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Stress and strain state in the segmental linings during mechanized tunnelling

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Abstract. The application of the mechanized tunnelling has been extended in recent years. There are at present different approaches that are used in the design of segmental tunnel linings supported in mechanized tunnels. Even though segmental lining is utilized for mechanized tunnels, its behaviour is still quite unclear under in situ stress and there is a lack of data regarding the distribution of stresses inside segmental linings. So far no single effective calculation method exists for segmental lining design. The lack of clear solutions makes the use of segmental lining to be more expensive due to the adoption of greater safety factors. Therefore, a particular attention must be given in order to obtain data from monitored tunnels which permits to validate design methods. In this study, strain measurements, which were conducted during the construction of twin tunnels in the Bologna-Florence railway line, have been presented. The behaviour of segmental lining during the excavation and the influence of a new tunnel excavation on an existing tunnel have been shown through the measured data. The data are then compared with the results obtained with Einstein and Schwartz's method and Duddeck and Erdmann's method, which permits to highlight the fact that the two analytical methods underestimate structural forces induced in the segmental lining and then must be used with caution.

Keywords: tunnel; tunnelling; monitoring; segmental lining; structural forces; strain gauges

1. Introduction

Segmental lining is a three-dimensional (3D) structure with characteristics that are very different from those of a monolithic cast-in-place concrete lining. As far as segmental concrete linings are concerned, its behaviour depends on many factors such as concrete segments, segmental joints in the lining and the ground-lining interaction (Takano 2000, Naggar and Hinchberger 2008, Teachavorasinskun and Chub-Uppakarn 2009, 2010, Thienert and Pulsfort 2011, Do *et al*. 2013 a, b, 2014a, b). Furthermore, according to a report performed by a Technical Committee of Japan Society of Civil Engineers (Sugimoto 2006), construction load is one of the

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main causes which leads to the damage of segmental lining installed in tunnels. It is then necessary to develop a segmental lining design method that allows one to take the construction loads during the tunnel excavation into consideration. As a result, the most important problem of segmental lining design is whether the design model can reflect the actual stress of segments.

At present, there are some methods applied to segmental lining design which have been developed and may be classified into three main groups: empirical methods, analytical methods and numerical methods (BTS 2004). Among them, numerical methods, 3D analyses in particular, are the only manner that allows the problem to be taken into consideration in a rigorous way (Dias and Kastner 2013, Dias and Oreste 2013, Dias *et al*. 2000, Do *et al*. 2013b, Hyang *et al*. 2006, Oreste 2013, Oreste and Dias 2012, Mollon *et al*. 2013). The application of 3D numerical models is still limited in special underground works due to the complexity and the time consumption. Analytical methods are actually the most commonly chosen solution for preliminary designs. Einstein and Schwartz (1979) and Duddeck and Erdmann (1985) have proposed interesting analytical solutions for the design of tunnel linings. One of the benefits to the designer is that these solutions are simple and quick to use. Using these methods, it is possible to obtain structural forces and deformations induced in the tunnel lining. In order to simplify the calculation process, both models assume a circular tunnel with strain plane conditions, elastic lining and to be embedded in an isotropic and homogenous elastic medium. Owing to these assumptions, structural forces predicted by these methods should be validated by comparing with field monitoring data acquired during the excavation of tunnels (Dias and Kastner 2013, Bilotta and Russo 2013).

In this paper, comparisons between the analytical results using the two above methods and the field measurements, which were conducted during the construction of twin tunnels in the Bologna-Florence railway line project, have been presented in order to estimate the behaviour of segmental linings during the excavation and also to evaluate the applicability of analytical methods proposed by Einstein and Schwartz (1979) and Duddeck and Erdmann (1985).

2. Bologna-Florence railway project

The Bologna-Florence high-speed railway line project was performed to modernize the Italian rail links and enhance the passenger capacity. In Bologna, the working part of the project involves the excavation of two tunnels with distance of 15 m from centre to centre. Each tunnel has an extension of about 6.2 km, an external excavation diameter of 9.4 m, and an internal diameter of 8.3 m for a useful section of 46 m^2 (Fig. 1).

The two tunnels were excavated at a depth of between 15 and 25 m below the ground surface. Totally, 8160 rings of precast concrete were used. The two tunnels were excavated through two formations: the alluvial deposits of the late Pleistocene-Pliocene era, which are mostly alluvial deposits of the river Savena, with clay, and sandy soil deposits (clayey sands and Pliocene clays) (Croce 2011).

