ULTIMATE LIMIT STATE DESIGN FOR LININGS OF BORED TUNNELS

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Abstract. According to modern design codes, such as the Eurocode 7, also Soil Retaining structures should be designed according to Ultimate Limit State analysis, see [1]. *From an economic point of view, the design of the lining, i.e. the thickness and reinforcement of it, would be optimal if the loading during construction is less critical than the loading during service life, i.e. to the overburden loading. If necessary one could take measures to prevent that any loading during construction becomes critical compared to the overburden loading. If this can be achieved, the structural design would be limited to establishing the overburden pressures, and calculating equilibrium between lining strength and overburden. For optimisation one could use Probabilistic theory and Risk Analytic techniques to establish a sufficient distance between actual loading and design parameters to get the most economic lining thickness and reinforcement.*

In order to evaluate our present situation with respect to lining design, some observations from engineering practice are discussed: the first, from the construction of the 2nd Heinenoord tunnel, where the damage to the lining during construction was above average; the second from the construction of the Green Hart tunnel where measurements show, that flexibility of the tube and the influence of interaction between structure and underground can also lead to critical loading conditions for the lining.

Finally the analyses and observations are generalized and some conclusions with respect to lining design are drawn.

1   INTRODUCTION

In structural engineering it is common practice to design the structural members of a building, based on an equilibrium stress state for the whole structure, and an elasto-plastic limit state analysis for the member's cross-section. The stress-state applied may well have been derived from an elastic analysis. This method is founded on, Drucker's postulate, see [2], which states that the equilibrium state of stress will guarantee a lowerbound for the failure load, i.e. the ULS bearing capacity. This methodology in general leads to a clear and well-defined

approach that is well understood and accepted and leads to safe buildings and an economic design.

With the introduction of Eurocode 7, one has tried to introduce a similar clear concept, also for geotechnical design. However, for buildings we have the advantage that in general there is a clear distinction between loads and strength, whereas for geotechnical engineering, loads and strength are less clear separated; more about this in the next section.

Furthermore with respect to design methodology; as the overburden pressure is the main loading for the tunnel during it's lifespan; from an economics point of view it would be advantageous, if overburden is also the critical loading compared to loading in the construction phase, otherwise one would need to introduce reinforcements to the concrete, that apart from the construction phase would never be loaded up to its design stresses afterwards. That would be a waist of material; better to introduce some adjustment to the construction method, to prevent that the construction loading is the critical loading.

In this paper an overview of present day engineering practice is given, where among other things it is observed that lining design is mainly based on characteristic values for the soil stresses and pressures on the lining. Further some critical observations from engineering practice are shown.

In addition to that, some Ultimate limit states are discussed that can be further developed in order to introduce ULS design for the lining. Only after the Ultimate limit states for tunnel design are established one can think of numerical modelling these; the ULS states themselves cannot be established within the framework of Numerical analysis only.

Further some observations from engineering practice are described and evaluated in the light of the Ultimate limit states given in section 3. Finally some conclusions are drawn.

2   PRESENT DAY ENGINEERING PRACTICE IN TUNNEL DESIGN

Engineering practice in tunnel design is that dimensions and reinforcement are mainly based on experience. The lining thickness is a constant ratio to the diameter. This lining thickness is used to analyse several structural mechanisms of the lining. Reinforcement is calculated according to building codes, i.e. applying a safety factor of 1.7. The critical construction stages usually identified are:

1. Axial jack forces
   - compressive strength of the concrete
   - splitting strength under and beside jacks
   - eccentricity analyses
2. Tangential (ring) forces
   - compressive and splitting strength of the longitudinal joints
   - bending moments
   - joint rotations
3. Radial forces
   - so-called coupling forces caused by dowel and socket or bolting systems

The structural mechanisms for tangential and radial forces are analysed in single and double ring modes. These are well known calculation schemes in literature. The goal of the models is to calculate values that can be (unity) checked: the tangential bending moment to be checked with the bending moment capacity; the joint rotation to be checked with the rotation capacity, etc. This method does not explicitly includes the check on the Ultimate Limit State of the structure as a whole. The question, "*When will the tunnel really fail*" is not answered; there is no clear safety concept. Some issues of the current design approach will be discussed further.

The structural ring models used in the analyses mainly consist of the following parts:

- Soil.
- Concrete segments in rings.

- Joints between segments in a ring.
- Structural couplings between adjoining rings.

