6. Dimensioning and design of the segmental lining

6.1 General

With regards to the dimensioning of segments it is to be distinguished between temporary loads acting during construction stages, and loads acting during the stage of operation due to rock mass pressure, water pressure, temperature changes, swelling pressure etc.

Temporary loads are loads resulting from storage, transportation, installation of the segments and loads the segmental lining is subjected to until the TBM and the backup system completely passed through. This includes loads due to jacking forces, grouting pressure, self-weight as well as loads resulting from bumping and stockpiling. The temporary loads usually do not need to be superimposed with the loads acting during operation because they do not occur at the same time.

In order to be able to take into account the load bearing behavior of the ground and to simulate the different steps of loading it is recommended to carry out numerical analyses, e.g. by means of the FEM (see Section 3.5.1). These analyses also allow for a detailed and realistic modeling of the geometry of the segments and the joints.

Simplified analysis methods, such as the modulus of subgrade reaction method (see Section 3.5.2), should be applied only for special cases or approximate calculations respectively, because they are based on oversimplifying approaches. The rock mass pressure for instance has to be applied as an external load when this method is applied. The interaction between the ground and the tunnel can only be accounted for by approximation by means of the bedding with elastic springs. The discontinuity fabric of the rock mass cannot be considered at all.

The stability of the segmental lining (stage of operation) normally can be proven by means of two-dimensional or pseudo three-dimensional analyses respectively. The analyses in most cases can be carried out considering only one segmental ring, since the coupling between adjacent rings, according to the results of comparative analyses, does not effect the results, as long as the modulus of deformation of the rock mass E is higher than 500 MN/m². Two-
dimensional FE-analyses can be carried out if the ground is isotropic. In case of anisotropic rock mass pseudo three-dimensional analyses may become necessary (see Section 3.5.1). Three-dimensional analyses are required e.g. for the assessment of the stability in the area of the temporary face and the machine area, as well as for the simulation of heading in a fault zone. Furthermore, three-dimensional analyses should be carried out for the design for some temporary load cases, such as partial area compression resulting from jacking forces.

6.2 Analyses for the segmental lining (stage of operation)

6.2.1 FE-mesh, boundary conditions and loads

In the following, analyses carried out for double tracked tunnels of the railroad line Genoa - Ventimiglia, Italy, are presented. The considered tunnel section is located in rock with an overburden of 250 m. The clear tunnel diameter amounts to 10.74 m. The groundwater table in the analyses is assumed to be located 60 m above the tunnel roof (Fig. 6.1).

The FE-mesh represented in Fig. 6.1 is used for the pseudo three-dimensional analyses, which are carried out to calculate the stress resultants in the segmental lining. The computation section consists of a 110 m wide, 110 m high and 1 m thick rock mass slice. This size is sufficiently large, to outrule a noteworthy influence of the boundaries on the analysis results (see Section 3.5.1).

The discretization of the segmental lining and the surrounding area of the tunnel is very fine. In order to reduce the calculating effort, the elements are widened and the mesh is discretized coarser with increasing distance from the tunnel.

At the upper boundary of the computation section a surface load is applied, which corresponds to the self-weight of the rock mass overlying the considered section. Thus, the size of the mesh can be reduced without a remarkable influence on the analysis results.

The boundary conditions are selected assuming an isotropic ground. Accordingly, the nodes on the lateral boundaries and on the lower boundary are fixed perpendicular to the corresponding boundary plane (see Section 3.5.1).
Fig. 6.1: Pseudo three-dimensional analysis, computation section, FE-mesh, boundary conditions and rock mechanical parameters
The rock mass is horizontally bedded and vertically jointed. The rock mechanical parameters, the analyses are based on, are given in Fig. 6.1. Because of the demands from the awarding authority the discontinuities are not accounted for and an isotropic stress-strain behavior is assumed for the rock mass.

The lining of the tunnel in the selected example consists of 40 cm thick segmental rings composed of 4 regular stones, two boundary stones and one keystone. The keystones are inserted into the area of the roof and the upper sidewall. In the analyses presented here, it is assumed that the keystone is installed in the area of the roof (see Fig. 6.1, detail tunnel).

Fig. 6.2: Design of the longitudinal joints between two regular stones
The design of the longitudinal joints between two regular stones is illustrated in Fig. 6.2. The joints are oriented radially. The width of the contact area between the segments amounts to 17 cm and thus approx. 43 % of the width of the segment. The centerline of the contact area corresponds to the segment axis so that the normal thrust within the joint can be carried without eccentricity. For the outside gasket a 44 mm wide groove is foreseen. The spacing between the groove and the external boundary of the segment results to 35 mm. The spacing between the groove and the contact area results to 36 mm. The distance of the contact area to the external as well as to the internal boundary of the segment thus amounts to 115 mm.

The geometry of the longitudinal joints was simplified in the FE-mesh (Fig. 6.3). The groove for the gasket normally not have to be modeled in analyses, which are carried out in order to determine the stress resultants in the segmental lining.

![FE-mesh, detail longitudinal joint](image)
Fig. 6.4: Design of the keystone
The keystone is trapezoidal in a developed view. The angle between the longitudinal joint of the keystone and its longitudinal axis in this example amounts to 10° (Fig. 6.4). In the cross-section the longitudinal joints between the keystone and the adjacent boundary stones are oriented parallel to the centerline of the keystone (Fig. 6.4). This design of the keystone enables an easier installation.

The longitudinal joints between the keystone and the boundary stones were modeled realistically, i.e. parallel to the centerline of the keystone (see Fig. 6.1, detail keystone). The trapezoidal shape of the keystone does not need to be modeled in two-dimensional and pseudo three-dimensional analyses respectively. Thus, the state of stress in the area of the keystone can be calculated with sufficient accuracy in the analyses for the determination of the stress resultants.

The parameters describing the stress-strain behavior of the lining segments and the longitudinal joints are summarized in table 6.1. For the concrete of the segments, elastic stress-strain behavior is assumed. On the basis of the analysis results it is checked, if and to what extent the admissible tensile and compressive strength as well as the shear strength are exceeded.

The longitudinal joints are modeled by elements which do not allow for tensile stresses. Compressive stresses can however be transferred. The shear strength parallel to the joints is described by the Mohr-Coulomb failure criterion. A cohesion of $c = 0$ and an angle of internal friction of $45°$ is assumed.

<table>
<thead>
<tr>
<th></th>
<th>Young's modulus E [MN/m²]</th>
<th>tensile strength</th>
<th>shear strength</th>
<th>compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete (C45/55)</td>
<td>37000¹)</td>
<td>elastic</td>
<td>elastic</td>
<td>elastic</td>
</tr>
<tr>
<td>longitudinal joints</td>
<td>37000¹)</td>
<td>0</td>
<td>$\varphi = 45°$</td>
<td>elastic</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$c = 0$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(parallel to the joints)</td>
<td></td>
</tr>
</tbody>
</table>

1) increased compared to DIN 1045-1 (conservative assumption)

Table 6.1: Parameters describing the stress-strain behavior of the lining segments
The annular gap can be backfilled using either mortar or pea gravel. The backfill is modeled by a separate row of elements (Fig. 6.3) and simulated by an elastic-viscoplastic stress-strain behavior. The shear bond between the annular gap filling and the concrete of the segmental lining must be selected in accordance with the properties of the backfill. When mortar is used the age of the mortar at the time of loading must be taken into account.

