dias, Guilloux (eds)
 ISBN 2-9510416-3-2

Analysis of monitoring data from a deep tunnel in a tectonized clay-shale (Raticosa tunnel, Italy)

D. Boldini, A. Graziani & R. Ribacchi
University of Rome "La Sapienza", Rome, Italy

ABSTRACT: The Raticosa Tunnel is one of the several tunnels currently under construction for the new Bologna-Florence high-speed railway line (Italy). The tunnel, having an overburden of up to 500 m, is located in the Apennine chain and half of its 10 km length crosses a tectonized clay-shale formation called Chaotic Complex. Full-face excavation was adopted with reinforcement of the tunnel face by means of fiber-glass dowels. The primary lining consists of a closed ring of shotcrete and steel sets. Due to the heavy squeezing ground conditions, predicted in the preliminary investigation phase, the excavation of the tunnel was performed under the strict control of an extensive monitoring system.
In this paper, the in situ measurements of face "extrusion" and tunnel wall convergence have been analyzed and correlated with the single construction stages and specific ground conditions. The deformation pattern near the tunnel face generally calls for core reinforcement and early invert closure so as to assure tunnel stability. Large plastic time-dependent deformations of the ground have generally occurred, which have determined local failure phenomena at some locations along the tunnel route. A tentative explanation of the observed behavior has been proposed on the basis of simplified analytical models. A more thorough back-analysis based on visco-plastic models is currently under way.

1 THE RATICOSA TUNNEL

The Raticosa tunnel (10 km) is one of the tunnels under construction for the new Bologna-Florence high-speed railway line (Italy) which crosses the Apennine Chain. From the Bologna side, half of the tunnel is excavated in a tectonized clay shale formation called Chaotic Complex which since the design stage represented one of the most critical points in the whole project, due to poor geotechnical conditions and a high overburden of up to 500 m (Fig. 1). The same formation is also crossed by the Osteria access tunnel which runs almost parallel to the main tunnel before turning to intersect it.

Due to the expected heavy squeezing conditions, full face-excavation required face reinforcement using fiber-glass dowels. A closed-ring primary lining consisting of a shotcrete layer and steel sets was adopted.

All the tunneling steps were carried out under the careful control of tunnel deformation and loading conditions of the lining in order to ensure compliance with the construction quality requirements, verify the design assumptions and calibrate reinforcement. Moreover the large amount of data gathered during construction represents a valuable tool for the back-analysis of the mechanical behavior of the tectonized clay-shales for which the geotechnical characterization in the preliminary investigation stage was difficult and affected by many uncertainties.

1.1 Geological conditions

Tectonized clay shale formations are rather widespread throughout Italy (Picarelli et al., 2000). The Chaotic Complex encountered during the excavation of the Raticosa tunnel belongs to the Liguridi units

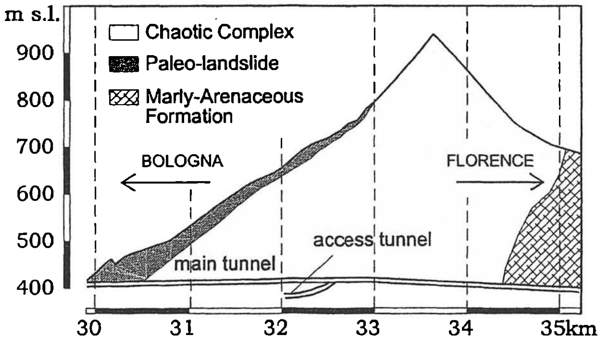

Figure 1. Geological profile along the tunnel route.

and reached the current position after intensive tectonic events which took place from the Miocene-Pliocene to the Plio-Pleistocene.

The chaotic structure of the formation is the result of such tectonic actions. Lithologically it is mainly composed by a pelitic matrix with dispersed lithic components. The pelitic matrix is constituted by an assemblage of clay scales of the size of millimeters to centimeters whose surfaces are curved, smooth and at times striated. The lithic components include calcareous, marly or arenaceous blocks for a total volume fraction varying between 0 and some tens %. Because of the low percentage of the lithic components, mainly in the form of disarranged strata and fragments, the mechanical behavior of the Chaotic Complex is governed by the clay shale matrix.