The tunnels were excavated using two earth pressure balance shields (EPB). The second tunnel drive was realized after the first tunnel in a period of 6 months. The first EPB have encountered major problems during the excavation that lead to a long inactivity period of the tunnelling machine. The second EPB, exploiting the information obtained during the excavation of the first tunnel, has operated without any major problem.

The linings of the two tunnels are composed of precast segments made of reinforced concrete. Each circular ring of 1.5 m length consists of 6 regular shaped conical blocks and a smaller key

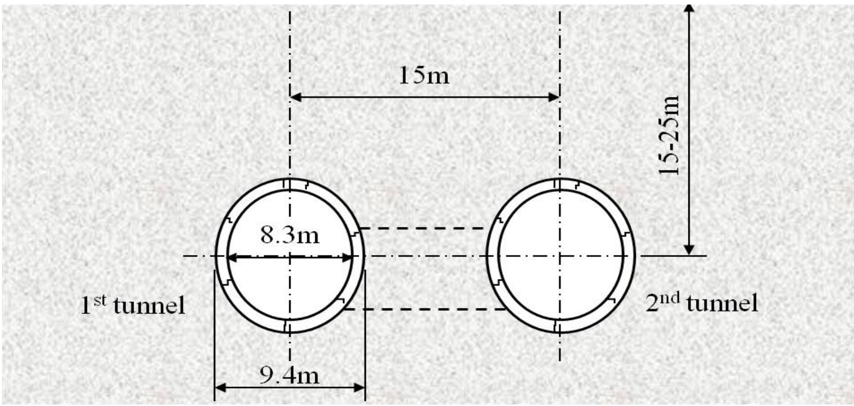

Fig. 1 Typical cross-section of the two tunnels

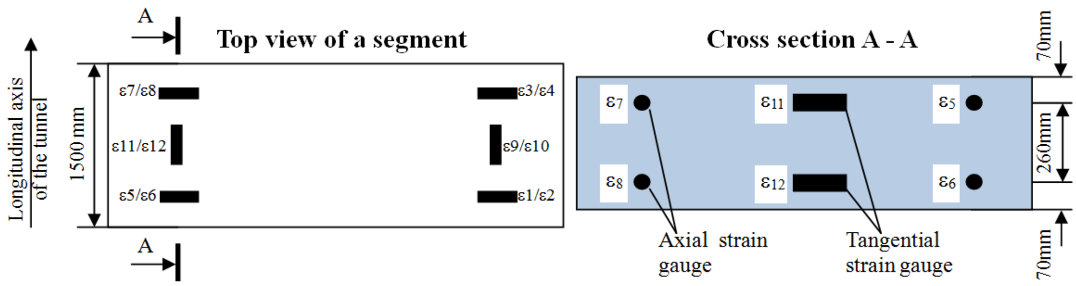

Fig. 2 Layout of the strain gauges (not scaled)

block. Each precast concrete ring has an extrados diameter of 9.4 m and a thickness of 0.4 m. The excavation speed of the machine has reached considerable peaks of 15-20 m/day.

3. Field monitoring

Field monitoring is a very important tool in tunnelling where the excavated medium are usually heterogeneous and uncertain. The main types of field measurements during tunnelling are displacements, loads, stresses, hydraulic pressures, temperature and vibrations.

In order to monitor the tunnel lining deformations, strain gauges were installed in the segments when they were made. The gauges used in this project are a vibrating wire type (3.000 με measuring range, sensitivity 1 $\mu\varepsilon$). A zero value was recorded immediately after the installation of the segments in the tunnel (Borgonovo *et al*. 2007).

Each segmental lining ring consists of six concrete blocks numbered *A*1, *A*2, *A*3, *A*4 and two blocks *B* and *C*, and the key block (named *K*). Two pairs of strain gauges were installed at every two blocks, which were oriented in the circumferential direction and were placed in the centre of segment with distance of about 25 cm from lateral edges of segments. There are 6 pairs of gauges installed in each every two blocks (Fig. 2). In this study, of the seven rings monitored, only the monitored data of ring 582 is considered.

