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Highlights

► Detailed study of tunnel-ground longitudinal force transmission mechanisms. ► Progressive loss of the tunnel longitudinal compression caused by lining creep. ► Engineering prediction of linings stress evolution and anchorage lengths at the ends. ► Longitudinal compression losses up to 65% depending on the conditions. ► Significant influence of plastic packers creep on linings stress loss.

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Longitudinal time-dependent response of segmental tunnel linings

O. Arnau^{a,*}, C. Molins^a, C.B.M. Blom^{b,1}, J.C. Walraven^{b,1}^a Department of Construction Engineering, Universitat Politècnica de Catalunya (UPC), Jordi Girona 1-3, C1, 08034 Barcelona, Spain^b Faculty of Civil Engineering and Geosciences, Delft University of Technology, Stevinweg 1, P.O. Box 5048, 2600 GA Delft, The Netherlands

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ABSTRACT

The longitudinal forces introduced by tunnel boring machines (TBMs) to the segmental tunnel linings influence their structural response. The analyses of the linings construction process and the ground–structure interaction mechanisms have shown the influence of the lining creep on the progressive loss of the initial longitudinal force. An analytical formulation to predict the remaining compression of the linings as a function of time is proposed, supported by means of a complete numerical model, which considers the effect of creep during the sequential construction process. An experimental program to determine the creep of plastic packers was developed, revealing its significant influence on the global lining creep factor and the evolution of the remaining compressive stresses.

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1. Introduction

The application of modern tunnel boring machines (TBMs) allows the construction of tunnels of larger dimensions at increasing depths under complex ground conditions. Its use is mostly associated with segmental concrete linings which provide the structural resistance to the ground and water pressure, further to provide the necessary reaction to generate the progress of the TBM. The improvement of the knowledge and comprehension of the structural response of segmental tunnel linings will entail the optimization of their design, providing economical, efficient and safer structures.

A segmental concrete lining is composed of multiple concrete rings that are sequentially placed as tunnel boring advances (Fig. 1). There exist different types of linings depending on the shape of the segments and the ring (JSCE, 2006; Guglielmetti et al., 2008). Concrete segments have to be designed in order to individually resist the casting and storage process and the forces coming from the TBM. The whole ring design is focused on the adequate resistance of all foreseeable ground loads, which can be determined by means of advanced numerical models that simulate the influence of the boring process into the ground stresses (Broere and Brinkgreve, 2002; Kasper and Meshke, 2004). The analysis of the lining internal forces is nowadays performed by means of finite element models that allow the consideration of the structural par-

ticularities presented in segmental linings (Plizzari and Tiberti, 2006; Blom et al., 1999). The joints existing between segments create a multiple-hinged structure that presents a complex structural behavior (Muir Wood, 1975; Blom, 2002; Teachavorasinskun and Chub-uppakarn, 2010). Research programs have been developed recently in order to determine, explain and model the different phenomena involved in segmental linings radial response (Molins and Arnau, 2011; Arnau and Molins, 2011), to optimize its structural resistance by means of steel fiber additions (de Waal, 2000; Kasper et al., 2007; Tiberti et al., 2008) or directly increase the structural resistance by applying composite sections (Zhang and Koizumi, 2010).

TBM's apply a large longitudinal force to segmental tunnel linings by means of multiple hydraulic jacks in order to produce their movement and to resist the excavation face pressure. Packing materials are commonly placed in circumferential joints (between adjacent rings) in order to regularize the contact surfaces and to center the TBM force into the segments height (Fig. 1). As a consequence, the structural collaboration between adjacent rings is governed by a packer-concrete friction mechanism that directly depends on the normal force applied on. A few tunnel projects include a dowel and socket system on their circumferential joints in order to confine the differences in deformation between adjacent rings (Blom, 2002). In these cases, the frictional mechanism only acts until the dimensional tolerance of the dowel and socket system is exhausted. Therefore, the structural analysis of segmental tunnel linings should include the longitudinal forces applied on the lining if its real three-dimensional behavior needs to be adequately considered (Blom et al., 1999; Mo and Chen, 2008).

Recent research programs employed advanced 3D structural models that consider the longitudinal force to analyze different

* Corresponding author. Tel.: +34 934017349; fax: +34 934054135.

E-mail addresses: oriol.arnau@upc.edu (O. Arnau), climent.molins@upc.edu (C. Molins), c.b.m.blom@tudelft.nl (C.B.M. Blom), j.c.walraven@tudelft.nl (J.C. Walraven).

¹ Tel.: +31 015 2784578; fax: +31 015 2787438.

