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Soil loads acting on shield tunnels: Comparison between bedded beam model and finite element calculations

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ABSTRACT: The Ministry of Transport and Public Works of The Netherlands has ordered the construction of a large diameter bored tunnel at Heinenoord. The tunnel will have an outer diameter of 8.3 m and will be the first large diameter shield tunnel in The Netherlands. The tunnel will be bored in soft soil conditions. For the design of the tunnel lining, calculations have been performed with the bedded beam model. A comparison between these calculations and finite element calculations has been made. It was concluded that soil stresses acting on the lining and behaviour of the lining according to the finite element calculations was different from the bedded beam model with respect to the influence of the water pressures (buoyancy of the tunnel) and the modulus of subgrade reaction.

1. INTRODUCTION

In 1994 and 1995 a design was prepared for the first large shield tunnel to be driven in The Netherlands, the Second Heinenoordtunnel. The tunnel contains two tubes with an outer diameter of 8.3 m and a lining thickness of 0.35 m. The tunnel crosses the Oude Maas river. Delft Geotechnics has executed the soil investigation for this project and has provided geotechnical recommendations. The design has been made by the TCH contractor combination on behalf of the Dutch Ministry of Transport and Public Works. The TBM is of the Slurry Shield type.

The soil conditions in the western part of The Netherlands differ at various locations from soil conditions in the surrounding countries. The top layers can be characterised as very soft and the ground water table is mostly high. Therefore extensive research has taken place and will be executed in the near future to investigate the applicability of foreign design methods for Dutch soil conditions. In this paper some results are presented of research, that has been executed by Delft Geotechnics.

Results will be presented with respect to the geotechnical parameters used in the bedded beam model to determine stresses in and deformations of the tunnel lining. Special attention is given to :

- effective soil stresses acting on the lining after construction of the lining;
- the determination of the modulus of subgrade reaction and distribution of the subgrade reaction of shallow tunnels;
- effective soil stresses on tunnels situated in soil, which is liable to settlement.

For this research 2D finite element calculations have been executed for two situations at the Second Heinenoordtunnel and a hypothetical situation, representing a Dutch soft soil profile. These calculations will be discussed in this paper.

2. CALCULATION MODEL

The calculation model, that has been used by the TCH to determine bending moments, shear forces and normal forces in the lining is the version of the bedded beam model developed by Wayss & Freytag. Two parallel lining rings are modelled, where each ring contains 7 elements with a width of 1.5 m and a thickness of 0.35 m. The second ring is rotated over half the segment length in comparison with the first ring. The two rings are connected by a 'shear spring'. The principle of this model is given in figure 1.

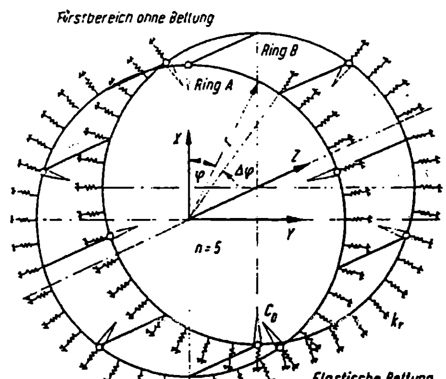

Figure 1, Wayss & Freytag's bedded beam model

The subgrade reaction and the effective soil stresses are schematised in accordance with ITA and Duddeck. (1980). This means that for shallow tunnels, no subgrade reaction is taken into account at the top of the lining over a total angle of 90 degrees. The fact that the vertical effective stress acting on the bottom of the lining is taken equal to the original vertical effective stress at the top of the lining, suggests that no stresses in the lining are generated due to decompression of the soil below the lining. The buoyancy is eliminated from the calculation, in accordance with Erdmann (1983), by assuming a sine distribution of the water pressures.

3. ANALYSED SOIL PROFILES

In this paper analyses will be discussed of tunnels in 3 soil profiles. The first two profiles were obtained from the Heinenoord project. The first profile is found at the northern starting point of the shield tunnel. The top layer mainly consists of loose sand, followed by firm peat and peaty clay layers with an undrained shear strength of about 50 kPa. These layers are lightly over consolidated. Below these layers a dense to very dense sand layer is found. The tunnel is located mainly in the peat and clay layers, while the bottom rests on the dense sand layer. The spacing between the tubes at this location is half the outer tunnel diameter. The ground water table is at approximately 4 m below ground surface.