The northern stretch of the Raticosa tunnel (Bologna side) crosses a large paleo-landslide area for approximately 500 m (Fig. 1). Deformation phenomena are still active: movements of more than 4 mm per year were measured at the surface of the landslide body.

1.2 Geotechnical characterization

The most critical task during the investigation stage was the assessment of the mechanical properties of the tectonized clay shales mainly because of the high sensitivity of the pervasively fissured material to sampling disturbance. A review of the basic data is given in Table 1.

The index properties do not vary significantly neither with the position along the tunnel route nor with depth: natural water content and plasticity index indicate that the tectonized clay shales fall in the domain of low-to-medium plasticity clays. Moreover, the index properties and the grain-size distribution strongly depend on the disaggregating technique adopted in preparing the samples. Only prolonged work with the spatula will destroy most of the interscale diagenetic bonds, thereby increasing the clay fraction (Picarelli et al., 2000).

The saturation degree is generally lower than unity also for very deep samples: this is quite a common finding in these formations, partly explained by the opening of the fissures due to sampling disturbance.

Compressibility was measured in the laboratory by means of oedometer compression tests: as normally occurs with scaly clays, also the Raticosa shales exhibit a very high swelling index and therefore a high ratio between the swelling index and the compression index. The dynamic stiffness can be considered an upper limit of the actual in situ stiffness of the rock-mass subjected to excavation-induced load. P-wave and S-wave velocities measured by the Authors directly on shale blocks taken at the face under an overburden of 480 m showed a dynamic shear modulus of around 2900 MPa. P-wave velocities ranging from 2000 to 2500 m/s were measured in boreholes at a depth of around 150 m.

The deformation and elastic moduli given by dilatometer tests appear to be strongly scattered: the deformation modulus is 140 MPa for a 0.5 to 2.5 MPa load increment and 3900 MPa for a 2.4 to 5.4 MPa load increment. Similarly, the elastic modulus is 250 MPa for a 2.5 to 0.5 MPa load reduction and becomes 3950 MPa for a 3.9 to 2.4 MPa load reduction.

The shear strength of the tectonized shales is strongly affected by the network of fissures. To evaluate the strength parameters of the shales of the Raticosa tunnel, triaxial as well as direct shear tests were performed. The samples were obtained from investigation boreholes as well as from blocks cut at the tunnel face during excavation (Fig. 2). Triaxial tests are deemed to afford the more reliable results. Peak shear strength values seem to indicate two different strength envelopes according to the depth of the sample, with a higher cohesion for the samples deeper than 200 m; no appreciable strain-softening is generally observed beyond peak.

At present no reliable information is available on the water table level and on pore pressure distribution inside the Chaotic Complex. Piezometers installed in the ground during tunnel excavation up to a distance of 15 m from the tunnel wall have not measured so far any pore water pressure. Also the in situ measurement of hydraulic conductivity by borehole pumping tests have failed because of the negligible water flow. Only for the more permeable loosened material of the paleo-landslide meaningful pumping tests could be executed, obtaining permeability values in the range $3 \cdot 10^{-7} \div 2 \cdot 10^{-9}$ m/s.

The lack of information about pore pressure distribution leads to much uncertainty about the effective stress at the tunnel depth and about the overconsolidation ratio of the material.

Table 1. Index and mechanical properties of the tectonized clay shale formation.

Properties	Range	Average
Unit weight of total volume (kN/m^3)	21.2 ÷ 23.6	22.7
Unit weight of solid (kN/m^3)	27.4 ÷ 27.7	27.5
Natural water content (%)	7 ÷ 19	10
Liquid limit (%)	32 ÷48	28
Plastic limit (%)	16 ÷ 22	18
Saturation degree S_r (%)	70 ÷ 91	82
Clay fraction CF (%)	11 ÷ 44	20
Compression index C_c	0.10 ÷ 0.17	0.11
Swelling index C_s	0.03 ÷ 0.06	0.05
P-wave velocity v_P (m/s)	2000 ÷ 2500	
S-wave velocity v_S (m/s)	1000 ÷ 1100	
Peak cohesion c'_p (kPa) *	16 ÷ 540	
Peak friction angle φ'_p (°)	15	

* the lower and upper limits refer to values obtained from samples taken at depths below or above 200 m.

