Stress and Strain State in the Segmental Linings during the Excavation by Tunnel Boring Machines

Do Ngoc Anh [1], Oreste Pierpaolo [2], Croce Antonello [2], Dias Daniel [3], Djeran-Maigre Irini [1], Locatelli Livio [4]

[1] Laboratory L.G.C.I.E., INSA de Lyon, Villeurbanne, France
[2] Department of Land, Environmental, Geotechnological Eng, Polytechnics of Turin, Italy
[3] Laboratory LTHE, Polytech'Grenoble, Joseph-Fourier University, Grenoble, France
[4] Golder Associates S.r.l., Milano, Italy

1. INTRODUCTION

Segmental lining is a three-dimensional structure with characteristics that are very different to that of a monolithic cast-in-place concrete lining. For segmental concrete lining, its behavior depends on many factors such as characteristics of the concrete segments, effects of joints in the lining and interaction with the soil mass [5,8,9]. Also, according to the report published by a technical committee of Japan Society of Civil Engineers (2005) [6], construction load is one of the main causes which leads to the damage of segmental lining installed in tunnels. It is then necessary to develop a segmental lining design method taking into account of the construction loads during construction. 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, many design methods of segmental lining have been developed and may be classified into three main groups including empirical methods, analytical methods and numerical methods [10]. Among them, numerical methods, especially three-dimensional numerical methods, are the only manner to take into consideration in a rigorous way the problem [1,4,10]. But due to their complexity, they seem to be only used in special underground works. In other words, analytical methods are in most of the cases used for preliminary designs. Einstein and Scwartz (1979) and Duddek and Erdmann (1982) have proposed very interesting analytical models to design segmental linings. One of the benefits to the designer is that the methods are simple and quick to use. Information is provided on the normal forces, bending moments and deformations. For simplification of the calculation process, both of the two models assume plane stress, isotropic and homogenous elastic medium and elastic lining for a circular tunnel. Due to these assumptions, member forces predicted by these methods should be validated by comparison with field monitoring data acquired during the excavation of actual tunnelling projects. In this paper, field measurements of a shield tunnel construction site, belonging to the Bologna - Florence Italian railway high speed line project, constructed in 2004, were carried out to estimate the behaviour of the segmental lining during the excavation and evaluate the accuracy of analytical models proposed by Einstein and Schwartz (1979) and Duddek and Erdmann (1982).

This project includes two shield tunnels excavated in parallel at a depth of 15 m below a densely populated urban area. The influence of the second tunnel on the first one will be also shown by the measurements.

2. BOLOGNA-FLORENCE RAILWAY LINE PROJECT

The Bologna-Florence railway high speed line project is one part of the Italian high speed railway network. It has been constructed to modernize the Italian rail links and enhance the passenger capacity. The total length of the Bologna-Florence railway high speed line is about 78.5 km in which over 93% of tunnels. This includes nine tunnels with a total length of 73 km. In Bologna, the working part of the project includes two tunnels with a distance spacing of 15 m between the two tunnel centers. Each has an extension of about 6.2 km, an external excavation diameter of 9.4m and an internal diameter of 8.3 m for a useful section of 46 m^2 (figure 1).

To ensure safety during the construction of tunnels, communication tunnels between the two tunnels are built every 500 meters tunnel length. Above 70% of the tunnel is excavated below the old railway line Milan-Naples located on the surface. The tunnels have been excavated at a depth comprise between 15 and 25 m below the ground surface. In total, 8160 rings of precast concrete have been used. The two tunnels have been excavated through mainly two formations: the alluvial deposits of late Pleistocene - Pliocene, mostly alluvial deposits of the river Savena with deposits of clay, and sandy soils (clayey sands and Pliocene clays).

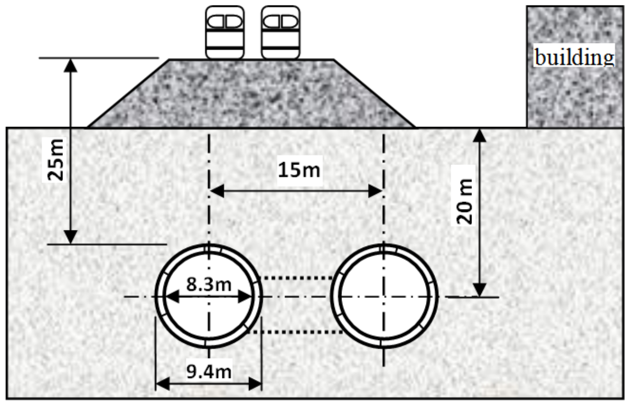


Figure 1 Typical cross-section of the two tunnels excavated below the railway

3. EXCAVATION METHOD

To meet requirements of design and construction during the works, the excavation has been done with mechanized machines. For the construction of the two parallel tunnels in Bologna, two Earth Pressure Balance Shields (EPBs) have been used (figure 2). 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 phase leading to a long inactivity period for the machine. The second EPB, exploiting the information obtained during the excavation of the first tunnel, has done its job without any major problems.