Nomenclature

A_c	concrete cross area	S_{cl}	lateral concrete surface of a ring
AR	lateral surface aspect ratio	S_{pl}	lateral ring surface covered by packers
E_c	concrete deformation modulus	t	time of analysis
$E_{c,28}$	concrete deformation modulus at 28 days	t_0	age of loading
E_s	ground deformation modulus	u	longitudinal displacement
F_i	initial force	u_p	perimeter in contact with the atmosphere
F_t	transferred force to the ground	u_{sl}	ground spring displacement for plastic response
$F_{x,n}$	force transmitted to the ground at ring n	x	variable of position
F_{RF}	force at tunnel initial reaction frame	ε_c	concrete strain
F_{TBM}	longitudinal force applied by the TBM	$\varepsilon_{c\sigma}$	concrete stress dependant strain according to CEB-FIP Model Code
f_{cm}	concrete compressive strength	ε_{el}	concrete elastic strain
h	notational size of the member	$\varepsilon_{gr,\Delta t_n}$	concrete creep strain at time period Δt_n
HR	relative humidity	ε_g	grout strain
K_l	longitudinal ground–structure interaction stiffness	ε_p	packer strain
K_r	radial ground–structure interaction stiffness	ε_{p-el}	packer elastic strain
K_t	tangential ground–structure interaction stiffness	ε_{p-cr}	packer creep strain
L_{an}	anchorage length	σ_c	concrete compressive stress
L_c	concrete ring width	ν	Poisson ratio
L_p	packer thickness	τ_{sl}	maximum tangential stress
L_t	total ring width	$\varphi_c(t, t_0)$	concrete creep coefficient for t_0 to t period
N	longitudinal force	$\varphi_p(t, t_0)$	packer creep coefficient for t_0 to t period
P_e	tunnel external perimeter	$\varphi_G(t, t_0)$	lining global creep coefficient for t_0 to t period
R	tunnel radius		
r^2	correlation coefficient of packers creep curve fitting		

scenarios during construction or in service (Blom et al., 1999; Klapers et al., 2006; Mo and Chen, 2008). In these cases, the force applied to the lining by the TBM is adopted, assuming that the long term ring forces remain close to their initial value. As this hypothesis entails the maximum structural interaction between adjacent rings, it is necessary to ensure its validity during the full tunnel service life. Additionally, a possible loss of the longitudinal compressive stresses involves a reduction of the closing pressure in the sealing devices (gaskets) placed in the circumferential joints, causing a reduction of the watertightness of the lining.

Koek (2004) performed a study, which focused on the influences of the lining and ground stiffness on the tunnel's longitudinal response in front of TBM force variations. This work includes a brief analysis assuming that the lining is subjected to a pure relaxation process, which implies that longitudinal stress losses can occur. As a consequence, it was shown that is necessary to carry out a thorough study and analysis of the ground–structure interaction mech-

anism, which can cause a transfer of forces from the lining to the ground, as well as of the influence of the segmental construction process. The knowledge of the force transfer mechanism will enable the determination of the stress development in segmental tunnel linings.

The conicity and the thickness of the TBM shield and the overcut produced by its cutting wheel produce an annular gap between the external radius of the lining and the ground, which width ranges between 13 and 18 cm (Thewes and Budach, 2009). The complete and fast fulfilling of the annular gap presents a paramount importance in order to minimize the surface settlements (Pelizza et al., 2010) and to lock the segmental lining into its definitive placement (Fig. 2). It is usually performed by means of cementitious mixes or two component grouts (Peila et al., 2011) which conforms a stiff material between the lining and the ground. As a consequence, the adequate study of the ground–structure interaction requires the consideration of the backfill grouting process.

This paper deals with the determination of the longitudinal force that remains in segmental tunnel linings during their service

Fig. 1. Segmental tunnel linings most common configuration.

Fig. 2. Forces and constraints of an ideal construction case.

lives. In the first part, the construction process and the **ground-structure** interaction mechanism are thoroughly analyzed, showing the dependence of the longitudinal force on the long term deformations of the lining. A formulation is proposed to predict the remaining compressing force on the lining at a certain time, based on its longitudinal creep coefficient (**remaining compression factor** approach, RCF). A **finite elements model** is used to simulate the construction process of a tunnel and its time-dependent response in order to verify the suitability of the proposed formulation and to study the particular influence of lining creep during the construction stage. The influence of the creep of the packers is also studied, presenting the results of a particular experimental program. Finally, a formulation for a global longitudinal creep coefficient of the lining (φ_c) which includes both the concrete and the packers creep is proposed.

2. Ground-structure longitudinal interaction mechanism and the remaining compression factor (RCF) approach

As a consequence of the construction process of segmental tunnel linings, the TBM is constantly applying a minimum longitudinal axial force in order to compensate the excavation face pressure, which mainly depends on the TBM face support method and the ground conditions (Maidl et al., 1996). At structural level, this is achieved by means of a sequential loading process based on the individual loading of each segment, generating a permanent longitudinal compression state of the lining.

In this section, the influence of the construction process on the stress state of the lining is analyzed. Firstly, the **ground-structure** interaction mechanisms are studied in order to determine the longitudinal force transfer that can cause the loss of the initial compression state of the lining. As a result, a formulation to predict the evolution of the remaining compressive stress of the lining is proposed based on its longitudinal creep (RCF approach).

2.1. Ground-structure interaction mechanisms

In order to present in a simple and understandable way the effect of the construction process on the longitudinal response of the lining, an ideal case is firstly analyzed. The increase of the longitudinal force that can be caused by the radial loading of the rings (produced by the existing confinement in the longitudinal direction) is neglected due to its usual minor influence on the longitudinal stress of the lining. Despite that, it can be easily contemplated by superposing it to the initial applied force. Concrete is assumed as a linear elastic material without time dependent deformations whilst packing materials are not yet contemplated.