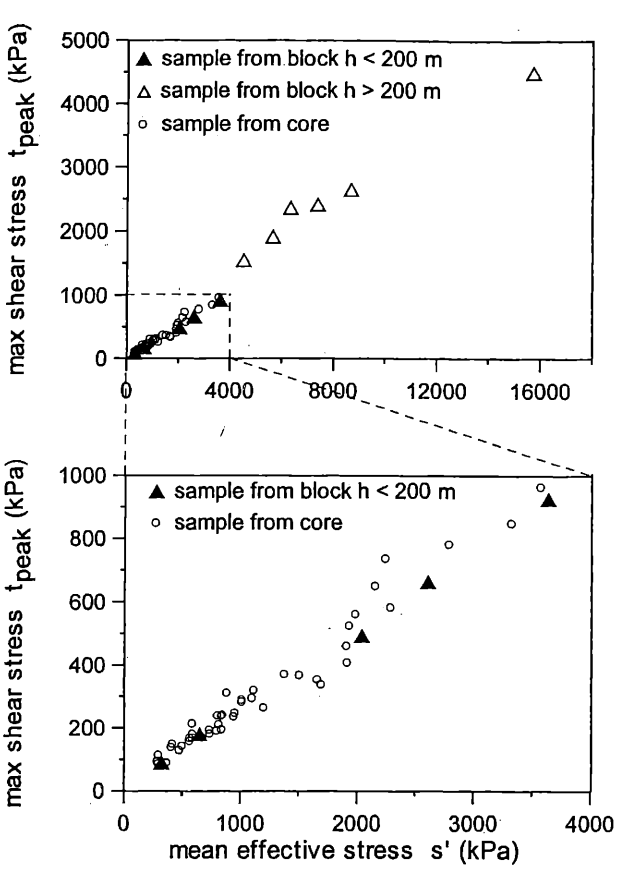

Figure 2. Synopsis of laboratory peak strength measurements of clay shale samples.

2 DESIGN AND CONSTRUCTON

The design of the tunnel was based on the ADECO-RS approach (Lunardi & Focaracci, 1999), whose distinctive feature is to highlight the key role of tunnel face stability for the overall static conditions of the tunnel; it also introduces the principles of the observational method into tunnel design.

Full face excavation was preferred for the operational advantages it secures, such as enabling a high level of industrialized construction and safe working conditions; it also provides static benefits, mainly the possibility of closing the support ring with an invert arch immediately behind the face and the extensive application of face reinforcement techniques.

Figure 3a shows the typical cross-section of the tunnel (area 120÷140 m^2), with the primary lining consisting of a shotcrete layer (thickness 25÷30 cm) and steel sets (including an arched strut in the invert), and the final lining made of reinforced concrete (invert and sidewalls) and plain concrete (vault).

Construction stages are better illustrated by the longitudinal section in Figure 3b: the face is reinforced by a set of 40÷50 fiber-glass dowels (length 20÷24 m) installed every 10÷12 m of face advance.

If geotechnical conditions are worse than predicted, additional reinforcement can be applied also beyond the excavation profile, by installing the outer dowels with a small outward inclination.

The higher rate of reinforcement was actually required only for the initial stretch of the tunnel that crosses at low depth the softened and remolded shale of the paleo-landslide area.

Also primary lining was adapted to the variable geotechnical conditions encountered along the tunnel route: after the high convergence experienced in the paleo-landslide area, heavier steel sets (HEA300/m instead of 2IPN220/m) embedded in shotcrete and closed by a steel strut at the invert were applied. A thin layer of shotcrete is applied also to the face when the excavation is stopped to allow the installation of fiber-glass dowels in safety conditions. The concrete invert is cast within a distance of one tunnel diameter from the face, generally the length of each cast segment is 11 m; finally the concrete lining is closed by the vault at about 30 m behind the face.