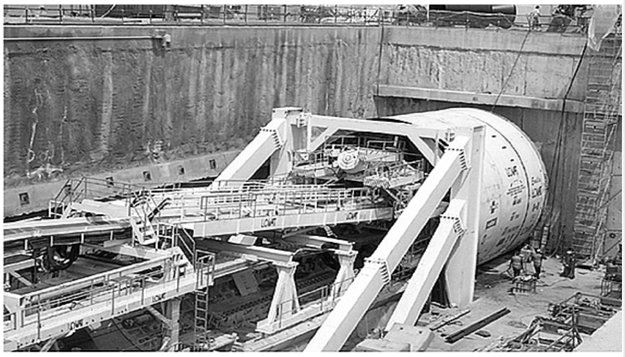


Figure 2 EPBs used at Bologna project

The lining of the two tunnels is composed of precast segments made of reinforced concrete. Each circular ring of 1.5 m length consists of 6 conical blocks of regular shape and a key block of smaller size. Each precast concrete ring has an extrados diameter of 9.1 m and a thickness of 0.4 m. The excavation speed of the machine has reached considerable peaks of 15-20 m/day.

4. FIELD MONITORING

Field monitoring is a very important tool, especially in tunneling, where environments are usually heterogeneous and uncertain. The main types of measurements for underground works are displacements, loads, stresses, hydraulic pressures, temperature and vibrations.

The underground monitoring system of the two tunnels undercrossing Bologna consists of 16 sections of geotechnical monitoring placed along the entire track perpendicular to the axis of the tunnel. The measurements consist of inclinometers, topographic instruments, motorized total stations, multipoint extensometers, and piezometers. For tunnel lining, to follow deformations, strain gauges were installed in the segments at the time of fabrication. The gauges are of vibrating wire type suitable to be embedded in the segmental lining (3.000με measuring range, sensitivity 1με).

Immediately after the implementation of the segments in the tunnel, the zero value was recorded.

Each ring consists of six blocks numbered (A1, A2, A3, A4 and two blocks B and C) plus the key block (named K). In the even blocks, two pairs of strain gauges were installed and oriented in a circumferential direction and located at the center of the segment and at about 25 cm from the lateral edges. The segments marked by an odd number involved 6 pairs of gauges (see figure 3).

In this paper, of the seven rings monitored, only the monitored data of ring 582 in the clayey sand is considered.

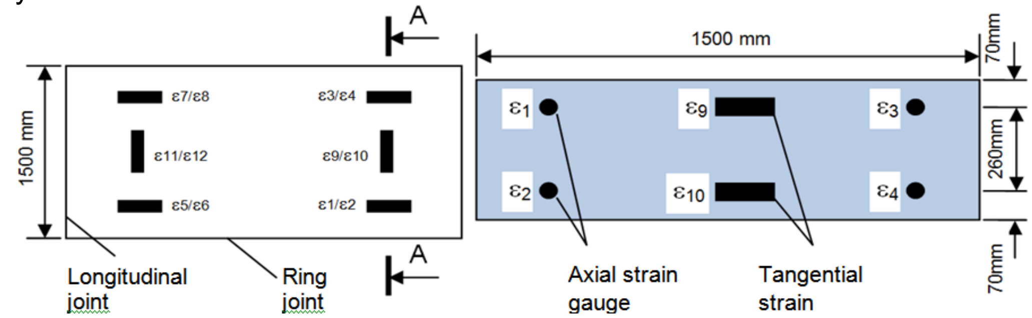


Figure 3 Layout of strain gauges in odd blocks (not scaled)

