

backfilling pressure (grout pressure) at the tail of the shield. Within the shield zone, the pressure distribution is either linearly interpolated between the face and tail pressures or applied uniformly, depending on the modeling assumptions. A continuous pressure distribution is assumed across the defined pressure zones. The tunnel lining, following the pressurized zone, is modeled using plate elements. Figure 2 illustrates the schematic distribution of pressure along the length of the TBM. The final support (SD) and grout injection pressures (MVPD) are determined based on the allowable ground surface settlement, heave, and tilt tolerances, particularly in relation to sensitive structures and their foundations. The implementation of the pressure model within the 3D finite element model is shown in Figure 3.

The tunnelling process is simulated using a step-by-step analysis approach. The excavation length per calculation phase corresponds to the width of one segment: 2.0 m for the VTR and 1.2 m for the ERS, as shown in Figure 2. Accordingly, the tunnel advances by one segment per phase. The analysis is performed under partially drained conditions, accounting for the excavation advance rate - 10 m/day for the VTR and 8.4 m/day for the ERS - so that each phase includes a consolidation component. For the Dbb and PT sections, this results in approximately 470 construction phases for a 230 m long model. Additional phases are introduced to simulate a stepwise reduction in face or air pressure in the event of an emergency access to the excavation chamber while the VTR is being driven beneath the Dbb pier foundations.

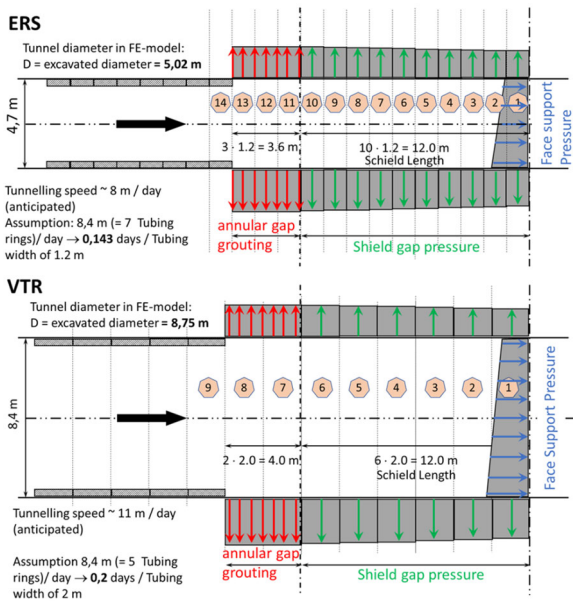


Figure 2. Schematic representation of the pressure model.

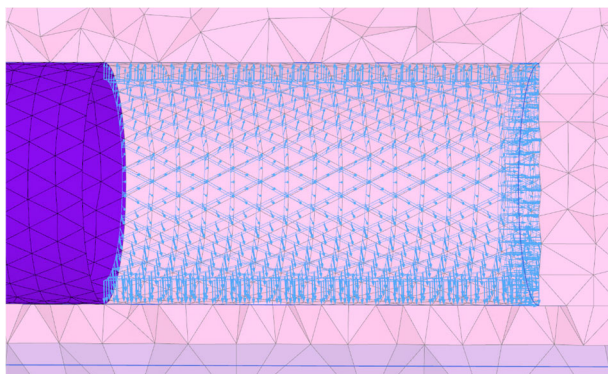


Figure 3. Implementation of the pressure model in the FE 3D model.

3.2 Modelling – Joints (VTR/ERS)

The segments are modelled as a homogeneous ring using 6-node triangular plate elements. These elements are defined as linear-elastic with a constant cross-section and reduced stiffness according to Muir Wood (1975). The bending stiffness is adjusted by considering the geometry of the longitudinal joints (i.e. the contact surfaces between individual segments). The resulting reduced moment of inertia is calculated as follows:

$$I = I_s + I_n \cdot \left(\frac{4}{m}\right)^2 \quad (1)$$

- I = reduced moment of inertia,
- I_s = moment of inertia of the contact area of the joints,
- I_n = Moment of inertia of the segment as a whole,
- m = number of the segments

3.3 Modelling – Existing Tunnels Joints

Depending on whether the tunnel segment or sprayed concrete lining (SCL) is modelled as a plate or as a solid volume, different approaches can be applied for modelling the joints of existing tunnels in PLAXIS 3D software.

