Structural design of linings for bored tunnels in soft ground

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Abstract: The increase of Bored tunnels in the Netherlands has raised the question how to design the tunnel structure in an efficient way. As a large part of the cost of a bore tunnel is related to the cost of the lining, it is important to design the lining in a cost-effective way. In the Netherlands it is customary to base the structural design of tunnel linings, on the use of models, validated models. Due to the increased flexibility in modelling quite often in engineering practice these models are numerical. Numerical models however are sometimes difficult to interpret, this contrary to the clarity of most empirical and analytical models. In this paper a number of, these simple models are used to evaluate different design aspects of the tunnel lining. These models are discussed in a comparison with measurements and results of more elaborate numerical analysis. The purpose of which is to show that tunnel lining behaviour can be understood, with relatively simple models. The advantage of simple models is that these can be used for a 1st order covering of the dominant mechanisms and for preliminary design. Empirical relations and analytical solutions generally give a good insight in the relevant issues. However to take into account the complexity of geometry, geology and construction method, in practice for a final design numerical models are necessary. Finally, a number of special issues, such as 3D effects, creep and longitudinal effects in the tunnel tube, and in addition to that, the effects of fire as well as some aspects of durability, are discussed.

Key words: underground, construction, tunnels, soft ground models

1 Introduction

In the Netherlands up to 1994 little attention had been given to the development of a philosophy for the design of bored tunnels. Since then, however, things have changed; in 1993, two pilot projects for bored tunnelling were started, the Second Heinenoord Tunnel for road transport and the Botlek Rail Tunnel for rail transport. An extensive monitoring programme accompanied both projects. At the time of writing this paper, the construction of the Second Heinenoord tunnel has finished and the tunnel is opened for the public whereas the construction of the Botlek rail tunnel has finished too, although that during the finishing of this paper, the tunnel itself had not been taken in operation. After those projects a number of other projects with bored tunnelling have been taken under design and under construction such as the Sophia tunnel under the Noord, the Westerschelde tunnel. The North South line for Metro in Amsterdam and the tunnel under the Pannerdensch Canal are under design.

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When in 1994 the design of the Second Heinenoord tunnel had to be undertaken, models and a design philosophy for the tunnel structure had to be adopted from abroad. Considering the soft soil conditions in the Netherlands with this a risk was taken because these, mainly empirical, models where validated for much stiffer soil conditions. For the lining the subgrade reaction model of Duddeck (1980) was adopted, though later on in addition to that, specific Finite element applications were developed and applied for evaluation of the different design issues.
In this paper the structural design of tunnel linings will be discussed, with the focus on lining stresses due to soil loading. It has to be understood, however that other loading conditions such as during transport of tunnel lining segments, or due to jack-forces during tunnel construction are not less important; these loadings have to be evaluated in addition to the soil loading.
The choice for a focus on soil loading is because the soil loading is everlasting during operation of the tunnel. The soil loading has to be supported with sufficient safety during the entire tunnel's lifetime, whereas the other loading conditions are mainly important for a short period during construction. Therefore the challenge is to find other means to support critical loading conditions during construction, as the soil loading will always be a "design driver".
The general outline of this paper is that it gives an overview of different issues that have to be considered in tunnel lining design. Where possible, both empirical and numerical approaches and sometimes, analytical approaches are discussed. The reason for this approach is that empirical and analytical methods are more easily handled in an evaluation of the relevant mechanisms in a preliminary design.
First, in the next section the influence of soft ground conditions on the tunnel lining design will be discussed.
In section 3, some of the principles for tunnel design are mentioned as well as a shortlist of the demands for tunnel lining design according to the ITA; the International Tunnelling Association. Subsequently some considerations with respect to plastic design and interaction will be discussed.
In section 4, which is the centre part of the paper, the cross sectional lining stresses due to soil loading will be discussed, including a back-analysis of measurements taken from the construction of the Second Heinenoord Tunnel.
In section 5, longitudinal stresses, Eigen stresses, due to the construction process, i.e. the TBM loading will be discussed.
Subsequently in section 6, some considerations are given with respect to thrust jacking forces and local stresses are given and further in section 7, and 8 some other exceptional loading situations, such as fire and durability aspects are discussed.
In section 9 concluding remarks are given.

2 Soft ground soil conditions

The top-section of the stratification of the western part of the Netherlands consists of soft ground; i.e. peat and soft to very soft clay. These deposits, formed during the Holocene (alluvial) period, are overlaying a thick layer of coarse Pleistocene sand and gravel formed during the glacial periods. At that time the water level of the North see was relatively low. During the interglacial times of high sea level, the West coast was below sea level and marine clays were deposited. About 5000 years ago, at the

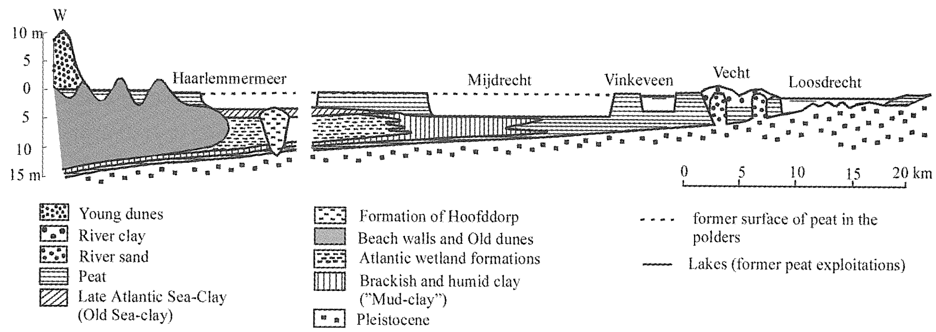

Figure 1. Geological cross-section of West-Netherlands, over Mijdrecht and near Delft, after Buisman (1978)

beginning of the Holocene period a coastal barrier system formed as dune formation began. In the areas behind the coastal barrier, out of reach of the aggressive sea, thick peat layers were formed. Locally, these layers were eroded by the sea again and were partly replaced by marine clay sediments. Because of the low stiffness and strength of the upper Holocene layers, the construction of bored tunnels in the Netherlands was for long regarded as too difficult. Since then the insight has changed, nowadays we know that also for the Dutch soil conditions tunnel boring is a feasible construction method. This change to feasible is not all that surprising if we realise that for the most part the tunnels that have been taken under design and under construction are located at a depth which places them for the larger part in the stiffer Pleistocene deposits.

2.1 *Typical problems of Bored tunnelling in soft soil*

According to Peck (1969), one of the main problems related to bore tunnelling is: "*Keeping the tunnel safe and operational during us*".

Although keeping the tunnel safe and operational will include much more than structural integrity of the structure, the latter is related to this purpose. The safety of a tunnel is related to the functions, which the tunnel has to perform, the tunnel's purpose. If the function of a tunnel becomes outdated however, still the structural safety of the structure is a concern, because of the possible effects on the soil surface if a tunnel structure would collapses.

Structural integrity is related to a balance between loads and structural reactions, and besides that the stability of this balance. The easiest approach for an evaluation of stability would be to take the soil loading and calculate the stresses in the structure, and evaluate the bearing capacity of the structure under these loads.

A drawback to this approach is, that soil loading of a tunnel is not an action, but a reaction, as will be discussed in section 3. The active soil stresses on a tunnel lining are partially determined by the initial stresses in the soil, the stresses acting before any tunnelling activity has taken place, and partially by the deformations which are induced by the tunnel construction and the deformations of the tunnel lining. Therefore the analysis of tunnel deformations and stresses has to include the surrounding soil, in order to evaluate the stability in an objective way.

3 Structural design of linings for bored tunnels

3.1 Basic principles for the structural design;

Among the basic principles for structural design are the demands that the structure has strength and stiffness and is stable during all the stages of construction and during operation and use. Besides that there are functional requirements that have to be met, such as the demand of water tightness of the lining. Functional requirements with respect to the operational use as a part of a transport system are obvious but disregarded here, because these lie beyond the scope of this paper.

Strength, stiffness and stability. After that the TBM has excavated sufficient space for the erection of a new tunnel-ring, a new lining ring is mounted at the end of the tube constructed earlier, and loaded during further excavation of the Tunnel boring machine. The internal forces developing in the ring have to balance the loading on the ring. To begin with, the internal stresses have to meet the requirements related to an evaluation of the ultimate limit states of a tunnel ring. This is a structural requirement with respect to the internal strength of a tunnel ring. As a starting point for the Ultimate Limit State it is assumed that each ring can make a balance of internal forces and soil loading independently; i.e. without additional support of adjacent rings.

Buoyancy. Besides the internal stability of a tunnel ring, the tube has to make balance with soil loading, and as the weight of the lining and the tunnel installation is less then the soil (including the groundwater) that is removed, the structure is initially not in vertical equilibrium. In order to gain equilibrium a slight upward movement, initiating stress redistribution above, and stress relief under the tunnel will occur, until vertical equilibrium is reached. The effects of this are related to the beam action of a tunnel, see also section 5.

Axial equilibrium; At the stage of entering the receiving shaft, after completion of a tunnel track, the axial stresses at the front, will diminish because the face pressure might be released when entering the free air. If no adequate measures would have been undertaken, the axial stresses in the tube may reduce significantly. The axial stresses in the tube however, contribute to the capacity of the tube to sustain shear forces, and therefore a procedure has to be developed to secure a sufficient shear force capacity between the rings.