Usually, the TBM first reaction point is a steel frame installed in the field or in the shaft where the tunnel starts as it is schematically shown in Fig. 2. The loading process of ring 1 is completed once it leaves the TBM shield and becomes surrounded by the liquid backshield grout or mortar. No constraints affect its longitudinal strain ε_c , being determined as $F_{TBM}/E_c \cdot A_c$, where F_{TBM} is the longitudinal force applied by the TBM, E_c is the concrete deformation modulus and A_c is the concrete cross area. Assuming that F_{TBM} remain constant during the different construction stages, when a new ring is placed (2), it is also subjected to the same initial strain ε_c . At this time, the grout over ring 1 becomes stiffer, relating its initial zero strain ($\varepsilon_g = 0$) to the segments deformed shape ($\varepsilon_c = F_{TBM}/E_c \cdot A_c$). Whilst the TBM force remains constant and permanently applied, the situation will be the same for all rings. Therefore, under the assumption of the ideal conditions described, the ground does not contribute to the equilibrium of longitudinal forces of the lining. The activation of the **ground-structure** interaction requires longitudinal lining movements after grout hardening.

Two main different circumstances can cause displacements of the lining: (1) changes in the applied forces (F_{TBM} or the reaction at the initial frame, F_{RF}) and (2) long-term deformations of the concrete segments.

The removal of the initial frame produces the disappearance of the reaction force (F_{RF}). In consequence, the lining starts its displacement, activating the transfer of forces from the lining to the ground until the lining is fully anchored. Severe changes in TBM forces generate a similar process, producing lining displacements until the force difference is fully transmitted to the ground. Both transfer mechanisms are local, only affecting a certain lining section depending on the necessary length to complete the transmission of forces.

As long as the lining and the ground properties remain linear elastic, the transfer of forces between them is governed by the problem of a bar embedded in an elastic media (Fig. 3). The differential equation that defines the equilibrium of forces is obtained from the scheme presented in Fig. 3 (Eq. (1)), where x is the variable of position, u is the longitudinal displacement, K_l is the longitudinal stiffness of the ground (N/mm^3) and P_e is the external perimeter of the tunnel (mm).

$$E_c A_c \frac{d^2 u}{dx^2} - K_l P_e u = 0 \quad (1)$$

$$\frac{d^2 u}{dx^2} - \alpha^2 u = 0 \quad \text{where: } \alpha^2 = \frac{K_l P_e}{E_c A_c} \quad (2)$$

Eq. (1) corresponds to a second order differential equation (Eq. (2)). The mathematical solution of Eq. (2) can be found in some publications (i.g. Bouma, 1989), providing the expression of the transferred force F_t (Eq. (3)). Assuming that a force F_i is initially applied to the tunnel, the evolution of the tunnel longitudinal force (N) corresponds to Eq. (4).

$$F_t(x) = F_i e^{-\alpha x} \quad \text{where: } \alpha = \sqrt{\frac{K_l P_e}{E_c A_c}} \quad (3)$$

$$N(x) = F_i - F_t \quad N(x) = F_i - F_i e^{-\alpha x} \quad (4)$$

$$L_{an} = \frac{\ln(0.05)}{-\sqrt{\frac{K_l P_e}{E_c A_c}}} \quad (5)$$

The force transmission zone (anchorage length) ends when the lining force $N(x)$ is equal to the initial applied force F_i . Eq. (4) presents an asymptotic behavior to F_i , and therefore, if it is solved assuming that F_t is equal to F_i , the value of x is infinite (∞). As a consequence, it is necessary to assume a tolerance value of the initial force F_i which determines that the lining is fully anchored.

Fig. 3. Schematization of the development of the force in the lining based on differential analysis.

Assuming a tolerance of 5% in the force ($F_{L_i} - N(x) = 0.05 \cdot F_i$), the resolution of Eq. (4) provides the expression for the anchorage length (L_{an}) shown in Eq. (5).

According to Eq. (5), for certain lining conditions (fixed E_c , A_c and P_e) the anchorage length only depends on the ground stiffness K_t . Therefore, as long as the ground (or the **ground–structure** interface) presents a linearly elastic behavior, the anchorage length (L_{an}) is independent of the initial force (F).

2.2. Effect of long-term deformations (RCF approach)

Creep and shrinkage are the essential time dependent deformations experienced by the structural concrete. In tunnel linings construction, concrete segments are precast in an industrial plant and subsequently stocked until their assembly. As a consequence, creep is the only time dependant deformation that significantly affects the behavior, because shrinkage is almost fully developed when the segments are placed.

The strain of concrete segments (ϵ_c) is composed by the elastic strain (ϵ_{el}) and the creep strain ($\epsilon_{cr,\Delta t}$). Creep is the increase in strain caused by a sustained load in a time period Δt (Nawy, 2008). As described in Section 2.1, the rings are initially fixed to

the ground according to their elastic strain. As time goes by, creep phenomenon produces a reduction of the ring size due to the described strain increase. As a consequence, the concrete lining undergoes longitudinal displacements that activate the **ground–structure** interaction. Whilst the ground tangential behavior remains linear, a larger movement results in a larger force transmitted to the ground. As can be observed in Fig. 4, the transmission of the force to the ground (F_g) implies that the force in the previous ring decreases, also diminishing its stress and its elastic strain (ϵ_{el}). Consequently, a stress/strain redistribution occurs in order to achieve a new equilibrium of forces.