3.3.1 Tunnel lining as plate element

There are two ways to simulate joints in the Plaxis software. These are (Figure 4):

- Predefined joints, the so-called 'connections',
- a small plate (e.g. 50 to 100 mm wide) with a realistic stiffness value of the joint material

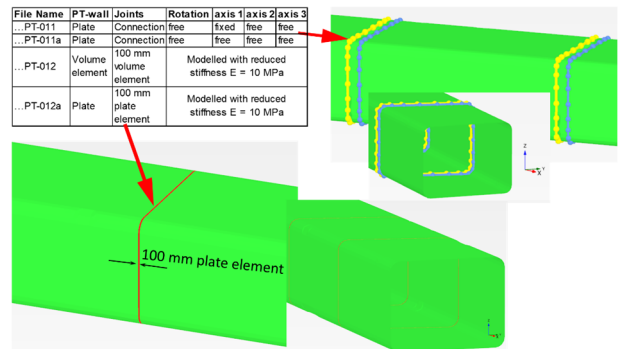


Figure 4. Schematic representation of the joints in FE-model - plate.

In a 3D finite element model of approximately 230 m by 100 m, incorporating a realistic joint with a width of 50 to 100 mm may cause mesh quality issues, potentially leading to numerical instability. A comparative study of two modelling approaches - namely the 'Connection' method and shell elements with a 100 mm width - demonstrates that the analysis results are comparable for box-shaped tunnels (Figure 5).

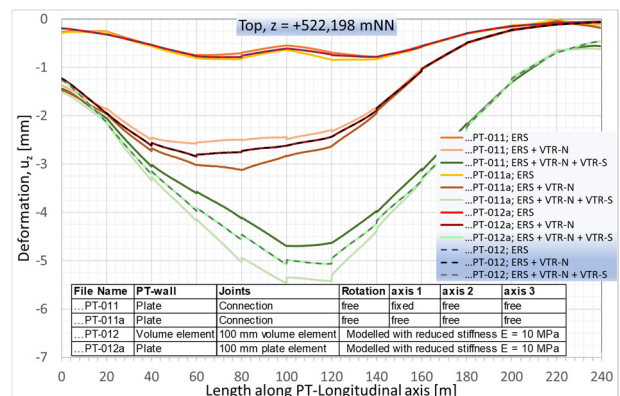


Figure 5. Comparison of joint simulations - lining as plate element.

For the sake of completeness, the use of an explicit joint model with a 100 mm-wide solid has been investigated, and the results are identical to those of an explicit joint model with a narrow shell (plate) element as shown in Figure 5.

3.3.2 Tunnel lining as a solid body

For existing tunnels, where the tunnel lining (SCL) is modelled as a solid volume, two approaches are commonly used to simulate the joints between different tunnel sections (Figure 6):

- A 50 to 100 mm wide solid element representing the joint with realistic stiffness properties of the joint material,
- Interface (contact) elements with stiffness parameters reflecting the joint material properties.

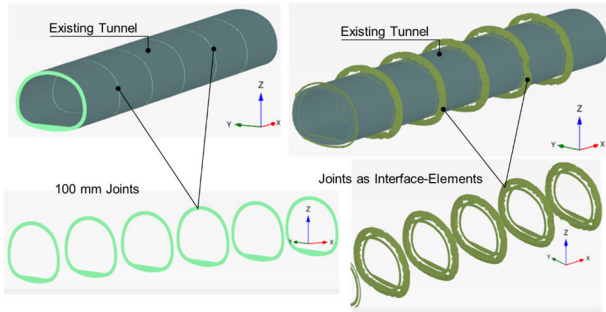


Figure 6. Schematic representation of the joints in FE-model - tunnel lining treated as a solid body.

Comparative analyses have demonstrated that using interface elements to simulate actual joints produces results comparable to those obtained with a 100 mm solid volume element (see Figure 7), given that the magnitude of relative deformations is very small ($\ll 1$ mm).