3.2 Design loads for tunnel linings

According to the ITA Guidelines for the design of Shield Tunnel lining, (2000), the following design loads have to be evaluated:

- Geo-static Loads; to evaluate load effects on lining segments and ground.
- Thrust jacking loads; to evaluate load effects distributed on segments by distribution-pads.
- Trailer and other service loads; Including main bearing loads, divided by the number of wheels.
- Secondary grouting loads; extending regular grout pressure.
- Dead load, storage and assembly loads; bending moment influence.

Here, in this paper, not all these loads will be addressed. The main focus in this paper will be put on geo-static loads, secondary to grouting loads and dead loads, the latter for as far as it is implicitly taken into account in a two dimensional analysis of the load situations on a tunnel ring.

This means that the discussion is limited. Especially the effects of Thrust jacking loads and assembly loads are only taken in consideration sideways. Though it is recognized that based on the back analysis to be discussed in section 4 it was found that the internal forces due to thrust jacking and assembly might be in the same order as those due to geo-static loads. In section 6 we will come back on this issue.

3.3 Modes of deformation and failure for a Tunnel Lining under soil loading

For a lined tunnel, which is safe with respect to buoyancy and breaking up (i.e. if the tunnel is sufficiently deep) the deformation mode to be expected is horizontal ovalisation. This is associated with a minor stress relieve above the tunnel and a greater one below. Due to compression of the soil at the sides of the tunnel, a lateral stress increase will be observed, which tends to diminish the difference between the vertical stresses and horizontal stresses. For a very flexible tunnel in stiff soil, theoretically the lining will deform up to the point that the bending moment vanishes and that the only remaining stresses are due to hoop compression. This process is illustrated in Fig. 2, where feasible plastic zones around the tunnel are indicated. Due to this process, bending moments in the lining would ultimately disappear, and the only mode of failure left would be compressive failure of the lining.

However, before we decide to design the lining to only bear the compressive forces, we have to consider that additional bending moments will develop due to large deformation effects. We have to consider that, the advantage of taking into account flexibility of the lining and calculating a decrease in bending moments due to that, might be overtaken by an increase in bending moments due to geometric effects.

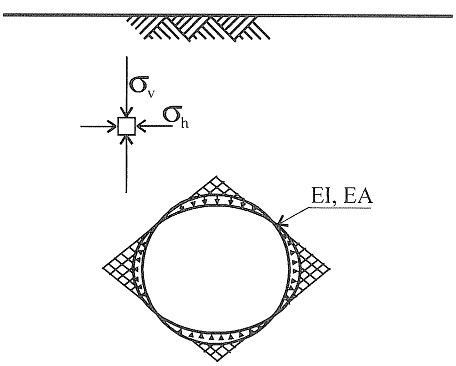

Figure 2. Compressive failure of a tunnel lining, in combination with soil failure around the tunnel. The feasible plastic zones are indicated.

To evaluate this mechanism, Bakker (2000), analysed the ultimate state of a tunnel ring with vertical and horizontal loading, assuming a mechanism with 4 double symmetrically placed plastic hinges in the lining. A relation between the necessary plastic moment and the deformation needed to derive equilibrium was found, according to:

$$M_{pl} = \frac{(1-K_o)}{4}\,\sigma'_v\, r^2 + \frac{(\sigma_v + \sigma_h)}{2}\, r\, u - \frac{E_g\, u\, r}{4} \qquad (1)$$

Where

r	=	tunnel radius
u	=	deformation; half the change in diameter at the tunnel axis
E_g	=	Young's modulus of the soil
K_0	=	Coefficient of neutral soil stress
$\sigma_{v,h}$	=	Soil stress

The first part of equation 1 is related to bending moments caused by the initial stresses in the soil, (i.e. the distortional part). The second part is related to the compressive part of the soil loading, (i.e. only contributing if the tunnel deforms), this part can be recognised as the contribution of second order deformations. The third part is related to the soil-structure response. Due to the compression of the soil at both sides of the tunnel, the difference between the magnitude of the soil stresses above and at the sides of the tunnel reduces.
To get a better understanding of what this means, an example has been worked out for a tunnel with a radius of $r = 4$ m, a depth ratio $h/r = 4$, a coefficient of neutral horizontal soil stress $K_0 = 0.5$, a wet soil weight of $\gamma_w = 20\ kN/m^3$ and a groundwater table near the soil surface (see Fig. 3). These parameters more or less refer to the situation of the Second Heinenoord tunnel.

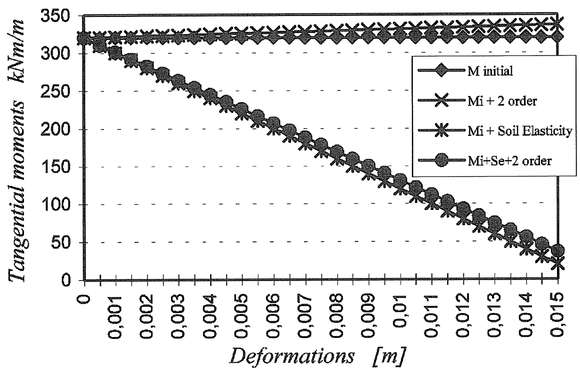

Figure 3. Bending moments as a function of deformations for a tunnel with a diameter of 8 m, with a depth ratio of h/r = 4, for a soil stiffness of E = 20000 kPa, for a $K_0 = 0.5$.

As one can observe the bending moment strongly reduces for a minor deformation of the lining. Based on this analysis, it must be doubted whether the choice for a high partial safety factor on the capacity for bending moments is a good investment. Maybe it would be more beneficial to assure that the lining has a high flexibility and that it can sustain the hoop forces.
Thus far it was assumed that the lining is homogeneous, or behaves as one. If the lining is composed of segments however, also local failure due to shearing has to be considered. In order to maintain a stabile configuration, the shear force in radial direction in the axial joint may not exceed the shear resistance.
Here, to evaluate this mechanism, a friction type of relation for shear capacity is proposed according to:

$$Q \leq N f \tag{2}$$

Further developing this relation, see Bakker (2000), assuming the analytical solutions given in section 4, equations 5 to 7, assuming the initial soil stresses based on the assumption that the effective horizontal stress is determined by K_0 and disregarding the stress reduction due to soil structure interaction we find:

$$\left(\frac{1+K_0}{2}\sigma_v' + u\right) f \geq \frac{1-K_0}{2}\sigma_v' \tag{3}$$

Where u is the groundwater pressure.
If u is zero, (i.e. for the situation without groundwater), the criterion becomes:

$$f \geq \frac{1-K_0}{1+K_0} \tag{4}$$

If the material is a concrete-concrete friction, we might expect a friction coefficient of $f \approx 0{,}5$, this means that we need to have $K_0 \geq 1/3$ for internal stability with respect to shearing. For the Dutch situation this criterion is normally met. However, equation 2 and 4 are based on very crude assumptions, disregarding any stress redistribution due to interaction between the lining and the soil. In practice, therefore it must be advised to evaluate shearing with equation 2, in combination with a derivation of the normal force and the shear force based on a more elaborate analysis.
Besides shearing at the axial joints also shearing at the ring joints has to be considered because this may damage the waterproof sealing between the tunnel segments.

3.4 The loading on the tunnel lining; as an interaction

From the section above we learned that the soil loading on a lining is not straightforward; in the sense that a set of explicit relations to calculate the soil stresses on a lining is not available. On the contrary, we have learned that the final loading on a tunnel also depends on the deformation of the structure.
Therefore, for a better understanding we need to have models that account for the interaction between soil and structure. In the next section the attention will be shifted to such models.

4 Stresses in the lining due to soil loading

Soil loading is not the only loading to be evaluated for the dimensioning of a lining. This section, however, is limited to the evaluation of soil loading, because failure to support the soil leads to an ultimate limit state with major consequences.
The analysis of the bearing capacity of a lining depends on the loading actions; the soil stresses which have an interaction with the stresses in the lining, in order to make equilibrium. If these soil stresses would not depend on the deformations of the lining; if there would not be any interaction, it would be relatively easy to calculate the stresses in the lining, and henceforth the bearing capacity of a lining.

4.1 Models for the analysis of stresses in the tunnel lining

One of the characteristics of the soil loading on a lining is that it depends on the interaction between the deformation of the lining and the deformation of the soil. For a good understanding of this phenomenon it is advantageous to know the analytical relations between loads and stresses on a circular tunnel lining. Here described in terms of the intermediate vertical soil stress and the horizontal soil stress at tunnel axis level.

With respect to the analytical solutions applied here, these implicitly assume that the vertical deformation needed to derive vertical equilibrium, (triggered by the buoyancy force), gives an equal stress increase at the crown of the tunnel as a stress relieve at the invert of the tunnel. Although it is known that in practice the stress relieve at the invert might be higher than at the crown. We assume this assumption to be conservative.

Based on analytical relations for loading of rings, derived by Bouma (1993), assuming both the radial stresses of the soil as well as the tangential stresses the following relations were found for the main Beam forces.