5. RESULTS AND ANALYSIS OF FIELD MEASUREMENT

In order to monitor the tunnel lining behaviour, and the effects of the second tunnel excavation process on the first one, the ring monitoring was carried out continuously and automatically. The measured strains of three gauges in block A1 installed on the right shoulder of the ring 582 are presented in figure 4. Strain gauges were monitored after the assembly of concrete blocks. The deformation progress of segmental lining shown in figure 4 can be divided into four principal phases. At the beginning time of the measurement period, very considerable deformations due to the thrust forces of the EPB hydraulic jacks and effects of injected grouting process were recorded (1st phase). When the tunnel face has advanced further beyond the ring, deformations in the blocks became relatively constant till the passage of the 2nd EPB. From figure 4, we can see the considerable changes of deformation in the lining with time in the initial hours after ring erection and especially at the date (5/6/2004) when the 2nd EPB cross the ring 582 installed in the first tunnel. It might be conclude that the excavation process of the second tunnel caused an impact on the mechanics statement of the first tunnel lining. This remark is highlighted by results of member forces in the lining presented below. Deformation velocity in lining increases significantly during the passage of the 2nd EPB and continues for 10 to 15 days after. Then, this velocity gradually reduces and the excavation process of the second tunnel has no more effect on the first one.

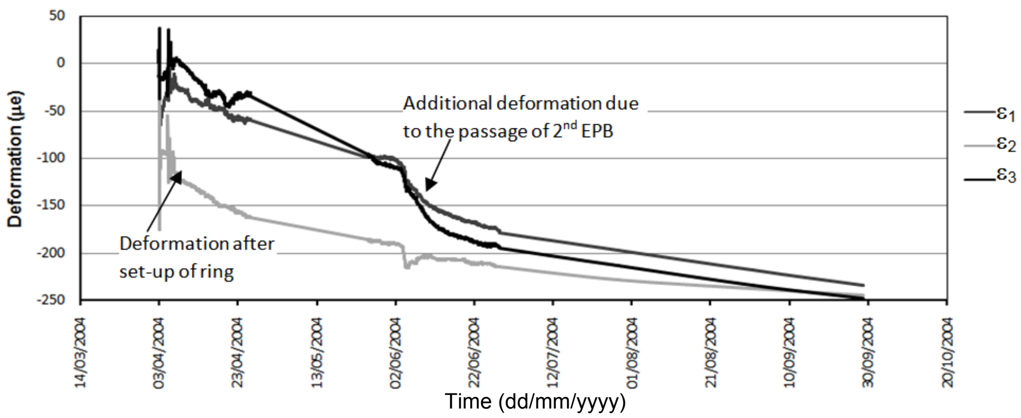


Figure 4 Deformation with time of the ring 582 - block A1

For each segment, the stresses in terms of bending moment M and normal force N are evaluated from measured strain. The maximum strain values are considered before and after the passage of the 2nd EPB. The evolution of the stresses in these two cases will be presented. In this paper, only the measurements made from strain gauges placed in the circumferential direction are considered (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 block). From each couple of strains values, for example ε_1 and ε_2, the average value permits to evaluate the normal strain of the segmental lining section. The difference of the two values can be also calculated in order to evaluate the bending strain of the segmental lining section. To calculate strain at the concrete surface, the strains have to be converted as being at the surface (the strain devices are covered with 70mm of concrete). Based on the normal strain and the bending strain, the normal force N and bending moment M acting in the lining at two sides of the ring 582 in the longitudinal direction (called " At the tunnel face" and "Behind the tunnel face", respectively) have been evaluated (see table 1 and figures 5 and 6).

Table 1 Member forces calculated based on field measurement

Block	θ (deg)	At the tunnel face					Behind the tunnel face				
		Strain gauge	M (MN.m/m)		N (MN/m)		Strain gauge	M (MN.m/m)		N (MN/m)	
			26/5/04	18/6/04	26/5/04	18/6/04		26/5/04	18/6/04	26/5/04	18/6/04
C	50	ε1-2	0.627	0.561	1.999	2.950	ε3-4	0.235	0.190	1.319	1.395
A1	86.5	ε5-6	0.112	0.171	1.964	2.732	ε7-8	0.167	0.212	2.248	2.806
A1	125.5	ε1-2	-0.063	-0.031	1.989	2.596	ε3-4	-0.063	-0.036	1.956	2.918
A3	206.5	ε5-6	0.060	0.032	2.529	2.772	ε7-8	0.038	0.013	0.920	1.096
A3	243.5	ε1-2	0.131	0.148	3.554	4.112	ε3-4	0.056	0.058	1.824	2.196
A4	285	ε1-2	-0.098	-0.054	2.055	2.815	ε3-4	-0.027	0.002	2.028	2.504
B	325.5	ε5-6	0.090	0.097	2.012	2.288	ε7-8	0.156	0.163	1.073	1.227
B	354.5	ε1-2	0.071	0.070	0.734	0.993	ε3-4	0.041	0.047	2.404	2.788