In the presented project the circumferential joints of the segmental lining are planar. Thus, there is no tongue-and-groove or cam-and-pocket design. Therefore, a coupling between adjacent rings is only possible by means of friction along the load transmission pads.

Comparative analyses carried out for coupled segmental rings have shown that the simulation of the coupling does not have a noteworthy effect on the results, if the modulus of deformation of the rock mass is higher than 500 MN/m². Therefore, for the considered example it is sufficient to carry out the analyses for a single segmental ring only.

As mentioned above, in most cases two-dimensional or pseudo three-dimensional analyses are sufficient to determine the stress resultants in the segmental lining for the stage of operation. Three-dimensional analyses for instance may however be required, if the ground cannot be assumed to be homogeneous in longitudinal direction or if the objective of the analysis is to assess the stability in the working area.

In Fig. 6.5 exemplarily a three-dimensional FE-mesh for a tunnel of the railroad line Genoa – Ventimiglia, which was already mentioned before, is represented. This mesh is used to simulate the driving of the tunnel through a steeply dipping fault zone with a thickness of 8 m which intersects the tunnel axis with an angle of 15°. The computation section is 90 m wide, 82 m high and 159 m long. The fault zone is modeled by elements which have a higher deformability and a lower strength than the elements modeling the sound rock mass. The rock mechanical parameters are given in Fig. 6.5. The boundary conditions and the surface load at the upper boundary are selected as already described for the pseudo three-dimensional FE-mesh. The segmental lining is modeled in the same way too.
The loads to be assumed in the analyses for the design of the segmental lining are regulated in the corresponding directives and guidelines for road and railroad tunnels. The loads for railroad tunnels specified by the Guideline 853 of the German Railroad (DB Netz, 2003) are summarized in Fig. 6.6.

The Guideline 853 (DB Netz, 2003) distinguishes between main loads, additional loads and special loads, which are to be superimposed in three different load cases, each considering the most unfavorable combination.

For road tunnels carried out by underground construction in Germany, the Additional Technical Contract Conditions and Guidelines for Engineering Services (ZTV-ING), Part 1, Section 1 (ZTV-ING, 2003) is to be applied. The loads given in this set of rules also can be applied to tunnels with a segmental lining. They correspond to a large extent to the loads given in the Guideline 853 (DB Netz, 2003).
Main loads

- permanent loads:
  - self-weight of the lining and all built-in units (load case self-weight)
  - loads from the rock mass incl. potential swelling (load case rock mass pressure)
  - loads from water pressure (load case water pressure)
  - self-weights from the track
  - concentrated loads from catenary facilities
  - loads from pre-stressing measures
  - effects due to shrinkage and creep of the lining (DIN 1045 and DIN 4227)
  - permanent loads at the ground surface and influences from adjacent openings

- steady traffic loads:
  - acc. to Guideline 804 and DIN 1072
  - aerodynamic pressure or suction

Additional loads

- effects of heat (load case temperature)
- temporary loads during construction, e.g.
  - backup system
  - belt conveyor
  - occasional traffic loads
  - temporary loads due to changes at the ground surface and from adjacent openings

Special loads

- impact loads
- breakage of the catenary and other lines
- impact of earthquakes on built-in units
- impact of potential subrosion
- fire load (acc. to EBA (2001))

Fig. 6.6: Loads according to Guideline 853 of the German Railroad (DB Netz, 2003)

Substantially, the following load cases were investigated for the tunnels of the railroad line Genoa – Ventimiglia:

- self-weight of the segmental lining (S),
- rock mass pressure (R),
- water pressure (W),
- temperature (T).

The load combination

self-weight (S) + rock mass pressure subjected to uplift (R) + water pressure (W) + temperature (T)

turned out to be decisive for the dimensioning of the segmental lining.
6.2.2 Load case self-weight

For the load case self-weight basically the self-weight of the lining is accounted for. The self-weight of built-in units as well as of the track normally does not lead to a noteworthy loading of the segmental lining. Therefore, they are usually neglected when evaluating the stress resultants. In each individual case it needs to be checked however, if such a simplification is admissible. In addition, the transfer of concentrated loads resulting from built-in units mounted on the segments generally has to be designed.

The loading due to self-weight of the lining can be evaluated in combination with other load cases such as rock mass pressure or water pressure. A complete circumferential bedding of the segmental lining can be assumed, if the annular gap between the segmental lining and the rock mass is backfilled immediately behind the shield, and if the load bearing capacity of the backfill is instantly available when loaded. Analyses for which such a complete bedding is assumed show that the loading due to self-weight of the lining is comparatively small. Thus, the load case self-weight only in most cases is not decisive.

If the segments are however only incompletely bedded after leaving the shield, separate analyses are required (see Section 6.2.7).

6.2.3 Load case rock mass pressure

For the load case rock mass pressure the loading of the segmental lining due to the action of the rock mass is evaluated. The computation steps for this load case for two-dimensional or pseudo three-dimensional analyses respectively, are represented schematically in Fig. 6.7. In the first step the stresses resulting from the self-weight of the rock mass are calculated (primary state). To take into account the influence of the deformations of the rock mass, which occur in the area of the temporary face and the shield before the segmental lining is installed, different techniques can be applied (Wittke, 2000). For the presented example it is assumed that the elastic deformations already have partly occurred before the lining is installed, because an elastic stress relief in the rock mass according to experience already takes place immediately after excavation.
Fig. 6.7: Load cases rock mass pressure and water pressure, computation steps, two-dimensional or pseudo three-dimensional analysis.
Viscoplastic deformations due to exceeding of strength in the rock mass however develop slower. Consequently a preceding stress relief is simulated in the 2nd computation step. For this a material is introduced in the cross section of the tunnel, which in comparison to the surrounding rock has a reduced modulus of deformation $E_{\text{red}} = \alpha_v \cdot E$. $\alpha_v$ is referred to as stress relief factor (Wittke, 2000). In this computation step only elastic deformation are allowed for.

In the 3rd step the excavation of the tunnel and the installation of the segmental lining are simulated. The lining is installed into the cross-section of the tunnel, which is already deformed corresponding to the 2nd computation step. Thus, it is loaded by the remaining elastic and the complete viscoplastic deformations of the rock mass.

A precondition for the application of this method however is the knowledge of the correct stress relief factor. This factor according to experience is not constant. It depends on the rock mass behavior, the primary state of stress as well as on the time sequence of excavation and installation of the lining.

![Diagram](image)

Fig. 6.8: Determination of the stress relief factor, elastic displacements due to excavation of an unsupported tunnel, result of a three-dimensional analysis (Wittke, 1990)
With the following approach the stress relief factor for the analysis of a segmental lining can be estimated. This approach is based on the elastic displacements due to excavation of an unsupported tunnel, which are evaluated as a result of a three-dimensional analysis (Wittke, 1990).

The result of this analysis shows that approx. 30% of the final elastic subsidence of the roof and approx. 20% of the total elastic heaving of the invert have already occurred ahead of the temporary face. Approx. one tunnel diameter behind the temporary face, already more than 90% of the elastic displacements due to excavation have occurred (Fig. 6.8).