It turns out that the soil loads the tunnel, the tunnel deforms and pushes (or releases) the soil and the soil therefore supports the lining. In fact, the soil loads and supports the concrete structure. Soil is found at the force and at the resistance side of the structural model.

The remaining structural forces are a result of the stiffness ratio between the ring and the soil. If the ring gets stiffer, the larger the forces that remain in the ring. If the ring gets weaker or less stiff, less force remains in the ring.

A somewhat peculiar result of this mechanism was described by Mendez Lorenzo [8]. Who found that for a given lining thickness that is marginally increased, the tangential bending moment also increases. However, the increase in moment capacity with the increased thickness was less than the increase in bending moment, leading to a lesser safe structure!! Based on this observation it would be advantageous to take a thinner lining! (up to the point that the lining fails in compression).

When looking at engineering practice it is found that in general the lining thickness still has a constant ratio to the tunnel diameter. Despite all engineering effort put into this, there still is no convincing argument found why the constant ratio is so dominant in the design. On the other hand some new research [e.g. 4] has given prove why this practical constant ratio fulfils. In a later section attention will be given to this phenomenon.

Another issue in the design is the safety concept. In axial direction the failure mechanism is unified clear. When axial jacking forces cannot be resisted by the concrete segments, the tunnel is pointed to fail. For the ring behavior (tangential forces) the philosophy of lining failure is less clear. Most heard argument is that the lining would collapse if no proper reinforcement is put in to bear the bending moments. This argument however is not supported by analysis or tests. In [3] and [4] arguments are given against this. Still engineering practice judges that the lining capacity fails short when the first plastic hinge occurs for the bending moment. Arguments against this are that the tunnel underground interaction is geometrically and physically highly non linear. In the above, arguments are given, to decrease the lining thickness in order tot increase structural safety, however we need to evaluate for what conditions these arguments are valid.

Moreover, not mentioned yet, it is debatable why coupling forces (dowel and notches, bolts) would have to be checked or incorporated in the analysis for the Ultimate Limit State. The main mechanism is ring behavior and coupling of rings would only locally disturb the internal forces. If a coupling fails, the ring structure will act as if couplings were never there. The main experts view on ring couplings is that these should act during construction.

With respect to safety, the application of partial safety factors turns out to be complex and therefore quite often one chooses to apply overall safety factors. This complexity can be explained on the ring behavior due to the soil load. In general the soil loading both causes a tangential normal force and a tangential bending moment in the segments in the ring. The tangential normal forces being beneficial to the bearing capacity for bending moments. For structural engineering in general, the regular safety approach is that load that is of benefit for the bearing capacity, is multiplied by a factor that lowers the influence of the benefit (e.g. $\gamma = 0.9$). Whereas load that causes internal forces that have to be beard are multiplied by a safety factors that increases the influence (e.g. $\gamma = 1.5$). One can understand that the dual role of soil loading on the lining complicates the application of this concept.

In case of reinforced concrete the material factor itself is also under discussion. Remember that the material factor for concrete (e.g. 1.1) might differ from the value for reinforcement steel (e.g. 1.15). Finally the overall safety factor on the bending moment is the multiplication of the material and the load factor (resulting in e.g. 1.7). In a similar approach the check for hoop force strength at compression is executed with an overall safety factor of (more than) 2, which is recognized from e.g. the DIN 1045.

It is concluded that the current design practice does not show an optimization process to decrease the lining thickness. The Soil structure interaction complicates the application of partial safety factors. In general dimensions are based on engineering judgment and experience. On the other hand the amount of concrete and reinforcement are of significant influence to the direct costs of a project and therefore should be optimized.

Finally despite all effort put into this issue, the segmental lining thickness still shows a constant ratio to the tunnel diameter.

3 THE MEANING OF ULTIMATE LIMIT STATE; PRINCIPLES

In [3], a number of limit states in the underground are described that combined with limit states in the lining may lead to an Ultimate Limit state for a tunnel. Based on this, in general two main mechanisms can be identified,

> a) For Deep tunnels,   h/D >> 3, horizontal ovalisation.
>
> b) For Shallow tunnels, vertical ovalisation.

Whereas deep tunnels in normally consolidated soil, i.e. with a K0 value < 1, are dominated by the overburden pressures, and have a tendency to flatten out. For shallow tunnels on the other hand, due to the nearness of the soil surface and due to its dimensions, the horizontal loading may dominate over the overburden, which leads to a horizontal compression of the tunnel.