As it was previously explained, as long as the maximum tangential stress of the ground (or the **ground–grout** interface) has not been exceeded, the length of the force transmission zone (L_{an}) is independent of the magnitude of the force. This means that outside the anchorage zone, the force transmission from the lining to the ground should be zero. This assumption implies that, once the surrounding grout has hardened, the displacement of each ring should be zero as well. The only possibility to accomplish this requirement is that the total strain of a ring at a certain time t , $\epsilon_c(t)$ is equal to its initial strain $\epsilon_c(t_0)$ (Eq. (6)). This phenomenon occurs when the values of the positive relative strains caused by creep are equal to the

Fig. 4. Schematization of force transmission due to lining creep.

Fig. 5. RCF evolution as a function of the concrete creep coefficient, $\chi(t, t_0) = 0.9$.

negative relative strains caused by the decrease of the compression stress. Therefore, the lining is subjected to a pure relaxation mechanism caused by its longitudinal creep that causes a reduction of its compressive stress in time.

$$\varepsilon_c(t) = \varepsilon_c(t_0) \quad (6)$$

$$\varepsilon_c(t) = \frac{\sigma_c(t_0)}{E_c} [1 + \varphi(t, t_0)] + \frac{\Delta\sigma_c(t, t_0)}{E_c} [1 + \chi(t, t_0) \cdot \varphi(t, t_0)] \quad (7)$$

The strain of the concrete at a certain time t is commonly evaluated by Eq. (7) (Bazant and Wittmann, 1988), where $\sigma_c(t_0)$ is the initial concrete stress, $\Delta\sigma_c(t, t_0)$ is the stress variation from the load time t_0 to t whilst $\varphi(t, t_0)$ and $\chi(t, t_0)$ are the creep coefficient and the aging coefficient related to the same time period respectively. The combination of Eqs. (6) and (7) directly allows the derivation of a Remaining Compression Factor (RCF) (Eq. (8)) which describes the proportion of the initial longitudinal lining stress that remains in the concrete lining after a certain time t (Eq. (9)).

$$RCF(t) = 1 - \frac{\varphi(t, t_0)}{[1 + \chi(t, t_0) \cdot \varphi(t, t_0)]} \quad (8)$$

$$\sigma_c(t) = \sigma_c(t_0) \cdot RCF(t) \quad (9)$$

During the construction process of the segmental lining, the concrete is loaded at a relatively advanced age (generally more than 60 days). As a consequence the value of the aging coefficient $\chi(t, t_0)$ is always near to 1 and can be considered constant for time periods longer than 1 year (Ghali et al., 2002). Fig. 5 shows the evolution of the RCF as a function of the creep coefficient $\varphi(t, t_0)$, assuming a constant value $\chi(t, t_0) = 0.9$. For small values of the creep coefficient (short time after loading) the RCF decreases relatively fast, achieving a value under 0.5 for a creep coefficient of 1.

3. Numerical simulation

A complete set of numerical analyses are employed to determine the validity of the proposed analytical formulations and the limitations imposed by the performed hypotheses. An advanced FE model that considers the longitudinal creep during the tunnel construction process is developed for such purpose. The analyses comprise a total number of nine simulations, reproducing three different concrete scenarios and three surrounding ground stiffnesses. To simplify the problem and to determine its minimum effect, only the concrete creep is considered (the effect of packers is not yet taken into account).

3.1. Case study and model description

The selected case study is a 900 m long tunnel with an external diameter of 10 m and a thickness of 0.35 m. Its construction

process is composed by the following steps: (1) the lining starts at a reaction frame which is modeled by means of longitudinal rigid supports at the first ring, (2) the tunnel segments are placed according to a construction speed of 9 m per day, (3) at the end of construction, the TBM force is not further applied and the initial reaction frame is disassembled, and (4) time proceeds until 10,000 days (27 years), which can reasonably be considered as the time of stabilization for the time-dependant properties of concrete.

Due to the symmetry of the circular tunnels and assuming a uniform distribution of the longitudinal stresses at every ring, it is possible to model the whole lining longitudinal behavior as a tunnel sector. Therefore, a rectangular cross section of 1×0.35 m is adopted. The longitudinal force applied by the TBM to the lining (F_{TBM}) is considered constant and permanent during the whole construction process. The selected value of F_{TBM} is 1750 kN, determined to produce a uniform compressive stress of -5 N/mm².

A bar elements model is proposed for the case study. The ground-structure interaction is modeled by means of distributed longitudinal springs (Fig. 3). Three different values of the ground-structure stiffness (K_l) are applied in order to cover a significant range of ground characteristics (Table 1). There is no generally accepted relation to determine the ground-structure longitudinal stiffness (K_l) from a certain tunnel and ground properties. In the numerical analyses carried out, K_l was assumed to be equal to the tunnel tangential stiffness (K_t), which is commonly accepted to be one third of the radial stiffness (K_r) (Eqs. (10) and (11)). These relations are obtained from the resolution of the ground distortion response and can be particularly adjusted to certain tunnel and ground conditions by means of finite element software as shown by Koek (2004).

$$K_r = \frac{E_s}{R \times (1 + \nu)} \quad (10)$$

$$K_l = K_t = \frac{K_r}{3} \quad (11)$$

The ground-structure interaction failure is considered using elastic-plastic behavior to define the response of the longitudinal springs (Fig. 6). The determination of the maximum tangential stress for each ground type (τ_{sl} in Table 1) is performed according to the formulations and recommendations described by Fleming et al. (1994), assimilating the tunnel lining to a single pile foundation and assuming the ground properties given in Table 1. The reduced tunnel pressure of Terzaghi (JSCE, 1996) was used to determine the effective ground pressure on the tunnel. The spring displacement that produces the start of the plastic response (u_{sl}) is determined by the expression of the ground-structure longitudinal stiffness:

$$u_{sl} = \frac{\tau_{sl}}{K_l} \quad (12)$$

Table 1

Ground types and characteristics employed in the numerical analyses.