On this basis of these results, the elastic displacements which occur in the rock mass before the segmental lining is installed can be estimated. For simplification the length of the shield is assumed to be equal to the tunnel diameter for the following evaluation (Fig. 6.9 and 6.10).

![Diagram showing evaluation of the stress relief factor.](image)

**Fig. 6.9:** Evaluation of the stress relief factor.

Case A: Annular gap between rock mass and shield remains open

\[
a = 0.92 \cdot \delta_{e,r,A-A} - 0.31 \cdot \delta_{e,l,A-A} \\
b = 0.96 \cdot \delta_{e,l,A-A} - 0.21 \cdot \delta_{e,l,A-A}
\]

\[a + b < 2 \cdot \Box \text{ (annular gap between the rock mass and the shield remains open)}\]

\[\Rightarrow \text{ displacements which occur before the installation of the segmental lining (}\delta_{2,1}\text{, see Fig. 6.7):}\]

\[
\delta_{2,1,\text{roof}} + \delta_{2,1,\text{invert}} = 0.92 \cdot \delta_{e,r,A-A} + 0.96 \cdot \delta_{e,l,A-A}
\]

\[\Rightarrow \text{ stress relief factor } \alpha \text{ must be selected correspondingly}\]
It is accounted for that a steering gap is existing between the rock mass and the shield. This gap can be achieved by a corresponding overcut. In addition the steering gap can be increased by means of a conical shape of the shield skin. For simplification this possibility subsequently is not considered. Basically two cases can be distinguished:

**Case A (Fig. 6.9):**

The elastic displacements of the unsupported tunnel contour in the area of the shield are smaller than the overcut. In this case the steering gap between the rock mass and the shield remains open and 90 % of the elastic displacements of the unsupported tunnel contour can occur before the segmental lining is installed. These displacements can be taken into account in the 2nd computation step by means of a preceding stress relief (Fig. 6.7). The stress relief factor $\alpha_v$ has to be selected correspondingly.

**Case B:**

- $a = 0.92 \cdot \delta_{el,J-A} + 0.31 \cdot \delta_{el,I-A}$
- $b = 0.96 \cdot \delta_{el,J-A} + 0.21 \cdot \delta_{el,I-A}$

$$\delta_{2-1, \text{roof}} + \delta_{2-1, \text{invert}} = 0.31 \cdot \delta_{el,J-A} + 0.21 \cdot \delta_{el,I-A} + 2 \cdot o$$

The rock mass is leaning against the shield skin
Case B (Fig. 6.10):

The elastic displacements of the unsupported tunnel contour in the area of the shield are larger than the overcut. In this case the rock mass is leaning against the shield skin. The elastic displacements of the rock mass which occur before the segmental lining is installed, in this case are composed of the displacements ahead of the temporary face and the overcut. These displacements also can be accounted for in the 2nd computation step by means of a preceding stress relief (Fig. 6.7). The stress relief factor $\alpha_v$ has to be selected correspondingly.

The flow chart represented in Fig. 6.11 illustrates the evaluation of the stress relief factor $\alpha_v$. In a first step the elastic displacements of the unsupported tunnel contour have to be evaluated by means of a separate analysis. On the basis of the results it can be checked, if case A or case B is decisive. Then, the required stress relief factor $\alpha_v$ must be determined iteratively with the aid of finite element analyses. A stress relief factor $\alpha_v$ is selected and the displacements resulting in the 2nd computation step are compared with the displacements which can occur before the installation of the lining (Fig. 6.8 and 6.10). The stress relief factor $\alpha_v$ then must be adjusted until the calculated displacements correspond approximately with the target values explained above.

For the two-dimensional analysis presented in section 6.2.1, a stress relief factor of $\alpha_v \approx 0.075$ was evaluated. Consequently, the modulus in the area of the tunnel’s cross-section must be reduced in computation step 2 by 92.5 %, to a value of 7.5 % of the modulus of the surrounding rock mass.

The results of this analysis are summarized in section 6.4.1.

Three-dimensional analyses enable a direct simulation of the heading and the evaluation of the loading of the segmental lining due to rock mass pressure under consideration of displacements, which occur ahead of the temporary face. The computation step for the example of a three-dimensional analysis for the simulation of the crossing of a fault zone (see Section 6.2.1, Fig. 6.5) are schematically illustrated in Fig. 6.12. The analysis was carried out in 23 computation steps. After evaluation of the primary state (1st computation step), the excavation of a 44.5 m long tunnel section is simulated in the 2nd computation step. In the same com-
putation step the installation of the segmental lining is modeled in the corresponding area. The 2nd computation step serves as the initial state for the simulation of the heading and lining installation, which is carried out in the computation steps 3 to 23 according to the 'step by step' method described in detail in Wittke (2000). Accordingly, in each computation step the excavation of the tunnel with a length of one segmental ring as well as the installation of a corresponding ring are simulated. The required number of computation steps in each individual case has to be adapted depending on the local boundary conditions, the computational effort and the demands on the accuracy of the analysis results.

Fig. 6.11: Determination of the stress relief factor, flow chart

\[ \delta_{\text{rel}} = 0.9 \cdot \delta_{\text{rel}, A} + 2 \cdot \delta_{\text{rel}, A} \]

- \( a, b \) and \( \sigma \) see Figs. 6.9 and 6.10
- total displacement (roof and invert)
Fig. 6.12: Crossing of a fault zone, three-dimensional analysis, computation steps

**computation steps**

1. **1st computation step**

2. **2nd computation step: initial state**

3. **3rd to 17th computation step:** excavation of the tunnel and installation of the segmental rings acc. to the 'step by step' method (Wittke, 2000)

4. **18th to 23rd computation step:** excavation of the tunnel and installation of the segmental rings acc. to the 'step by step' method (Wittke, 2000)

**check of the displacements in the unsupported area:**

1. $\delta_{23} - \delta_{17} \leq 0$ (overcut) $\rightarrow L_{\text{unsupported}} = L_{\text{shield}}$ (length of the shield)

2. $\delta_{23} - \delta_{17} > 0$ (overcut) $\rightarrow L_{\text{unsupported}}$ must be selected smaller so that $\delta_{23} - \delta_{17} = 0$
The shield skin is not modeled in these analyses. The first section behind the temporary face thus remains unsupported. The length of this section initially is selected corresponding to the length of the shield. It must however be checked, whether the displacements in the unsupported section are greater than the overcut. In this case the rock mass would lean against the shield's skin and thus, the maximum possible displacements which can occur before the installation of the segmental lining are smaller than the calculated displacements. In such a case, new analyses with a shorter unsupported section would have to be carried out. The resulting displacements due to excavation in this section then must be equivalent to the overcut.

The load case self-weight of the segmental lining (see Section 6.2.2) is included in the analyses for the load case rock mass pressure, since the unit weight of concrete is assigned to the elements of the segmental lining.

The loading due to swelling pressure can be simplified by applying a surface load in the area of the invert. Furthermore, analysis procedures taking into account the stress-strain behavior of swelling rock mass in a realistic way are available. These methods are presented in detail in Wittke (2000).