For the deep tunnel the horizontal deformation leads to the activation of horizontal soil stresses, redistributing the load, see Fig. 1. Due to these deformations, a renewed equilibrium situation will develop, that may give lower bending moment. It is common practice to account for this effect, e.g. applying the models proposed by Duddeck [7], or variations of this concept.

For typical Dutch soil conditions the deflection related to this mechanism may lead to a reduction for the bending moments of about 50 %. The lateral deformations related to the interaction are in the order of, $\delta = r/500$, which with regard to the tunnel function is a more or less negligible deformation.

The question remains whether this method may be regarded as an Ultimate limit state. Would the structure collapse if the capacity calculated acc. to this concept would be exceeded? The obvious answer is; maybe not, if the soil itself does not also yields a limit state, and if the lining has some ductility, there still may be equilibrium states beyond this deformation..
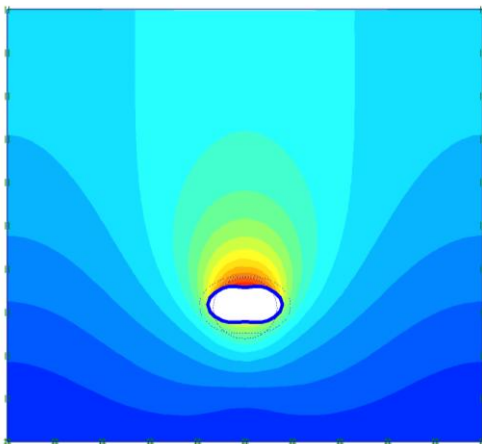

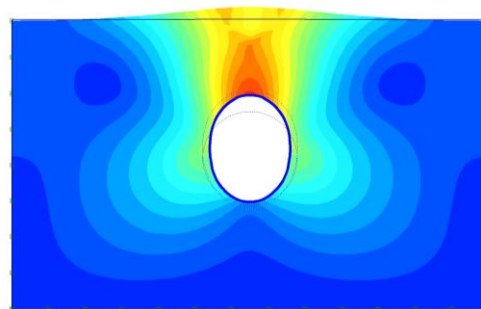Figure 1   ULS Deformation of a deep tunnel, where $K_0 < 0$

Figure 2   ULS Deformation of an undeep tunnel or a tunnel with $K_0 \geq 0$
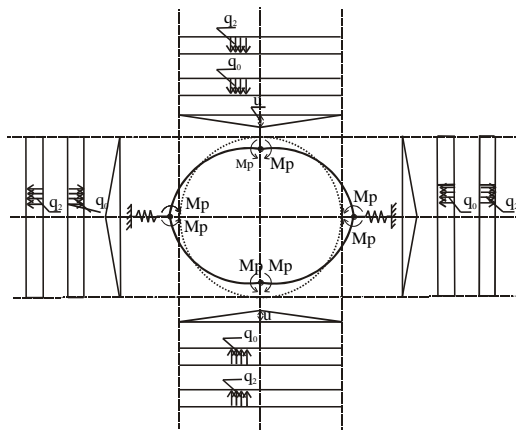
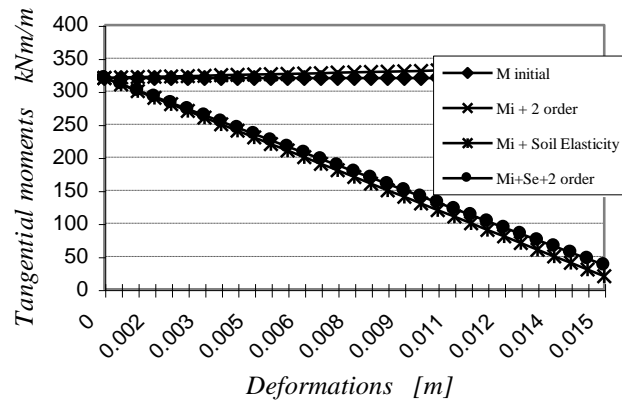Figure 3   Simplified model for the ULS states of a deep tunnel

Figure 4   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 $K_0 = 0.5$

Based on this assumption, an analytic model was developed, and evaluated for typical Dutch soil conditions, see Fig. 3, and Fig. 4. The model shows that the bending moment in the ULS state may depend on the deformation. For an even moderate wall deflection, i.e. of about u/r < 250, the bending moment may have nearly vanished, and the lining stresses that remain will mainly be the hoop forces.