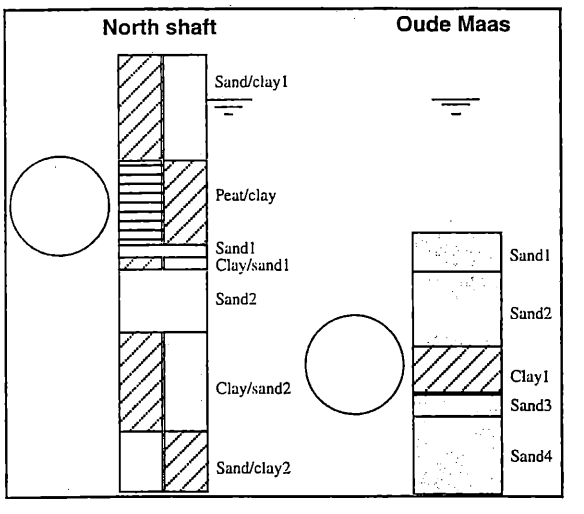

Figure 2. Soil profiles Second Heinenoordtunnel

The second profile is situated at the river crossing. During high tides and/or high river flow a water level of approximately 14 m above the river bed occurs. At this location the soil above, adjacent to and below the tunnel is dense to very dense sand. The spacing between the tubes equals about 1 diameter. These two soil profiles are given in figure 2. A detailed description of the soil parameters and layer thickness is given in section 4.

Additional calculations have been executed for a typical Dutch soft soil profile. The top 10 m consist of very soft peat and peaty clay layers, with an undrained shear strength increasing from approximately 3 kPa at ground surface to approximately 12 kPa at 10 m depth. The ground water table is located at approximately 0.8 m below ground surface. The saturated unit weight of these layers varies from 10.2 kN/m^3 of the peat layers to 14.5 kN/m^3 of the clay layers. The average unit weight of the soft layers is about 12 kN/m^3. Below the soft layers a very dense sand layer is found with a thickness of about 12 m. Below the sand layers a stiff clay layer is found with an undrained shear strength of approximately 150 kPa. In this profile two tubes with an outer diameter of 9.8 m and a lining thickness of 0.45 m was modelled. The profile is visualised in figure 3.

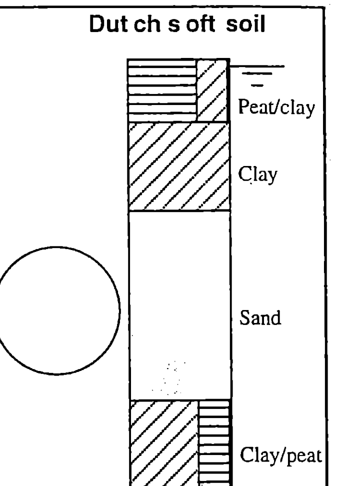

Figure 3. Dutch soft soil profile

4. CALCULATIONS

The aim of the calculations was to simulate the stress changes in the soil due to the tunnelling process for the normative situations. The calculations were 2D finite element calculations performed with the Pluto package which has been developed by Delft Geotechnics. In the calculations, the following process was modelled :

- first, the unit weight of all soil layers is applied, to generate the initial stress state in the soil layers;
- after that, the first tube is installed in the mesh;
- the unit weight of the concrete lining is applied, the effective weight of the excavated soil is removed and the water pressures acting on the tube, including the buoyancy force, is applied;
- finally, a second tube is installed in the same way as the first tube.

Table 1. Soil parameters northern shaft Second Heinenoordtunnel (1)

Soil type	Top of layer (m to NAP)	Unit weight (kN/m^3)	E (kPa)	ν (-)
sand/clay1	+4.5	17	7000	0.28
peat/clay	-5.2	14.5	3300	0.28
sand1	-12.9	20	41000	0.25
clay/sand1	-14.0	15.5	6000	0.30
sand2	-15.0	20	47000	0.26
clay/sand2	-20.5	19.7	25000	0.27
sand/clay2	-29.4	20	40000	0.26

For the calculations, performed for the design of the Second Heinenoordtunnel, use has been made of a Mohr-Coulomb failure criterion for all soil layers and a linear elastic stiffness until failure. The stiffness of the soil layer below the tunnel has been increased in comparison with the measured values to incorporate a relatively stiff behaviour under decompression. The soil layers and soil parameters are given in tables 1 and 2 for the calculation, executed for the situation just behind the northern shaft, and in tables 3 and 4 for the calculation, executed for the situation at the Oude Maas crossing.

