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Time-dependent behaviour of bore tunnels in soft soil conditions; a numerical study

Comportement dépendant du temps des tunnels creusés en conditions de sol mou; une étude numérique

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ABSTRACT: In recent years a lot of material, experimental and numerical research has been performed on bore tunnels in (Dutch) soft soil conditions. In the current paper a study is described concerning the time-dependent soil behaviour due to creep around a bored tunnel in soft soil by means of a finite element analysis. In the analysis the soft soil behaviour is modelled using the Soft Soil Creep model. The purpose of the research is to evaluate the long term development of lining forces. The creep process causes a continuing compaction of the soil around the relatively stiff tunnel. Regardless of the conditions during the construction of the tunnel, the lining will finally carry more than just the soil weight directly above the tunnel. As a result, the lining forces (bending moments and axial forces) increase with time. These observations were explained and verified by means of stress path analyses in different points in the soil around the tunnel during construction and subsequent creep phases. A comparison of the lining forces is made with Duddeck's solution for the calculation of structural forces in tunnel linings.

RÉSUMÉ: Depuis quelques années beaucoup de recherche expérimentale et numérique a été fait sur des tunnels creusés en sol très mou aux Pays-Bas. Le présent article décrit une étude en éléments finis concernant le comportement dépendant du temps à cause du fluage d'un tunnel creusés en sol mou. Dans cette analyse le comportement à fluage du sol mou est modelé avec le 'Soft Soil Creep' model. Les forces dans le revêtement (les forces axiales, les moments) grandissent avec le temps, et finalement le revêtement supportera plus que le poids propre du sol juste au-dessus du tunnel.

1 INTRODUCTION

Large diameter shield tunnelling is a rather new way of tunnel construction in the Netherlands, whereas in other countries it is more common practice. This is due to the soft soil conditions and the high water table, especially in the western part of the Netherlands, which complicates the tunnel boring process. In recent years, several research programmes have been executed on various aspects of shield tunnels and the tunnel boring process.

One particular aspect has still obtained little attention, namely the influence of long-term creep settlements of the surrounding soil on the forces in the tunnel lining. This aspect was the topic of a preliminary numerical study at the University of Delft. The results of this study are described in this paper.

The main part of the study was performed with the finite element program PLAXIS for geotechnical engineering applications. This program includes a model for time-dependent behaviour of soft soils, named the Soft Soil Creep model, as described by Vermeer & Neher (1999). In Chapter 2 a short review is given of this model. The situation as considered involves a 6m diameter bore tunnel in soft soil, as described in Chapter 3. The soil properties in terms of Soft Soil Creep parameters were selected arbitrary, but they are realistic for soft soil behaviour.

The construction process of a shield tunnel can be divided into different phases. PLAXIS allows for a convenient and realistic simulation of the construction process. The phasing as used in this study is described in Chapter 4.

The results of the 2D finite element analysis in terms of lining forces (bending moments and normal forces) are described in Chapter 5. When evaluating results from finite element analyses it is always interesting to study stress paths of specific points of interest. In the current study, stress paths of points around the tunnel were effectively used to explain the evolution of lining forces.

In the last part of the study, the numerical results were compared with Duddeck's solution for the forces in a tunnel lining. This part is described in Chapter 6, after which Chapter 7 gives the final conclusions of this preliminary study.

2 SOFT SOIL CREEP MODEL

The Soft Soil Creep model is an elastic viscoplastic model, formulated as a relastionship between stress rates and (total) strain rates. Total strain rates are decomposed into elastic strain rates and creep strain rates:

$$\dot{\underline{\varepsilon}} = \dot{\underline{\varepsilon}}^e + \dot{\underline{\varepsilon}}^c = \underline{\underline{D}}^{-1}\dot{\underline{\sigma}}' - \frac{1}{\alpha}\frac{\mu^*}{\tau}\left(\frac{p^{eq}}{p_p}\right)^{\frac{\lambda^*-\kappa^*}{\mu^*}}\frac{\partial p^{eq}}{\partial \underline{\sigma}'}$$

In Eq. (1) $\underline{\underline{D}}^{-1}$ is Hooke's law of isotropic elasticity with a linear stress-dependent stiffness, or in terms of principal stresses and strains:

$$\underline{\underline{D}}^{-1} = \frac{\kappa^*}{3p'(1-2\nu_{ur})}\begin{bmatrix} 1 & -\nu_{ur} & -\nu_{ur} \\ -\nu_{ur} & 1 & -\nu_{ur} \\ -\nu_{ur} & -\nu_{ur} & 1 \end{bmatrix} \quad (2)$$