Continuing on this approach, it can be shown that for shallow tunnels, compared to [3], the plastic hinges move upward to the "shoulder". Using a numerical version of the model, it was shown; see [4] that the ring will not fail due to exceeding strength of the lining but to loss of equilibrium. Due to large deformations, the ring cannot act as a ring anymore.

The diagram with Fig. 5 illustrates the snap through of the tunnel. At first, after load increases, the ring acts linearly, then geometrical non linear behaviour develops and at a certain load the first plastic concrete hinge occurs; in this case at the bottom of the ring. It turns out that the tunnel ring still remains capable to bear the load. After the load is further increased a collapse is triggered when at the same time two more plastic hinges occur at the topside of the ring. After that, equilibrium is lost. This is called the snap through. Analysis of the internal hoop forces shows that the segments themselves are still capable to resist the normal force. The final collapse is caused by the disability to act as a ring.

This analysis shows that the actual failure occurs at a load that is over three times the load that causes the first plastic hinge. Projecting this result on the current design practice where it is assumed that the ring structure fails at the activation of a first plastic hinge, this would mean that the implicit safety factor is (more than) 3. This shows that the current design philosophy for tunnels is a safe approach but very conservative. In [4] it is proposed to put up criteria for the maximum deformation that can be allowed, in order to prevent UL states to be triggered due to large deformations, although the deformations related to such a Limit state are much higher than normally observed and in the order of $\delta/D = 1/20$.
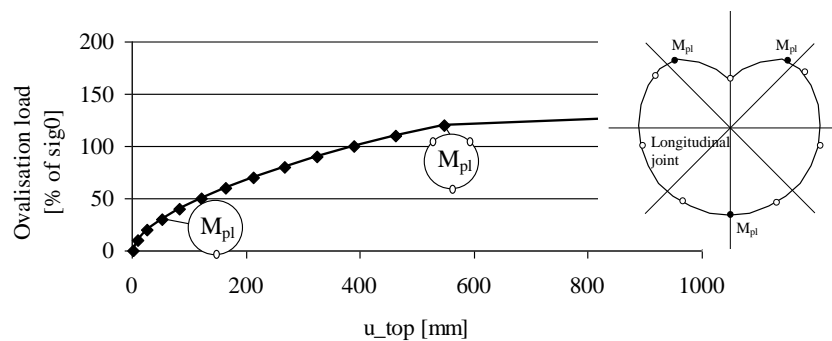

Figure 5   Result of snap through analysis (tunnel diameter ~10 m).

Based on the above one might argue, that if sufficient criteria are put up that restrict the allowable deformations, there would not be a necessity to account for bending moments in the lining, and the lining thickness would not necessarily have to be thicker than necessary to bear the intermediate compressive forces, i.e. σ_0. For that situation it would suffice to know the overburden stress; i.e. the depth and soil weight to derive the dimension of the lining thickness. Based on a simple analysis, considering the maximum compressive stresses in the lining only, and considering a partial safety factor of 2, one finds that a relative thickness of $d/D = 1/72$ would suffice. However, such a thin lining is not observed in engineering practice and as will be shown in section 4.1 even thicker linings have unacceptable damage.

One of the reasons that these results do not coincide is the neglect of large deformations; i.e. buckling is not included in the analysis. For a first estimate one might apply the equations from the elastic solution for the limiting load, disregarding bedding reactions, according to the Euler large deformation analysis, for a circular arch is used:

$$\sigma_{0crit} = \frac{E_c \, d^3}{4 \, r^3} \qquad (1)$$

Where

E_c	= The Concrete Young's modulus
γ_w	= Wet soil weight

Table 1	parameters for the Second Heinenoord tunnel	
$D = 2\,r$	= 8.3 m,	(Outer Diameter)
d	= 0.35 m	(lining thickness)
h/D	= 3 – 5	(rel. depth ratio)
K_0	$\cong 0.5$	

This would yield a lining/diameter ratio of:

$$d / D = \tfrac{1}{2} \sqrt[3]{\frac{4 \, \gamma_u \, \gamma_w}{E_c} \frac{h}{D} D} \qquad (2)$$

For a tunnel under typical Dutch soil conditions, see Table 1, with a partial safety factor for buckling of $\gamma_u = 2.5$ and with a lowest estimate for the Young's modulus of the concrete of about 15000 mPa this would give a relative thickness ratio of about 0.031; i.e. $d/D = 1/32$, which is much lower and compares better with the empirical rule of thumb of $d/D = 1/22$.