Normal force:

$$N = -\frac{(\sigma_v + \sigma_h)}{2} r + \frac{(\sigma_v - \sigma_h)}{2} r \cos(2\theta) \tag{5}$$

Shear force:

$$Q = \frac{\sigma_v - \sigma_h}{2} r \ \sin(2\theta) \tag{6}$$

Bending moment:

$$M = -\frac{(\sigma_v - \sigma_h)}{4} r^2 \cos(2\theta) \tag{7}$$

Where σ_v = vertical soil stress, σ_h = horizontal soil stress and r = the radius of the tunnel.

Finally the radial deformation of the lining with respect to its centre can be found according to:

$$w = -\left(\frac{\sigma_v + \sigma_h}{2}\right)\frac{r^2}{EA} - \left(\frac{\sigma_v - \sigma_h}{12}\right)\frac{r^4}{EI}\cos(2\theta) \tag{8}$$

These equations assume that the soil stresses on the lining are known, and have a simple form. For that situation a first indication of the lining stresses can be derived quite easily.

The advantage thus is that these equations give a very quick insight into the main mode of deformation and of stress development. Secondly if one has little information, but might guess what the stress situation after construction might be, or if one has measurements related to this issue, a preliminary design of the lining and reinforcement is possible.

In practice however, the soil stresses relate to the relative stiffness ratio between the tunnel and the soil. To overcome this shortcoming, subgrade reaction models have to be used, to relate initial stresses to final loading, which issue will be elaborated in the next section.

4.2 *Subgrade reaction type models*

When the tunnel tube liner and the soil interact, a reaction has to be accounted for. In that case an analytic solution would be difficult to derive. Numerical solutions such as those derived by Duddeck (1980, 1984 & 1991) are more appropriate then. These solutions can be shown with graphs such as given in Fig. 4.

In order to clarify these solutions, here an evaluation will be given of the influence of the soil stiffness on the circumferential stresses in the lining.

For the situation that $\sigma_h = K_0\sigma_v$ and $K_0 < 1$, for a tunnel which is situated not too shallow, disregarding any pre-stressing due to tail void by grouting, the tunnel will take an oval shape in the horizontal direction, giving an increase in horizontal soil stresses and a reduction in the stress difference.

In the limiting case of a very weak soil the subgrade reaction model fits in the results of the analytical model given by Bouma. The solutions can be understood in such a way that the effect of the relative stiffness on the bending moments and normal forces in a lining, can be worked out to give a reduction factor on the solution when initial stresses are applied in combination with the analytical models by Bouma, presented in the former section. This reduction is a function of two elasticity parameters:

$$\alpha = \frac{E_g r^3}{E_b I_d} = \frac{E_g D^3}{8 E_b I_d} \qquad (9) \qquad \text{and} \qquad \beta = \frac{E_g r}{EA} = \frac{E_g D}{2Ed} \qquad (10)$$

where

E_g	=	The Young's modulus of the soil
D	=	the diameter of a tunnel; $D = 2r$
E_b	=	The Young's modulus of the concrete wall
I_b	=	the moment of inertia of the cross-section of the tunnel wall; $l_b = {}^1/_{12} d^3$
d	=	the thickness of the concrete liner

The evaluation is done for horizontal stresses given by $\sigma_h = K_0\,\sigma_v$ and using an adapted notation,

where m is introduced according to $m = \frac{(1 - K_0)}{4}$. With this a relationship between circumferential

bending moment and soil loading according is found according to $M_{max} = m\sigma_v r^2$ (which can be compared with equation 7).

The solution is displayed in Fig. 4 for $K_0 = 0.5$, and for a homogeneous soil. Originally Duddeck indicated a region of application between $5 < \alpha < 200$. Which means somewhere in the midst of the declining branch for the bending moment in the right-hand picture.

In comparison to a stress analysis applying the initial in-situ stresses based on K_0, this would mean a reduction in the bending moment between 20 and 80 %.

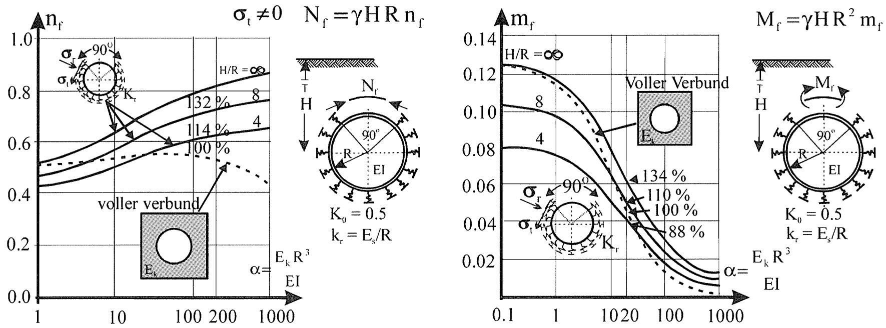

Figure 4 Results of the Duddeck model for tunnel analysis as a function of the subgrade reaction ratio.

For engineering practice Vrijling further simplified the relations derived by Duddeck, into a pair of empirical relations, applicable for a pre-design.
For the circumferential normal force, we find:

$$N = -C_0^N \frac{(\sigma_v + \sigma_h)}{2} r + C_2^N \frac{(\sigma_v - \sigma_h)}{2} r \cos(2\theta) \tag{11}$$

And for the circumferential bending moment that operates in the tangential cross-sections we find:

$$M = -C^M \frac{(\sigma_v - \sigma_h)}{4} r^2 \cos(2\theta) \tag{12}$$

where:

$$C_0^N = \frac{2}{2+1.54\beta} \quad (13), \qquad C_2^N = \frac{2(1+0.064\alpha)}{2+0.171\alpha} \quad (14), \qquad C^M = \frac{4}{4+0.342\alpha} \quad (15)$$

The advantage of the subgrade reaction approach is, that it includes an important part of the soil structure interaction in some simple equations, which can be applied in a pre-design stage for the tunnel. However, the limitations on the other hand are, that this approach disregards soil layering, staged construction effects and volume loss. The latter especially should not be overlooked, as it may have a large effect both on Tangential normal force and on the bending moments.
With volume loss V_s here is meant, the volume of excavated space (per unit of tunnel length), divided by the total of excavated material, that is not filled up, either by the tunnel; i.e. the outside of the lining or the tail void grout material. Beware, that though V_s is a volume, in practice it is sometimes customary to discuss the volume loss as if it is a ratio; the ratio with respect to the total of excavated material (per unit of tunnel length). The volume loss V_s causes a relieve of radial stresses in the surrounding soil, also affecting the loading on the tunnel lining.
In order to calculate this effect mentioned above, numerical analysis, such as Finite element analysis has to be applied.

4.3 *Finite element analysis*

One of the features enabled by Finite Element analysis is the ability to model the staged construction of a tunnel structure. According to Fujita (1993), see Fig. 5, a number of staged construction steps are recognized. As mentioned before, one of the important aspects of the staged construction process is the volume loss V_s. The volume loss is partially dependent on the conic shape of the TBM and partially on the tail loss of the Tunnel Bore Machine. The total volume loss per unit of length is usually in the order of 0.5 to 1.5 % of the total amount of excavated material (Cross section times unit of length). Though this volume loss is more or less compensated by grouting of the tail void, consolidation and creep effects contribute to an additional volume loss, therefore in the end a volume loss of 0.5 to 1.0 % cannot easily be avoided. Volume losses cause surface settlements, and a relieve of soil stress. The latter is important for the analysis of stresses in the lining. For the analysis of this effect a continuum approach is proposed; i.e. Finite Element analysis.

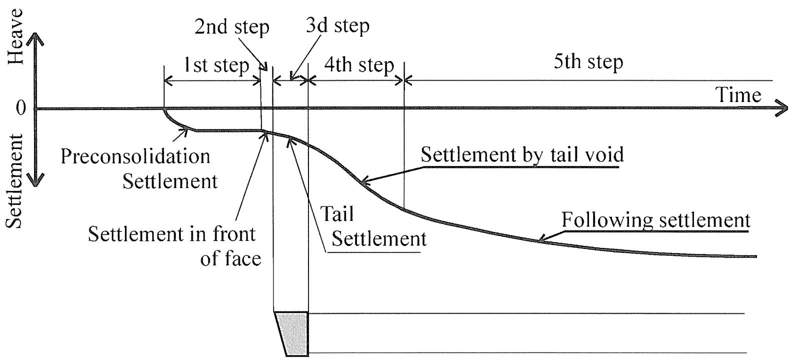

Figure 5. Five steps in ground displacements (Fujita 1993).

The development of the construction process, such as indicated in Fig. 6, can be modelled by applying staged construction features, (i.e. contraction and de-watering), and ensuring equilibrium of the structure.

In 2D Finite Element analysis, the following stages of analysis are normally taken into account:

Stage I: Initial stresses are calculated using a K_0 procedure.

Stage II: The tunnel lining is activated while simultaneously the soil inside the tunnel is de-activated. This implicitly de-activates the soil weight. The water stresses and weight remain active.

Stage III: The groundwater weight is removed. To start with, the weight difference between groundwater and bentonite-clay is normally neglected. After this step the tunnel is 'dry'. The extra weight of the TBM is mostly neglected.