Furthermore:

$$\alpha = \frac{\partial p^{eq}}{\partial p'} \quad (3)$$

$$p^{eq} = p' + \frac{q^2}{M^2 p'} \quad (4)$$

= equivalent isotropic stress based on the Modified Cam-Clay yield function

The model involves the following parameters:

- κ^* = Modified swelling index (determines reversible strains)
- λ^* = Modified compression index (determines total strains)
- μ^* = Modified creep index (det. time-dependent strains)
- ν_{ur} = Poisson's ratio for unloading / reloading
- τ = Intrinsic time parameter (1 day)

M = Determines the steepness of the creep function (similar as the inclination of the Critical State Line in the Modified Cam-Clay model) and, as a result, determines the σ'_h/σ'_v stress ratio in one dimensional compression.

p_p = Isotropic preconsolidation stress (a state parameter)

The evolution of the preconsolidation stress, p_p, is controlled by the creep strains:

$$p_p = p_p^0 \exp\left(\frac{-\varepsilon_v^c}{\lambda^* - \kappa^*}\right) \tag{5}$$

where

$$\varepsilon_v^c = \int \dot{\varepsilon}_v^c dt \quad \text{and} \quad \dot{\varepsilon}_v^c = \frac{\mu^*}{\tau}\left(\frac{p^{eq}}{p_p}\right)^{\frac{\lambda^*-\kappa^*}{\mu^*}} \tag{6}$$

As a result, the creep strain rate and the evolution rate of the preconsolidation stress depends on the inverse of the overconsolidation ratio, $OCR^{-1} = p^{eq} / p_p$. If the stress state remains unchanged, the creep process continues and the preconsolidation stress keeps on increasing, but at a decreasing rate. For more details on the Soft Soil Creep model see Vermeer & Neher (1999).

3 FINITE ELEMENT MODEL

In order to numerically evaluate the influence of creep in a soft soil layer on the lining forces in a shield tunnel, an imaginary situation of a 20m deep soft clay layer was considered with a 6m diameter tunnel at a depth of 15m below ground surface. The behaviour of the clay in the finite element model was simulated with the Soft Soil Creep model. The soil behaviour was taken as drained in order to avoid intermediate consolidation calculations, just to keep the phasing simple. An overview of the selected model parameters is given in Table 1.

A 2D finite element mesh composed of 220 high-order cubic strain elements was used to model one symmetric half of the situation (Fig. 1). The tunnel boring machine (TBM) and tunnel

Table 1. Model parameters of clay layer (Soft Soil Creep model).

Parameter	Symbol	Value	Unit
Saturated unit soil weight	γ_{sat}	18.0	kN/m³
Modified compression index	λ^*	0.10	-
Modified swelling index	κ^*	0.02	-
Modified creep index	μ^*	0.005	-
Poisson's ratio	ν	0.15	-
Stress ratio in 1D compression	K_0^{nc}	0.70	-
Inclination of Critical State Line	M	1.03	-
Effective friction angle	φ'	26.0	°
Effective cohesion	c'	1.0	kN/m²

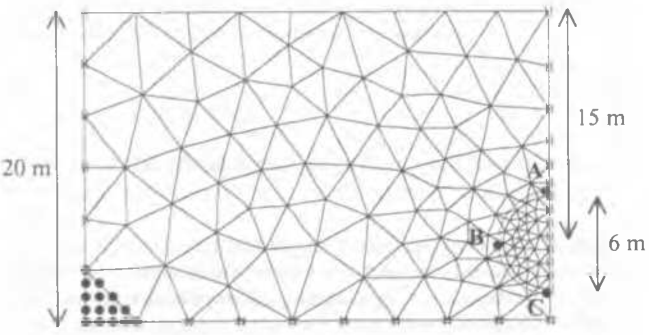

Figure 1. Finite element mesh and dimensions of the situation.

Table 2. TBM and lining properties.

Parameter	Symbol	Value	Unit
Axial stiffness	EA	$9 \cdot 10^6$	kN/m
Flexural rigidity	EI	$30 \cdot 10^3$	kNm²/m
Equivalent thickness	d	0.2	m
Distributed weight	w	5.0	kN/m²
Poisson's ratio	ν	0.1	-

lining were modelled by means of compatible beam elements. The properties of the TBM and lining were taken equal, as listed in Table 2. Around the tunnel, interfaces were included to model the soil-structure interaction. A reduced interface strength of 0.7 times the soil strength was used in these elements.

Pore pressures were generated on the basis of a phreatic level, which was set at the ground surface. The initial stress state was generated from the effective soil weight and a K_0-value of 0.7.