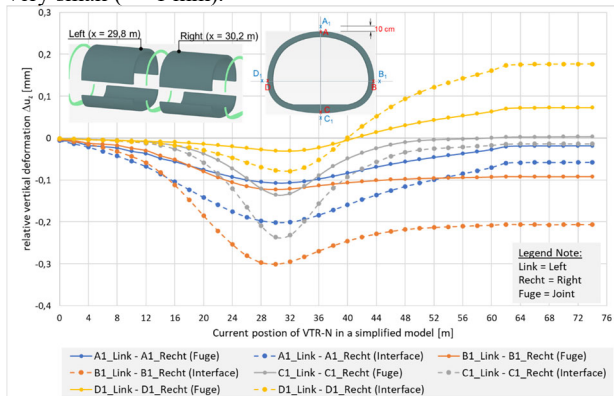


Figure 7. Comparison of two types of joint simulation: tunnel lining treated as a solid body.

4 EXAMPLES

4.1 Donnersberger Bridge und Post-Tunnel (Dbb & PT)

As shown in Figures 8 and 9, the 3D-FE-model is 230 m x 100 m x 40 m in size and contains all the relevant structures and loads within the influence zone of the tunnels for the Dbb & PT area: The calculation model consists of:

- 2 x traffic tunnels - VTR-N & VTR-S (2nd main railway line, new),
- 1 x Exploratory and Emergency Tunnel - ERS (2nd main railway line, new),
- Donnersberger Bridge (Dbb) Pier foundation (existing)
- Post tunnel (PT) - open cut tunnel (existing),
- 2 x injection shafts (2nd main railway line, new) - for lifting the PT if required,
- Various train loads at surface levels (existing),
- Building loads (existing).

The soil volume elements are assigned material properties according to the Hardening-Soil-Small-Strain (HSS) constitutive model, as recommended in the geotechnical interpretation report.

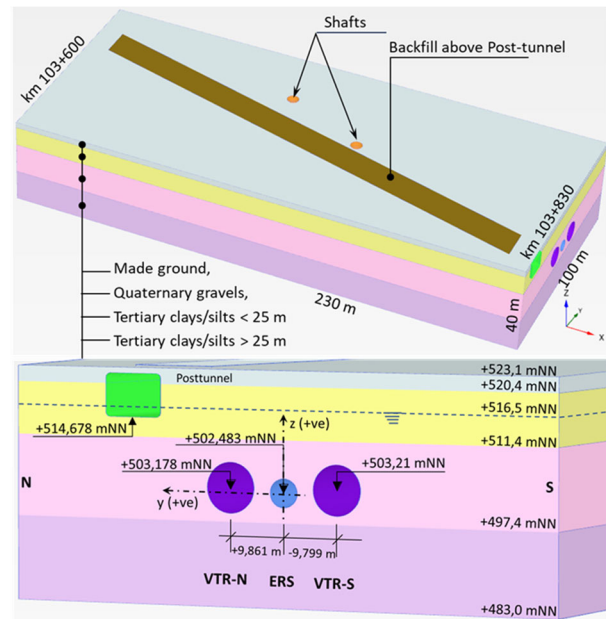


Figure 8. 3D FE-Model overview – Dbb & PT

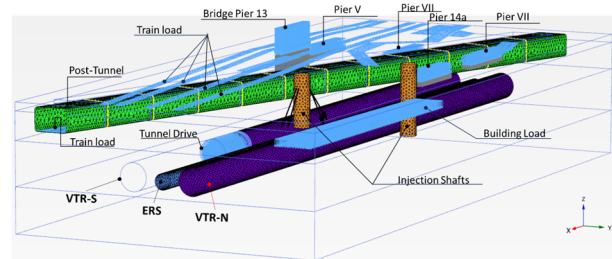


Figure 9. 3D FE-Model - Dbb & PT: structural elements and loading

The post tunnel (PT) is modelled using plate elements with linear-elastic behavior. The joints between the tunnel blocks are modelled with 'Connections', i.e. as hinges with free degrees of freedom in all directions, see section 3.3.1.

