Geotechnical Aspects of Underground Construction in Soft Ground – Yoo, Park, Kim & Ban (Eds)

TBM face stability & excess pore presssures in close proximity of piled bridge foundations controlled with 3D FEM

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ABSTRACT: The Amsterdam North/South Metro line is a challenging project in an unfavourable urban environment of very soft soil and over 1000 historical buildings founded on wooden piles close to the tunnels. The two shield driven tunnels are 6.5 m in diameter. At Bridge 404 the tunnels are stacked, thus the tunnels pass under the bridge at different depths. Bridge 404 is a historical bridge founded on brick abutments, which in turn are supported on driven wooden piles 13 m below surface level. The crown of the deep tunnel is situated 12 m below the pile tips but the shallow tunnel has a clearance of only 1.5 m below the pile tips. As the cover under the canal is reduced and the loads of the piles have a significant impact on the stability of the bore front, face stability was not guaranteed by means of the standard analytical methods. The design team was faced with a technically not feasible TBM passage. Therefore, advanced 3D numerical simulations were used to develop the TBM process parameters, making the passage technically feasible and to design the mitigating measures. Advanced monitoring results of excess pore pressures originating from the TBM face of the deeper Western tunnel were integrated in the 3D FE models for the Eastern tunnel. The paper describes the comparison between the analytical methods and 3D numerical simulations of the face stability, the results of the excess pore pressure measurements and the settlement performance of the TBM.

1 INTRODUCTION

The Amsterdam North/south Metro scheme includes 3.8 km of twin tunnels with a diameter of 6.5 m. The tunnels are constructed using a mixshield tunnel boring machine to reduce volume loss and disturbance to the environment. Along the tunnel route approximately 1000 historical buildings, mainly founded on wooden piles, are present. The piles are driven through the very soft clays to the 1st sand layer. At locations where the tunnels are curving, volume loss was expected to be higher and the design recognized that at these locations, despite the use of a very advanced TBM, mitigation of tunnelling settlements would be needed (Kaalberg et al, 1999). In total seven locations required the use of compensation grouting to mitigate settlements (Kaalberg et al, 2011). In general the crown of the TBM was at least 4 m below the pile toes, however at Bridge 404, the crown of the east tunnel passes only 1.5 m below the pile toes and a special strategy needed to be developed. At this location both permeation grouting and compensation grouting were envisaged in order to improve the ground and to heave the bridge in case of unacceptable settlements (Kaalberg et al, 2012). Besides the challenge in mitigating the settlements of the bridge, the short distance between the crown of the TBM and the pile tip faced the engineers with a theoretically unfeasible passage (i.e. potentially collapse of the bridge due to instability of the excavation face) of the bridge when the generally accepted face-stability calculations by the DIN 4085 would have been used.

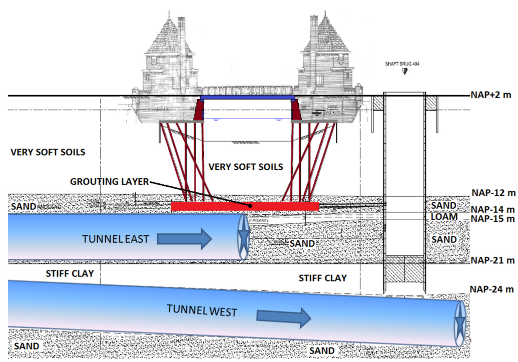

Figure 1. Geometry Bridge 404.

2 THE ENGINEERING CHALLENGE OF TUNNELING UNDER BRIDGE 404

With respect to the TBM Transit situation three issues needed to be prevented at this location:

- Structural bridge damage due to settlements caused by tunneling
- Face instability due to low face pressure
- Surface blow out caused by a high face pressure

Regarding the 1st issue a 3D bridge response analysis was carried out to evaluate the soil structure interaction of the bridge abutments in order to determine if the predicted settlements would be reduced due to the structure interaction (Kaalberg et al. 2012). Initial calculations run with a flexible foundation indicated that the predicted settlements of the bridge abutments would reach the predicted 40 mm (virgin) subsoil settlement at pile toe level. When the pile stiffness and deck structure were introduced, settlements of the abutments were significantly reduced, but still ground improvement and compensation grouting above the tunnel crown were considered to necessary, as at earlier full scale trials it was found out that wooden piles with end bearing capacity can settle more than the subsoil due to stress relieve around the pile toes (Kaalberg et al., 2005). As this is difficult to model, this effect was not incorporated in the above simulations. The 3D soil structure interaction modeling, the design of the mitigating measures as well as the monitoring of the settlement performance of the TBM and the compensation grouting were described in (Kaalberg et al. 2012) and will not be elaborated in this paper.