Stage IV: The last stage; volume loss at the tail, including the effects of the tail grouting process is modelled by applying a cylindrical contraction, with a specified percentage of the volume of the tunnel, to model a volume loss V_s. In addition to this a volume strain might be.

Stage V: Consolidation effects (up to the passage of a feasible second tunnel) are calcu-lated.

Stage VI: For a second tunnel, the stage II, III, IV and V are repeated.

Stage VII: Calculation of creep effects on the long run.

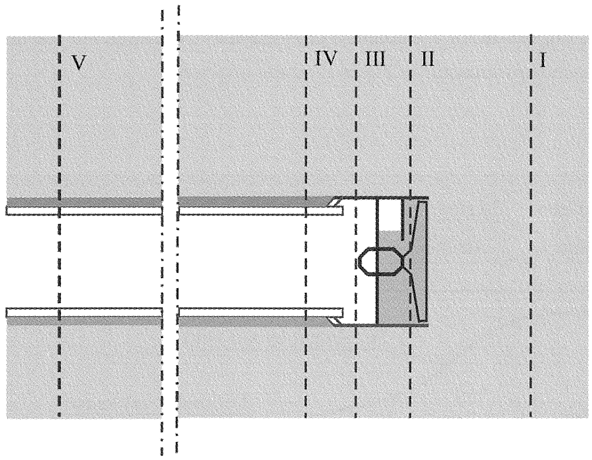

Figure 6. Phased analysis of tunnel construction.

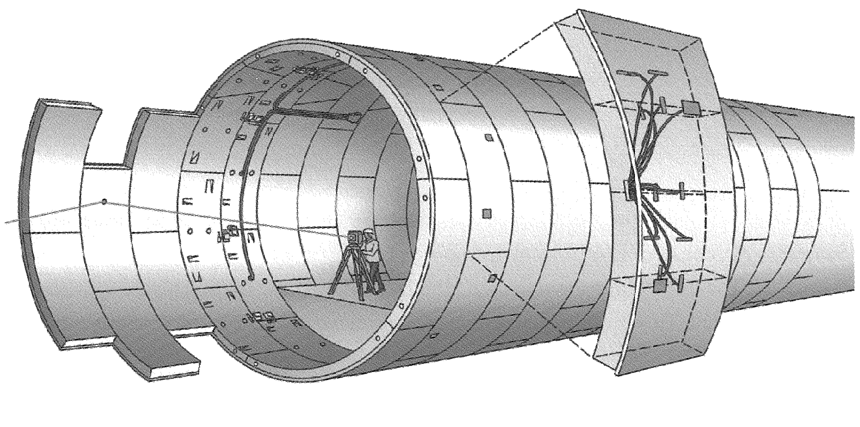

Figure 7. Measuring ring and instrumentation.

Before drawing conclusions on the applicability and advantages of the 2D Finite Element analysis, in the next section a back-analysis with this method on the data of the Second Heinenoord tunnel will be given.

4.4 Evaluation of the measurements at the Second Heinenoord tunnel

During the construction of the Second Heinenoord tunnel a complimentary research programme was executed in order to gain experience and do research into Tunnelling in soft soil. The second Heinenoord tunnel was the first of two pilot projects on Bored tunnelling in the Netherlands. In order to evaluate the design models available, measurements were made both in the TBM and during passage of two measuring fields, one on the North Bank, and one on the South Bank of the River Oude Maas. Under both measuring fields, a tunnel-lining ring was equipped with strain gauges placed in various different directions (ten strain gauges per segment).
All seven segments in a ring were instrumented to measure the entire stress distribution in the ring (see Fig. 7), as a function of time and distance behind the TBM. Pressure cells were put on the outer surface of the segments, (two pressure cells per segment on 7 segments), to compliment the strain gauges.
During the tunnel boring, measurements of circumferential normal forces and bending moments in the lining were made. After construction of the tunnel, the measurements where evaluated. The measurements where compared with the predictions before tunnel construction. The number of models applied during the predictions, made it difficult to discuss the advantages and disadvantages of the different models in a comprehensive way, therefore instead of that a back-analysis with the analytical models such as described in section 4.2 and a back-analysis with PLAXIS will be discussed here.

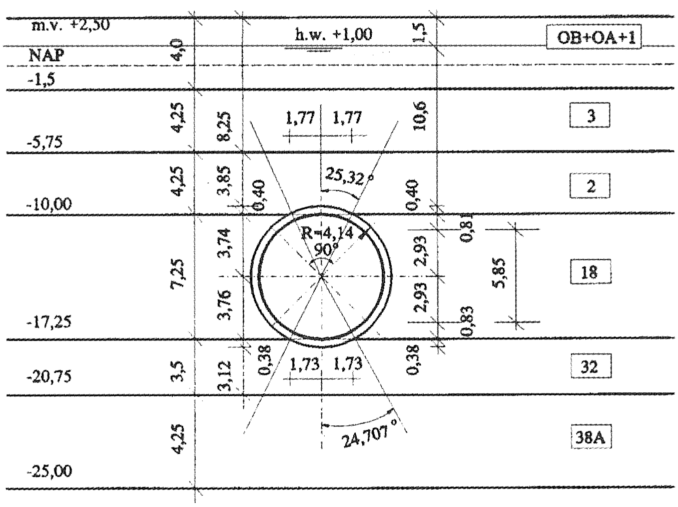

Figure 8. Cross-section measuring field North.

Table 1. Description of layers and soil parameters for the North Bank

Symbol	Soil type	Top of layer [m] N..A.P	γ_{sat} (γ_{dry}) [kN-/m³]	c_u [kPa]	c' [kPa]	φ' [-]	ν [-]	E_{oed} [mPa]	K_0 [-]
OA/1/-OOB	mixture of sand and clay	+ 2.50	17.2- (16.5)	-	3	27	0.34	5.2	0.58
3	sand, local parts of clay	- 1.50	19.5	-	0	35	0.30	26	0.47
2	sand with clay	- 5.75	19.0	-	0	33	0.31	25	0.47
18	sand, local parts of clay	- 10.00	20.5	-	0	36.5	0.30	40	0.45
32	sand, gravel	- 17.25	20.5	-	0	36.5	0.30	60	0.50
38A	clay, local parts of sand	- 20.75	20.0	140	7	31	0.32	16	0.55
38F	sand	- 25.00	21.0		0	37.5	0.30	80	0.55
38A	clay, local parts of sand	- 26.50	20.0	140	7	31	0.32	16	0.55

4.4.1 Back analysis for measuring ring 'North Bank' using Duddeck's model.
A summary of the main soil data from the 'K100 data set for predictions' is given in Table 1.
For the analytical model, the stresses, σ_v, and σ_h, in the soil at the level of the tunnel axis are calculated. In this model, these stresses are determinate for the bending moments in the lining. These stresses are calculated, summing up the soil weight, and accounting for the water pressure. The effective horizontal soil stresses are calculated using the K_0 relationship.

Based on the geology as indicated in Table 1 and Fig. 8, the horizontal and vertical soil stresses at the tunnel axis are calculated as: $\sigma_v = 300\ kPa$ and $\sigma_h = 218\ kPa$.
Subsequently, the flexibility of the lining is evaluated calculating the parameters α, and β from equation 9 and 10, which gives $\alpha \cong 35$, and $\beta = 0.023$
For which the coefficients for the empirical model are calculated as:

$$C_0^N = 0.982 \qquad C_2^N = 0.81 \qquad C^M = 0.25$$

If we combine this with the equations for bending moment and normal force we find:

$$M(\theta) = 81\cos(2\theta) \quad [kNm/m] \qquad \text{and} \qquad N(\theta) = -1004 + 132\cos(2\theta) \quad [kN/m]$$

According to this model we find that due to the interaction, the amplitude of the bending moment reduces, whereas the normal force is influenced only slightly.
In order to analyse the effects of the volume loss, in addition to this a number of 2D finite elements calculations were performed. The results of these are given, together with the measured data in the Figures 9, 10 and 11. For the Finite element analysis the axial stiffness of the wall was assumed to be equal to that of a homogeneous wall, whereas the bending stiffness of the wall was reduced by 50 % to account for the fact that the lining is composed of segments.

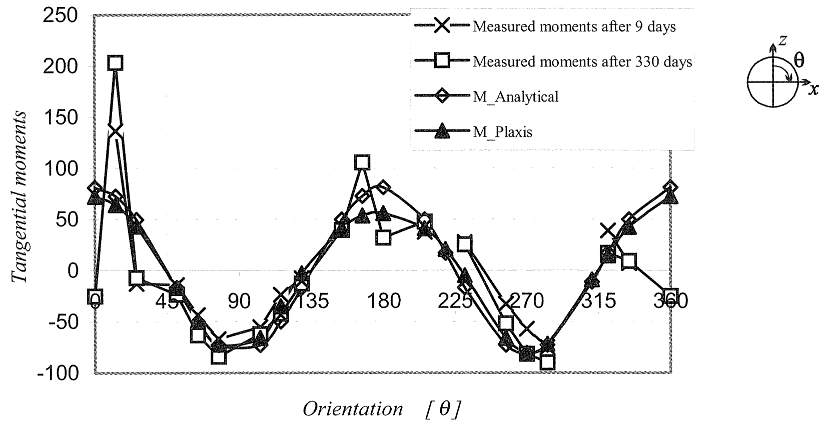

Figure 9. Bending moments as measured and back calculated for the measuring ring on the test site at the North Bank.