(*θ - angle measured clockwise in degrees from the tunnel right side*)

The general tendency of the internal force changes in the segmental concrete lining due to the passage of 2nd EPB presented in table 1 are relatively decreased compared to that of before. The average magnitude of the decrease of the bending moment and normal force are 47,2% and 30,6%, respectively. Whereas, bending moment at the right shoulder and normal force at the left shoulder of the tunnel are considerably increased with the average magnitude of 70% and 200%, respectively. These phenomena can be explained by the movement of the ground towards the second tunnel and then followed by an additional downward displacement of the ground above the first tunnel which causes the increase of vertical external loads acting on the lining rings installed in the first tunnel.

6. ANALYTICAL METHODS OF EINSTEIN AND DUDDEK

These two analytical models are based on a circular lined excavation in a uniformly stressed continuum. These models also assume that the ground is a semi-infinite medium and therefore they should only be used for tunnels where the axis is deeper than two tunnel diameters below the surface [10]. In the case of Bologna project, two tunnels have been excavated at a depth of near 20-25 m below the surface so the two above methods can be considered.

Einstein uses two ratios: the compressibility ratio C* and flexibility ratio F* to take into account the interaction between the tunnel lining and the surrounding ground medium using symmetric loading conditions and anti-symmetric loading conditions, respectively. The results of bending moment M and normal force N are given considering with and without bonding forces between the tunnel lining and the ground, corresponding to the no-slip case and the full-slip case as mentioned below [3].

The member forces for the no-slip case can be calculated using formulas:

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

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

The member forces for the full-slip case can be calculated using formulas:

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

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

Where θ = angular location (counterclockwise with respect to horizontal), radial; R = tunnel radius, m; σ_v = vertical stress, MN/m^2; K = ratio of horizontal-to-vertical stress; E = Young's modulus of the ground mass, MN/m^2; and a^*_0 , a^*_2 , b^*_2 = dimensionless coefficients.

In fact, the annular void behind the segment ring is grouted. Considering this practical and to take into account the effect of longitudinal joint in the ring, in this paper, the member forces obtained from the methods of Einsteinand Schwartz and Duddeck and Erdmann are calculated based on equivalent parameters of a composite lining, including segmental lining with longitudinal joints and grout layer outside. From which, we can determine the member forces acting in the segmental lining only.

The bedding beam model proposed by Duddek and Schwartz (1982) is a generally accepted structural design model in a lot of countries [5,7,10]. The member forces in the lining are dependent on the stiffness of the lining relative to that of the ground that surrounds it. They are evaluated in considering and not considering the bonding force between the tunnel lining and the ground [2].

The member forces for the no-slip case can be calculated using formulas:

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

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

The member forces formulas for the full-slip case can be calculated using formulas:

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

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

Where θ = angular location (clockwise with respect to horizontal; R = tunnel radius, m; σ_v = vertical stress, MN/m^2; K = ratio of horizontal-to-vertical stress; E, E_{sup} = Young's modulus of the soil and the tunnel lining, respectively, MN/m^2; ν = Poisson's ratio of the soil; J = the moment of inertia of the cross-section of the tunnel lining, m^4; and A = cross-section area of the tunnel lining per unit length along the tunnel axis, m^2.

Geomechanical parameters at location of segment lining ring 582 used as input in the Einstein and Schwartz method and Duddeck and Erdmann method are presented in table 2. Figures 5 and 6 show the results of bending moment and normal force obtained by using the two analytical methods in the same diagram with the field measurements. The maximum values of member forces acting 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	Thickness of grout layer h_m (m)	0.15
Cohesion c (MPa)	0.02	Width of segmental lining b (m)	1.5
Internal friction angle φ (degree)	32	Young's modulus of soil E (MPa)	250
Unit weight γ (MN/m^3)	0.017	External tunnel radius R (m)	4.7
Young's modulus of concrete E_{sup} (MPa)	35000	Poisson's ratio of soil ν	0.3
Young's modulus of grout E_q (MPa)	12000	Poisson's ratio of concrete ν_{sup}	0.15
Thickness of segmental lining h_c (m)	0.4	Lateral pressure coeffecient K	0.85