4.1.1 Preliminary Investigations

The finite element model shown in Figure 8 is computationally intensive, comprising approximately 500,000 elements and over 470 calculation phases (construction stages), including consolidation effects. Completing the full analysis may require several days of computation time. Therefore, a simplified model with dimensions of 130 m x 100 m x 40 m and relatively coarse mesh discretization was developed to perform parametric studies and to identify the limit support pressures required to satisfy the project-specific deformation and tilting criteria. The following variants were investigated:

- Variant I (V-I): Lower bound of pressure at crown level from stability analysis:
 - ERS: SD = 148 kPa; MVPD = 188 kPa
 - VTRs: SD = 162 kPa; MVPD = 177 kPa
- Variant II (V-II): Upper bound of pressure at crown level from stability analysis:
 - ERS: SD = 305 kPa; MVPD = 383 kPa
 - VTRs: SD = 262 kPa; MVPD = 352 kPa
- Variant III (V-III): The minimum pressure at crown level required to fulfill the project requirement:
 - ERS: SD = 305 kPa = MVPD

4.2 Frauenkirche

The Cathedral of Our Dear Lady (Frauenkirche) is one of Munich's most iconic landmarks, located in the city center at Frauenplatz (Figure 14). The Cathedral was constructed in the first half of the 13th century as a three-aisled, transeptless pillar basilica with two west towers. The upper closure of the interior space consists of net vaults (ribbed vaults) that span each bay between the rows of interior piers and the wall piers of the outer walls. The building is approximately 109m long and 40m wide, with its two distinctive western towers reaching a height of about 99m.

The cathedral's walls and interior piers consist of brick masonry. The foundations mainly comprise brickwork and mortared gravel. The foundation pressures beneath the 16 m × 16 m towers reach up to approximately 770 kPa, while the pressures beneath the outer walls and interior piers reach about 560 kPa and 310 kPa respectively. The sacristy, located in the north-eastern part of the cathedral, was constructed in the second half of the 15th century together with the main structure. Due to its historic and architectural significance, strict settlement and tilt limits apply during nearby tunnelling works. At the outset of the analysis, the deformation of the towers was of particular interest.

The VTR-S tunnel runs closest to the Frauenkirche, with a horizontal clearance of about 10 m beneath the sacristy. The closest horizontal distance to the north tower is approx. 34 m. The vertical distance between the crown of the VTRs and the lowest foundation point is approx. 29m.



Figure 14. Frauenkirche, view from southeast

The 3D finite element model of the Cathedral of Our Dear Lady (Frauenkirche) spans 192 m × 183 m × 67 m. The entire brick foundation of the cathedral is represented within the model, with the Marienhof station beginning at the eastern boundary (see Figure 15). All relevant structural components of the cathedral's foundation are explicitly modelled, including their geometry, stiffness, loading conditions, and interface elements along their boundaries. Other secondary loads - such as those from rail traffic, surface loads, or adjacent buildings - are incorporated as distributed flexible loads applied at the corresponding foundation depth. This approach is justified by the tunnel crown being located approximately 34 m below ground level.

4.2.1 Preliminary Investigations

The required support and grout pressures were determined following the same method as described in section 4.1.1 with a simplified model. Chosen calculation results of the preliminary analyses are shown in Figure 16 and Table 3. The magnitude of