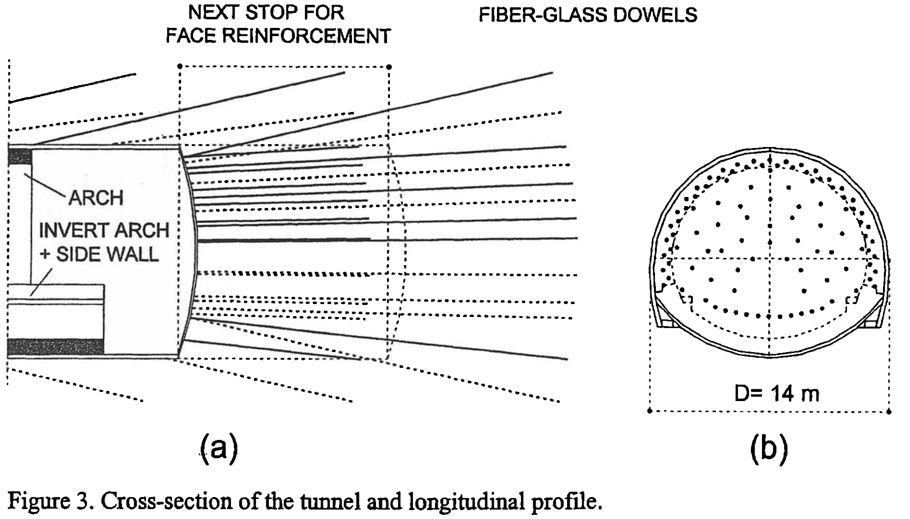

Figure 3. Cross-section of the tunnel and longitudinal profile.

3 ANALYSIS OF MONITORING DATA

The whole construction process has been performed under the constant control of a monitoring system, mainly based on the geodetic survey of tunnel wall displacements (a measuring section every 30 m with 5 optical targets installed close to the face) and on deformation measurements carried out by a sliding micrometer along pipes installed from the face in the direction of the tunnel axis.

3.1 Extrusion measurements

Figure 4 shows two typical extrusion measurements recorded at two different chainages in the Raticosa tunnel.

The first (Fig. 4a) comes from the paleo-landslide zone (overburden 30 m). The instrument length decreased from the initial 33 m (zero reading) to the final 20 m, corresponding to the last D and E readings, taken after 13 m of face advance. Every measurement exhibits an "extrusion" (i.e., axial displacement of the measuring pipe) which reaches a maximum at the tunnel face; extrusion values are progressively higher as the face advances, thus indicating that excavation-induced deformations affect the ground core for a distance significantly greater than 13 m.

A different trend is shown by the "extrusion" measured in the Chaotic Complex underneath a high overburden (Fig. 4b). The maximum extrusion is reached after a face advance of about 10 m (reading D) with no further increase in the next steps (reading E). It is therefore conceivable that inside the stiffer shale formation the influence zone of the face is limited to about 10 m.

From Figure 4, also the separate effect of time-dependent deformation can be appreciated: at chainage 30+171, no excavation occurred between readings D and E, meanwhile a face extrusion increase of about 21 mm/day was measured; a much smaller deformation of only 1.2 mm/day was measured at chainage 33+676, during a face stop between readings A and B.

Maximum extrusions δ_{max} recorded by each instruments installed along the tunnel route are represented in Figure 5a, as a function of tunnel chainage. At low tunnel depth, i.e. in the paleo-landslide area, maximum extrusion measurements, even larger than 200 mm, can be interpreted as the sign of actually unstable face conditions, as confirmed by the high deformation rate during face stops. As mentioned earlier, the face could have experienced an even larger extrusion than that recorded by the measurement system. On the contrary, inside the Chaotic Complex the maximum extrusion values are always of the order of 50 mm, almost independently of tunnel depth, thus denoting substantially stable face conditions.