6.2.4 Load case water pressure

In the load case water pressure the loading of the segmental lining due to water pressure is evaluated. If the spacing of the water-filled discontinuities is small compared to the tunnel diameter, and if the total of the contact areas of the discontinuity walls is small in comparison to the total area of the discontinuities, the application of the water pressure as surface load over the entire circumference of the segmental lining is a good approximation. This assumption is described in detail in Wittke et al. (2004) and in section 3.5.1 of this volume for a segmental lining with a grouted annular gap in a jointed rock mass.

The evaluation of the loading due to water pressure for the presented example of the tunnels between Genoa and Ventimiglia is illustrated in Fig. 6.13. The analysis is carried out using the pseudo three-dimensional mesh (Fig. 6.1), already described in Section 6.2.1. For this analysis the entire computation section is assumed to be weightless, since loading due to the rock mass and
the self-weight of the lining is not to be considered. The boundary conditions consist of horizontally sliding supports for the nodes on the upper and lower boundaries and of vertically sliding supports for the nodes on the lateral boundaries. The rock mass is assumed to be elastic. Thus, the stiffness of the rock mass, which effects the bedding of the lining is modeled. The water pressure as described above is applied by a surface load on the segmental lining. The transmission of tensile forces between the lining and the rock mass is eliminated in the analysis.

Fig. 6.13: Load case water pressure, assumptions regarding the loading and bedding of the segmental lining

The results of this analysis are presented in section 6.4.1.
6.2.5 Superposition of rock mass and water pressure

As already mentioned, the application of the rock mass pressure and the water pressure on the lining of machine-driven tunnels is described in detail in Wittke et al. (2004). Accordingly, if traffic tunnels are located in sedimentary rocks with horizontal bedding and vertical jointing, the loads due to rock mass pressure subjected to uplift and water pressure have to be superimposed for the design of the lining, if the spacing of the discontinuities is small in comparison to the tunnel diameter and if the total of the contact areas of the discontinuity walls is small in comparison to the total area of the discontinuities.

To consider the load combination "rock mass pressure + water pressure" the stress resultants from the load case "rock mass pressure" (Fig. 6.7) are superimposed with those from the load case "water pressure" (Fig. 6.13). In this case the unit weight of the rock mass subjected to uplift \( \gamma' \) must be selected in the analysis for those parts, which are located underneath the groundwater table.

Another possibility to evaluate the loading of the lining due to rock mass and water pressure is to apply an additional surface load on the segmental lining, which corresponds to the existing water pressure. This can be done in a fourth computation step, following the three computation steps which are required for the load case rock mass pressure (see Fig. 6.7). If this procedure is selected, the elements located below the groundwater table must be assigned with the unit weight. Furthermore, a transmission of tensile forces between the lining and the rock mass then has to be prevented. Thus, the loading of the segmental lining due to rock mass and water pressure can be evaluated in one analysis. Such an analysis enables to consider the interaction between the segmental lining and the rock mass in a more realistic way than the separate analyses for the load cases rock mass pressure and water pressure, which were described in sections 6.2.3 and 6.2.4.

The superposition of the stress resultants obtained from separate analyses for the load case rock mass pressure and water pressure normally leads to a higher loading of the lining and therefore is conservative.
6.2.6 Load case temperature and superposition with other load cases

In the load case temperature the loading of the segmental lining due to temperature changes, which occur after the installation of the lining is evaluated. In Fig. 6.14 the procedure applied for the tunnels between Genoa and Ventimiglia is represented. The analysis is carried out using the pseudo three-dimensional mesh (Fig. 6.1) already described in section 6.2.1. The entire computation section is assumed to be weightless as it is also doing for the load case water pressure, described in Section 6.2.4. The rock mass is assumed to be elastic in order to simulate the bedding of the lining. The boundary conditions are also selected in the same way as for the load case water pressure.

For this example the temperature changes are selected according to the Guideline 853 of the German Railroad (DB Netz, 2003). The temperature changes for the load cases 'summer' and 'winter' are specified for different distances of the analysis cross-section from the tunnel portal. Similar specifications are contained in the ZTV-ING, Part 5, Section 1 (ZTV-ING, 2003).

In the initial state all nodes of the FE-mesh corresponding to the Guideline 853 (DB Netz, 2003) are allocated with the temperature of installation of 10 °C. Afterwards the temperatures given in Fig. 6.14 for the external, central and internal plane are assigned to the corresponding nodes of the segmental lining.

The stresses in the segmental lining, which result from the partly prevented temperature strains, are dependent on the temperature change compared to the initial state, the coefficient of thermal expansion of the concrete as well as the bedding and the stiffness of the lining. A radially uniform temperature change within the lining leads to normal thrusts in the lining, if deformations are impeded. Non-uniform temperature changes lead to forced moments, if a change of the curvature of the lining is prevented. The transmission of tensile forces between the lining and the rock mass must be eliminated in the analysis.

The results of these analyses are presented in Section 6.4.1 for the load case 'summer'.
To consider load combinations, the stress resultants from the load case temperature (Fig. 6.14) are superimposed with those from other load cases, such as rock mass pressure (Fig. 6.7) and/or water pressure (Fig. 6.13). However, the stress-strain behavior of temperatures to be applied acc. to Guideline 853 (DB Netz, 2003):

<table>
<thead>
<tr>
<th>railroad tunnels (except for urban railway tunnels)</th>
<th>plane</th>
<th>summer [°C]</th>
<th>winter [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>covered tunnel between portal and 200 m from portal</td>
<td>external plane $T_E$</td>
<td>+15</td>
<td>-5</td>
</tr>
<tr>
<td></td>
<td>central plane $T_C$</td>
<td>+20</td>
<td>-10</td>
</tr>
<tr>
<td></td>
<td>internal plane $T_i$</td>
<td>+25</td>
<td>-15</td>
</tr>
<tr>
<td>covered tunnel from 200 m to 1000 m from portal</td>
<td>external plane $T_E$</td>
<td>+10</td>
<td>+5</td>
</tr>
<tr>
<td></td>
<td>central plane $T_C$</td>
<td>+15</td>
<td>0</td>
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<tr>
<td></td>
<td>internal plane $T_i$</td>
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<td>-5</td>
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<td>covered tunnel more than 1000 m from portal</td>
<td>external plane $T_E$</td>
<td>+10</td>
<td>+5</td>
</tr>
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<td>internal plane $T_i$</td>
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<tr>
<td>urban railway tunnels</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>covered tunnel between portal and 200 m from portal</td>
<td>external plane $T_E$</td>
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<tr>
<td></td>
<td>central plane $T_C$</td>
<td>+15</td>
<td>+7.5</td>
</tr>
<tr>
<td></td>
<td>internal plane $T_i$</td>
<td>+20</td>
<td>+10</td>
</tr>
</tbody>
</table>

temperature of installation $T_0 = 10^\circ$ C

Fig. 6.14: Load case temperature, computation assumptions
the longitudinal joints resulting from the superimposed loading needs to be checked in advance.