Property	Hard/rock mass	Medium/gravel	Soft/soft clay
Longitudinal stiffness, K_l (N/mm ³)	0.1	0.01	0.001
Deformation modulus, E_s (N/mm ²)	1950	195	19.5
Poisson ratio (ν)	0.3	0.3	0.3
Rock strength, q_u (N/mm ²)	10		
Friction angle, ϕ (°)		30	
Density, γ (kN/m ³)		24	
Cohesion, c (kN/m ²)		50	
Undrained shear strength, c_u (kN/m ²)			50
Tangential strength, τ_{sl} (kN/m ²)	500	179	25

Fig. 6. Model for ground-structure interaction springs response.

3.2. Numerical model results

Fig. 8 presents the longitudinal lining stress distributions for Model A and $K_l = 0.01 \text{ N/mm}^3$ at different times. As well known, the concrete creep effect is largest immediately after loading of the structural member, starting the loss of force during the construction process. When the last ring has been placed (just before the initial reaction frame disassembly, Cons End Pre-RFD in Fig. 8), the longitudinal stress at the start of the lining is -3.79 N/mm^2 , representing a reduction of 24.2% (2.69% per 100 m of tunnel).

In 500 days after the start of construction, almost the whole lining sustains a stabilized stress of -3.13 N/mm^2 , diminishing to a value of -2.37 N/mm^2 after 10,000 days. As a consequence, 71.1% of the total stress losses occur during the first 500 days.

At end of construction, when the TBM force stops and the initial reaction frame is disassembled (Fig. 8, Cons End Post-RFD), the length of the anchorage zones can be clearly appraised. The anchorage length does not present significant variations during the stress relaxation process.

Fig. 9 shows the longitudinal stresses obtained at 10,000 days for Model A concrete and the different ground conditions. As explained in Section 2.1, the surrounding ground stiffness does not show a significant influence on the remaining longitudinal compression of the lining. The small differences that can be appraised in Fig. 9 are related to the sequential construction process. Depending on the ground stiffness, the stress-strain redistributions that constantly occur during the tunnel construction due to the lining creep slightly differ. Then, the lining stress distribution at construction end is also different (Fig. 10), conditioning the creep and stress evolution of the lining.

Diana 9 software is employed for the resolution of these staged construction analyses. This software includes the creep formulation proposed in the CEB-FIP Model Code (1993), which is adopted for the numerical analyses. Eq. (13) shows the stress dependent strain (ε_{cs}) proposed by CEB-FIP Model Code (1993). Creep coefficient φ_c mainly depends on the compressive strength of concrete, the dimensions of the member, the relative humidity to which the member is exposed and the age of loading. Table 2 shows the properties and parameters of the concrete scenarios analyzed, which provide the evolution of the creep coefficient $\varphi_c(t, t_0)$ shown in Fig. 7, and determine values of $\varphi_c(t, t_0)$ at 10,000 days from 1 to 2.

$$\varepsilon_{cs}(t, t_0) = \sigma_c(t_0) \left[\frac{1}{E_c(t_0)} + \frac{\varphi_c(t, t_0)}{E_{c,28}} \right] \quad (13)$$

Table 2
Properties and parameters of the concrete scenarios analyzed.

Concrete properties/parameters	Model A	Model B	Model C
Concrete compressive strength, f_{cm} (N/mm ²)	50	40	30
Deformation modulus, E_c (N/mm ²)	38,606	36,246	33,550
Concrete age of loading, t_0 (days)	100	60	30
Concrete area, A_c (m ²)	0.35	0.35	0.35
Perimeter in contact with the atmosphere, u_p (m)	1	1	1
Notational size of the member, $h = 2A_c/u_p$ (mm)	700	700	700
Relative humidity, HR (%)	75	54	48
Resulting creep coefficient at 10,000 days, $\varphi_c(t, t_0)$	1	1.5	2

Fig. 7. Concrete creep coefficient evolution for the analyzed scenarios according to the CEB-FIP Model Code (1993).

Fig. 8. Evolution of longitudinal lining stress for Model A and $K_l = 0.01 \text{ N/mm}^3$. Fig. 9. Longitudinal lining stress at 10,000 days for Model A and different K_l values.

Fig. 10. Distribution of the longitudinal stress for Model A and different K_I values at construction end just before the reaction frame disassembly.

Fig. 11. Longitudinal lining stress for different concrete models and the RCF prediction. $K_I = 0.01 \text{ N/mm}^3$.

The ground stress analysis reveals that the maximum tangential stress (τ_{st}) is not achieved in any case. Consequently, the anchorage lengths (L_{an}) obtained with the proposed formulation (Eq. (5)) should accurately fit the numerical results. As can be observed in Fig. 9, the analytical results of the anchorage lengths for the three different ground conditions (represented by the dotted lines in Fig. 9) show excellent agreement with the lining response obtained with the numerical model.