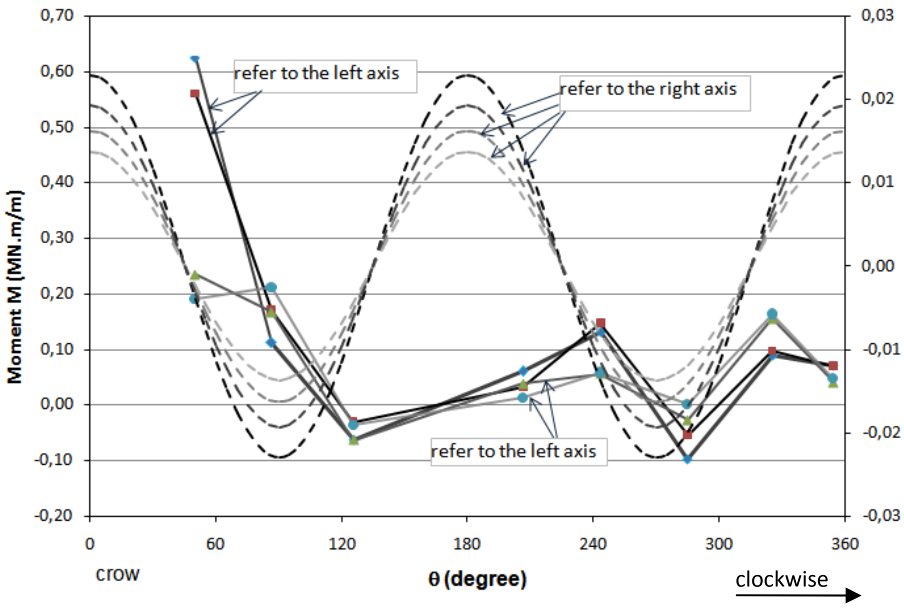
Table 3 Maximum member forces in the segmental lining ring 582

	Field measurement		Einstein's method		Duddek'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 (MN.m/m)	0.627	0.561	0.016	0.014	0.023	0.019
(%)	100	89.5	2.6	2.2	3.7	3.1
N (MN/m)	3.554	4.112	0.900	0.912	1.348	1.401
(%)	100	115.7	25.3	25.7	37.9	39.4

7. DISCUSSION

From figures 5 and 6, we can show that the results of member forces in the segmental lining based on Duddek and Erdmann method are in fair agreement with values obtained by Einstein and Schwartz method. But the values of both of the two above methods are very low compared to that evaluated using the field measurements.

These differences are mainly due 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 of the disturbances occurring in the medium induced by construction process, especially after the set-up of segments on the tunnel's periphery (grouting pressure, jacking forces, etc), the heterogeneity of soil or the impact of joints. In general, the analytical methods represent simplified ones due to their initial assumptions. In addition, in case of these two tunnels excavated in parallel, the rule of thumb is that the distance between the two tunnels should be higher than one tunnel diameter. In this case, the distance is less than one tunnel diameter, the design should then take into account of the second tunnel construction effect.