Table 2. Soil parameters northern shaft Second Heinenoordtunnel (2)

Soil type	Top of layer (m to NAP)	ϕ' (degr.)	c' (kPa)	K_0 (-)
sand/clay1	+4.5	27	1	0.55
peat/clay	-5.2	22	4.5	0.55
sand1	-12.9	34	0	0.55
clay/sand1	-14.0	22.5	3	0.55
sand2	-15.0	34	0	0.45
clay/sand2	-20.5	29	4	0.45
sand/clay2	-29.4	32	2	0.45

Table 3. Soil parameters Oude Maas crossing Second Heinenoordtunnel (1)

Soil type	Top of layer (m to NAP)	Unit weight (kN/m^3)	E (kPa)	ν (-)
sand1	-10.7	20	7500	0.30
sand2	-15.0	20	24000	0.26
clay1	-21.8	19.5	9500	0.27
sand3	-26.0	20	19000	0.26
sand4	-28.0	20	50000	0.26

Table 4. Soil parameters Oude Maas crossing Second Heinenoordtunnel (2)

Soil type	Top of layer (m to NAP)	ϕ' (degr.)	c' (kPa)	K_0 (-)
sand1	-10.7	30	0	0.40
sand2	-15.0	34	0	0.40
clay1	-21.8	28.5	5	0.40
sand3	-26.0	32.5	0	0.40
sand4	-28.0	32.5	0	0.40

In these tables E is Young's modulus, ν is Poisson's ratio, ϕ' is the angle of internal friction, c' is the effective cohesion and K_0 is the coefficient of effective horizontal stress at rest.

The top level of the tubes near the northern shaft is NAP - 5.1 m and the bottom level NAP - 13.4 m. The spacing between the tubes was 4.4 m. The ground water table is taken at NAP + 0.4 m.

The top level of the tubes at the Oude Maas crossing was NAP - 19.4 m and the bottom level NAP - 27.7 m. The spacing between the tubes is 8.3 m. The water level in the river is taken at NAP + 0.5 m.

To incorporate the effect of reduced lining stiffness due to segmentation a Young's modulus of the lining of 12 GPa was used, which is approximately 30% of the concrete stiffness. The unit weight of the concrete lining has been taken 24 kN/m^3. A full bond between tunnel and soil was assumed.

4.1 *Results northern shaft*

At the northern shaft a water pressure of 56 kPa was acting on the top of the tubes and 139 kPa at the bottom of the tubes.

In figure 4 the effective, radial soil stresses acting on the lining at the end of the calculation are given for the first tube to be installed and the second tube to be installed.

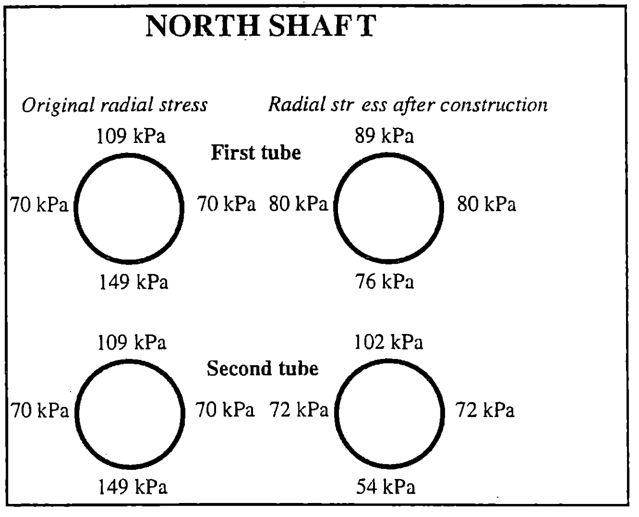

Figure 4. Effective radial soil stresses before and after tunnel installation at northern shaft

The results show a horizontal ovalisation of the tunnel i.e. the horizontal diameter increases and the vertical diameter decreases due to deformation of the tubes. Some influence was found of the buoyancy of the tunnel. Due to the buoyancy some soil reactions are mobilised at the top of the tube, which enlarges the horizontal ovalisation. The difference in the (vertical) water pressure acting on the top of the tube and the (horizontal) water pressure acting on the side of the tube at centreline causes a relative vertical ovalisation. The dead weight of the tube and the effective soil stresses cause a relative horizontal ovalisation.