For the comparison, the stresses measured after 9 days where taken, as it was observed that after that time, the changes in stresses were only minor, and it was assumed that any 3D effect was passed after the length being constructed within that period. See also Fig 14.

To begin with the results for the bending moments are evaluated. Here our attention is drawn to the fact that both the empirical solutions as well as the Finite element Solution, on the average, give a good indication of the stresses and the stress distribution in the lining. Except for the observation that the stress distribution is not smooth as would be expected according to theory, but that peak stresses have been measured at several positions around the circumferential of the lining.

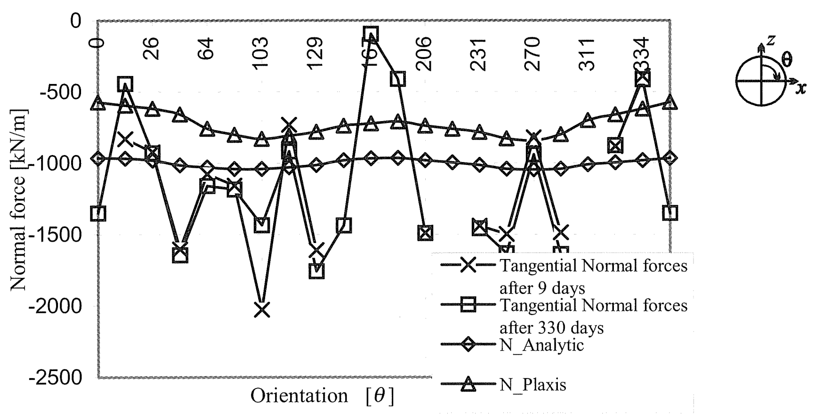

Figure 10. Comparison of Back-analysis for the Normal forces with Measurements at the North Bank.

One has to consider however, that the Finite Element analysis includes the effect of Volume Loss, whereas the empirical model only implicitly takes this effect into account. If the effect of Volume loss is not included in the Finite Element analysis, the agreement between Finite Element analysis and Empirical model is less good. For that situation Finite Element analysis calculates a tangential bending moment of M = 65 kNm/m, instead of 81 kNm/m. This effect will be discussed in more detail further on in this section.
Besides that, a general observation from the various calculations was that in comparison to the results of empirical models, which indicate that the stresses for a rough wall always lead to higher bending moments, the results from finite element analyses are contrary. With finite elements the highest bending moments are found for a smooth wall. Though this result was recognised by Erdmann & Duddeck (1986), it is not included in the analytic solution as given by the equations 11 and 12, which assume a rough wall.
If we look at the level of soil stresses from the finite element analysis, and compare this with the measured soil stresses shortly after installation, the finite element analysis gives a much better agreement than the analytical approach, see Fig. 10. Apparently there is a large influence on the soil stresses by the volume loss, which asks for a more refined analysis.

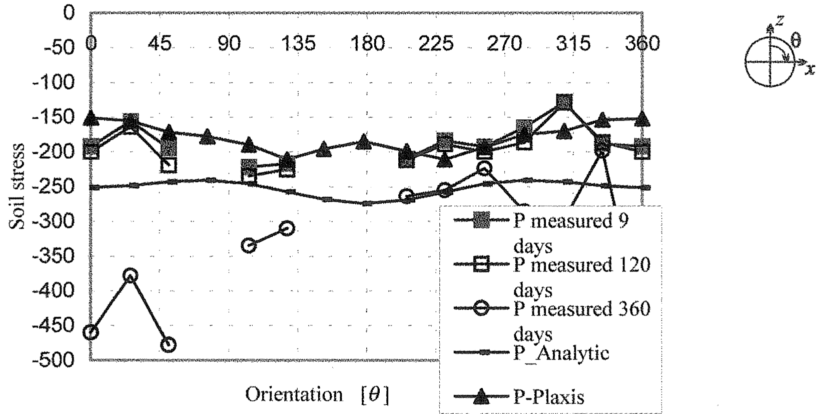

Figure 11. Radial soil stresses on the tunnel lining.

Looking at the normal forces in the lining, (the hoop forces) where the analytical solutions do not give an adequate fit, finite element analysis did not lead to improvement. In a comparison of both types of analysis with the measurements, the FEM analysis of hoop forces, see Fig. 11 is less good. On the one hand there is some doubt about the stress measurements, on the other hand at the time of analysis, the influence of grout-injection pressures might not have been understood good enough, and taken into account adequately. With respect to the stress measurements, it is as if the low soil stresses measured in the pressure gauges are not compatible with the normal forces in the lining being measured with the strain gauges. A possible explanation for this effect might be that the measured soil pressures are related to an earlier time step of stress development than the normal forces. It is as if the stress level directly after grouting is 'frozen in' by the hardening of the grout. Further increase of the effective soil stresses might be correctly measured in the tangential strain gauges but not by the pressure gauges due to over-bridging effects in the hardened grout. Therefore the accuracy of the soil pressure gauges is unclear, especially with respect to their capability to monitor the stress development. The strain gauges, on the other hand, are thought to have had a higher degree of reliability. Other analyses by van Oosterhout (1999), taking into account measurements over a much longer period, i.e. one and a half-year give some weight to this assumption.

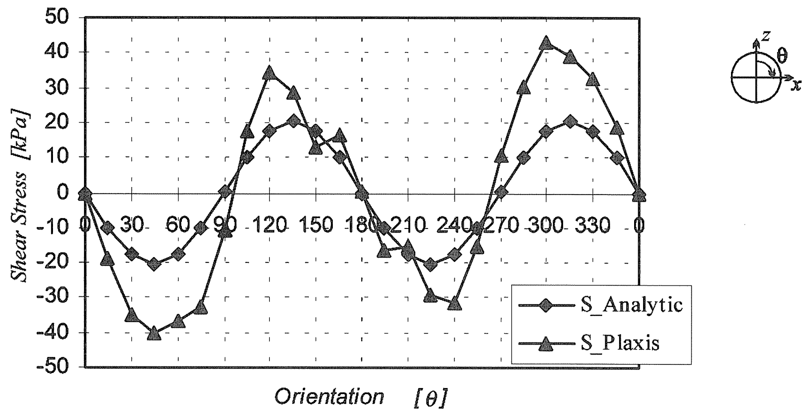

Figure 12. Comparison of Shear stresses around the tunnel, between the analytical model and according to PLAXIS.

Next the vertical equilibrium and the effect of circumferential shear stresses is evaluated. Looking into the stress distribution from PLAXIS it was found that the integrated normal stresses around the tunnel only, do not generate a vertical equilibrium. The stress level at the invert of the tunnel is much higher than at the crown. This might be explained, however, if we evaluate the shear stress distribution. For the analytical model and for the finite element model, see Fig. 12. It is observed that the shear stresses calculated with PLAXIS, at the crown of the tunnel, are higher and therefore also contributes to the vertical equilibrium of the tunnel.
As discussed above, the agreement for the bending moments seems reasonable, although at some points, distinct differences are observed. These differences are attributed to the variations in the normal force, and interaction between adjacent tunnel rings. According to Visschedijk (1996), interaction at the joints in a ring might increase the bending moments by a factor up to 1.6. In addition to that, the variations in the normal force combined with variations in the deformations of the midst of the segments, which have been observed to exceed 0.02 m, might add to that. This additional moment in estimated to be approximately;

$$M_{ad} = N\delta \approx 1500 * 0.02 \approx 30kNm/m$$

This suggests that the differences between the back-analysis and the measurements can reasonably be explained by interaction between segments and the effects of installation.

4.4.2 Bending moments as a function of the volume loss

In the back-analysis, it was assumed that the volume loss was 0.5 %. The value of 0.5 % was estimated from the volume loss as observed at the soil surface, where a volume loss of approximately 0.8 % was measured. For small volume losses, assuming the analytical solutions, such as those of Sagaseta (1987), the volume loss at the tunnel is back analysed to be less by a factor of 1.6.

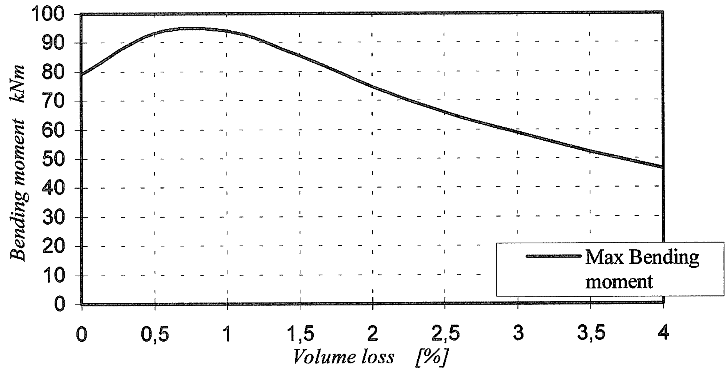

Figure 13. Bending moments as a function of the volume loss in %, for measuring field South (PLAXIS-analysis).