Discounting for the safety factor, this analysis is more or less in agreement with the work by Mendez-Lorenzo for ITM, that was mentioned before. Who looked into the design of Steel Fibre Reinforced tunnel lining, for the relation between the reliability index as a function of the lining thickness. It was found that below a thickness ratio of about $d/D = 1/40$ the reliability index drops due to large deformation effects, see [8]

In the next section some observations from engineering practice are discussed in order to evaluate the preliminary conclusions as formulated above, compared to the observations.

4   OBSERVATIONS FROM ENGINEERING PRACTICE

4.1   Lining damage as observed during construction of the 2nd Heinenoord Tunnel

At the start of the tunnel boring process, (with a constructed tunnel length less than 100 m.), damage to the lining was higher than expected and not acceptable. After investigating the damages, Leendertse [9], reported on the observations and drew some conclusions:

Damage patterns
1. *Many ring joints were subjected to differential deformations that displace up to 30 mm. The differential deformations on the longitudinal joints within one ring are much smaller.*
2. *In many places there was leakage between adjacent segments. There is not a clear relationship between the observed differential deformations and the amount of leakage water.*
3. *Some of the segments showed slight cracking, over about half of their length. This cracking extended over the whole width of the segment. Some of these cracks were diagonal. There*

was some slight leakage through these cracks, but this leakage appeared to self heal.

4. *Some segments show darkening (wetting) of the concrete surface. Without any visible cracks.*

5. *At some places corners of segments were broken. The broken corner was always on the side facing the tunnel boring machine.*

6. *In many places along the tunnel lining edges of segments have snapped off. In all cases this occurred at the TBM side of the segment. The damage concentrated near the notches and dowel zones. The thickness of shales coming of from the ring surface might be up to 0.1 m. At some places reinforcement steel is visible due to this type of damage*

7. *On more than one occasion, the edges of segments adjacent to the key segment (the closing segment of the tunnel ring), became damaged. Often the damage to edges of the key segment has extended to the entire segment width.*

8. *The observed damage was not exclusively concentrated near the key segment but appeared, to a lesser degree at the sides of the tunnel too. Apart from situations where edges snapped of, most of the damage was concentrated on the dowel and notch locations. There was a strong feeling that the locations of leakage were correlated with places where the back wall of a notch was overloaded and damage to the outer side of the tunnel was supposed, creating a shortcut for percolating water, behind the rubber water sealing.*

Each ring at its curved side is provided with two dowels (and notches), to create a system of interlocking segments. This ensures that there is capacity to transfer shear forces between tunnel rings. On the dowel, Kaubit stripes are placed to reduce the interaction forces.

Kaubit is a very soft material meant to reduce friction (if there is any). In the design configuration the dowel and notch system has a free deformation of 6 to 7 mm. If this space is exceeded, the dowel is loaded. If the loading exceeds the capacity, either the dowel breaks off or the notch fails, often combined with spalling at the lining surface. Here the design had resulted in a system where the dowel was stronger than the notch. This caused the breaking of fragments on either side of the wall; depending on the direction the dowel was loading the side of the notch. The low friction of Kaubit was pointed as one of the causes for the damage.

In order to reduce further damage, the Kaubit was replaced by triplex wood plates to reduce the stresses during ring building. This measure further has performed satisfactorily and was repeated for other tunnels, although there is still doubts about the durability of the triplex and questions to what extent this may have influence the long-term behaviour of the tunnel.