However, by implementing the ground improvement and the compensation grouting the face stability and settlement issues were not completely resolved. Following discussions with the contractor, a 6 m grouting exclusion zone in front, and behind, of the TBM was agreed in order to avoid any face stability issues.

3 ANALYTICAL FACE STABILITY CALCULATIONS

All the face-stability calculations of the 3.8 km North/south metro were initially made according to the German standard, DIN 4085:1987. To calculate the minimum required slurry pressure, this method assumes a predetermined 3D failure wedge. In a stable situation the active soil pressure on the tunnel-face equals the bentonite pressure in the TBM. The soil pressure on the wedge strongly depends on the arching effect in front of and above the TBM. The amount of arching is taken in consideration by the factor μ_{agh}, which depends on the tunnel depth and diameter. The more recent DIN 4085:2007 has a different approach on determining μ_{agh}. Here the factor also depends on the angle of internal friction. However, this version of the DIN is slightly more optimistic in this case.

By applying the design approach according to the DIN4085, the minimum allowed slurry pressure directly below the pile tips was (much) higher than the maximum allowed slurry pressure in the middle of the canal. This effect was sharpened as the bottom of the canal was at a slope even under the abutments and the front row of the piles were beyond the front line of the abutment, thus creating an extra high pressure on top of the tunnel crown (in the arch) whereas the cover on the crown was app. 7 m less than behind

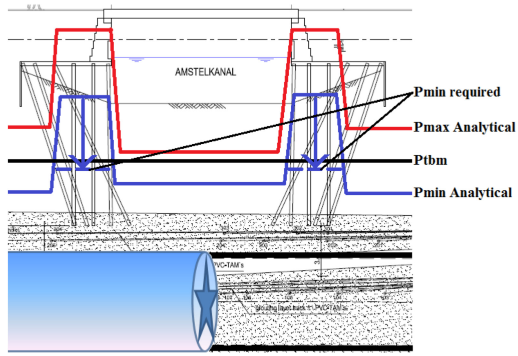

Figure 2. Result analytical face stability calculations: $p_{min} > p_{max}$.

the abutments. This made the TBM passage not feasible since it was considered too much risk to apply very substantial discrete adjustments in face pressure at the transition from abutment to the canal in order to avoid a blow out or cave in, as adapting this face pressure too late/early is likely to happen. On top of this, the TBM control system also required a difference of at least 20 kPa between the minimum and maximum face-pressure. At the early stages of the design this problem was not recognized in such a detail as the vertical alignment was altered in a later stage due to unforeseen rail associated requirements and therefore the space between the foundations and the tunnels was reduced. As the alignment at the time was fixed and tunneling had already started this placed the Clients design team of the bored tunnel for an exceptional engineering challenge, as the vertical alignment of the tunnel was the Clients responsibility. The result of the analytical calculations is shown schematically in figure 2.

From many options explored during brain storm sessions, the design team considered two options viable to make this TBM passage theoretically feasible:

- to increase the strength and density of the soil by means of ground improvement under the canal as well as under the foundations (figure 1 and Kaalberg et al. 2012).
- to decrease the minimum required soil pressure by means of application of advanced 3D FEM calculations (blue arrows in figure 2).

In the end both options were needed to create a safe passage of the TBM. The 3D FEM calculations were conducted in Plaxis 3D 2010. To show that the minimum required face-pressure calculated by the 3D FEM had the same level of safety as the analytical calculations, the entire safety philosophy of the DIN 4085:1987 was implemented in the 3D Plaxis calculations. Since this is not entirely possible some assumptions had to be made. These are described in the following chapter.

The maximum allowable face pressure however was not calculated by means of 3D FEM since this failure mechanism depends mainly on the heterogeneity of the soil. Blow-out can occur when just a small stream of bentonite reaches the surface (f.e. along the pile shafts). Because this in turn would lead to a very rapid pressure drop inside the excavation chamber, then active failure is likely to occur. The maximum allowable face pressure at a blow-out mechanism indicated by 3D FEM will therefore most likely be too optimistic in reality. The maximum allowable face pressure was therefore determined with the analytical method according to the DIN 4085:1987. In this (simple) method the vertical total stress of the soil above the crown of the tunnel has to be more than the isotropic face pressure.