4. Results and analysis of the field monitoring

In order to better understand the behaviour of tunnel linings and the effect of the second tunnel excavation process on the existing tunnel, a monitoring was carried out continuously and automatically. The measured strains of three gauges in block *A*1 assembled on the right side of the ring 582 are presented in Fig. 3. Strain gauges were monitored right after the assembly of concrete blocks. The deformation progress of segmental lining, which is shown in Fig. 3, can be divided into four principal phases. At the beginning of the monitoring period, considerable deformations due to the action of thrust forces originated from hydraulic jacks and the effect of injected grouting process were recorded (phase 1). When the advancing tunnel face was far from the monitored lining ring, deformations induced in the blocks became relatively constant (phase 2) till the passage of the 2nd EPB (phase 3) in the second tunnel. From Fig. 3, considerable changes in the deformation of the lining during initial hours after being assembled in the tunnel can be seen, especially at date (5/6/2004) when the 2nd EPB passes over the monitored ring 582, which was previously installed in the first tunnel. It is reasonable to conclude that the excavation process of the second tunnel caused a great impact on the first tunnel lining. This conclusion is highlighted by the results of the structural forces in the lining, as presented later on. The deformation velocity induced in the lining increases significantly during the passage of the 2nd EPB and still continues for 10 to 15 days after that (see phase 3 in Fig. 3). The deformation velocity then reduces gradually and the excavation process of the second tunnel has no more effect on the first tunnel (phase 4).

For each segment, the stresses in terms of bending moment (*M*) and normal force (*N*) are evaluated on the basis of measured strains. The maximum strain values obtained before and after the passage of the 2nd EPB are considered. The evolutions of stresses in these two cases are presented. In this study, only the measurements made from strain gauges placed in the circumferential direction (ε_1-ε_2, ε_3-ε_4, ε_5-ε_6, ε_7-ε_8 of odd blocks and ε_1-ε_2, ε_3-ε_4 of even blocks) have been taken into account. From each couple of strain values, for example ε_1 and ε_2, their average value allows one to evaluate the normal strain of segmental lining section. The difference of the two values has also been calculated in order to evaluate the bending strain of lining sections. In order to calculate the strain values at the two concrete surfaces (i.e., intrados and extrados sides),

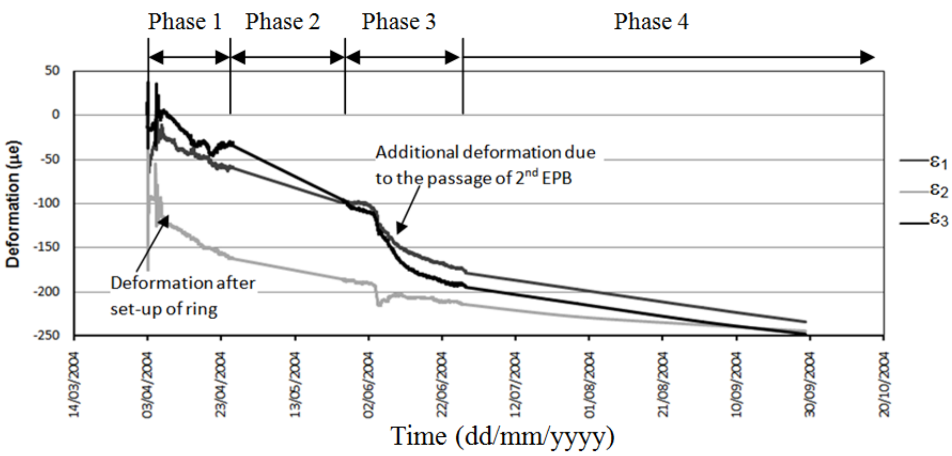

Fig. 3 Deformation in time of the ring 582 - block A1

Table 1 Structural forces in lining ring 582 (calculated based on field measurement)

Block	(deg)	At the tunnel face					Behind the tunnel face				
		Strain gauge	M (MNm/m)		N (MN/m)		Strain gauge	M (MNm/m)		N (MN/m)	
			26/5	18/6	26/5	18/6		26/5	18/6	26/5	18/6
A1	7	5-6	0.112	0.171	1.964	2.732	7-8	0.167	0.212	2.248	2.806
B	88	1-2	0.071	0.070	0.734	0.993	3-4	0.041	0.047	2.404	2.788
B	127	5-6	0.090	0.097	2.012	2.288	7-8	0.156	0.163	1.073	1.227
A4	165	1-2	-0.098	-0.054	2.055	2.815	3-4	-0.027	0.002	2.028	2.504
A3	247	5-6	0.060	0.032	2.529	2.772	7-8	0.038	0.013	0.920	1.096
A1	323	1-2	-0.063	-0.031	1.989	2.596	3-4	-0.063	-0.036	1.956	2.918