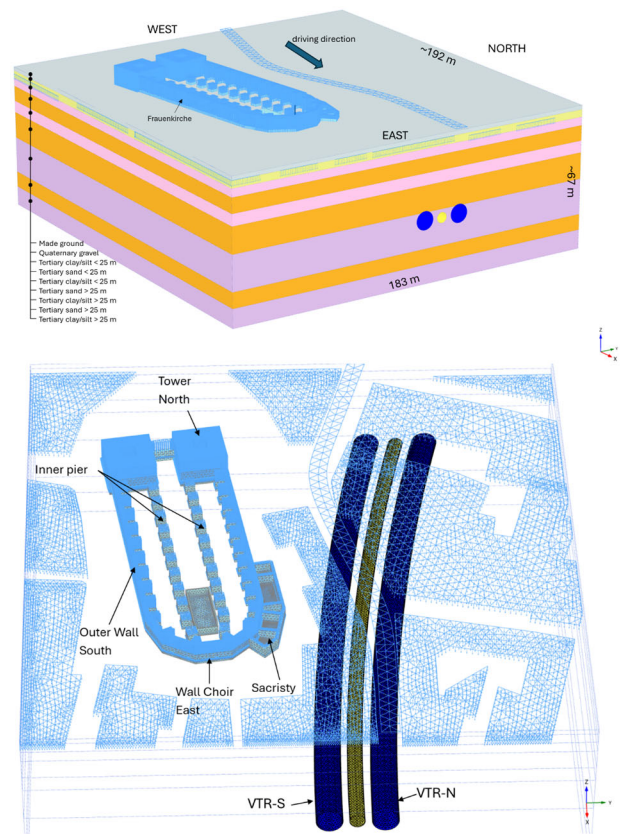


Figure 15. 3D FE-Model overview – Frauenkirche

the support and injection pressure has a significant influence on the surface settlements. As anticipated, the maximum settlement of approximately 14 mm occurs when applying the minimum pressures required to maintain face stability. To meet the settlement requirements at ground level, the pressures must be increased to the values shown in Table 2, designated as 'Var I'. These are approximately 0.8 bar lower than the maximum permissible pressures derived from the stability analyses. The settlement curves in Figure 16 assume that SD = MVPD.

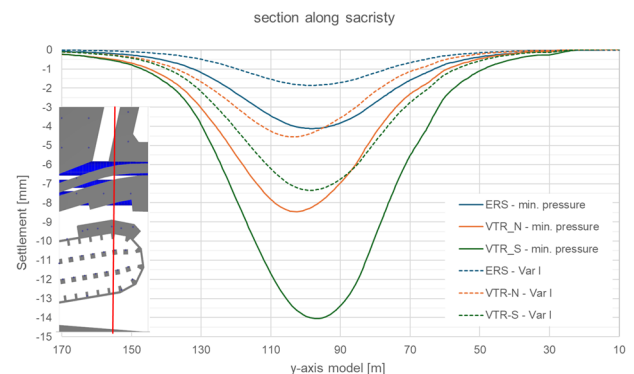


Figure 16. Settlement troughs with minimum pressure for stability requirements and required pressure for settlement limitation

Table 3. Summary of selected calculation results for the Frauenkirche model

Frauenkirche (the Dom)	SD		MVPD		max. settlement	max. gradient of tangent	volume loss factor		
	ERS	VTR	ERS	VTR			ERS	VTR-N	VTR-S
	kN/m ²								
min SD, min MVPD	165	300	193	346	14.1	n.a.	1.09%	0.35%	0.51%
max SD *)	550	516	550	516	4.7	~1/6600	0.39%	0.12%	0.14%
Var I *)	470	400/415	470	400/415	7.4	~1/4250	0.48%	0.22%	0.26%

*) MVPD = SD

4.2.2 Results

The final large-scale FE model was calculated using the support and grout pressures derived from the preliminary investigations. The model comprises of approximately 460,000 elements and 400 calculation phases (construction stages), including consolidation effects. The results are consistent with the findings of the preliminary analyses and confirm that deformations and distortions at ground level remain within the allowable limits.

In addition to the settlements at ground level, the deformations of the foundation and the underlying soil strata were evaluated in more detail. Figure 17 shows the deformed mesh (scaled up 2000 times) of the immediate bearing strata and the foundation at the end of all tunnel drives. It can be concluded that the towers will be only minimally affected by the tunnel construction. The main impact is expected at the sacristy and the north-eastern outer wall.

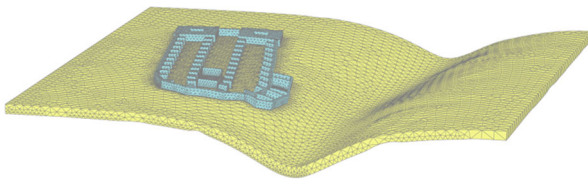


Figure 17. Deformed mesh (scaled up 2000 times)

The calculated spatial deformations of all foundations were documented, allowing for the determination of strains within the foundation structures. Figure 18 illustrates the evaluation of settlements at selected foundation points throughout the various tunnel drives. The maximum settlement was calculated to be less than 4 mm.