4.2   Observations from the Green Hart tunnel on deformations of the lining

Contrary to expectation the deflection of the tunnel lining was rather a vertical than a horizontal ovalisation. Looking into more detail to the loading situation during construction,
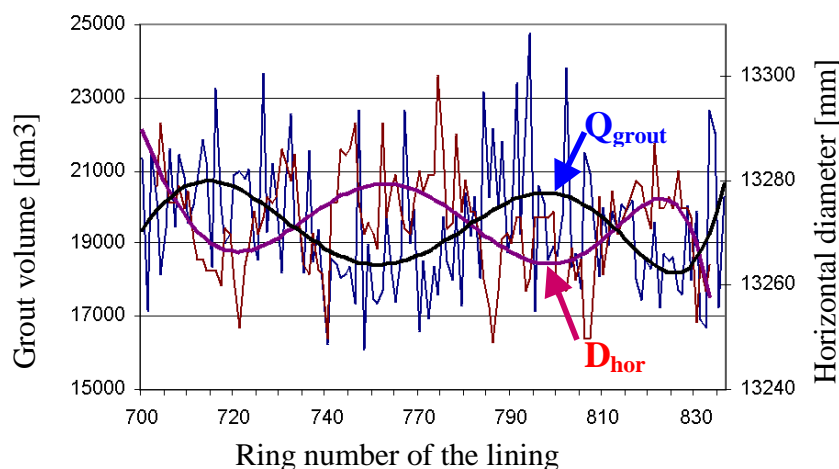
Figure 6 Relation between injected grout volume and deformation of the tunnel lining, acc. to Warmerdam [11].

i.e. the tail void grouting, it was supposed that the grouting pressure may have exceeded the weight of the soil above the crown of the tunnel, triggering the deformation mode that normally is considered for a shallow tunnel, vertical ovalisation of the cavity, see Fig 2. For that situation, a different mode of failure would have to be evaluated, i.e. a breaking up of the soil above the tunnel, see [3]

Apart from the tail void grouting, the local geology may also have contributed to this mechanism. Although the tunnel is located firmly in the stiff Pleistocene sand layers, it is overlaid by a 10 to 15 m thick layer of very soft and relatively light Alluvial peat and clay. Locally this soft upper layer may have lacked the weight to resist vertical ovalisation.

The influence of grouting can also be illustrated by the work of Warmerdam [10]. Who analysed segmental damages in relation to the construction stage. Fig. 6. shows the correlation found between injection volume of grout and the deformation of the lining. The same correlation was also found in [4].

A detailed evaluation of all Ultimate limit states that might correlate to tail void grouting is beyond the scope of this paper, but a partial mechanism that might be triggered in the process would be an overloading of the friction in one of the longitudinal joints due to the local shear force in this joint.

5   EVALUATION OF THE STRUCTURAL OBSERVATIONS AND FURTHER RESEARCH RESULTS

For a better understanding of the observed damages at the 2nd Heinenoord tunnel, three mechanisms are recognised and analysed:

- *Compression of the ring due to loading, when a tunnel ring leaves the tail of the TBM machine.*
- *Ovalisation of the ring due to the distortion part of the loading on the ring.*
- *Inaccurate installation of the segments of a tunnel ring*

With respect to quantitative comparison between these deformations, see [3], it was found that the radial deformation due to compression of the ring is negligible, less than $\delta < 0.00025$ m, and the ovalisation to adjust to the K_0 soil stresses may give to an ovalisation in the order of 0.005 m, which is not negligible but small. As these two mechanisms can't explain the differential deformations as observed, it was concluded that inaccurate installation must have been the cause of the damages, see also [11]

This preliminary conclusion, and considering that at that moment, more than 6 more bored tunnels where under design in the Netherlands, made the Ministry of Public Works decide to do additional Numerical and physical testing on tunnel lining behaviour.

These analyses showed that the largest displacements are triggered by inaccurate installation, and to a lesser degree by ovalisation. In the tunnel itself, tilting of segments with respect to the axis perpendicular to the tunnel axis, in the horizontal plane, was also observed. This mechanism also contributes to the displacements. Tilting might be triggered by the step-wise development of the grouting pressure on to a segment as the tail of the TBM moves forward.