Fig. 11 presents the results of the three different concrete models after 10,000 days for $K_I = 0.01 \text{ N/mm}^3$ compared with the results obtained with the proposed formulation for the RCF (Eqs. (8) and (9)). It can be concluded that the accuracy achieved by the RCF approach for the prediction of the long term longitudinal stress is excellent. As the RCF formulation is obtained from the static analysis of the segmental tunnel lining, the time effects during the construction stages are not considered. As a consequence the accuracy for short times is low (Fig. 12). For the case study

Fig. 12. Evolution of longitudinal lining stress of Model A and $K_I = 0.01 \text{ N/mm}^3$, RCF prediction and its error versus the numerical model.

Fig. 13. Modeling scheme adopted for grout analyses.

regarded (Model A with $K_I = 0.01 \text{ N/mm}^3$), the error of the RCF becomes lower than 10% after 750 days (2.05 years).

3.3. Grout influence

Another set of analyses was carried out in order to study the backfill grout behavior and its influence on the longitudinal response of the lining. Three different grout modulus of elasticity were used in the analysis for each different ground condition described in Section 3. An assessment of the influence of grout shrinkage was also performed by assuming a value of 0.05 mm/m according to the favorable curing conditions. The previously described model was completed by means of the addition of plane strain elements between the bar elements (lining) and the springs (ground) to simulate the grout (Fig. 13).

The results showed that: (1) the grout modulus of elasticity does not present a significant influence on the lining axial stress, (2) along the anchorage zones the grout is mainly subjected to an axial stress that can cause its tensile cracking for very stiff grouts, and (3) the lining creep and the grout shrinkage do not significantly influence the grout tensile stress for general tunnel conditions. Backfill grout cracking should not have a significant effect on the radial structural capacity of the segmental tunnel lining, but it can diminish the watertightness of the lining.

3.4. TBM force variations

The hypothesis of constant longitudinal force was assumed in order to simplify and clarify the study of the construction process and the time-dependant response of segmental tunnel linings. In real tunnel construction, the TBM force is not a constant value and can undergo significant variations mainly, caused by severe changes in ground conditions. Another set of numerical analyses of the tunnel construction process was carried out applying different TBM longitudinal forces at different lining sections. The analysis of the results led to two main conclusions: (1) when a TBM load variation occurs, a transmission zone is created around that point in order to transfer the force difference from the lining to the ground. This force transmission mechanism is the same as described in Section 2.1 and, therefore, its length is independent of the magnitude of the force and can be accurately determined by means of Eq. (5). (2) Outside of the described anchorage or transmission zones, the RCF approach (Eqs. (8) and (9)) allows a correct prediction of the compression stress evolution of each tunnel section subjected to different longitudinal loads. It is only necessary to apply the proposed formulation to the initial compression stress and the creep properties of the section analyzed.

4. The influence of creep of the packers

Concrete segments are not the only structural members subjected to permanent compression in the longitudinal direction of

segmental tunnel linings. A packing material is used to be placed at the circumferential joints (between adjacent rings) in order to control and regularize the longitudinal force transmission between the rings (Fig. 1). The lateral surface area covered by packers (S_{pl}) is always smaller than the concrete surface area (S_{cl}) and, as a consequence, they are subjected to higher stresses than the segments (aspect ratios $AR = S_{cl}/S_{pl}$ from 2 to 3 are common in current linings design). Despite their small thickness (around 2 mm) which is 1000 times smaller than common ring widths (1500–2000 mm), their modulus of elasticity is around 150 times lower (about 30.000 versus 200 MPa). The combination of these factors with the higher compressive stress on the packers can lead to a situation that the packer creep represents a significant part of the total longitudinal creep of the lining.

Packers are usually made of plastic compounds or wood, where the first option is the most commonly used in current tunnel projects. Plywood packers can undergo some volume reductions in time due to the material rotting, especially if the tunnel is subjected to severe groundwater conditions (De Waal, 2000). This phenomenon is of another nature than creep but, in practice, it has a similar effect on the force in the lining.

The creep behavior of plastics is well known and is considered in the design of plastic components and pieces (Crowford, 1998; Brydson, 1999). Its characterization is commonly supplied by the main manufacturers of plastics for the original polymeric compound. Plastic packers are usually made of polyethylene, polyurethane or polypropylene and can be produced from recycled materials. Plastic's mechanical behavior is affected by multiple parameters (material properties, manufacturing process, temperature, stress type, etc.) and, therefore, it is necessary to determine the particular creep of plastic packers if it has to be considered in the lining response.

4.1. Experimental program to determine the creep of plastic packers

In order to characterize the creep deformations of the plastic packers applied in segmental linings, an experimental program was carried out at the laboratory of structures technology (UPC, Barcelona). The plastic packers applied in three different Spanish tunnel projects were tested under compression stresses of 4 and 20 N/mm², representing a common stress range with regard to different TBM forces and lateral ring surface aspect ratios. Table 3 summarizes the main characteristics of the packers tested.

The experimental set-up was based on columns of stapled specimens as commonly used in concrete creep testing (Fig. 14a). Five concrete cubic blocks of 150 mm side were piled up in order to create four interfaces between the blocks; plastic packers were allocated in three of them whilst the remaining joint represented a direct concrete contact to characterize its own time dependant deformations (Fig. 14b). The maximum planned stress exerted on the packers (20 N/mm²) is large enough to generate concrete micro-cracking or non-linear creep. In order to avoid stress concentrations in the block corners and high concrete stress levels, the surface of the packer specimens was limited to 100 × 100 mm² (Fig. 14c). Table 4 presents the packer and concrete average compression stresses, the applied axial force and the necessary

Fig. 14. Packer creep test configuration.