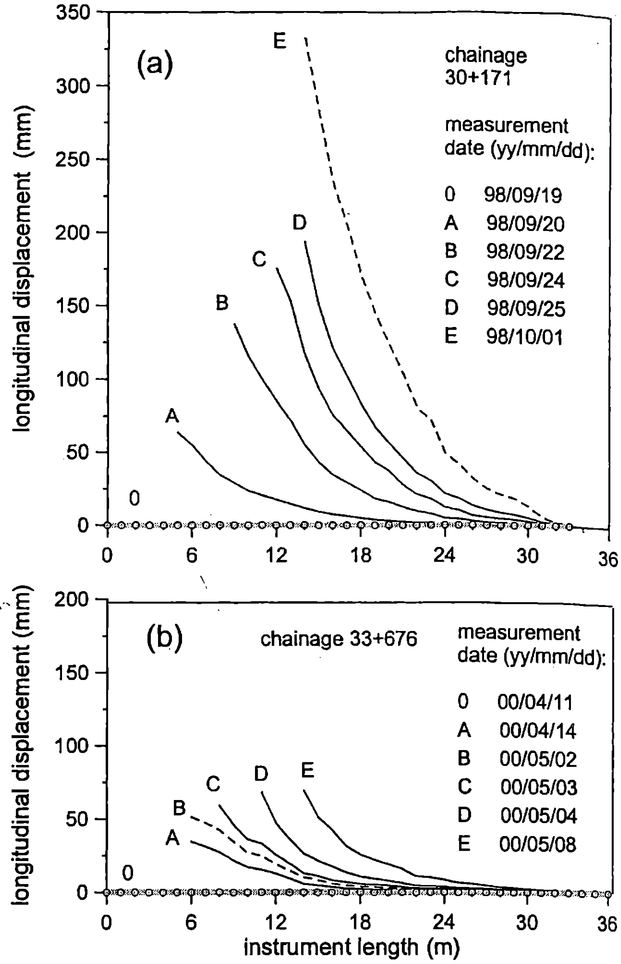

Figure 4. Longitudinal displacements ("extrusion") measured during face advance along a pipe installed at two different chainages.

3.2 Convergence measurements

Figure 5b summarizes the maximum convergence measured by each monitoring section along the tunnel route. Apart from maximum measured convergence values exceeding 120 mm in the paleo-landslide area, the average convergence is around 40 mm in the tectonized shale.

Data scatter is higher for convergence than for extrusion: this cannot be explained only by the different levels of accuracy of the geodetic survey. A major role must be attributed to interaction with the primary lining, which is quite sensitive to installation unevenness.

Despite the data scatter, the monitoring data (Fig. 5b) show a small but significant influence of the overburden on convergence (an increase of about 5 mm every 100 m of depth).

While a comprehensive analysis of all the monitoring data is still under way, a first interpretation of the convergence data has been based on the curve-fitting technique proposed by Sulem et al. (1987).

Tunnel closure is represented by an empirical law C(x, t), which takes into account the effect of both face distance x and the time-dependent behavior of the ground:

$$C(x,t)=C_{\infty,x}\left\{1-\left[1+\frac{x}{X}\right]^{-2}\right\}\cdot\left\{1+m\left[1-\left(1+\frac{t}{T}\right)^{-n}\right]\right\} \quad (1)$$

where t is the time elapsed since the face passed through the monitoring section.

Relationship (1) depends on five parameters: $C_{\infty,x}$ ("instantaneous closure") and X (a distance related to the tunnel radius R), which control the face effect; m, T and n for the time-dependent part.

The free parameters can be determined by a nonlinear regression of experimental data, for each monitoring section where a set of at least K=15÷20 readings were available, starting from a "zero" reading made close to the tunnel face (distance x_o) at time t_o. The set of k equations to be solved, in the sense of minimum squares, assume therefore the form:

$$\Delta C_{i,\,measured}=C(x_i,t_i)-C(x_o,t_o) \qquad i=1,k. \qquad (2)$$

To reduce problems of ill-conditioning of the system of equations (2), exponent n was given the value of 0.3, as suggested by Sulem et al. (1987). Moreover an iterative procedure was applied to refine the solution.

Figure 6 shows two examples of convergence curve fitting by means of Equation (1), respectively for a monitoring section located in the completely softened shale (chainage 30+343, overburden 50 m) and deep inside the Chaotic Complex (chainage 32+998, overburden 365 m). In the lower part of the figures the advance of the tunnel face as a function of time is also represented to make the interpretation of the observed behavior easier.