Using the Soft Soil Creep model, the initial OCR does not just determine the preconsolidation stress, p_p, but it also determines the initial creep strain rate, $\dot{\varepsilon}_v$, as can be seen from Eq. (6). Hence, in situations where the stress state is dominated by initial stresses, care has to be taken with the proper selection of the initial OCR. Considering normally consolidated soil, one would normally take OCR=1.0, but this will give very high initial creep strains, resulting in 100 mm of settlement in a few days (without additional loading). In reality, soils that are known as 'normally consolidated' may show realistic OCR-values around 1.5. In the current situation the initial OCR was taken 1.4.

4 SIMULATION OF TUNNEL BORING PROCESS

In a 2D finite element simulation of a shield tunnel, the tunnel boring process can be divided into four sequential calculation phases. Starting from the initial stress state, the TBM is installed, the soil inside the tunnel is excavated and the water pressure is removed. This all can be simulated in a single calculation phase. In the second phase shrinkage (contraction) is applied to the tunnel contour to simulate the radial reduction (conicity) of the TBM towards its tail. In the current situation a contraction of 0.5% of the cross section area was applied. In the third phase the TBM is de-activated and a pressure is applied on the soil around the tunnel to simulate the grout injected at the TBM tail. In the fourth phase the grout is considered to be hardened and the final tunnel lining is activated. One could consider undrained soil behaviour and consolidation after each of the above phases, but this has not been done in this study. Moreover, the consolidation and hardening of the grout layer, resulting in a final volume loss, has also not been considered here.

In all four phases an arbitrary time interval (duration) of 1 day was specified to allow for irreversible deformations. After the execution of the four basic calculation phases, a subsequent period of 1000 days of creep was added to each individual phase in order to study how the respective stress state and force distribution is influenced by creep.

A summary of all calculation phases is listed in Table 3.

After the analysis, stress paths were drawn for the stress points just above (A), just left (B) and just below the tunnel (C), as in-

Table 3. Calculation phases and sequence.

Phase No & Identification	Start phase	Activate	De-activate
0 - Initial phase	-	Gravity	
1 - Excavation & Dewater.	0	TBM	Soil & water inside
2 - Contraction	1	Contraction	
3 - Grout pressure	2	Pressure inside	TBM
4 - Final lining	3	Lining	Pressure inside
5 - Creep after Initial phase	0	Time, 1000 d.	
6 - Creep after Excavation	1	Time, 1000 d.	
7 - Creep after Contraction	2	Time, 1000 d.	
8 - Creep after Grouting	3	Time, 1000 d.	
9 - Creep after Lining	4	Time, 1000 d.	

dicated in Figure 1. It is the authors' opinion that stress paths are a very useful instrument to analyse computational results.

5 COMPUTATIONAL RESULTS

Figure 2 gives an overview of the development of the structural forces at the end of different calculation phases.

From the results in Figure 2 it can be seen that the bending moments are significantly increased (generally more than a factor 2) by the creep process. Also the axial forces are increased, but not so significantly (roughly 10%). The increased structural forces are caused by the fact that the tunnel acts as a stiff object in the soil whilst the surrounding soil settles due to creep. As a result, the soil will 'hang on' to the tunnel, causing more soil weight to be attracted to the tunnel, which results in larger structural forces. Indeed, the stress paths of points A and C generally show increasing vertical stresses in the creep phases, whereas point B shows a decreasing vertical stress and a more or less stable horizontal stress in these phases (Fig. 3).

When comparing Figure 2b with Figure 2a it can be seen that the bending moments slightly increase during contraction. This corresponds with the observations of Bakker (2000), considering the fact that a relatively small contraction is applied. The axial forces, however, slightly reduce, which can be explained by arching in the surrounding soil. This phenomenon can also be explained from the stress paths.

From Figure 2c it can be seen that just after the grouting process the direction of bending moments are opposite compared to the previous situations. This can be explained from the model by the vertical ovalisation (0) that occurs due to the relatively high horizontal pressure in the soil, which is later carried by the lining after dissipation of the grout pressure. However, it is questionable whether this situation is realistic or that it is just a resul from the modelling sequence. Nevertheless, after creep the moments return to a realistic distribution.