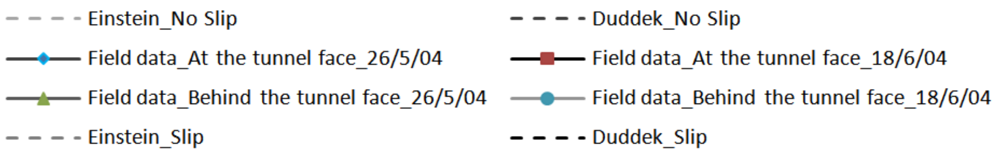


Figure 5 Bending moment diagram in the lining ring 582

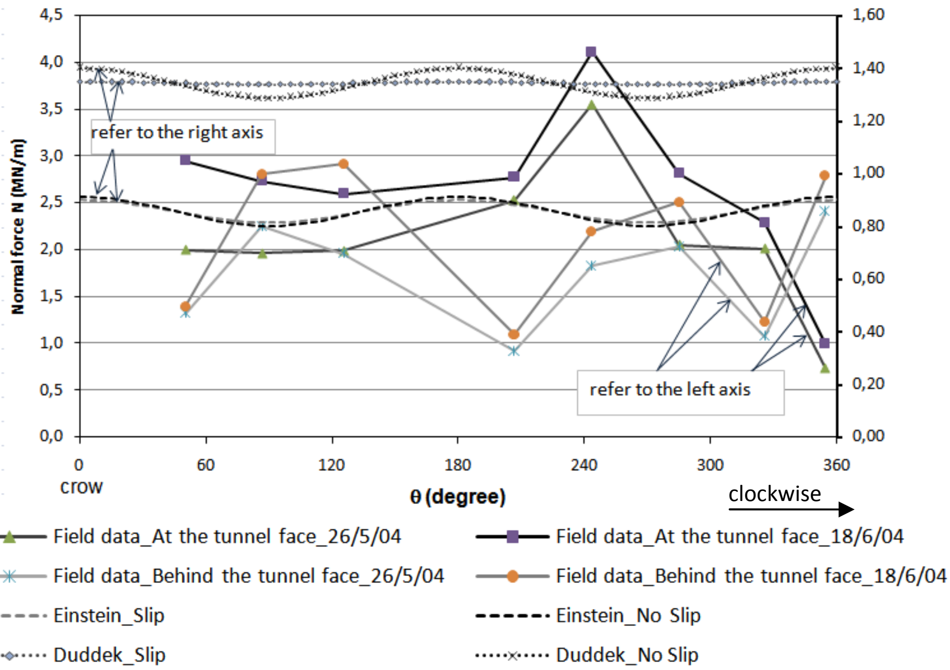


Figure 6 Normal force diagram in the lining ring 582

The same differences are also found in results presented by Bakker (2003) based on the comparison of experimental data obtained during the construction of the Second Heinenoord bored tunnel in Netherlands with analytical results computed using Duddek and Erdmann method and of a 2D FEM analysis (taking the effect of volume loss into account and using the reduced bending stiffness to simulate the effect of joints in a equivalent continuous lining). Although along the major part of tunnel's periphery, there is a relatively good agreement in shape of member force lines, but the maximum difference of bending moment and normal force obtained by analytical and 2D numerical methods are about 250% and 200% lower comparing to measured data, respectively [1].

Based on above results, we can conclude that the Duddek and Erdmann method and Einstein and Schwartz method underestimate the actual member forces during construction that means they are not safe for the design of segmental lining and need to be improved.

8. CONCLUSIONS

With reference to two tunnels excavated using earth pressure balance shields, monitoring was performed systematically to control the ground movements around the tunnels, and more specifically for the strains in segmental lining. By comparing member forces evaluated based on the field measurements with those obtained by using the two analytical methods of Duddek & Erdmannand Einstein & Schwartz, it can be concluded that these two methods underestimate the actual member forces during construction that means they are not safe for the design of segmental lining and need to be improved. To improve the design of this structure type, it will be necessary to take into account the complexity of the geometry, of the geology, of the construction process, and of the effect of joints. To develop a more precise design method, it will be necessary to set up a complete three dimensional numerical model to take into account of all the parameters, then to validate it with experimental data. Based on this database, it will be simpler to evaluate the impact of the several parameters and to develop an improved design method.

REFERENCES

[1] Bakker K.J., 2003, "*Structural design of linings for bored tunnel in soft ground*", Heron, Vol. 48, N0 1. pp.33-63.

[2] Duddek H. and Erdmann J., 1985, "*Structural design models for tunnels*", Underground Space Vol/Issue: 9, pp.5-6.

[3] Einstein H.H. and Schwartz C.W., 1979, "*Simplified analysis for tunnel supports*", Journal of the geotechnical engineering division, pp.499-517.

[4] Zheng-Rong H., Wei Z., Jing-Hua L., Jian L., et Rui J., 2006, "*Three dimensional numerical modelling of shield tunnel lining*", Tunnelling and Underground Space Technology 21, N^o. 3-4. pp.434-434.

[5] I.T.A. Working Group No. 2, International Tunnelling Association, 2000, "*Guidelines for the design of tunnels*", Tunnelling and Underground Space Technology, Vol. 15, Issue 3, pp.303-331.

[6] Mitsutaka S., 2006, "*Causes of shield segment damages during construction*", International Symposium on Underground Excavation and Tunnelling, Bangkok. pp.67-74.

[7] Roland D.W., 1999, Ph.D thesis, "*Steel fibre reinforced tunnel segment for the application in the shield driven tunnel*", ISBN 90-407-1965-9, DUP, Delft, Netherlands.

[8] Teachavorasinskun S., Chub-Uppakarn T., 2009, "*Experimental Verification of Joint Effects on Segmental Tunnel Lining*", Electronic Journal of Geotechnical Engineering, Vol.14.

[9] Teachavorasinskun S., Chub-Uppakarn T., 2010,"*Influence of segmental joints on tunnel lining*", Tunnelling and Underground Space Technology, Vol. 25, N^{o}4. pp.490-494.

[10] The British Tunnelling Society and The Institution of Civil Engineers, 2004, "*Tunnel lining design guide*", Thomas Telford Publishing, ISBN: 0 7277 2986 1, London.