Looking at the development of the stress in the lining, the force volume-loss relationship, it is observed that the (maximum) bending moment increases with the volume loss. For the given stiffness ratio between structure and soil, for the South measuring field, the relationship is given in Fig. 13, which indicates that for this situation the bending moment displays a maximum somewhere in the range between 0.5 and 1.0 % volume loss. For volume losses larger than this the bending moment appears to reduce.

For a typical tunnelling situation the volume loss is usually greater than 0.5 %. If the tunnel-boring machine is well driven it should be possible to limit the volume loss to 1.0 %. For the design of the tunnel ring, an assumed volume loss between 0.5 - 1.0 %, is thought to be reasonable.

4.5 3 Dimensional effects in the analysis

Thus far in this paper it was assumed that for tunnel design a 2D analysis is appropriate for the evaluation of stresses and strains in a lining. It has to be considered however that directly behind the tunnel boring machine this might be unrealistic, there the loading might be influenced by 3 dimensional effects due to the local character of the deformation.

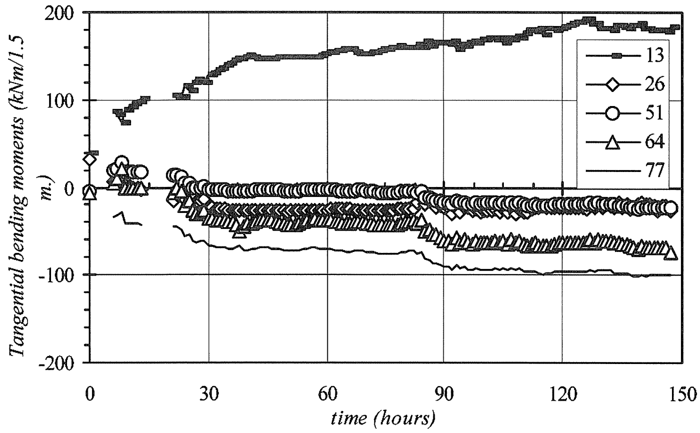

Figure 14. Bending moments in the tunnel ring 1; 3/4/97 up to 9/4/97, the numbering indicates the orientation in degrees with respect to the vertical axis.

Behind the boring machine the pressures at which the tail void is grouted are determinate for the loading on the tunnel and also on the surrounding soil. Due to the local character of the grouting a 3 dimensional stress situation develops. After that the grout hardens and tunnel construction advances, this 3 D stress state smoothes out and will assume a more 2 dimensional character. Based on an average speed of approximately 10 m per day it is assumed that the 3D effect is limited to a length of 3 to 4 times the tunnel diameter, see also Fig 14.
A similar result was also produced using PLAXIS 3D see Vermeer (2001). For design purposes it is assumed that a 2D analysis is sufficient. Vermeer argues that "2D bending moments are slightly larger than the ones from the 3D step-by-step installation, but differences are modest".

4.6 Tunnels in Clay, creep effects

Up to this point, the discussion is based on a short-term analysis, disregarding any time dependent effects, but for the fact that the analysis might include consolidation effects. For tunnels in sand or other hard soils such a time independent analysis is thought to be appropriate. For softer soils such as clay the time dependency cannot be disregarded. The analytical means to analyse such soil behaviour until recently were not available for practical purposes.

Since the implementation of a creep model in PLAXIS such an analysis has become more easily available. With respect to tunnel behaviour in clay, Brinkgreve (2001) has analysed the effects of creep on the development of lining stresses in more detail.
Brinkgreve modelled the creep behaviour of the soft soil by means of the Soft-soil Creep model. In the Soft-soil Creep model, the creep strain rate is partially determined by the initial over-consolidation ratio, the OCR.

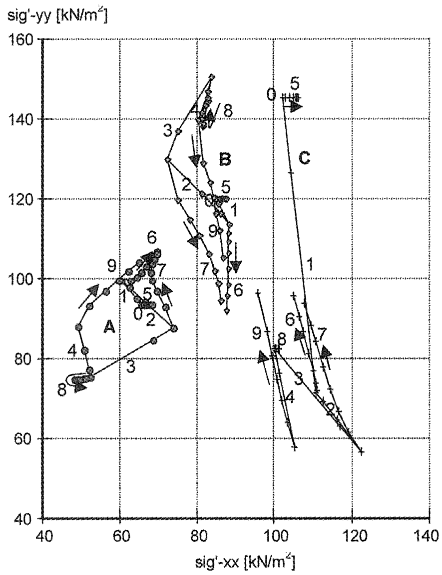

Figure 15. Stress paths during construction, contraction and creep, for points at the crown A, besides the tunnel B, and at the invert C.

For a characteristic situation for Holocene soil in the western part of the Netherlands, his numerical study indicates that bending moments might increase more than a factor of 2 compared to short-term values, whereas normal forces increase slightly, in the order of 10%. The results could well be explained by stress path analyses of points around the tunnel, see Fig 15, where it can be observed that the effect of short term stress redistribution due to interaction, on the long run is overruled by the effect of creep deformations.

Therefore for creep susceptible soil, this effect must be taken into account. If this effect is overlooked, the final bending moments in the lining might be largely under-estimated. Instead of a large reduction of lining bending moments due to interaction, the lining stresses might be in the range that would be found applying the initial soil stresses as input for the analysis of the soil loading on the tunnel lining.

5 Longitudinal action and beam action

Up to this point, the main discussion in this paper is focussed on the lining stressing in circumferential direction; Tangential normal forces and tangential bending moments in the ring. Due to the staged development of the construction and due to the Jack-forces continuously acting on the tube, the lining is also loaded in longitudinal direction. Due to this process, bending moments and shear forces develop in the tunnel tube. The development of these stresses is influenced by the construction method.

The construction of a segment tunnel might be characterised as adding tunnel rings onto the earlier constructed tube. These rings are subsequently loaded by the groundwater due to the buoyancy forces, and by forces from the tunnel-boring machine. The actions from the tunnel-boring machine might be, an (eccentric), axial load, and a vertical (shear) force, see Fig. 16. For the analysis of the tube bending moments, the mean axial loading by the tunnel-boring machine is not considered further, as the mean axial force does not contribute to the longitudinal bending moment. The eccentricity however, is important and is modelled here as an external bending moment.

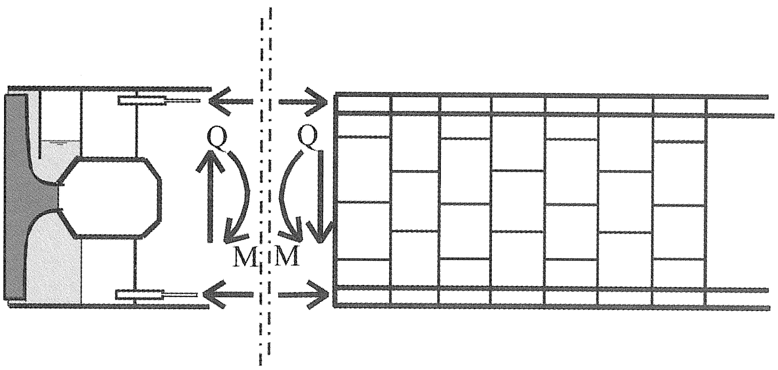

Figure 16. Loading of the tunnel beam by the tunnel-boring machine.

5.1 Quantitative analysis of the beam action applying a subgrade reaction model

For this loading situation, Bogaards (1999) developed a method for the analysis of the axial bending stresses and it's distribution in the lining. In order to illustrate the scheme he applied, in Fig. 17, the addition of subsequent segments (tunnel-rings), is sketched. The assumption is, that after each successive stage of construction, equilibrium of forces and stresses is required. We can recognise in Fig. 18, where 18A is the old stage of construction, and 18C is the new stage of construction, that 18B has to be added to the system of forces, to derive the desired equilibrium state. That is, putting the external forces on the one side of the new segment, and releasing the forces on the other side of the segment. This scheme implies that to derive the stress state at a certain stage of construction, the former stages of construction have to be summed up, as illustrated in Fig. 17.

In Fig. 18 it is also clear that this summation of forces constitutes of three partial contributions, which are:

1. *The distributed load **q** due to buoyancy*
2. *The shear force Q produced by the tunnel boring machine*
3. *The eccentric axial force introduced by the tunnel-boring machine, modelled as an external bending moment M.*

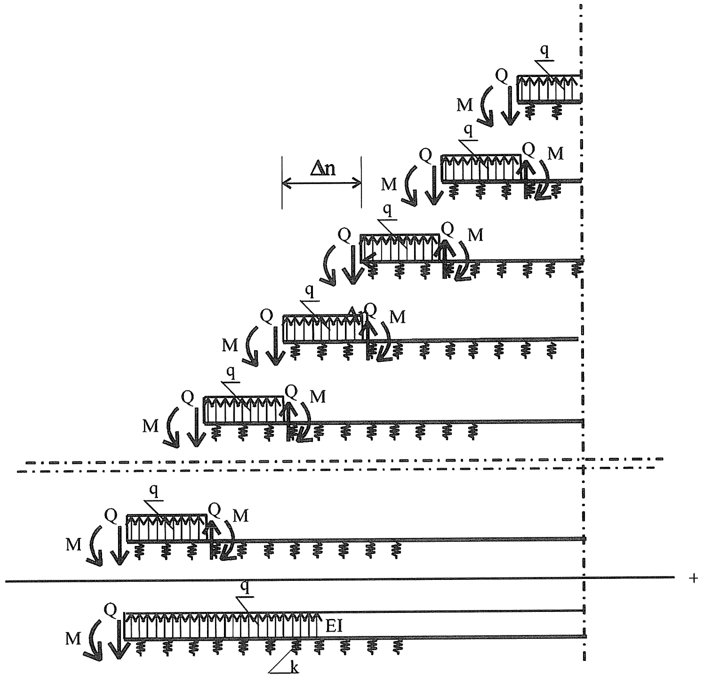

Figure 17. Conceptual models for the analysis of longitudinal stresses in a tunnel lining.