Table 4
Stress, force and pressure applied on the test columns.

Packer stress (N/mm ²)	Concrete stress (N/mm ²)	Axial force (kN)	Hydraulic Jack pressure (kPa)
4	1.78	40	1273
20	8.89	200	6366

pressure at the 200 mm hydraulic jack of columns for the two different test configurations.

Concrete blocks were casted at the same laboratory applying a C30/37 concrete which strength was controlled by means of standard uniaxial compression tests carried out on a 100 × 100 × 100 mm cubic specimens at 7 and 28 days.

The deformation of the packers was measured at three different faces of the concrete blocks in order to adequately determine the deformation planes. Initially, a continuous measurement was performed by means of nine displacement transducers (six LVDT and three OMEGA transducers) connected to a data acquisition system. Once the measurements achieved a high level of stabilization, the transducers were replaced by DEMEC points. The application of a mechanical strain gauge with digital display supplies the evolution of joints movements in a discrete manner.

With the experimental set-up chosen the real packer-concrete interface behavior can be measured. On the other hand, the measurement devices are fixed in the concrete blocks and, as a consequence, the influence of the concrete creep and shrinkage has to be adequately considered and subtracted from the total measurement.

The evaluation of the time-dependant deformations of the concrete was measured by means of DEMEC points bridging the concrete-to-concrete joints (Fig. 14). The results were compared with the analytical creep and shrinkage values according to the CEB-FIP Model Code (1993), in order to eliminate the influence of the surface contact. The calculations only include the drying component of shrinkage, because the blocks were loaded after finishing the curing process: a very good agreement with the experiments was observed.

4.2. Test results

Figs. 15 and 16 show the test results for the packer creep coefficient (φ_p), which is evaluated as the relation between the creep strain (ε_{p-cr}) and the elastic strain (ε_{p-el}) (Eq. (14)). For small compression stresses (4 N/mm²), the creep evolution shows an almost linear behavior whilst for high compression (20 N/mm²) the creep coefficient shows an early increase suggesting an exponential

Table 3
Main characteristics of the packers subjected to creep testing.

Tunnel project	Thickness (L_p) (mm)	Measured modulus of elasticity (E_p) (N/mm ²)
Metro of Line 9 (Barcelona)	2	202.1
Metro of La Cella (Madrid)	2	216.8
Road tunnel in M30 (Madrid)	2	188.7

Fig. 15. Measured packer creep coefficient (ϕ_p) at 4 N/mm².

Fig. 16. Measured packer creep coefficient (ϕ_p) at 20 N/mm².

response (similar to those of concrete materials). At the end of the test (330 days), the La Cela packer presents a large creep coefficient of 1.34 at 20 MPa whilst 0.98 is obtained at 4 MPa. On the contrary, Line 9 and M30 measurements do not present significant variations according to different stresses. At 330 days, Line 9 packer presents a creep coefficient around $\phi_{p,19} = 0.76-0.88$ whilst M30 presents a value between $\phi_{p,M30} = 0.42-0.44$. The small negative values obtained for M30 packer at the start of the 4 MPa test may be caused by an adaptation phenomenon of the samples. This measurement dysfunction was not presented for the 20 MPa test, showing that higher stresses provide more reliable results.

$$\phi_p = \frac{\varepsilon_{p-cr}}{\varepsilon_{p-el}} \quad (14)$$

The 20 MPa results are employed to perform a curve fitting based on Eq. (15), which corresponds to the usual formulation used in the definition of the concrete creep coefficient. Table 5 summarizes the obtained values for A, B and C parameters and its corresponding correlation factor r^2 , showing the agreement of the packers response to the selected expression (Fig. 17). Assuming that packers creep evolves as expressed in Eq. (15), the expected creep coefficients for long term analysis (10,000 days) are 1.50 for the La Cela packer, 1.05 for Line 9 packer and 0.53 for M30 packer.

$$\phi_p(t-t_0) = A \cdot \left[\frac{(t-t_0)}{B+(t-t_0)} \right]^C \quad (15)$$

Table 5
Values obtained for packers creep coefficient curve fittings.

Packer	Value of parameters			Correlation coefficient r^2
	A	B	C	
Metro of Line 9 (Barcelona)	1.0565	233	0.349	0.9874
Metro of La Cela (Madrid)	1.512	96.37	0.495	0.9963
Road tunnel in M30 (Madrid)	0.5355	210.2	0.4127	0.9932

Fig. 17. Curve fittings of packers creep coefficient.