4 SAFETY PHILOSOPHY FACE STABILITY IN PLAXIS 3D

As the design team was aware that only academical comparison studies between analytical methods and 3D design methods were available and no precedent was known where TBM face stability was really designed according to 3D FEM, it placed a high risk on the shoulders of the design team. The aim of the 3D FEM calculations was therefore to show that even though a less conservative model was used, the level of safety would at least be equal to that of the analytical method. To achieve this, all model input was mirrored to the analytical method.

4.1 *Geometry, soil strength and water levels*

There was no difference between the two models on these points. Only the cohesion of 3 kPa that could be applied to the loam layer in front of the TBM shown in figure 1 was reduced to zero in the Plaxis calculations because the sensitivity analysis showed that cohesion had a very (to) favorable influence on the calculation results. Furthermore both models applied the same water level.

4.2 *Safety factors*

The factor on the horizontal earth pressure, h_E was 1.5 in the analytical method. In the Plaxis calculation, this factor was the minimum required result of the φ-c reduction analysis. In the specific case of Bridge 404 this was possible, because only cohesionless soils were present. Therefore, within the relevant reach of the TBM, the lateral active earth pressure coefficient (Rankine, 1856) has a nearly linear relation with the angle of internal friction.

The safety factor on the water pressure, η_w was 1.05 in both models. Both models applied the lower limit values for soil strength and soil weight. The upper limit value for soil weight was used only above the tunnel crown. However, the analytical method did not take any load spreading below the pile tips into account, whereas this would automatically follow from the Plaxis calculations. Therefore, an additional

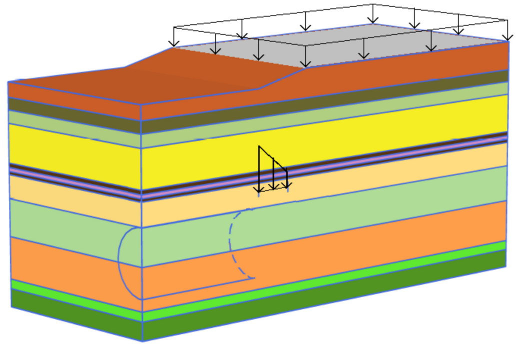

Figure 3. Load distribution applied in the FEM calculations at the pile tip level.

safety factor on the loads from the pile tips was introduced. This factor ηLOAD404 was 1.2.

4.3 *Load from bridge 404*

The load in the analytical calculation had a uniform distribution over the pile tips. This was not the case in Plaxis 3D. For these 3D calculations it was recognized that most of the load from the bridge was transferred to the piles closest to the canal (figures 1 & 2). This resulted in an applied load distribution as shown in figure 3. Note that the top of the model was removed and replaced by a surface load. This was done to simplify the model and prevent the modeling of the actual bridge abutments and piles. Also, all of the load from the piles was placed (conservatively) at the pile tip level, thus no shaft friction along the pile was modeled, providing a higher actual stress above the tunnel crown.

4.4 *Influence soil improvement above tunnel crown*

This was taken to be unfavourable in the analytical calculation, by assuming that no stress distribution would occur. Thus the loads from the pile tips were directly applied to a level just above the tunnel crown in the analytical calculations. In the 3D FEM analysis a sensitivity analysis resulted in a safe design approach in which the effects of the soil improvement with respect to the minimum required face pressure were ignored and the properties of just the virgin sand layer were used. It was also investigated numerically if this ground improvement between tunnels and pile toes would contribute to the face stability, but this effect was not significant as the added stiffness to the subsoil was relatively low compared to the bridge loads. Also the contribution of the ground improvement at the tunnel face to the stability was numerically investigated. It appeared that ground improvement at the bore front did not significantly increase the factor of safety, although it was noticeable that settlements were smaller before the numerical model became instable. It must be stated that the stiffness parameters for the ground improvement were taken conservatively, as it is rather difficult to assess the achieved in-situ stiffness and homogeneity of the injections.