In the calculations an uplift of the tunnel of approximately 20 mm is found. This uplift is partially caused by swelling of the soil layers below the tunnel due to decompression as a consequence of the removing of the effective soil weight in the tubes. Because the layers directly below the tunnel are sand layers, it is likely that this part of the uplift has already occurred before hardening of the tail void grout.

A distinct interaction was found between the two tubes, although the spacing between the sides of the tubes was more than 0.5 times the outer tunnel diameter. Due to horizontal ovalisation of the first tube, the effective horizontal stresses increase at the location of the second tube before installation of this tube. This results in lower horizontal deformations of the second tube in comparison with the first tube and therefore lower bending moments. A horizontal ovalisation of the first tube of approximately 2 times 9 mm was found and approximately 2 times 5 mm of the second tube. Due to the buoyancy of the second tube, vertical effective stresses on top of the first tube are reduced. Also the distribution of the stresses acting on the first tube changes due to the installation of the second, tube which causes an increase of the maximum bending moments in the continuous beam model by approximately 10%.

4.2 *Results Oude Maas crossing*

At the Oude Maas crossing a water pressure of 199 kPa was acting on the top of the tubes and 282 kPa at the bottom of the tubes.

In figure 5 the effective, radial soil stresses acting on the lining at the end of the calculation are given for both tubes.

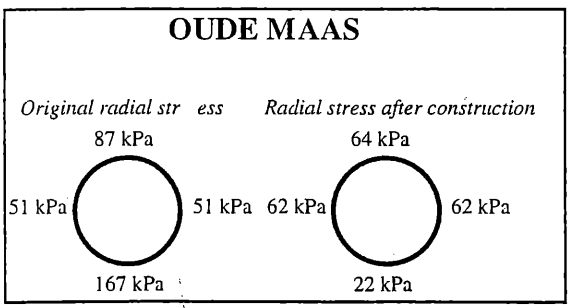

Figure 5. Effective radial soil stresses before and after tunnel installation at Oude Maas crossing

In this calculation a horizontal ovalisation of the tube was found of approximately 2 times 6 mm. The bedded beam model would predict a vertical ovalisation. Due to the buoyancy and the dense layer overlying the tunnel, a horizontal ovalisation was found. The spacing between the tubes was 1 outer tube diameter. No noticeable interaction between the tubes was found. The uplift of the tubes was about 20 mm. Due to the buoyancy force effective soil stresses at the bottom of the tunnel reduces to a low value of 22 kPa.

4.3 *Results Dutch soft soil profile*

The calculations for the Dutch soft soil profile have been executed with more advanced soil models. The clay and peat layers were analysed with the modified Cam-clay model. For the sand layer a trilinear model was used containing two parallel Mohr-Coulomb fractions in order to make a more adequate schematisation of the behaviour under triaxial conditions. This is visualised in figure 6a.

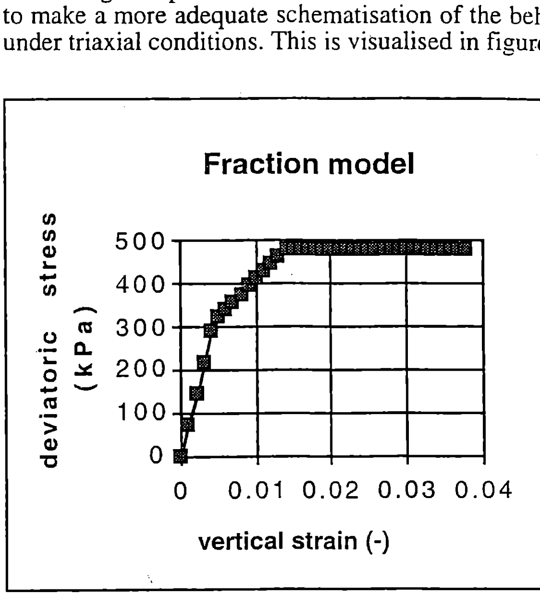

Figure 6a. Results of triaxial test simulated with 2 parallel Mohr-Coulomb fractions

Table 5. Soil parameters Dutch soft soil profile (1)

Soil type	Top of layer (m to NAP)	ϕ' (degr.)	c' (kPa)	K_0 (-)
peat/clay	-1.1	21	3	0.65
clay	-6.0	22.5	5	0.60
sand	-13.0	35	0	0.50
clay/peat	-27.8	29	10	0.70

Table 6. Soil parameters Dutch soft soil profile (2)