(*θ - angle in degrees measured counter clockwise from spring line on the right*)

the measured strains have to be converted as being determined at the concrete lining surface (the strain gauge devices were covered with 70 mm of concrete). On the basis of the normal strain and bending strain, the normal force (*N*) and bending moment (*M*) that induced in the lining at the two sides of the lining ring 582 in the longitudinal direction, which are hereafter called "At the tunnel face" and "Behind the tunnel face", respectively, have been evaluated (see Table 1 and Figs. 4 and 5).

Owing to the passage of the 2nd EPB, the normal forces and bending moments induced in the segmental concrete lining of the first tunnel are generally increased compared with those determined before the interaction (see Tables 1 and 3). The increases in maximum normal forces and bending moments are equal to 15.4% and 26.3%, respectively. As far as the bending moment is concerned in detail, the results in Table 1 show a decrease trend occurred at the tunnel bottom, and an increase occurred at the right side of the tunnel lining, which is close to the new tunnel, due to the passage of the new tunnel. The maximum decrease of the bending moment determined at the tunnel bottom is equal to 92%, and the maximum increase induced at the tunnel right side is equal to 52% (Table 1). The bending moments determined at the tunnel crown show a slight increase. These phenomena can be explained by the movement of the ground between the two tunnels towards the new tunnel and then followed by an additional downward displacement of the ground above the existing tunnel, which causes an increase in vertical external loads acting on the lining rings installed in the first tunnel.

5. Analytical methods of Einstein & Schwartz and Duddeck & Erdmann

The two analytical models assume a circular tunnel embedded in a uniformly stressed continuum. These models also assume that the ground is a semi-infinite medium and they should therefore only be used for tunnels which are two tunnel diameters below the ground surface (BTS 2004). In the case of the Bologna project, two tunnels were excavated at a depth of 15 ÷ 25 m below the ground surface. The two above methods can therefore be utilized.

Einstein and Schwartz (1979) use two ratios, that is, compressibility ratio C^* and flexibility ratio F^*, to take into account the interaction between the tunnel lining and the surrounding ground using symmetric loading and asymmetric loading conditions, respectively. This method allows both cases, with and without the bonding forces between the tunnel lining and the ground, to be taken into account. These two cases correspond to the no-slip case and the full-slip case as mentioned below (Einstein and Schwartz 1979).

The results of bending moment (M) and normal force (N) for the no-slip case can be calculated using the following formulas

$$\frac{N}{\sigma_v R} = \frac{1}{2}(1+K)\left(1-a_0^*\right) + \frac{1}{2}(1-K)\left(1+2a_2^*\right)\cos 2\theta \tag{1}$$

$$\frac{M}{\sigma_v R^2} = \frac{1}{4}(1-K)\left(1-2a_2^*+2b_2^*\right)\cos 2\theta \tag{2}$$

The results of bending moment (*M*) and normal force (*N*) for the full-slip case can be calculated using the following formulas

$$\frac{N}{\sigma_v R} = \frac{1}{2}(1+K)\left(1-a_0^*\right) + \frac{1}{2}(1+K)\left(1-2a_2^*\right)\cos 2\theta \tag{3}$$

$$\frac{M}{\sigma_v R^2} = \frac{1}{2}(1-K)\left(1-2a_2^*\right)\cos 2\theta \tag{4}$$

where θ = angle measured counter clockwise from spring line on the right, radial; R = tunnel radius, m; σ_v = vertical stress, MN/m^2; K = lateral earth pressure coefficient; E = Young's modulus of the ground, MN/m^2; and $a^*{}_0$, $a^*{}_2$, $b^*{}_2$ = dimensionless coefficients; M = bending moment (MNm/m); N = normal forces (MN/m).

The bedding beam model proposed by Duddeck and Erdmann (1985) is generally adopted in many countries (Roland 1999, Takano 2000, BTS 2004). The structural forces in the lining are dependent on the stiffness of the lining and that of the surrounding ground. Like Einstein & Schwartz's method, Duddeck and Erdmann's method allows both cases, with and without the bonding force between the tunnel lining and the ground, are taken into account.