4.5 *Excess pore pressures*

However, the analytical calculations according to DIN4085 (even the 2007 version) do not take the excess pore pressures into account, whereas it was proven earlier in Dutch soil conditions that these excess pore pressures do really occur (as described by Bezuijen et al., 2001 and Broere, 2001). As this DIN 4085 was applied in many tunnel projects world wide and no actual face collapse was published due to this phenomenon, it is likely there is a hidden safety in the analytical models, which compensates for this excess pore pressures (i.e. reducing the real safety factor). When applying 3D FE models this hidden safety could be significantly reduced, and as the responsibility was high it was decided to specifically apply these excess pore pressures in the 3D FE models. This is described in the following chapter.

5 DETERMINING EXCESS PORE PRESSURES

The face pressure is always higher than the surrounding water pressures in normal, static, conditions. The difference is called the excess face pressure. The excess face pressure is fully or partly transferred to the soil skeleton by means of a bentonite cake. To describe this, the factor α is introduced (Bezuijen et al., 2001). This factor determines which part of the excess face pressure manifests itself as excess pore pressures. The factor α is related to the permeability of the soil, the viscosity of the slurry, the actual speed of the TBM and to some extend to the shape of the cutter wheel. Since the tunnel at bridge 404 is fully situated in sandy/silty soils this effect had to be determined for these specific conditions.

Because of the continuous drilling, a flow of water is created which can, in porous conditions, be measured up to a distance over 50 m from the tunnel face. To model this phenomenon simple groundwater flow calculations can be used (Bezuijen et al., 2001). In his theory the increase in piezometric head in front of the tunnel face decreases exponentionally with the distance.

This assumed relation showed a very good fit with measurements of the 2nd Heinenoord tunnel shown in figure 4. To find a proper value for, the maximum excess pore pressures at the tunnel face, it is important to know a realistic value for α. This becomes clear when formula (1) is considered.

$$\varphi_0 = \alpha\,(ps - pw) \qquad (1)$$

Where:

$\varphi 0$ = excess pore pressures at the tunnel face
α = relation between the face- and excess pore pressure
$p_s - p_w$ = the excess face pressure (= the face pressure - the 'static' pore water pressure)

Unfortunately there was not a lot of data available to find or calculate a realistic value for α. This is because

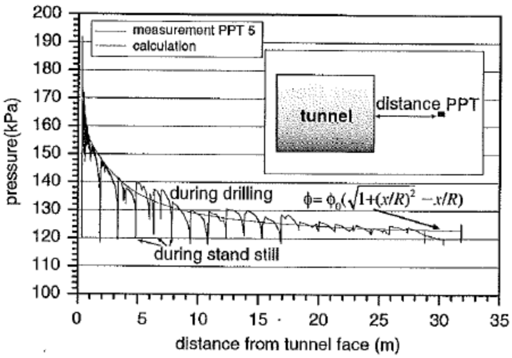

Figure 4. excess pore pressures in front of the 2nd Heinenoord slurry shield and approximation [2].

both the face-pressures and the excess pore pressures should be measured and plotted versus the position of the TBM. For a rough estimate, the measurements from the 2nd Heinenoord tunnel, as presented by Bezuijen, 2001, were used to perform a first generic back analysis. The results of the pore pressure gauges in the centerline of the tunnel at the 2nd Heinenoord tunnel are presented in figure 4. From this figure and the fit with formula (1), $\varphi 0$ was determined. The groundwater level at this position was known and the applied face pressure was estimated by analytical calculations of the minimum and maximum face pressures. This resulted in $\alpha = 0.4$.

The minimum soil pressure strongly depends on the pore pressures in the surrounding soil and therefore also on the excess pore pressures. With $\alpha = 0.4$, it was concluded from the 3D FEM calculations, that a safe tunnel passage at bridge 404 was feasible. The value for α however, as there was a high risk at stake, needed a much stronger argumentation. Fortunately the 1st tunnel was constructed at a lower level than the critical shallow tunnel. Therefore it was decided to install two vertical piezometer arrays (Geobeads by Alert Solutions) between the center lines of the tunnels, ahead of the start of the TBM for the deeper tunnel. The advantage of vertical monitoring arrays was that the piezometric head could be measured at multiple positions at the same spot, and therefore all at the same distance from the tunnel face. The first array was installed at app. 300 m. from bridge 404, here the deeper tunnel was at the same depth (and groundlayers) as the shallow tunnel near bridge 404. The second array was installed right in front of bridge 404, monitoring the pore pressures at both levels of the tunnels. The positions and locations of the piezometers are shown in figure 5.