If, for instance, the normal thrusts are large in comparison to the bending moments, the entire contact area of the longitudinal joints is compressed. Thus, the joints do not open. An opening of the joints, in the load case temperature therefore also has to be prevented. This can be achieved, if elastic stress-strain behavior is assigned to the joint elements (see Fig. 6.3). If, however, an opening of the joints is to be expected for the considered load combination, no tensile strength should be allowed for the joint elements in the load case temperature (see table 6.1).

Such a distinction of cases is not needed, if the loading due to temperature is applied directly as an additional load in the analyses for the load case rock mass pressure and/or water pressure. Thus, the loading of the segmental lining due to a load combination which includes the load case temperature can be evaluated by one single analysis. The joint elements as already described, in such a case can be modeled assuming a viscoplastic stress-strain behavior, not allowing for tensile stresses across the joints.

6.2.7 Special load cases

Fire

According to ZTV-ING, Part 5, Section 1 (ZTV-ING, 2003) the required structural fire protection is to be assured by means of the compliance with design requirements or with the aid of calculative design procedures. In Guideline 853 (DB Netz, 2003) the fire protection is regulated similarly. The corresponding specifications are given in Section 2.2.4.

Earthquakes

For the evaluation of the loading due to earthquakes, normally the equivalent load method is used. In such analyses, an equivalent horizontal load is applied, which can be selected according to the German Standard DIN 4085, Supplement 1 (DIN 4085, 1987). Furthermore, dynamic analysis procedures are available, but only rarely used.
**Insufficiently bedded segmental ring, effectiveness of gaskets**

The tightness of the joints is of great importance, if the single lining construction method is used. The tightness may be endangered as a consequence of insufficient bedding of the segmental ring, as demonstrated by the following example.

The segments are installed and screwed under the protection of the shield by means of the erector. During the further advance the segmental ring is leaving the area of the tail-skin. Immediately behind the tail-skin the filling of the annular gap is carried out. If the annular gap is not completely filled, the segmental ring may be insufficiently bedded after leaving the tail-skin. In the most unfavorable case, the annular filling is missing along the entire periphery of the segmental ring.

Due to the insufficient bedding, the loading from self-weight of the segmental ring and possibly also additional loads from falling rock, lead to deformations. The segmental ring deforms into a lying ellipse (Fig. 6.15). As a consequence, tensile strains occur at the outside of the sidewalls of the segmental ring, and thus in the area of the gaskets.

Subsequently, a segmental ring with gaskets is considered, which has a joint gap width of 21 mm in the non-compressed state (Fig. 6.16a). During the installation of the adjacent segment and the subsequent screwing, the gaskets are pressed together. Both gasket frames are compressed by approx. 10 mm each and thus pre-stressed (Fig. 6.16b). This pre-stressing must be maintained during the subsequent stages of construction in order to achieve a sufficient tightness of the segmental ring. Potential tensile stresses, as they result the area of the sidewalls of the segmental ring in case of a deformation as represented in Fig. 6.15 are not allowed to lead to a noteworthy reduction of the pre-stressing of the gaskets. This will be further illustrated by means of the following example.

The example is based on a tunnel cross-section with an internal radius of 4.15 m and a segment thickness of 70 cm. The geometry and the loading due to self-weight are symmetrically to the vertical plane through the tunnel axis. Simplifying the loading due to outbreaks above the roof of the tunnel are also assumed to be symmetrically to this axis. Therefore, only one half of the segmental
ring is modeled by the FE-mesh (Fig. 6.17a). The thickness of the FE-mesh is 1 m. The longitudinal joints are modeled each with two rows of elements (Fig. 6.17b). In the area of the joints the FE-mesh is discretized very fine, because of the high stress differences which are to be expected in case of an opening of the joints.

Fig. 6.15: Deformation of the segmental ring due to self-weight and a potential load at the roof if the annular gap is not filled, not to scale

Plane strain conditions are assumed for the segmental ring. All nodes along the plane of symmetry, which is vertical through the tunnel axis, are fixed in horizontal direction (Fig. 6.17a).
Fig. 6.16: States of deformation of a gasket in the longitudinal joint in the area of the sidewall

A bedding of the segmental ring in the area of the invert is simulated by means of elastic truss elements (Fig. 6.17a). The modulus of subgrade reaction $K$ is evaluated using the formula $K = E/R$. 
where $E$ is the modulus of deformation of the rock mass and $R$ is the radius of the centerline of the segmental ring ($R = 4.50$ m). The stiffness of the truss elements is selected according to the evaluated modulus of subgrade reaction $K$.

The selected parameters for the concrete, the joints and the gaskets are given in Fig. 6.17. Because of the expected high loading of the segmental lining in the final state, which is not discussed in detail here, a high-strength concrete of grade B65, corresponding to C55/67 is selected for the segments. The corresponding Young's modulus amounts to 40500 MN/m². The elements of the longitudinal joints have the same elastic constants as the concrete, however no tensile strength ($\sigma_t = 0$). Thus, an opening of the joint can be simulated in the FE-analyses. The shear strength parallel to the joints is described by the Mohr-Coulomb failure criterion. An angle of friction of 45° and no cohesion ($c = 0$) are assumed. The gaskets are assumed to be practically weightless. The selected Young's modulus of the gaskets ($E = 2.40$ MN/m²) is comparatively large. The force-displacement relationship of the gaskets normally is non-linear. The assumption of a constant Young's modulus for the gaskets therefore is an approximation.

Fig. 6.17: FE-mesh: a) segmental ring and boundary conditions; b) joint (detail)
The Young's modulus of the gaskets, however, is much smaller than the Young's modulus of the concrete. The deformations of the segmental ring therefore are practically determined only by the

**Fig. 6.18:** Insufficiently bedded segmental ring, computation steps

- Installation of the concrete in the deformed system
- Mounting of screws (nodal forces are removed)
- Application of the roof load $p$
stiffness of the concrete and are independent of the stiffness of
the gaskets. Only the stress changes in the gaskets as a conse-
quence of deformations are dependent on the Young's modulus of the
gaskets. The unloading of the gaskets due to deformation however,
is overestimated in the finite element analysis, because of the
comparatively large Young's modulus.

The screwings of the longitudinal joints are modeled by truss ele-
ments, which can only transfer normal thrusts in axial direction
(Fig. 6.17).

In order to simulate the pre-stressed gaskets a computation step
in which the gaskets are subjected to a pre-stressing is required
(1st computation step) before the self-weight of the segmental
ring and an additional roof load \( p \) is applied (2nd computation
step). Both computation steps are illustrated in Fig. 6.18.

The pre-stressing of the gaskets is simulated in the first step
with the aid of nodal forces \( F \), which are applied along the whole
surface of all gaskets. The nodal forces are selected in a way
that the required force for a compression of the gaskets by 20 mm
is achieved. In order to assure that these forces induce the des-
ignated stresses only in the gaskets and not in the segments, the
Young's modulus for the elements of the segments is selected to
\( E = 0 \) (air) in this computation step.

In the second step the installation of the segmental ring is simu-
lated by the so-called "stressless" installation of the concrete
\( (E = 40500 \text{ MPa}) \) in the deformed system. At the same time the nodal
forces applied in the first computation step for pre-stressing the
gaskets are removed. This leads to a loading of the contact areas
between the gaskets and the concrete. Furthermore, the truss ele-
ments which simulate the screws are installed and an additional
roof load is applied in the second computation step (Fig. 6.18).
Thus, the segments are loaded by their self-weight and the addi-
tional roof load.