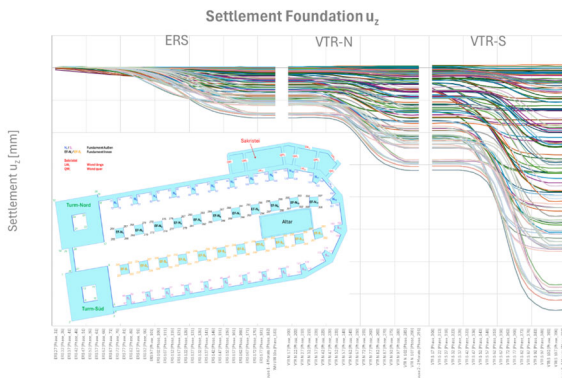


Figure 18. Foundation settlement throughout tunnel drives

The maximum tilt due to tunnel excavation was calculated to be less than 1:26,000 at the northern tower foundation and less than 1:4,400 at the northern outer wall adjacent to the sacristy. Soil strains were also determined at various levels beneath the foundations. As an example, Figure 19 shows the total Cartesian strain ϵ_{yy} immediately below the tower foundation. These results confirm that the major impact is not expected at the towers themselves, but rather at the sacristy and the adjacent northern outer wall.

The calculation results are provided to a client-appointed expert for the Cathedral of Our Dear Lady, who will assess the acceptability of the calculated deformations and verify the structural integrity of this landmark.

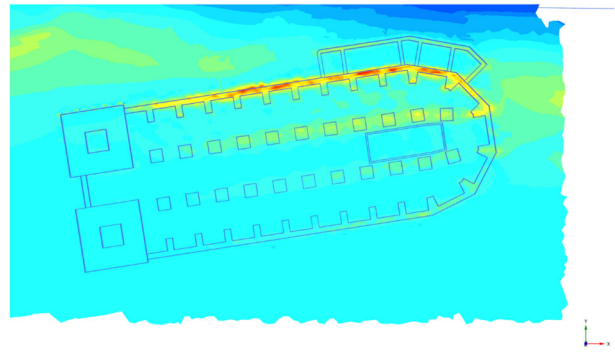


Figure 19. Total cartesian strain ϵ_{yy}

Further results of the FE model not included in this paper comprise an assessment of the effects of a potential emergency incident during VTR-S tunnelling beneath the Frauenkirche, which could necessitate access to the excavation chamber. In this scenario, the impact of a gradual loss of support or air pressure on the cathedral's foundations was investigated. Figure 20 shows an examined location close to Frauenkirche, where the air pressure was gradually decreased.

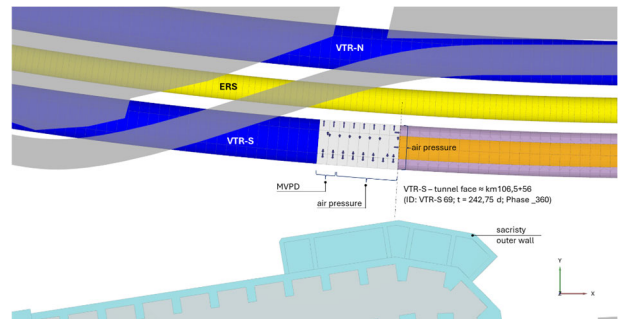


Figure 20. Examination of incident beneath Frauenkirche

5 APPREVIATIONS AND SYMBOLS

- ERS: Exploratory and Emergency tunnel
- VTR: Railway tunnel
- VTR-N: North Railway tunnel
- VTR-S: South Railway tunnel
- VTRs: the twin railway tunnels
- TBM: tunnel boring machine
- Dbb: Donnersberger Bridge (Donnersberger Brücke)
- PT: Post-Tunnel
- SD: support pressure (slurry) along the front face of the excavation
- MVPD: backfilling pressure (grout pressure) along the shield and tail of TBM
- HSS: Hardening-Soil-Small-Strain (HSS)
- FE: Finite Element

6 REFERENCES

- Muir Wood, A. M.: The circular tunnel in elastic ground. In: Geotechnique 25, No. 1 115-127, (1975).