The results of bending moment (*M*) and normal force (*N*) for the no-slip case can be calculated using the following formulas

$$M = \sigma_v (1-K) R^2 \frac{1}{4 + \dfrac{3-2\nu}{3(1+\nu)(3-4\nu)} \dfrac{ER^3}{E_{\sup} J}} \cos 2\theta \tag{5}$$

$$N = \frac{\sigma_v (1+K) R}{2 + (1-K)\dfrac{2(1-\nu)}{(1-2\nu)(1+\nu)} \dfrac{ER}{E_{\sup} A}} + \frac{\sigma_v (1-K) R}{2 + \dfrac{4\nu\, ER^3 / E_{\sup} J}{(3-4\nu)\left(12(1+\nu) + ER^3 / E_{\sup} J\right)}} \cos 2\theta \tag{6}$$

The results of bending moment (*M*) and normal force (*N*) for the full-slip case can be calculated using the following formulas

$$M = \sigma_v (1-K) R^2 \frac{1}{\dfrac{10-12\nu}{3-4\nu} + \dfrac{2}{3(1+\nu)(3-4\nu)} \dfrac{ER^3}{E_{\sup} J}} \cos 2\theta \tag{7}$$

$$N = \frac{\sigma_v (1+K) R}{2 + (1-K) \dfrac{2(1-\nu)}{(1-2\nu)(1+\nu)} \dfrac{ER}{E_{\sup} A}} + \frac{\sigma_v (1-K) R}{\dfrac{10-12\nu}{3-4\nu} + \dfrac{2}{3(1+\nu)(3-4\nu)} \dfrac{ER^3}{E_{\sup} J}} \cos 2\theta \tag{8}$$

where θ = angle measured counter clockwise from spring line on the right, radial; R = tunnel radius, m; σ_v = vertical stress, MN/m^2; K = lateral earth pressure coefficient; E, $E_{\sup}$ = Young's modulus of the ground and tunnel lining, respectively, MN/m^2; ν = Poisson's ratio of the ground; J = the inertia moment of the tunnel lining cross-section, m^4; and A = cross-section area of the tunnel lining per unit length along the tunnel axis, m^2; M = bending moment (MNm/m); N = normal forces (MN/m).

Since the lining is made of segments, a reduction in flexural stiffness of the tunnel lining is expected (Muir Wood 1975, Liu and Hou 1991). However, the reduction of the lining stiffness caused by the presence of segmental joints has not been considered in this study. This assumption has been adopted in order to consider the worst case for the calculated lining stress state. Geomechanical parameters determined at location of segment lining ring 582, which have been used as input data in Einstein and Schwartz's method and Duddeck and Erdmann's method, are presented in Table 2.

Figs. 4 and 5 show the results of the bending moment and normal force obtained using the two above analytical methods presented in the same diagram with the measured data. The maximum values of structural forces that induced in the segmental concrete lining evaluated by all methods are presented in Table 3.

Table 2 Geomechanical parameters at location of lining ring 582

Description	Value	Description	Value
Tunnel depth H (m)	20	Young's modulus of ground E (MPa)	150
Internal friction angle φ (degree)	37	External tunnel radius R (m)	4.7
Unit weight γ (MN/m^3)	0.017	Poisson's ratio of ground ν	0.3
Young's modulus of concrete $E_{\sup}$ (MPa)	35000	Poisson's ratio of concrete $\nu_{\sup}$	0.15
Thickness of segmental lining h_c (m)	0.4	Lateral earth pressure coefficient K	0. 5
Width of segmental lining b (m)	1.5		

Table 3 Maximum structural forces in the segmental lining ring 582

	Field measurement		Einstein and Schwartz's method		Duddeck and Erdmann's method	
	Before the passage of the 2nd EPB	After the passage of the 2nd EPB	Full-slip	No-slip	Full-slip	No-slip
M (MNm/m)	0.167	0.212	0.128	0.116	0.126	0.117
(%)	100	126.3	76.7	69.5	75.2	69.8
N (MN/m)	2.529	2.918	1.392	1.656	1.340	1.663
(%)	100	115.4	55.0	65.5	53.0	65.8

6. Results and discussions

It can be seen from the results presented in Figs. 4 and 5 and Table 3 that the structural forces induced in the lining determined using Duddeck and Erdmann's method are in good agreement with the results obtained with Einstein and Schwartz's method. However, both methods give structural force results which are low compared with the measured data.