The measurements of the piezometric head during the 1st TBM passage are shown in figure 6. The measurements were taken with a high resolution of 1 measurement every 10 seconds. Also the face pressures inside the TBM were measured with a high frequency. Because the exact time of each measurement was known the front pressure could be linked to the position of the TBM face, the face pressure and the

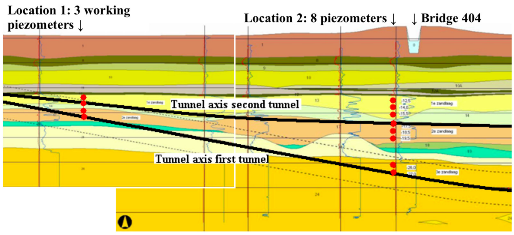

Figure 5. Piezometers North/southline to examine the excess pore pressures.

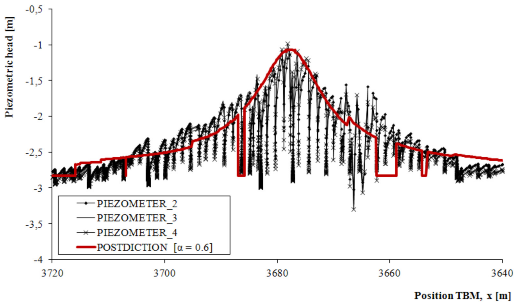

Figure 6. Measurements of the piezometric head of the first tunnel at location 1.

excess pore pressure. The postdiction was performed according to the method of Bezuijen, creating the best fit with $\alpha = 0.6$. The postdiction curve has a round peak at chainage = 3683 m, because the piezometers were located several meters next beside to the tunnel track (and not swallowed by the TBM like in Bezuijen, 2001). The measurements also show that the increase in pore water pressure in front of the TBM is (surprisingly) more or less symmetrical to the increase behind the tunnel face, although one would expect a more significant effect in the front of the tunnel.

6 PREDICTION RESULTS

The analytical model and the 3D PLAXIS model where used to determine the upper and lower limits for the face pressure to be applied.

6.1 *Maximum allowable face-pressure*

The middle of the canal obviously was considered as the critical position of the TBM regarding the maximum face-pressure. This calculation was performed analytically only and resulted in a maximum allowable face pressure of 181 kPa. Since the TBM needed at least a control range of 20 kPa, the maximum minimum required face pressure thus was 161 kPa. The 3D FE models therefore had to show that a face pressure of 161 kPa fulfills all safety criteria.

6.2 *Minimum required face-pressure*

The analytical predictions produced a minimum required face-pressure of 194 kPa (with safety factor

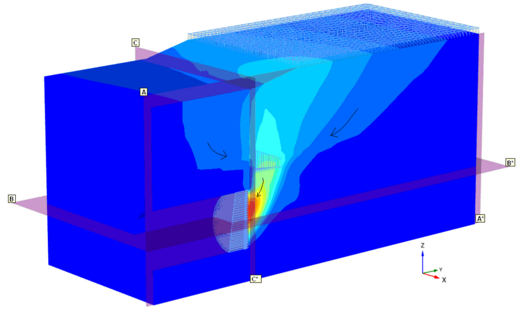

Figure 7. Active failure in 3D Plaxis calculation – note load concentration below pile tips.

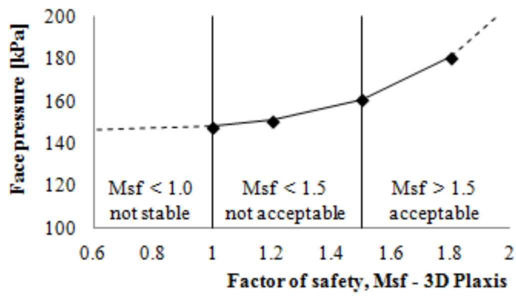

Figure 8. Result stability calculations in 3D Plaxis in ULS.

according to the DIN). An iterative 3D FEM analysis sequence (fig. 7), calculating a φ-c reduction at several face pressures, showed that at a face-pressure of approximately 160 kPa in ULS, a safety factor of 1.5 in φ-c reduction method is reached (fig. 8).