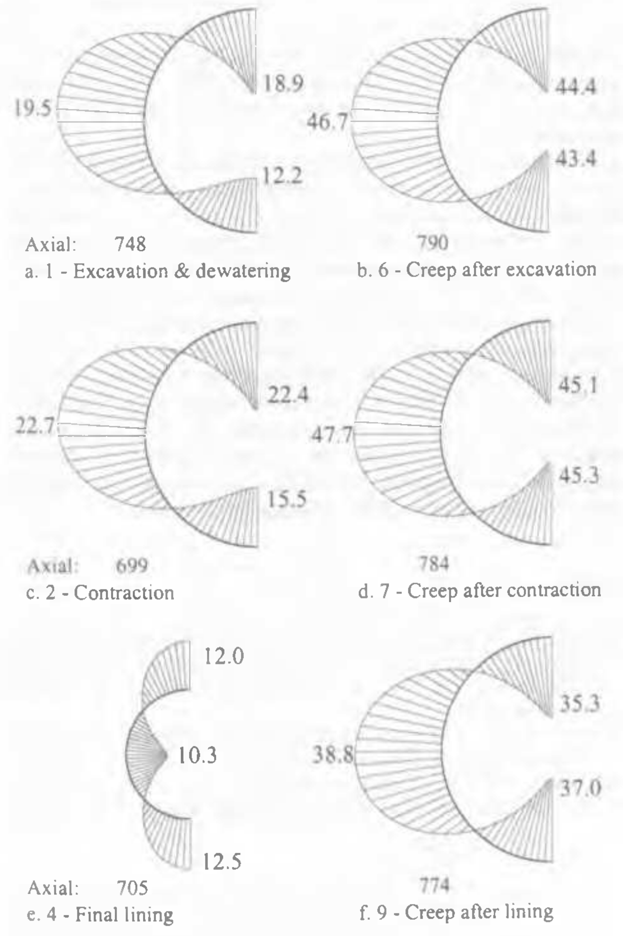

Figure 2. Overview of bending moments from different calculation phases (values in kNm/m). Indication of axial forces in kN/m.

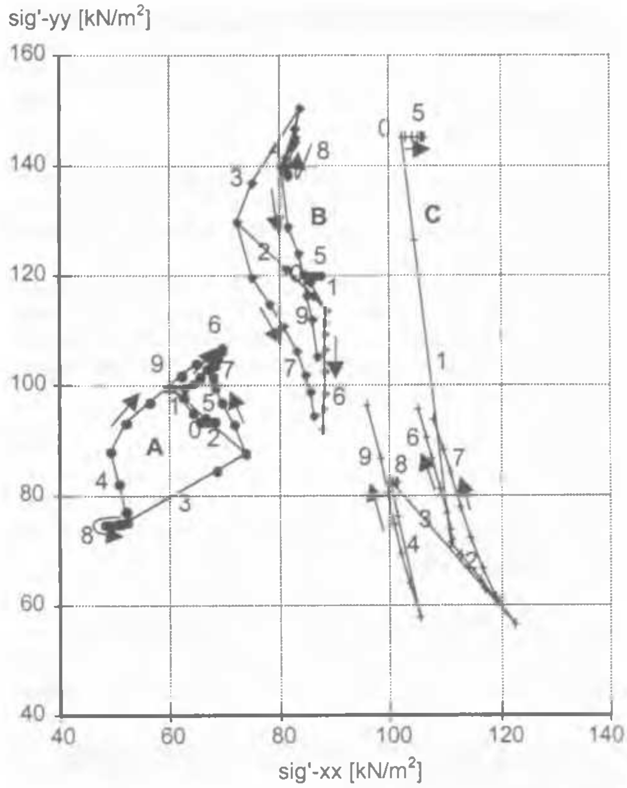

Figure 3. Effective stress paths of points A (above), B (left) and C (below the tunnel). The numbers refer to the calculation phases. The arrows indicate the stress development during the creep phases.

6 EVALUATION

In order to evaluate the lining forces as calculated with the finite element model, a comparison is made with the empirical model of Duddeck (1991). The Duddeck's model evaluates the tangential Normal forces and Bending moments as a function of a flexibility parameter α, where

$$\alpha = \frac{E_g R^3}{EI} \tag{7}$$

and

E_g = Young's modulus of the soil
R = Radius of the tunnel; $D = 2\,R$
EI = Flexural rigidity of the lining

Duddeck gives the Normal forces and the Bending moments as a function of this flexibility parameter. Here the focus is put on the analysis of the bending moments, since the largest influence is observed for these quantities.

An approximate analytical relation for the bending moment given in graphs by Duddeck is found using the relation:

$$M = -C_m \frac{(\sigma_v - \sigma_h)}{4} r^2 \cos(2\theta) \tag{8}$$

where C_m is a correction factor on the basic situation in which it is considered that the initial soil stresses act directly on the tunnel lining. Hence, Equation (8) is derived for the situation of full bonding in the soil-structure interaction, i.e. it is assumed here that shear stresses will develop corroborrating with the initial soil stresses. The coefficient may be approximated to be in the order of:

$$C_m = \frac{4}{4 + 0{,}342\,\alpha} \tag{9}$$

This empirical model can be used to compare the bending moments with the results of the Soft Soil model, for the situation afer phase 1 (Table 3), as it is not possible to take further phases into account in this simple approach.