These differences could be attributed to the fact that the two above analytical methods are structural design models which are subjected only to the external loads determined in normal conditions (primary loads), and do not take into account the disturbances that occur in the surrounding ground induced by the construction process, especially after the installation of segments on the tunnel periphery (i.e., grouting pressure, jacking forces, etc.), the heterogeneity of the ground or the presence of the joints. In general, analytical methods represent simplified ones due to their initial assumptions. In addition, in the case in which two tunnels are excavated in parallel, a rule of thumb is that the distance between the two tunnels should be higher than one tunnel diameter (Do *et al*. 2014a). In this case, the distance is less than one tunnel diameter; the design then should take into account the effect of the second tunnel construction process on the existing tunnel.

The same differences are also found in results presented by Bakker (2003), Bilotta and Russo (2012). Bakker (2003) showed a comparison between experimental data obtained during the construction of the Second Heinenoord bored tunnel in Netherlands and analytical results computed using Duddeck and Erdmann's method and that of a two-dimensional FEM analysis, in which the effect of volume loss and a reduced bending stiffness caused by the existence of the joints are taken into consideration. Although along the major part of tunnel periphery there is a relatively good agreement in shape of structural force lines, maximum differences of the bending moment and normal force obtained using analytical and two-dimensional numerical analyses are