The estimated values of regression parameters, also reported in the graphs, indicate a remarkable amount of time-dependent convergence, whose asymptotic values, controlled by the parameter m, is equal to 1.8÷2.4 times the convergence due to the face advance.

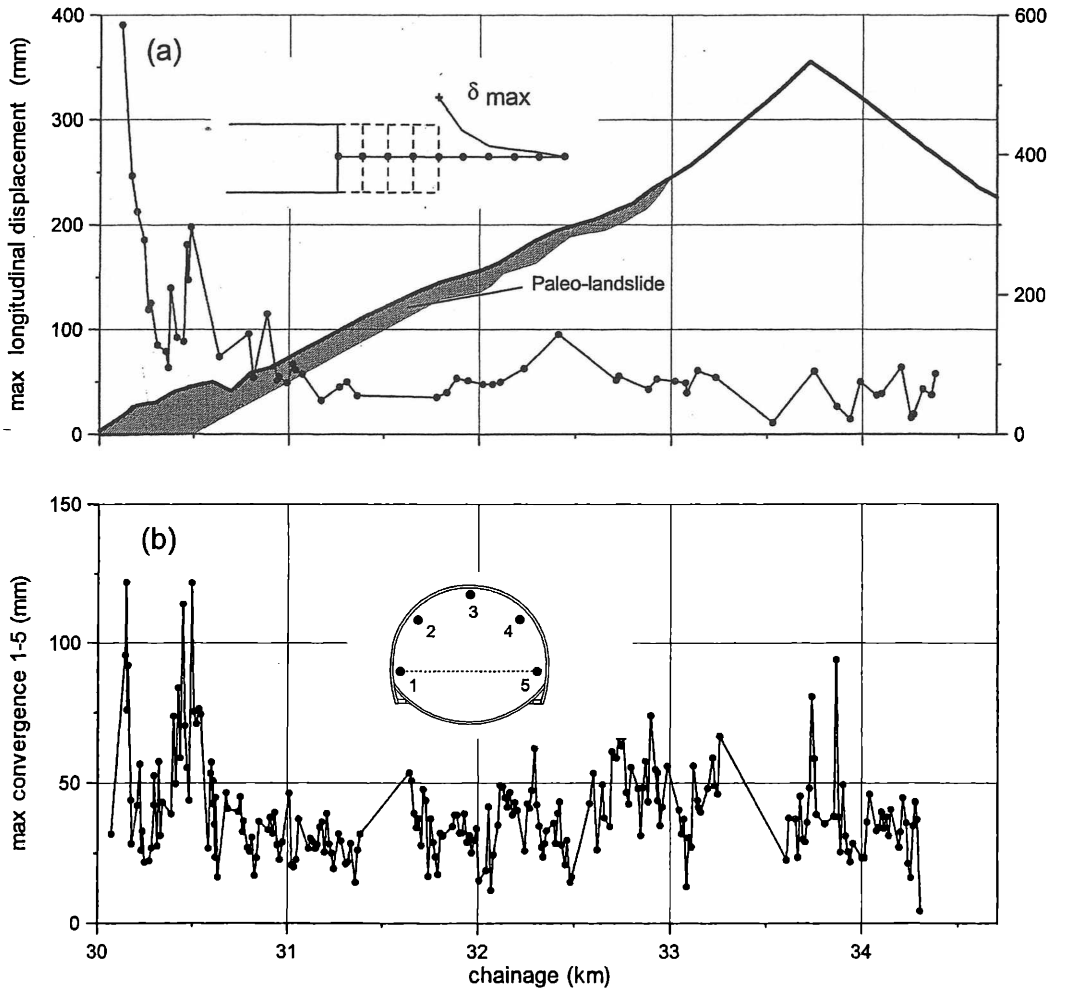

Figure 5. Profile of maximum values of convergence and extrusion measured along the tunnel route.