Soil type	Unit weight (kN/m^3)	κ (-)	λ (-)	ν (-)
peat/clay	11	0.40	1.6	0.2
clay	14	0.14	0.55	0.2
Soil type	**Unit weight (kN/m^3)**	**E_1 (kPa)**	**E_2 (kPa)**	**ν (-)**
sand	20	55000	17000	0.30
Soil type	**Unit weight (kN/m^3)**	**κ (-)**	**λ (-)**	**ν (-)**
clay/peat	20	0.009	0.055	0.35

Young's modulus for deviatoric stresses between 0 and 50% of the maximum deviatoric stress is equal to the sum of the Young's moduli of both fractions; Young's modulus for larger deviatoric stresses is equal to the Young's modulus of the second fraction. The magnitude of the soil parameters, used in this calculation, is given in tables 5 and 6.

In these tables κ represents the swelling index and λ the compression index. E_1 and E_2 represent Young's modulus of the first and second fraction. For the shallow peat/clay layer and the clay layer an OCR of 1.05 has been applied; for the deep clay/peat layer an OCR of 1.6 was adopted.

The ground water table was taken at NAP - 1.9 m. In the deep sand layer the piezometric head is approximately 3 m higher than the ground water table.

The concrete tube has a diameter of 9.8 m. The top level of the tubes was NAP - 16.2 m and the bottom level NAP - 26.0 m. The spacing between the tubes was 9.8 m. To incorporate the effect of reduced lining stiffness due to segmentation a Young's modulus of the lining of 10.8 GPa was used, which is approximately 1/3 of the concrete stiffness. The unit weight of the concrete lining has been taken 24 kN/m^3. A full bond between tunnel and soil was assumed.

In this situation the vertical effective stress at the top of the tubes is low, so the safety with respect to buoyancy is low. In figure 6b the radial effective stresses before and after the installation of the tunnel are given.

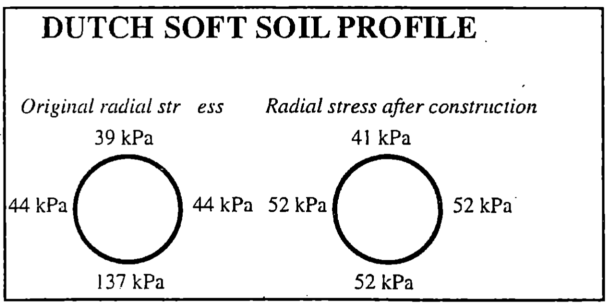

Figure 6b. Effective radial soil stresses before and after tunnel installation for Dutch soft soil profile

The results indicate a horizontal ovalisation of the tubes. Because the loading is relatively low, the calculated horizontal deformation is less than 1 mm. A vertical deformation of approximately 20 mm was found. Significant decompression of the soil below the tubes occurs due to the removal of the effective soil weight in the tubes and the buoyancy force.

The effect of decompression due to volume loss before hardening of the tail void grout is not modelled here. The soil below the tubes consists of stiff clay with a very low permeability of approximately 10^{-11} m/s. Calculations show that the vertical deformations of the tunnel due to decompression of the clay occurs predominantly within a period of approximately 2 years. Therefore the clay layer below the tunnel will behave almost undrained before hardening of the grout, so most of the swelling will occur afterwards.

The vertical effective stress at the top of the tubes is increased, despite the horizontal ovalisation. This is caused by the buoyancy of the tubes. The increase in vertical effective stress at the top is low (about 2 kPa). The vertical effective stresses at the bottom of the tubes is almost equal to the sum of the original vertical effective stress at the top of the tunnel and the stresses, caused by the dead weight of the concrete lining. Because a full bond between tube and soil was assumed, a shear stress at the side of the tunnel of almost 20 kPa was generated. If the influence of the shear stresses is removed, the distribution of the stresses is almost in accordance with the assumptions of the bedded beam model for shallow tunnels.

To obtain quantitative information of the modulus of subgrade reaction due to horizontal ovalisation, a horizontal, outward deformation of 20 mm was imposed on the two nodes at the level of the centreline of the tube at the end of the aforementioned calculation steps. The calculated deformations and stress changes at the interface between lining and soil have been used to determine the magnitude of the modulus of subgrade reaction. The results are given in figure 7.