4.3. Lining global creep coefficient

A formulation of a global creep coefficient (ϕ_G) is proposed in order to consider the long term deformation of packers in the longitudinal behavior of segmental tunnel linings. It is obtained by analyzing the total long term deformation of a set composed by the ring and the corresponding packer (Fig. 18). Its reference to the concrete elastic deformation (Eq. (16)) will allow its direct use on the RCF formulation (Eq. (8)) by replacing the concrete creep coefficient.

$$L_{t,t} - L_{t,t0} = \varepsilon_{c,t0} \cdot L_t \cdot \phi_G(t-t_0) \quad (16)$$

$$L_{t,t} - L_{t,t0} = \varepsilon_{c,t0} \cdot L_c \cdot \phi_c(t-t_0) + \varepsilon_{p,t0} \cdot L_p \cdot \phi_p(t-t_0) \quad (17)$$

The total long term deformation of a ring and packer set is described in Eq. (17). It is combined with Eq. (16) and the strain is expressed as function of the stress, providing Eq. (18). The use of the lateral surface aspect ratio (AR) allows determining the packer stress as a function of concrete stress. The final expression for the global creep coefficient of the lining $\phi_G(t-t_0)$ is presented in Eq. (19).

$$\frac{\sigma_{c,t0}}{E_c} \cdot L_t \cdot \phi_G(t-t_0) = \frac{\sigma_{c,t0}}{E_c} \cdot L_c \cdot \phi_c(t-t_0) + \frac{\sigma_{c,t0} \cdot AR}{E_p} \cdot L_p \cdot \phi_p(t-t_0) \quad (18)$$

$$\phi_G(t-t_0) = \frac{E_c}{L_t} \cdot \left(\frac{L_c \cdot \phi_c(t-t_0)}{E_c} + \frac{L_p \cdot AR \cdot \phi_p(t-t_0)}{E_p} \right) \quad (19)$$

This expression (Eq. (19)) can be simplified by assuming that the total ring width (L_t) is equal to the width of the concrete ring

Fig. 18. Deformation of a segment-packer pack at different stages.

Table 6

Parameters adopted for the global creep coefficient calculation of Model A with Line 9 packer.

Properties/parameters	
Concrete deformation modulus, E_c (N/mm ²)	36,246
Packer deformation modulus, E_p (N/mm ²)	202.1
Concrete ring width, L_c (mm)	1800
Packer thickness, L_p (mm)	2
Total ring width, L_t (mm)	1802
Concrete-packer aspect ratio, AR	2.5

Fig. 19. Global creep coefficient of Model A with and without Line 9 packer.

process that involves a gradual loss of the longitudinal compressive force initially introduced by the TBM.

A formulation to predict the remaining compression stress (RCF approach) is proposed. It is based on the longitudinal creep coefficient of the lining, presenting an excellent agreement with numerical results for mid and long term analyses. Significant reductions of more than 50% of the initial stress are obtained after 10.000 days (27.39 years) for favorable tunnel conditions. Numerical simulation allows the complete determination of the tunnel longitudinal stress evolution, since the construction process until the long term stationary conditions.

Additionally, a formulation to determine the length of the anchorage zones (L_{an}) at the end of the lining or under substantial changes of the TBM force is given. This formulation shows excellent agreement with the numerical results, as long as the ground and its interface with the structure remain in the linear elastic stage.

The longitudinal long term deformation of a lining depends on the response of both the concrete and the packer. An experimental program was specifically designed to determine the creep of different plastic packers, obtaining significant long term strains which can be analytically described similar to concrete creep. The combination of concrete and packers creep in a global creep coefficient of the lining (ϕ_G) shows the significant influence of the packers despite their reduced thickness. The consideration of packers creep produces a further increase of the global lining creep, reducing the long term compression stress that remains in the tunnel lining.

The description of the time-dependent longitudinal response of segmental tunnel linings allows the determination of the mechanisms and the variables that cause the longitudinal force loose in segmental tunnel linings. As a consequence, it is possible to adapt the construction process, materials and designs if the force loose wants to be reduced. The proposed formulations to predict the remaining compression factor and the anchorage lengths define an interesting tool for design engineers in order to predict the longitudinal stress state of the lining and, then, to assess the interaction capacity between adjacent rings and the consequent three dimensional response of the lining.

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(L_c), giving a more compact formulation which allows an easier and faster comprehension of the involved parameters and their effect:

$$\phi_G(t - t_0) = \phi_c(t - t_0) + \frac{L_p \cdot AR \cdot \phi_p(t - t_0) \cdot E_c}{E_p \cdot L_c} \quad (20)$$

4.4. Influence of packer creep in a tunnel lining

In order to assess the influence of the packer creep on the remaining compression of the lining, the global creep coefficient of Model A tunnel (Table 2) with Line 9 packer is initially determined. The concrete creep (CEB-FIP Model Code, 1993), the packer creep (Eq. (15)) and the tunnel properties listed in Table 6 are applied to Eq. (19). The inclusion of the packer creep produces a significant increase of the global longitudinal creep of the lining (Fig. 19). For the long term analysis of the case study (10,000 days), the global creep coefficient increases from 1 to 1.524 (52.4%), producing the reduction of the RCF value from 0.474 to 0.357. This means that disregarding the packers creep in the case study would produce an overestimation of 32.7% of the lining remaining compression.

5. Conclusions

The particular configuration of segmental tunnel linings produces that their structural response depends on the existent longitudinal force. As a consequence, it is necessary to analyze the longitudinal ground-structure interaction in order to determine the longitudinal force remaining on the lining during its service life.

The theoretical and numerical studies carried out demonstrate that segmental tunnel linings present a longitudinal time-dependent behavior which depends on the combination of the particular construction process and the long term deformations of their constitutive materials, i.e. concrete and packers. The longitudinal creep deformations of the lining produce a stress relaxation

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