Based on the pore pressure monitoring results of the 1st TBM passage an excess pore water pressure based on $\alpha = 0.6$ ($\varphi_0 = 32$ kPa at 160 kPa face pressure) was included. The safety factor is reduced to 1.0 when a minimum face pressure in the 3D FEM model of app. 147 kPa is applied. Although this φ-c method is widely recognized in geotechnical design it remains somewhat abstracted from tunneling construction practice, so a more practice based method to determine the safety factor in SLS was used as well. Now the face pressure in the 3D model was reduced in steps until face collapse occurred, applying SLS soil parameters. This iterative method indicated an SLS face pressure where instability would occur at 125 kPa. The difference of 35 kPa with the minimum required 160 kPa in ULS was considered safe enough during the tunneling process. Although both design procedures are not academically validated yet, it was felt that the outcome of both prediction procedures gave enough confidence to consider the result as sufficiently reliable.

6.3 *To be applied face-pressure inside the TBM:*

According to the 3D FEM predictions the minimum required face pressure was 160 kPa (with safety factor of 1.5). As the maximum allowed face pressure was only 181 kPa, the available control bandwidth was only

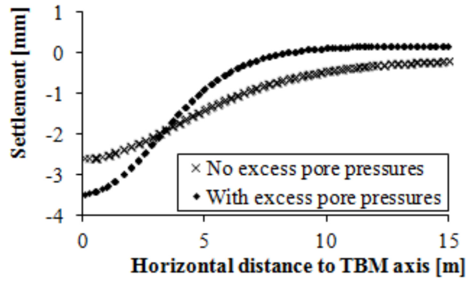

Figure 9. Settlement trough of bridge abutments in front of the tunnel face, perpendicular to the tunnel axis.

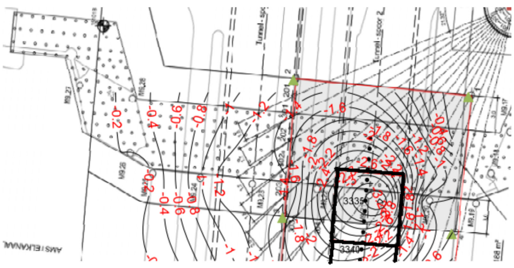

Figure 10. Settlement contours at surface level of southern bridge abutment in front of the tunnel face.

20 kPa. Therefore it was advised to apply a face pressure of 171 kPa in the TBM during the 2nd (shallow) passage, right in between p_{max} and p_{min}.

6.4 *Mitigating measures to be implemented*

In order to reduce the risks for blow out it was advised to implement ground improvement under the canal and abutment. In order to avoid face instability it was advised that heavy trams and lorries were not allowed to pass the bridge during TBM transit.

6.5 *Settlement predictions*

The 3D FEM analysis also produced settlement predictions related to face stability (fig. 9). The predicted settlements at surface level were below 5 mm, which was acceptable according to the bridge abutment response analysis given the fact that in the exclusion zone no compensation grouting was allowed. The effect of excess pore water pressures on settlements was considered not really significant.

7 TBM TRANSIT RESULTS

Towards TBM transit all above mentioned recommendations were acknowledged by the Contractor and implemented. During the 2nd TBM approach of the bridge the Geobeads sensors were monitored again and the monitored excess pore pressures were exactly the same as the 1st transit. Therefore the adopted face pressure was remained the same as the predicted value. During transit the face pressure was perfectly maintained at the required 171 kPa by the Contractor. It was conservatively envisaged that TBM transit would take one week, but the bridge was passed in 3 days. Even the monitored settlements in front of the TBM appeared to be in the range as the predictions (app. 3 mm at center line).

8 DISCUSSION

As good results were achieved while maintaining lower face pressures the theorem is posited that analytical face stability prediction methods do contain a hidden safety margin. In porous soils excess pore water pressures around the TBM can be significant and they should be implemented in the 3D FE models in order to approach the reality as much as possible and remain a safety margin. Implementing the excess pore water pressures in the analytical approach is considered to be too conservative.

9 CONCLUSIONS

The result of the shallow TBM transit showed that the result of the 3D FEM predictions showed to be reliable, as the bridge did not collapse. As the monitored settlements were in the same range as the predictions it can be concluded with prudence that the remaining safety probably was in the range of the targeted and predicted value. Forced by circumstances the groundbreaking adoption of 3D FE models in tunnelling practice created the unique opportunity as well as necessity to validate these 3D FE models in practice. The 3D prediction method appeared to be successful in Amsterdam, although it is acknowledged to carefully adopt them at many more cases before 3D FEM design guidelines can be widely accepted.

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