Distributed load: For the distributed load due to buoyancy an analytical solution, by Bogaards (1998) is available:

$$M_q(m) = -EI\beta^2 \sum_{n=1}^{m} e^{-\beta n\Delta_n}(-2C_1 \cos(\beta n\Delta_n) + 2C_2 \sin(\beta n\Delta_n)) \tag{16}$$

where

$$C_1 = -\frac{q}{k} e^{-2\beta\Delta_n} \sin^2(\beta\Delta_n) \tag{17}$$

and

$$C_2 = \frac{q}{k}[(\sin(\beta\Delta_n)\cos(\beta\Delta_n) - \tfrac{1}{2})e^{-2\beta\Delta_n} + \tfrac{1}{2}] \tag{18}$$

Here, Δ_n is the element length, (i.e. the width of a tunnel ring which is being constructed).

Eccentric load: For the eccentric axial loading, in this simplified model, a constant Eigen moment is introduced in the tube by the tunnel-boring machine

$$M_{exc} = M_{TBM} \tag{19}$$

Shear force: For the shear force introduced by the TBM, the following relation is derived:

$$M_Q(n) = -\sqrt{2}\sum_{m=1}^{n} Q\Delta_n e^{-\beta m\Delta_n}\left[\sin(\beta m\Delta_n + \tfrac{\pi}{4})\right] \tag{20}$$

The bending moment at segment n, can than be calculated by summing up the three partial contributions:

$$M(n) = M_q + M_{exc} + M_Q \tag{21}$$

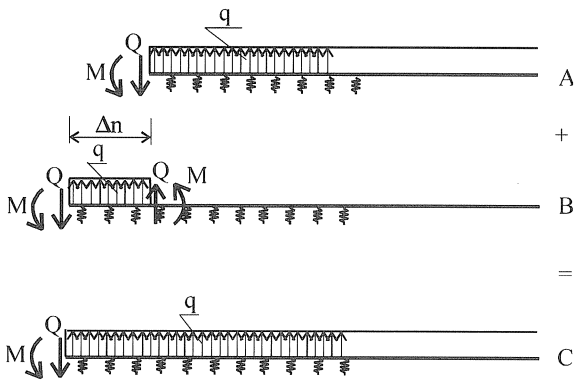

Figure 18. Staged construction of a tunnel tube, adding segments.

5.2 Measurements of longitudinal bending moments

The theory described in section 5.1 was applied by Bogaards for the analysis of the measurements from the Second Heinenoord tunnel. The result of this study is illustrated in Fig. 19. As can be observed, the magnitude of the measured and calculated bending moment is of the same order, although the predicted distribution is not accurate.

Based on the research of Bogaards in was concluded that to improve the analysis, the following steps should be taken:

1. *the forces due to the TBM, and of the Trailer train should be more closely monitored*
2. *the subgrade reaction of the hardening grout material in the zone just behind the TBM should be better understood.*

In a follow up study by Hoefsloot (2002), for the design of the Hubertus tunnel in The Hague, the model was improved to include the effect of the weight of the trailer train.

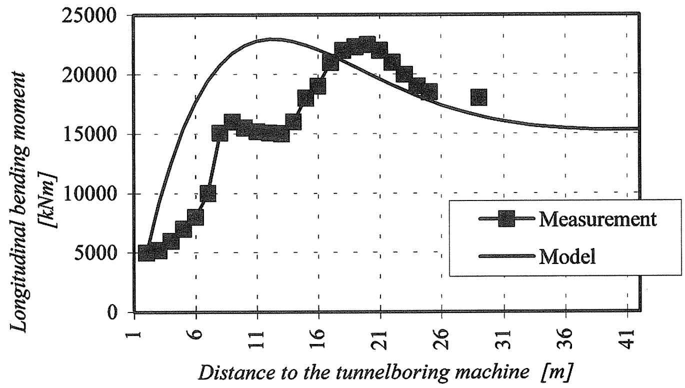

Figure 19. Comparison between measurements and analysis for longitudinal bending moments.

In addition to that Debrauwer (2003), did research on the development of Grout-stresses, including the effect of longitudinal forces.
With these studies the longitudinal stresses become better understood. Still, the Shear forces by the TBM, are an uncertain factor in the analyses. Generally the argument from the contractor's side is that the Jack-forces must be, and cannot be different than axially directed. The gradient in the measured bending moment however cannot be explained if this would be the case.

In practice for the structural design in Ultimate limit state, the longitudinal effect is of no importance. For the Serviceability limit-state however the situation is different, the longitudinal stresses might be determinate for the change of joint leakage. For the situation of an Extruded Concrete liner such as proposed for the Hubertus tunnel in the Hague (in design), this issue was recognized as important for the change on water bearing cracks, and therefore determinate for the distance of joints to be created between different tunnel sections.

6 Loads due to erection of the lining and due to thrust jacking

At the time that the predictions for the Second Heinenoord tunnel where made it was assumed without argument that the erection of the lining would not cause stresses in the lining of any significance, therefore to begin with these stresses where disregarded. In addition to that it was assumed that the Thrust jacking forces would only cause additional stresses close to the place where these forces where acting, and would not add up to the geo-static forces due to soil loading.
After that, at the start of the project, i.e. the first 100 m of construction, the tunnel showed some severe damage to the lining, and after the evaluations of the first instrumented tunnel ring was evaluated, our opinion on this subject had changed.
As among other things can be observed, i.e. as the peak stresses in Fig. 9, additional stresses might develop in this phase of construction, which contrary to the first assumption do add up to the stresses due to soil loading, doubling magnitude of the representative stresses in the lining.
Besides that, additional deformations between segment, and subsequent forces must have been

present, which caused cracking and spalling of tunnel lining segments.
The main thing to consider here is that apparently due to these effects Eigen stresses are introduced in the lining for which we have to recognize that these may not be diminished after that the tunnel-boring machine moves forward. A characteristic of Eigen stresses is that for a redundant structure, they do not contribute to failure in the ultimate Limit State. For failure calculations, therefore, it is only necessary to consider the stresses due to soil loading. On the other hand Eigen stresses may cause early cracking in the lining and therefore affect the durability of the structure.
After evaluation of this phenomenon it was concluded that just after the segments are placed and still within the tail of the TBM machine the lining is relatively vulnerable. At this position the lining is not yet subjected to the pre-stressing support of the soil. When the Thrust Jacking forces are put on the lining than, during excavation any irregularity might induce additional stresses, especially if there is any ill positioning of the lining with respect to its ideal position. This may cause irreversible deformations in the joints, and even may cause damage.
Based on these observations, for the Green Hart tunnel, in jointly cooperation with TNO and Delft University of Technology, a research program was started and a test facility, see Fig. 20, was build in order to do research on the structural behaviour of a segmented lining during the construction stage. The goals of the projects where: To get a better understanding of the stresses and deformation in a segmented lining, during the construction stage and to derive data for the validation of new 3D FEM models. This research is discussed in more detail by Blom (2002).

More details on this project are given in the other contributions of this issue of Heron.

7 Exceptional loading conditions, such as fire

Due to accidents during operation of a tunnel, especially if transport of flammable goods is not prohibited, there is always the risk of fire or even explosions.
In general the pressures induced by an explosion are beyond the bearing capacity of a tunnel; most tunnels in the Netherlands are only dimensioned for only a minor overpressure of 100 kPa, whereas a real explosion might be an order beyond that loading. For the situation of fire however, it is a starting point for the structural design that a lining has to withstand the fire induced by a burning lorry. Without going into detail about the temperature loads related to this, accidents as in the Channel tunnel and also in the Störebaelt tunnel, have shown that during these fire situations, spalling of the tunnel wall might occur. Reports say that at the Störebaelt at the most damaged zones half of the lining thickness was lost or severely damaged, though without causing a critical situation.