Due to this loading and the selected boundary conditions the seg-
mental ring, as expected, deforms into the lying ellipse repre-
sented in Fig. 6.15. The resulting normal stresses in joint 2,
which is located at the sidewall (see Fig. 6.15 and 6.17) as well
as the resulting normal thrusts in the gasket and the joint are
given in Fig. 6.19.
In the areas where the normal stresses are $\sigma_N \approx 0$, the joints are opened. In joint 2 this is the case at the outside of the segmental ring (Fig. 6.19). This leads to a stress relief in the gasket. The corresponding drop of the compression of the gasket in comparison to the pre-stressed state immediately after installation of the segmental ring converges to $\delta'_{KV} = 4.8$ mm in computation step 2 (Fig. 6.20).

Fig. 6.19: Normal stresses in joint 2 and normal thrusts in joint 2 and the associated gasket, computation step 2

Fig. 6.20: Change of thickness of the gasket in joint 2 in the course of the viscoplastic iterative analysis, computation step 2
According to the force-displacement diagram of the selected gasket, approx. 66 kN/m are required to achieve the initial compression of 20 mm (Fig. 6.21). Under consideration of the non-linearity of the force-displacement line of the gasket the residual compressive restoring force after deformation of the segmental ring results to approx. 37 kN/m (Fig. 6.21). Based on this result it needs to be checked in each particular case, if the tightness of the used gasket frame is still given with the reduced restoring force for the existing water pressure (see also section 6.4.4). The resulting tensile forces in the screws can also be evaluated by this finite element analysis. They are, however, not represented here.

![Force-displacement diagram of the gasket](image)

**Fig. 6.21:** Evaluation of the restoring force acting on the gasket after deformation of the segmental ring with the aid of the force-displacement diagram
Insufficiently bedded segmental ring, loading due to bending caused by self-weight

If the annular gap is only partially filled, a uniform circumferential bedding is no longer guaranteed. This may lead to a relatively high bending load, even in case of loading due to self-weight only. On the basis of analyses carried out for the segmental ring, which was already described in section 6.2.1, the loading resulting from this special load case are exemplarily evaluated. Furthermore, the resulting required reinforcement is compared with the reinforcement required for the other load cases. These analyses were carried out for different extents of the grouted sections of the annular gap. In the following the case with the smallest bedded section, which is located at the invert of the tunnel, is considered (Fig. 6.22).

Fig. 6.22: Insufficiently bedded segmental ring, loading due to self-weight, computation assumptions

The analyses are carried out using the two-dimensional mesh (Fig. 6.1), already described in section 6.2.1. The entire computation section except, the elements modeling the segmental ring, is assumed to be weightless, since only the loading of the ring due to self-weight is to be investigated. The rock mass is assumed to be elastic. A bedding of the ring is only possible in the area of the invert, where the annular gap is modeled to be grouted with mortar.
Fig. 6.23: Insufficiently bedded segmental ring, computation steps

1st computation step
pre-stressing

$\sigma_H = \frac{\nu}{1-\nu} \cdot \sigma_V$

2nd computation step
"stressless" installation of the concrete
=> restoring forces
The remaining elements modeling the annular gap are assigned to a Young's modulus of $E \approx 0$ (air). The screws connecting the segments are simulated by truss elements with their true stiffness. The gasket simplifying is modeled rectangular. The assumed dimensions correspond to those of the compressed profile at the time of installation.

As the analyses presented before, by which the effectiveness of the gaskets were investigated, the analyses are carried out in two computation steps (Fig. 6.23). In the first step the pre-stressing of the gasket and the screws is simulated. In the second step the "stressless" installation of the concrete is modeled and the pre-stressing forces are removed. This leads to a loading of the concrete due to its self-weight and the restoring forces. Thus, the loading of the segments is modeled realistically.

![Insufficiently bedded segmental ring](image)

**Fig. 6.24:** Insufficiently bedded segmental ring, loading due to self-weight, displacements
The segmental ring deforms due to the loading and the reduced bedding, similar to the analyses regarding the effectiveness of the gaskets presented before, into a lying ellipse. For this load case, as expected, large displacements of 19 cm at the roof and 16 cm at the sidewalls result (Fig. 6.24).

Furthermore, the segments are considerably overloaded. In Fig. 6.25 the required reinforcement resulting from a dimensioning with factors of safety according to standards is colored in red. Fig. 6.26 shows the required reinforcement resulting from a dimensioning with a factor of safety of 1.0. The required reinforcement, which was evaluated for a circumferential bedding of the segmental ring for the load combination 'self-weight + rock mass pressure + water pressure + temperature' is represented in blue in figures 6.25 and 6.26 for comparison. This reinforcement corresponds to the minimum required reinforcement according to the German Standard DIN 1045, Part 1 (2001). Even if the factor of safety is reduced to a value of \(\eta = 1.0\), the minimum reinforcement is not sufficient to withstand the bending load of the segmental ring, when it is bedded only at the invert (Fig. 6.26). Similar results are also obtained for greater grouted sections of the annular gap.
The analyses show that a complete filling of the annular gap in the area of the tail-skin is necessary to avoid damages and leakage.

![Diagram of segmental rings with load combinations and dimensions](image)

**Fig. 6.26:** Insufficiently bedded segmental ring, loading due to self-weight, dimensioning with a factor of safety of $\eta = 1.0$

### 6.3 Analyses for the individual segment (loads resulting from installation, storage and transportation)

#### 6.3.1 Loads due to jacking forces

During heading the segments are loaded at the front faces by the jacking forces. At the back sides of the segments these forces are transferred into the segmental rings already installed by means of load transmission pads, which in most cases are made of hardboards (see Section 2.2.2, Fig. 2.48). Due to the lateral extension of the loads applied by the jacks, transverse tensile forces are induced, which must be carried by an adequate reinforcement.
Fig. 6.27: Loads due to jacking forces, three-dimensional FE-mesh, boundary conditions, parameters and loading

In order to evaluate the tensile splitting reinforcement, three-dimensional finite element analyses are suitable, in which the three-dimensional state of stresses can be modeled. In the follow-
ing, as an example, the corresponding analysis for the tunnels of the railroad line Genoa - Ventimiglia will be dealt with.

In the FE-mesh the regular stone is modeled (Fig. 6.27). At the front face of the FE-mesh the loads from the jacking forces are applied. The segment is assumed to be loaded by three double jacks with a force of 3716 kN each. These loads are applied as local surface loads on partial areas, with dimensions of 17 cm x 90 cm. The dimensions are determined by the height of contact areas in the circumferential joint and the width of the jack shoes.

The hardboards are modeled as support at the back side of the segment and in this project they extend over nearly the whole thickness of the segment. Both lateral boundary planes in the area of the contact surfaces are held by sliding supports in circumferential direction. At the bottom side of these boundary planes additional sliding supports are located. The concrete is characterized by the same parameters as explained in Section 6.2.1.

The results of these analyses are presented in Section 6.4.3.