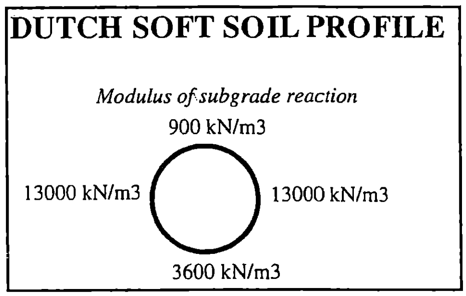

Figure 7. Calculated modulus of subgrade reaction

The results show that the modulus of horizontal subgrade reaction at the sides of the tubes is lower than the expected value of E_{oed}/R, where E_{oed} the oedometer modulus represents and R the radius of the tube (Ahrens, 1982). In this case only 65% of the expected value was calculated. This may be caused by the limited thickness of the sand layer above the top of the tunnel (3.2 m) in comparison with the tube diameter. This type of calculation has been executed for several soil conditions. In general, for shallow tunnels a modulus of horizontal subgrade reaction is found of between 0.5 E_{oed}/R and E_{oed}/R.

The modulus of vertical subgrade reaction at the bottom of the tubes is relatively low. In the calculations the behaviour of the deep clay/peat layer was described with the modified Cam-clay model. This means that stiffness and settlement/swelling is proportional with the isotropic effective stress. Although the soil below the tunnel behaves as relatively stiff, over consolidated soil, the stiffness reduces with decreasing isotropic effective stress.

4.4 *Tunnel in soil liable to settlement*

For the Dutch soft soil profile an analysis has been made of the consequences of applying a tunnel with the aforementioned dimensions, which is partially located in the soft layers. This situation occurs just behind shafts in order to limit the depth of the shafts. In that case the weight of the soil overlying the tunnel is insufficient to withstand buoyancy of the tunnel. One of the solutions is to increase the weight of the overlying soil by placing a sand fill or replacing the soft layers by sand. Placing of a sand fill on ground surface causes significant settlement. Also, the soil around the tunnel would be compressed significantly, even if excess pore water pressures are almost eliminated by using vertical drainage and a certain preload time. For instance due to a sand fill with a thickness of 3 m and a preload time of 1 year, using vertical drains, a settlement after installation of 0.20 m due to secondary compression would be expected at the top of the tunnel and almost no additional settlement of the deep sand layer, assuming that half of the 9.8 m diameter tunnel is located in the soft layers. Because the bottom of the tunnel is located in very dense sand, the bottom of the tunnel will not follow the vertical deformation of the soft layers. The consequence is a high load at the top of the tunnel and/or large deformation of the tubes.

Removal of the soft layers would be a better solution. However, if all soft layers would be removed and replaced by sand, flooding of the polder would occur, because the piezometric head in the deep sand is higher than ground surface. Therefore approximately 1 m of soft, impermeable layers will have to remain. The solution, in which all soft layers except for the deepest 1 m are removed and a preload during 1 year is applied, still leads to a differential settlement between the soil at the top of the tunnel and the dense sand of approximately 30 mm.

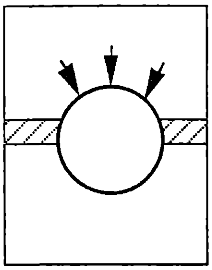

Figure 8. Tunnel in soil, liable to settlement

Due to the differential settlement between the soil at the top of the tunnel and the dense sand, additional vertical effective stresses are generated at the top of the tubes (see figure 8). If the tunnel would be infinitely stiff, the total effective stress on top the tubes can be larger than the neutral soil loads, due to arching. A fair estimation of the additional stresses was found by multiplying the calculated deformation of 30 mm by the modulus of subgrade reaction determined for a situation with a sand fill overlying the tunnel. The calculated additional stress corresponds with the stresses obtained from Leonhardt's method (ATV, 1978), applied for dimensioning of pipelines in soil which is liable to settlement. The sum of the original vertical effective stress and the additional vertical effective stress due to settlement has to be applied in the bedded beam calculation. Due to ovalisation of the tubes, calculated with the bedded beam model, the loads at the top of the tunnel may reduce.

5. CONCLUSIONS

Some remarks were placed on the bedded beam model, based on experiences obtained from results of finite element calculations. The results show that the buoyancy force may be important for the design of the lining. Also, neglecting the subgrade reaction at the top 90 degrees of the tubes is not always conservative. Various calculations show that the modulus of subgrade reaction may be significantly lower than according to the bedded beam formulae. It is concluded that the performance of finite element calculations for normative situations is recommendable in order to determine the modulus of subgrade reaction and to investigate complex phenomena.

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