Further looking into the construction process in more detail, during TBM progress the lining is simultaneously loaded by the Jack forces and by the viscous grout material that is pumped into the tail void to avoid moving in of soil into the tail void gap. This construction stage is analysed in more detail and described in [4] for saturated soil and results in the so-called 'uplift load'. When the tail void is fully filled up with grout, the tunnel behaviour turns out to be quite similar to a tunnel embedded in soil. However deviations might occur, e.g. if the grout is not able to be quickly de-saturated (remains soft viscous during a longer period) or in case that the void around the tunnel is not fully filled up with the grout material (incomplete grouting). In that case the Archimedes forces load the ring. The equilibrium of forces is found after displacement of the tunnel due to buoyancy. Since in this situation the
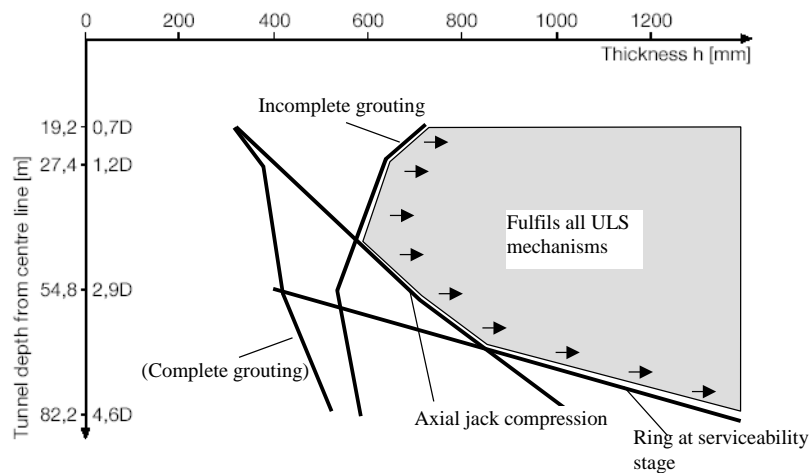
Figure 12 Illustration of ULS mechanisms that are decisive at different load conditions (internal tunnel diameter 14.9 m) [5].

soil is not properly able to support the ring, the internal forces might increase as a function of the deformations that are necessary to find the equilibrium against the buoyancy forces. Extended numerical analyses in [4] showed that this load case will result in a required lining thickness of d/D = 0.05.

Based on the different ULS theories for axial directions, construction stage with uplift forces and the final situation during serviceability, in [5] a large diameter tunnel has been analysed. It turns out that at different overburden situations, different structural mechanisms are normative, see Fig. 7.

From the graph it is clear that there are three mechanisms that can determine the minimum lining thickness. With low overburden the incomplete grouting situation is decisive. At medium overburden the compression strength by the jack forces is decisive and at high overburden the serviceability stage gets normative. Furthermore it should be observed that this analysis has been performed for a tunnel with an internal diameter of 14.9 m. Based on the rule of thumb, the required minimal lining thickness should be around 700 mm. It is concluded that up to a depth ratio of 3 times the diameter this rule of thumb is a safe approach.

6   CONCLUDING REMARKS

The cost for a tunnel is for a large part, sometimes up to 40 % taken by the cost of the lining. Any cost saving in the lining will lead to a cost saving for the user of the infrastructure; therefore it is attractive to look at cost savings in the lining.

In section 4 of this paper it is explained that for the Ultimate Limit state under overburden pressure only, the preliminary conclusion was drawn that for overburden loading only, there is no necessity to account for lining bending moment; accounting for the hoop forces would suffice.

However, in section 5 it is explained that during construction, compared to the loading by overburden pressures, the loading may be more complex, and the designer does not always has control on the loading during construction. The contractor in its efforts to avoid too much surface settlement might apply grouting pressures that are higher than considered in the design phase, pressures that may lead to stresses in the lining much higher than caused by the overburden pressures only. Loading situations during construction, that up to now is the prerogative of the contractor and out of the control of the designer. And if the latter does not have a grip on the construction loading a conservative approach is understandable.

Due to the limitations as mentioned at present, the construction loading is often dominant over the soil loading, and therefore the indicative design rules that relate overburden pressures with lining thickness cannot be applied. What remains is that these equations explain that in general the thickness ratio should increase with the depth ratio, and with the diameter.

For the ULS verification at least three structural mechanism should be analyzed: concrete compression due to jack forces, tangential bending due to incomplete grouting and tangential bending due to soil loads at the serviceability stage.

It has been shown that for tunnels with a depth ratio h/D less than 3, that the current design practice to apply a linear elastic approach and the design rule of d/D = 0.05 is conservative.

In that light the present engineering practice to apply a relatively high partial safety factor in DA2 for the cross-sectional loading of a concrete wall is understandable. On the other hand if it would be possible to get a better control on the other loading conditions, i.e. during construction, it would be a challenge to optimize the applied safety factor with respect to the bending moments that is considered for the soil loading.

Finally, if the choice is made for ULS design, before starting any development of structural engineering models, one should determine the real mechanisms that cause failure. It should be checked that the models that are applied in design are capable to describe these limit states. One may not rely upon the expectation that any numerical model with a non-linear constitutive relation for the soil, will deliver the Ultimate Limit states that the designer forgot to think about.

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