6.3.2 **Storage and transportation**

It has to be proven that the individual segments remain undamaged during all stages of loading and storage from striking to installation. For this punctual, linear and laminar supports of the segments need to be considered. Punctual and linear supports for instance occur during the storage of the segments after striking. A laminar support for example occurs, when extracting the segments from the framework with the aid of vacuum suction plates, and during the installation of the segments using a vacuum erector. In all proofs the existing strength of the concrete at the particular point in time of loading must be taken into account. Additional loads resulting from impacts must also be accounted for. These can be considered by multiplying the static loads by a vibration coefficient \( \varphi \). According to the German Standard DIN 1055, Part 3 (2002) the vibration coefficient can be selected to \( \varphi = 1.4 \).

Subsequently, two examples for analyses to prove the loading of individual segments due to storage and transportation are presented. Punctual and linear supports respectively are taken into account.
The segments for the tunnels of the railroad line Genoa – Ventimiglia are seated upon a rig which enables their transportation inside of the production hall after striking. This rig provides four supports of the segments with a spacing of 1.6 m in longitudinal direction and 1.2 m in transverse direction. The concrete quality at the time of striking corresponds to a concrete grade of at least C 12/15 (Fig. 6.28a).

For the computation of this example a simplified system is used. A longitudinal, 40 cm wide strip is considered, which is located above the supports. If the segments are sufficiently reinforced in the transverse direction, the stress resultants can be evaluated for a beam on two supports. Then the 40 cm thick segment can be dimensioned for the loading due to self-weight of half of the segment (Fig. 6.28a).

As a second example, the storage of the segments in a stock, after leaving the production hall is considered. In such a stock all seven segments of a ring are stacked on top of each other. Between the individual segments and underneath the complete stack, wooden beams with a spacing of approx. 2.75 m are located. The concrete quality at the beginning of the storage corresponds to a concrete grade of at least C 20/25 (Fig. 6.28b).

If the wooden beams are lying exactly upon each other, the segments are loaded by their self-weight only. In practice however, not all beams are stacked perfectly on top of each other. Therefore, the unfavorable case is considered, that one beam between the lowest two segments is shifted outwards by maximal 10 cm. This eccentricity leads to an additional single load on the lowest segment, which leads to an additional bending moment and shear force in the segment. The corresponding statical system and loading are represented in Fig. 6.28b.
Fig. 6.28: Load cases storage and transportation, examples for computation assumptions: a) storage and transportation in a pre-casting plant; b) stacking of segments
6.4 Static proofs

6.4.1 Dimensioning for stress resultants normal thrust (N), bending moment (M) and shear force (S)

The stress resultants for the load combination "self-weight + rock mass pressure", "water pressure" and "temperature" (see Section 6.2.2 to 6.2.6), which were evaluated for the segmental rings of the tunnels for the railroad line Genoa – Ventimiglia, are represented in Figures 6.29 to 6.31. The load case "water pressure" leads to the highest loading of the segmental ring (Fig. 6.30). The normal thrust of 4050 kN/m is comparatively high. The maximum bending moment results to approx. 80 kN/m. The stress resultants for the load combination "self-weight + rock mass pressure" are considerably smaller (Fig. 6.29). Fig. 6.31 shows exemplarily the results for the temperature load case "summer". An almost uniform loading due to bending moments and normal thrusts over the entire cross-section results.

Fig. 6.29: Stress resultants for the load combination "self-weight (S) + rock mass pressure (R)"

The dimensioning of the segmental lining should be carried out on the basis of the new European and national standards for geotechnics and reinforced concrete construction according to the concept of partial safety factors. Comments with regards to the dimensioning of railroad tunnels based on the concept of partial safety according to the updated Guideline 853 of the German Railroad, which will be published in a short time, can be taken from Schuck and Städing (2005). The application of the concept of partial safety to the dimensioning of road tunnels is explained in the reports of the German Federal Highway Research Institute (BAST, 2003) and Friebel et al. (2004).
Accordingly, in order to prove the ultimate limit state the dominant loads due to rock mass pressure and water pressure have to be multiplied with the partial safety factors according to the German Standard DIN 1054, Table 2 (DIN 1054, 2005). Consequently, a partial safety factor of $\gamma_p = 1.35$ must be selected for permanent loads and states lasting for the operational time of the tunnel (final state).

The partial safety factor for temperature loads according to ZTV-ING, Part 5, Section 2 (ZTV-ING, 2003) and Schuck and Städing (2005) can be selected to $\gamma_T = 1.0$. The input parameters of the finite element analysis such as unit weight, stiffness, strength,
water pressure etc. are specified with their characteristic values. As a consequence non-linear stress-strain relationships and failure criteria can be applied without restrictions and unchanged (Schuck and Städing, 2005; BAST, 2003). For dimensioning the calculated stress resultants therefore are multiplied with the corresponding partial safety factors.

The characteristic resistances and the corresponding partial safety factors for structures and components of reinforced concrete are to be specified according to the German Standard DIN 1045, Part 1 (2001). Accordingly, the partial safety factors for concrete and steel have to be selected as $\gamma_c = 1.5$ and $\gamma_s = 1.15$ respectively for permanent and temporary loads.

For dimensioning the stress resultants of the single load cases must be superimposed (Fig. 6.32). The stress resultants from the load case "temperature" are reduced by the factor 1/1.35, because they, as mentioned above, only need to be dimensioned with the reduced factor of safety of $\gamma_t = 1.0$. The dimensioning is carried out for the superimposed stress resultants with a factor of safety of $\gamma_s = 1.35$. Under consideration of the specifications given in the aforementioned example (concrete quality C45/55, steel quality BSt 500, thickness of segments $h = 40$ cm, distance of the reinforcement from the edge $d_1 = 6.5$ cm) the dimensioning for the decisive load combination leads to the result, that neither a bending reinforcement nor a shear reinforcement is statically required.

Fig. 6.32: Stress resultants for the load case "S + R + W + T_s"

**remark:** The stress resultants for the temperature load case "summer (T_s)" were reduced by the factor 1/1.35.
Additional proofs regarding the ultimate state of the serviceability are to be carried out. In connection with the dimensioning of the segmental lining particularly the proofs concerning the minimum reinforcement and the limitation of crack width are of importance. Requirements with regards to the minimum reinforcement can be found in the German Standard DIN 1045, Part 1 (2001), in the Technical Report No. 102 of the DIN (2003) as well as in the Guideline 853 of the German Railroad (DB Netz, 2003). The proof of the limitation of crack width can be carried out according to the German Standard DIN 1045, Part 1 (2001). Specifications for the admissible crack widths can be found in Guideline 853 (DB Netz, 2003) and in ZTV-ING, Part 5, Section 1 (ZTV-ING, 2003). For the single segmental lining the requirements for waterproof concrete are valid (see section 6.6.1).

**assumptions:**

DIN 1045, Part 1 (2001); failure load; \( h = 40 \text{ cm} \); \( d_1 = 6.5 \text{ cm} \);
BSI 500; \( \gamma_G = 1.35 \); \( \gamma_C = 1.5 \); \( \gamma_S = 1.15 \)

**Fig. 6.33:** Dimensioning for the load combination "\( S + R + W + T_s \)"
In the presented example the segments have to be reinforced at the inside and the outside in circumferential and longitudinal direction, respectively, by a minimum reinforcement corresponding to 0.15 percent of the cross-sectional area of the concrete. With a thickness of the segments of \( h = 40 \text{ cm} \) this leads to a minimum reinforcement of \( a_s = 6 \text{ cm}^2/\text{m} \) per side and direction (Fig. 6.33).