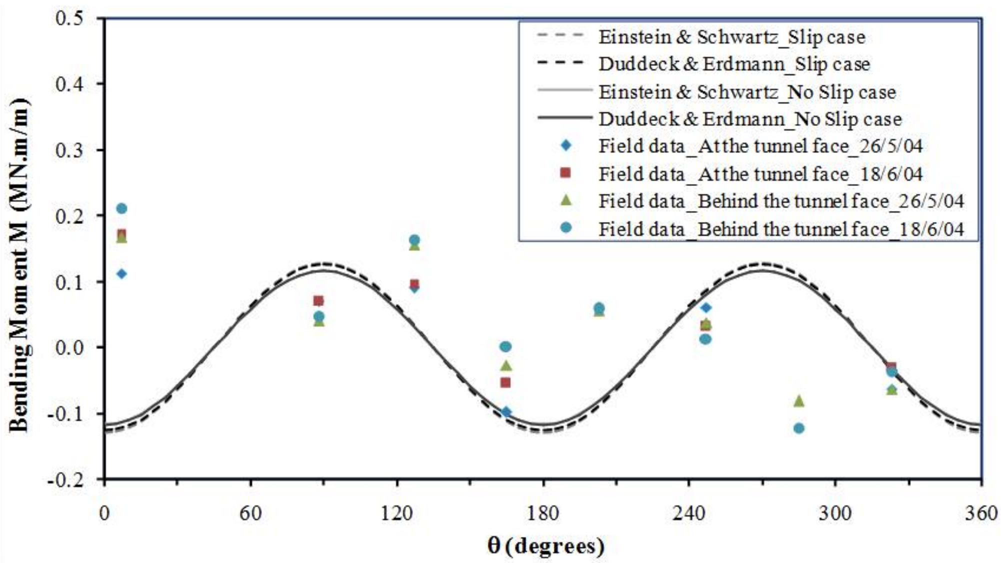

Fig. 4 Bending moment diagram in lining ring 582

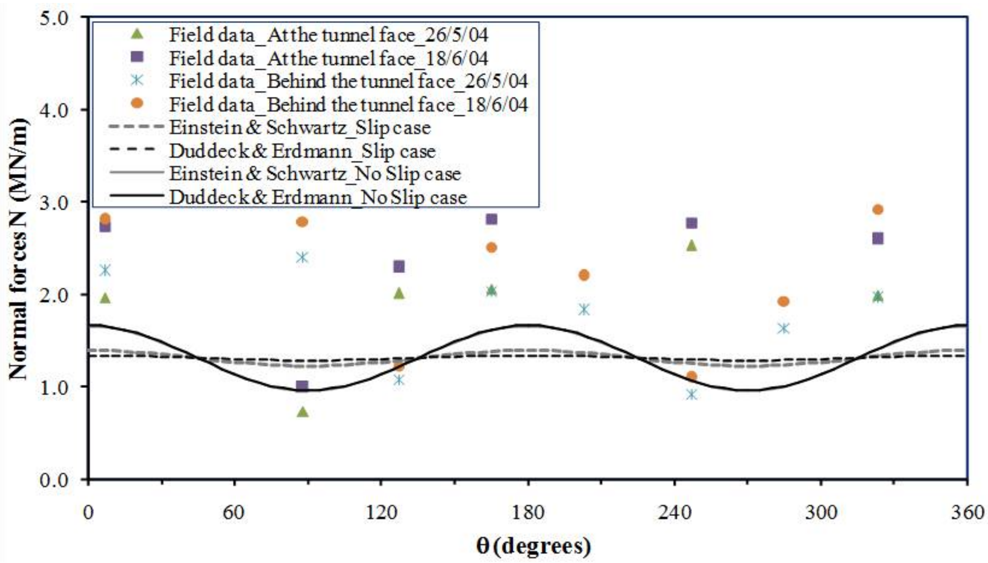

Fig. 5 Normal force diagram in lining ring 582

about 250% and 200%, respectively, lower than measured data. Similarly, Bilotta and Russo (2012) also presented a large difference between the measured data and the analytical results using a bedded ring model for shallow tunnels.

On the basis of above discussions, it is reasonable to conclude that Duddeck and Erdmann's method and Einstein and Schwartz's method underestimate the actual structural forces during the shield tunnel construction in this case studied. This means that these methods are not safe for the design of segmental lining and need to be improved. In addition, interaction between tunnels cannot be considered by these analytical methods. Numerical analysis should be used for this purpose (e.g., Hefny *et al*. 2004, Do *et al*. 2014a, b).

7. Conclusions

The change tendency in the structural forces induced in the segmental concrete lining of an existing tunnel due to the passage of a new tunnel has been shown through the measured data obtained at the Bologna-Florence high-speed railway line project. The results highlighted that, due to the passage of the second EPB tunnel, the maximum normal force and bending moment increases in the existing tunnel lining of respectively 15.4% and 26.3%. As far as the bending moment is concerned, the results indicated a general trend of decrease for the bending moment located at the tunnel bottom, and an increase occurred at the tunnel right side lining which is close to the new tunnel.

On the basis of the comparison between the structural forces evaluated from the field measurement and those obtained using the two analytical methods, it can be concluded that, for the studied case, the two above analytical methods underestimate the actual structural forces induced in the tunnel lining. Therefore, they are not safe for segmental lining used in mechanized tunnels and need to be improved.

References

Bakker, K.J. (2003), “Structural design of linings for bored tunnel in soft ground”, *Heron*, **48**(1), 33-63.

Bilotta, E. and Russo, G. (2012), “Back calculation of internal forces in the segmental lining of a tunnel: The experience of Line 1 in Naples”, *Geotechnical Aspects of Underground Construction in Soft Ground – Viggiani* (*ed*) *© 2012* Taylor & Francis Group, London, UK. ISBN: 978-0-415-68367-8, 213-221

Bilotta, E. and Russo, G. (2013), “Internal forces arising in the segmental lining of an EPB bored tunnel”, *J. Geotech. Geoenviron. Eng.*, **139**(10), 1765-1780.
DOI: 10.1061/(ASCE)GT.1943-5606.0000906

Borgonovo, G., Contini, A., Locatelli, L., Marco, P. and Ramelli, E. (2007), “Monitoring used as an alarm system in tunnelling”, *Rapid Excavation & Tunnelling Conference and Exhibit*, Toronto, Canada, June, pp. 381-395.

BTS (The British Tunnelling Society and The Institution of Civil Engineers) (2004), *Tunnel Lining Design Guide*, Thomas Telford Publishing, London, UK. ISBN: 0 7277 2986 1

Croce, A. (2011), “Analisi dati di monitoraggio del rivestimento della galleria del passante ferroviario di Bologna”, Degree Dissertation, Polytechnics of Turin, Italy. [In Italian]

Dias, D. and Kastner, R. (2013), “Movements caused by the excavation of tunnels using face pressurized shields - Analysis of monitoring and numerical modelling results”, *Eng. Geol.*, **152**(1), 17-25.
DOI: 10.1016/j.enggeo.2012.12.002

Dias, D. and Oreste, P.P. (2013), “Key factors in the face stability analysis of shallow tunnels”, *Am. J. Appl. Sci.*, **10**(9), 1025-1038.