The separate effect of time-dependent deformations can be clearly evidenced during a period of excavation pause, as the 20 day period in Figure 6b. In this case, when the face stopped at a distance of about 26 m from the monitoring section, the corresponding increase of convergence was about 15 mm; this measured value is well reproduced by the fitting procedure. This kind of behavior, observed also in other monitoring sections, stresses how important it is to closely respect the construction time-schedule in order to prevent overloading phenomena of the primary lining before the final concrete lining becomes effective.

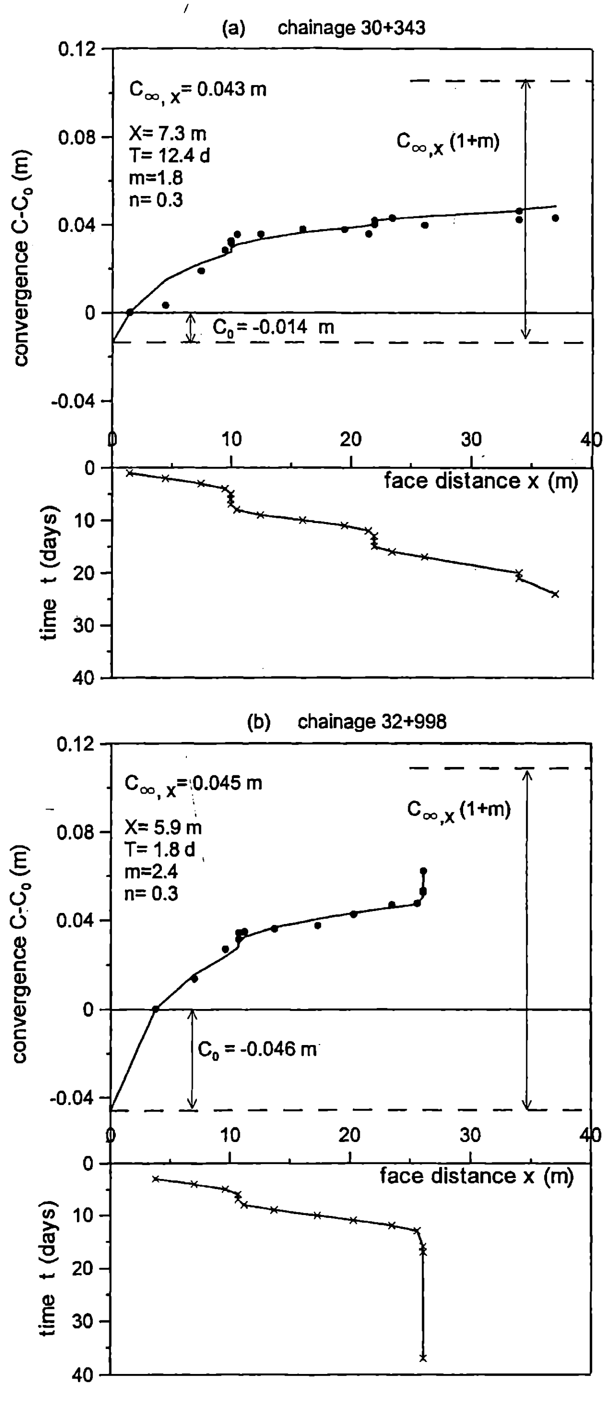

Figure 6. Curve fitting of convergence measurements at two monitoring sections.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Dott. Castellani, Ing. Meistro and Dott. Carrara of CAVET for having provided the monitoring data from the Raticosa tunnel and Ing. Gambelli and Ing. Cascino of ITALFERR for their cooperation during the visits at the tunnel site.

REFERENCES

Gambelli., C. 1999. Bologna-Florence high speed railway project – cooperation between client and contractor. *Felsbau* 17, 5: 367-373.

Lunardi, P. & A. Focaracci 1999. The Bologna to Florence high speed railway line: progress for underground. *In Challenges for the 21st Century, Alten et al. Eds*: 585-593.

Picarelli, L., L. Olivares, C. Di Maio & G. Urciuoli 2000. Properties and behaviour of tectonized clay shales in Italy. *In The Geotechnics of Hard Soil- Soft Rocks, Evangelista & Picarelli Eds*: 1211-1241.

Sulem, J., M. Panet & A. Guenot 1987. Closure analysis in deep tunnels. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* 24, 3: 145-154.