To make this comparison we have to derive Young's modulus of the Clay from the parameters given in Table 1. For a linear elastic calculation (and here for this first order approximation, with very small strains around the tunnel as a first assumption) we consider elasticity, in which case

$$E_g = 3(1 - 2\nu)K \tag{10}$$

where

K = Bulk modulus
ν = Poisson's ratio

In general, for the Soft Soil Creep model we may assume that

$$K^{ref} = \frac{p^{ref}}{\kappa^*} \tag{11}$$

For a reference stress p^{ref} = 100 kN/m^2, using the parameters in Table 1, this would give

$$K^{ref} = \frac{100}{0.02} = 5000\ kN/m^2$$

For the tunnel indicated in Fig. 1 the effective isotropic stress p at a depth of 15m might be approximated as

$$p = h\,(\frac{1+K_0}{2})(\gamma_n - \gamma_w) =$$

$$= 15\,(\frac{1+0.7}{2})(18-10) = 102\ kN/m^2$$

Which means that the average bulk modulus of the soil around the tunnel may be approximated as:

$$K \cong \frac{102}{100} * 5000 = 5100\ kN/m^2$$

In Duddeck's model, Young's modulus is used, which can be found according to

$$E = 3(1-2\nu)K = 3(1-2*0.15)5100 \cong 10700\ kN/m^2$$

Before we can proceed we approximate the soil stress differences above and at the sides of the tunnel for the initial situation, calculating the initial stresses at the level of the tunnel axis (implicitly accounting for the weight of the tunnel lining) as:

$$\sigma_v = h\gamma_w + h(\gamma_n - \gamma_w) = 15*10 + 15*8 = 270\ kN/m^2$$

and

$$\sigma_h = h\gamma_w + K_0 h(\gamma_n - \gamma_w) = 15*10 + 0.7*15*8 = 234\ kN/m^2$$

The flexibility index α and the correction factor C_m are subsequently calculated as:

$$\alpha = \frac{10700 * 3^3}{30.000.000 * \frac{1}{12} * 0.2^3} = 14.5$$

$$C_m = \frac{4}{4 + 0.342 * 14.5} = 0.45$$

With this result the bending moments after construction, but before contraction may be approximated as:

$$M_{max} = 0.44\,\frac{270-234}{4}\,3^2 = 36\ kNm/m$$

In comparison with the finite element results with the initial stresses, it seems that a further reduction of the bending moments occurs. Maybe the fact that for the numerical analysis a reduction factor of 0.7 for the soil friction interaction was taken further reduces the bending moments. For a situation with and without bond a factor of 2/3 is found between bending moments. This would explain a part of the difference observed here.

If, on the other hand, we compare this result with the results after creep, the order of magnitude has a reasonable correspondance. One might even argue that the empirical results are not allways safe if soil creep has to be considered underestimating the bending moments by a factor of 0.75, i.e. by 25%.

7 CONCLUSIONS

This paper describes a numerical study after the effects of time-dependent (creep) settlements in soft soils on the structural forces in a tunnel lining.

The creep behaviour of the soft soil was modelled by means of the Soft Soil Creep model. The initial situation involves the input of the initial overconsolidation ratio, OCR, in order to set the initial preconsolidation stress. In the Soft Soil Creep model, the initial OCR determines the initial creep strain rate in situations where the stress state is dominated by the initial stresses rather than by loading. Hence, care must be taken in these cases with the selection of the initial OCR-value, especially for normally consolidated soils. Values in the order of 1.5 are more likely to be entered, which are actually not unusual for 'normally consolidated soils'.

The numerical results indicate that bending moments may increase more than a factor 2 compared to short-term values, whereas normal forces increase roughtly by 10%. The results can well be explained by stress path analyses of points around the tunnel. It is the authors' opinion that stress paths are, in general, a very useful instrument to analyse computational results.

The order of magnitude of the bending moments in the tunnel lining was compared with Duddeck's solution. This solution gives bending moments that are higher than the values obtained for the first phase (initial excavation, which is in principle a comparable situation) and they are lower than the values as obtained for the situation after creep. Nevertheless, the order of magnitude is comparable. The precise value may be influenced by the roughness factor in the soil-structure interaction.

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