Dias, D., Kastner, R. and Maghazi, M. (2000), “Three-dimensional simulation of slurry shield tunnelling”, *Geotechnical Aspects of Underground Construction in Soft Ground*, *Kusakabe*, *Fujita and Miyazaki* (*eds.*), Balkema, Rotterdam, pp. 351-356.

Do, N.A., Dias, D., Oreste, P.P. and Djeran-Maigre, I. (2013a), “2D numerical investigation of segmental tunnel lining behavior”, *Tunnel. Undergr. Space Tech.*, **37**, 115-127.

Do, N.A., Dias, D., Oreste, P.P. and Djeran-Maigre, I. (2013b), “Three-dimensional numerical simulation for mechanized tunnelling in soft ground – The influence of the joints”, *Acta Geotechnica*.
DOI: 10.1007/s11440-013-0279-7

Do, N.A., Dias, D., Oreste, P.P. and Djeran-Maigre, I. (2014a), “2D numerical investigations of twin tunnel interaction”, *Geomech. Eng., Int. J.*, **6**(3), 263-275.

Do, N.A., Dias, D., Oreste, P.P. and Djeran-Maigre, I. (2014b), “Three-dimensional numerical simulation of a mechanized twin tunnels in soft ground”, *Tunnel. Undergr. Space Tech.*, **42**, 40-51.
DOI: 10.1016/j.tust.2014.02.001

Duddeck, H. and Erdmann, J. (1985), “Structural design models for tunnels”, *Undergr. Space*, **9**(5-6), 246-253.

Einstein, H.H. and Schwartz, C.W. (1979), “Simplified analysis for tunnel supports”, *J. Geotech. Eng. Div.*, **105**(4), 499-517.

Hefny, A.M., Chua, H.C., Jhao, J. (2004), “Parametric studies on the interaction between Existing and new bored tunnels”, *Tunnel. Undergr. Space Tech.*, **19**(4-5), 471.

Huang, Z.R., Zhu, W., Liang, J.H., Lin, J. and Jia, R. (2006), “Three dimensional numerical modelling of shield tunnel lining”, *Tunnelling and Underground Space Technology*, **21**(3-4), 434-434.

Liu, J.H. and Hou, X.Y. (1991), *Shield-Driven Tunnels*, China Railway Press, Beijing, China.

Mollon, G., Dias, D. and Soubra, A. (2013), “Probabilistic analyses of tunneling-induced ground movements”, *Acta Geotechnica*, **8**(2), 181-199.
DOI: 10.1007/s11440- 012-0182-7

Muir Wood, A. (1975), “The circular tunnel in elastic ground”, *Géotechnique*, **25**(1), 115-127.

Naggar, H.E. and Hinchberger, S.D. (2008), “An analytical solution for jointed tunnel linings in elastic soil or rock”, *Can. Geotech. J.*, **45**(11), 1572-1593.

Oreste, P.P. (2013), “Face stabilization of deep tunnels using longitudinal fibreglass dowels”, *Int. J. Rock Mechan. Min. Sci.*, **58**, 127-140.

DOI: 10.1016/j.ijrmms.2012.07.011

Oreste, P.P. and Dias, D. (2012), "Stabilisation of the excavation face in shallow tunnels using fibreglass dowels", *Rock Mech. Rock Eng.*, **45**(4), 499-517.
DOI: 10.1007/s00603-012-0234-1

Roland, D.W. (1999), "Steel fibre reinforced tunnel segment for the application in the shield driven tunnel", Ph.D. Thesis, DUP, Delft, Netherlands. ISBN: 90-407-1965-9

Sugimoto, M. (2006), "Causes of shield segment damages during construction", *International Symposium on Underground Excavation and Tunnelling*, Bangkok, Thailand, February, pp. 67-74.

Takano, Y. (2000), "Guidelines for the design of shield tunnels", *Tunnel. Undergr. Space Tech.*, **15**(3), 303-331.

Teachavorasinskun, S. and Chub-Uppakarn, T. (2009), "Experimental verification of joint effects on segmental tunnel lining", *Electr. J. Geotech. Eng.*, **14**.

Teachavorasinskun, S. and Chub-Uppakarn, T. (2010), "Influence of segmental joints on tunnel lining", *Tunnel. Undergr. Space Tech.*, **25**(4), 490-494.

Thienert, C. and Pulsfort, M. (2011), "Segment design under consideration of the material used to fill the annular gap", *Geomech. Tunnel.*, **4**(6), 665-680.

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