In the Netherlands the policy with respect to fire is to protect the concrete lining from the most severe temperatures by introducing an insulation material; preventing any spalling within the time span of 2 hours. From the point of Ultimate Limit state analysis, still the question had to be answered, for what remainder of a lining thickness a tunnel lining might become critical.
If a lining becomes affected by fire, due to spalling the lining thickness might decrease. This will increase the flexibility of the lining. In section 2 it was argued that due to flexibility, the lining might adapt to the soil loading up to the point that equilibrium is reached again. We have to

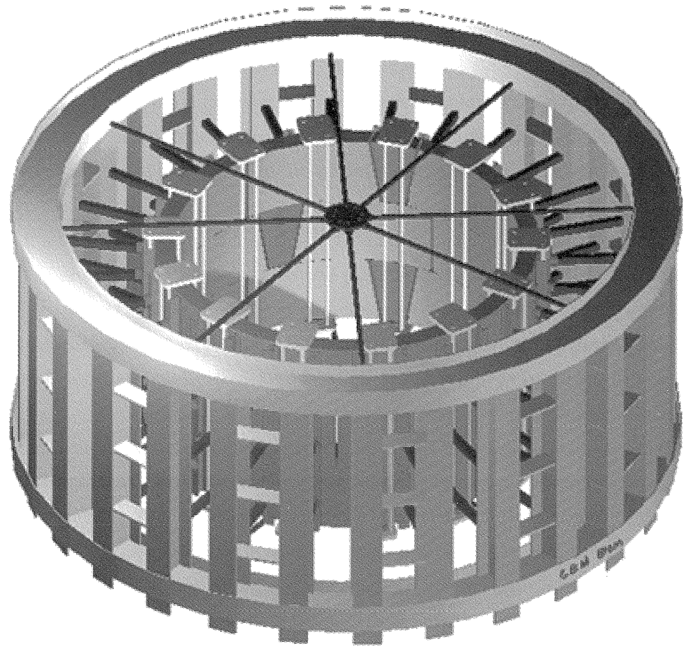

Figure 20. Bore tunnel test Facility at Delft University, Blom (2002).

remember however that in cross-sectional equilibrium, the circular lining of a tunnel will react both developing circumferential hoop forces, as well as bending moments. In that sense the lining is comparable to an eccentrically loaded column.
The latter is, for the ultimate Limit State, evaluated including second order large deformations. According to Timoshenko (1936), in the ultimate Limit State, the critical circumferential load (or hoop-force), for the lining is

$$N_{cr}=\frac{3EI}{r^3} \tag{22}$$

where r is the tunnel radius.
According to, among others, Besseling (1975), the bending moments including second order effects, can be approximated by:

$$M^2=M^1+\frac{n}{n-1}N^1\delta^1 \tag{23}$$

where

$$n = \frac{N_{cr}}{N^1}$$

M^1 = the first order, (small deformation) bending moment

N^1 = the acting normal force

δ^1 = the first order amplitude in deformation

Based on this assumption, Mendez Lorenzo (1998) related the reliability index to the lining thickness, for a homogenous tunnel ring theory, using probability theory. She made her analysis for a standard tunnel with a radius of r = 4.7 m, a wall thickness of 0.4 m, concrete with a compressive strength of 45 mPa, (B45 according to the Dutch code of practice).

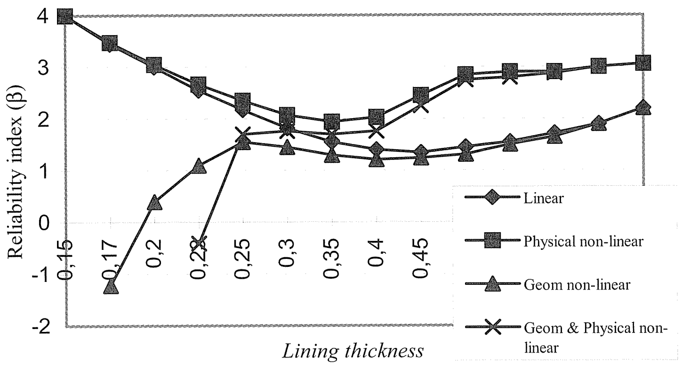

Figure 21. Reliability index as a function of lining thickness, according. to Mendez Lorenzo (1998) for α_{SFRC} = 0.05;B45 and R = 4.7 m.

Mendez Lorenzo found that the reliability index ß at first increased for a decreasing lining thickness, see Fig. 21, and decreased beyond a certain thickness. Second Order effects overtook the favourable effect of flexibility and stress exchange between lining and soil. In the example she analysed, Mendez Lorenzo established a critical lining thickness of 0.3 m. for a tunnel with a diameter of 9.4 m. i.e. (d/D = 0.033).

8 Durability of tunnel linings

Without going into detail, in order that a tunnel can maintain its functions, to begin with the structural integrity of the lining has to be secured; i.e. the tunnel strength, and apart from that, the ability of the lining to prevent water and soil to flow into the tunnel.
If we compare the linings of a bored tunnel, i.e. composed of segments, with the lining of an immersed tunnel, the first thing that comes under our attention is the length of joints in a segmented lining. If we make a comparison, an immersed tunnel of about 1.5 km of tunnel, may have a joint length in the order of one kilometre, whereas the joints of a segmental tunnel, (of the same length) may have a total joint length that may exceed that with a factor of about 30.

All these joints, in a bored tunnel, rely for the water tightness on a rubber sealing of only limited dimensions. Besides that, experience learns that during tunnel construction, it is very difficult to avoid that segments are loaded without some incidental cracking in the concrete. Sometimes these cracks even affect the sides of the elements, and if there is spalling at the sides or the corners of a segment, the rubber sealing might be damaged.
Nowadays it is a standard procedure to back-grout the tail void behind the lining. For the water sealing of the lining it is advantageous that the grout hardens after injection, developing cohesion and decreasing the permeability of the grouted material. Though in practice this advantage is not taken into account the positive effect of this is observed because joints leaking water directly after construction and back grouting, often show a self-healing behaviour.
From experience with immersed tunnels it is known that it is hardly possible to construct a tunnel without any leakage. On the other hand we have learned to handle minor leakages, and to adjust our dewatering system of the tunnel in such a way that this does not affect the tunnel functions. The conclusion therefore is that minor leakages do not affect the use of a tunnel.
A different attitude has to be taken for the situation that a leakage is transporting sand or soil. Any transport of material in the end will result in settlements of the tunnel. Experience has learned that it is very difficult, to stop a sand-leakage. In practice the relative impermeability of the tail void grout, in combination with its cohesion, give a second barrier to prevent sand carrying leakage.
At some tunnelling projects there is a trend to reduce the amount of cement for what is called "weekend-grout", in order to prevent any start-up problems on the Monday morning. It is even heard that sometimes, this cement free "grouting" material is used as the standard grouting material. If the tunnel construction is carried out in the relative impermeable soil such as the Boom-Clay, for the Westerschelde, this may not be a problem. If the surrounding soil would be sand, this would lead to an increased risk for the durability of the tunnel. If the tunnel would develop a leakage due to longer term settlements, sand-transport would not be unlikely for which the structure would not have any second defence system, and might collapse due to loss of embedment.
A more common evaluation of a harmful effect is the penetration of chloride into the concrete. If the chloride penetrates up to the point where it reaches the reinforcement of the concrete, corrosion may be triggered, giving severe damage to the lining. The use of high quality pre-cast concrete based on blast furnace cement considerably reduces the speed of chloride penetration in comparison to cast in situ concrete based on Portland cement. Analyses up to know indicates that, with minor additional measures, the 100 year design life for segmental tunnels should not be a problem see de Vries (2000).

9 Concluding remarks

Based on the evaluation of the stress measurements at the Second Heinenoord tunnel it is concluded that even with relatively simple models the stresses in a lining can be approximated sufficiently accurate, and can be used for a preliminary structural design.
Although, it has to be recognized, that for a more refined understanding of the stresses and strains, e.g. to take into account the effects of volume losses and or tail void grouting, two dimensional analysis, i.e. with Finite elements gives a better description. The latter approach also enables the

analysis of time dependent phenomena such as consolidation and creep, and is advised therefore for a final structural design.
An aspect to consider in more detail is the influence of flexibility on the tunnel lining design. In section 2 it was discussed that in plastic design, the final decision for an ultimate limit state, depends on criteria for the deformation. In comparison, a more flexible lining will yield the lower bending moments. A doubtful conclusion might be to choose a relatively thin lining, which is flexible and to design for as low a bending moment as possible. It was argued that in the context of stresses in the construction phase, or in case of other hazards such as fire, that a lining, which is too thin, is unprofitable.
In addition to the consideration of fire hazard, there is also the feasible effect of soil creep. In section 4.6 it was shown, based on numerical analysis, that for tunnels in clay, or for creeping soil in general, a design based on a short-term analysis might underestimate the bending stresses in the lining. Due to soil-creep the soil stresses might come back closer to their initial state than anticipated with an analysis only based on the interaction between the flexible lining and elastic soil behaviour.
Considering the durability of the rubber sealing and the securing of the water retaining function; too much differential deformation between tunnel segments is unwanted. In addition to that, it would be desirable if the back-grouting material, pumped in the tail void, would combine flexibility with some cohesion and if its final permeability after cementation would be low.

Finally, maybe the future is not far away that the tunnel lining designer may be using an integrated design system. Such a system might include features of importing design data from CAD, and GIS data of the soil surface, as a base for the topology and meshing of 3D Finite Element analysis. The Finite Element analysis methods however should be enhanced to include update meshing for geometric non-linear effects, time dependent phenomena such as consolidation and creep, and features for mathematical strength reduction; the latter in order to verify the structural safety of the tunnel design. Until that time, and even after that, the empirical models and more simple numerical models outlined here are important for education and combined with engineering judgement and experience, important for the design of tunnels today.

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