### 6.4.2 Partial area compression and tensile splitting reinforcement at the longitudinal joints

#### Partial area compression

The normal thrusts acting in circumferential direction are transferred by the longitudinal joints of the segmental ring. An eccentricity of the normal thrust leads to a bending load of the joint.

Due to the reduction of the cross-sectional area of the concrete (see Fig. 6.2) increased partial area compression at the contact areas occur. In Fig. 6.34 the maximum normal stresses acting in the longitudinal joints are represented for the load combination "S + R + W + T_s", which is decisive for the presented example. At the edges of the contact areas stress peaks occur, which amount up to \( \sigma = 94.6 \text{ MN/m}^2 \). The average stress results to \( \bar{\sigma} = 39 \text{ MN/m}^2 \).

As already mentioned, the analyses were carried out assuming an elastic stress-strain behavior for the concrete. Due to the fine discretization of the joint elements, the stress peaks are determined up to a distance of only 1 cm from the edges of the contact areas. In areas with smaller distances, higher stresses can occur and may lead to plastification. The geometry of the joints and the position of the gasket frames must be selected in a way that these plastification does not lead to damages and leakages.

For the geometry of joints represented in Fig. 6.2 and the provided concrete quality C45/55 an admissible partial area compression of \( \bar{\sigma} = 34.1 \text{ MN/m}^2 \) can be determined according to DIN 1045, Part 1 (2001). Thus, the calculated value (Fig. 6.34) exceeds the admissible value (see Fig. 6.34).

The proof according to DIN 1045, Part 1 (2001) is however based on an uniaxial state of stress. The strains in longitudinal direction of the tunnel in the area of a segmental ring are however confined by the adjacent segments. In radial direction the bedding of the
rock mass prevents an outward displacement. Under consideration of this three-dimensional state of stress the partial area compressions which are acceptable in reality are higher than the admissible value according to DIN 1045, Part 1 (2001).

Fig. 6.34: Maximum partial area compressions in the longitudinal joints, load combination "S + R + W + Tₜ"

In the course of the planning and construction of road tunnels, amongst others for the 4th tube of the Elbe-Tunnel in Hamburg (Schreyer and Winselmann, 1998; STUVA, 1996) and for the Weser-Tunnel (TU Braunschweig, 1992), tests on lining segments were carried out. As a result of these tests it could be shown, that the
stresses which can be transferred by the longitudinal joints without any damages are much higher than the admissible values according to DIN 1045, Part 1 (2001). The transverse tensile stresses resulting in the area of the joints, however, have to be covered by a corresponding tensile splitting reinforcement in any case.

**Tensile splitting reinforcement**

Transverse tensile stresses occur due to the redistribution of the normal stresses from the contact area in the longitudinal joints to the total width of the cross-sectional area of the segment.

Fig. 6.35: Transverse tension in the area of the longitudinal joints, principal normal tensile stresses, load combination "S + R + W + T_s"
In Fig. 6.35 the calculated principal normal tensile stresses in the area of the longitudinal joints for the decisive load combination \( S + R + W + T_s \) are represented. Transverse tensile stresses of up to \( \sigma_t = 2.6 \text{ MN/m}^2 \) occur. The distribution of the transverse tensile stresses in selected sections is shown in Fig. 6.36. The required reinforcement can be graded corresponding to the distribution of stresses. The dimensioning according to DIN 1045, Part 1 (2001) leads to a total required tensile splitting reinforcement of 12.5 cm\(^2\)/m (Fig. 6.36).

**Fig. 6.36:** Tensile stresses in selected sections and required tensile splitting reinforcement req. \( a_s \), load combination \( S + R + W + T_s \)
Fig. 6.37: Proof regarding the tensile splitting reinforcement at the longitudinal joints according to DAfStb, Issue 240: a) Resulting tensile splitting force due to an applied centric compressive force; b) example under consideration of an applied eccentric compressive force

**proof:**

\[ \gamma_G = 1.35 \quad \gamma_s = 1.15 \quad \gamma_c = 1.5 \]

\[ e = \frac{M}{N} = 0.006 \text{ m}; \quad d = 0.170 - 2 \cdot e = 0.158 \text{ m}; \quad d_s = 0.4 - 2 \cdot e = 0.388 \text{ m} \]

\[ Z_{s,d} = \gamma_G \cdot Z_s = 1.35 \cdot Z_s \]

\[ Z_s = 0.25 \cdot N \cdot \left(1 - \frac{d_1}{d_s}\right) = 0.25 \cdot 6612 \cdot \left(1 - \frac{0.158}{0.388}\right) = 980 \text{ kN/m} \]

\[ Z_{s,d} = 1.35 \cdot 980 = 1323 \text{ kN/m} \]

\[ Z_R = N \cdot \left(\frac{e}{d_s} - \frac{1}{6}\right) = 6612 \left(\frac{0.006}{0.40} - \frac{1}{6}\right) < 0 \text{ (compression)} \]

\[ Z_{s2} = 0.3 \cdot Z_R < 0 \text{ l (compression)} \]

\[ A_{s,d} = \frac{Z_{s,d}}{f_{yd}} = \frac{1323 \text{ kN/m}}{50/1.15 \text{ kN/cm}^2} = 30.4 \text{ cm}^2/\text{m} \]
The required tensile splitting reinforcement alternatively can also be evaluated according to Issue 240 of German Committee for Reinforced Concrete (DAfStb, 1991), where formulas for the calculation of the tensile splitting forces are given (Fig. 6.37a). The calculation of the tensile splitting forces using these formulas, is based on truss models, by means of which the actual stress distribution in the areas of the application of load is taken into account. Thus, it is an approximation procedure. In Fig. 6.37b, as an example, the required tensile splitting reinforcement for the longitudinal joints according to DAfStb, Issue 240 (1991) is evaluated for the stress resultants which were calculated for the decisive load combination of the presented example. The required tensile splitting reinforcement for the joint geometry given in Fig. 6.2 results to approx. 30 cm²/m. This is more than two-times of the required reinforcement which was evaluated on the basis of from the results of the finite element analyses.

The dimensioning according to DAfStb, Issue 240 (1991) is conservative. In order to avoid unnecessary high reinforcement it is therefore recommended to evaluate the required tensile splitting reinforcement by means of finite element analyses, which enable to calculate the state of stress adjacent to the longitudinal joints more realistically. The layout of the reinforcement should be adapted to the calculated distribution of the transverse tensile stresses.

6.4.3 Partial area compression and tensile splitting reinforcement at the circumferential joints

Partial area compression

The jacking forces are introduced into the already installed segmental rings using jacks. These forces are applied by jack shoes along partial areas in the circumferential joint at the front face of the segments. In order to transfer these loads at defined positions to the next segmental ring, in most cases timber sandwich layers consisting of hardboard are arranged at the back sides of the segments.

In the area of the jack shoes and of the sandwich layers for load transmission, partial area compressions occur. The corresponding stresses in the circumferential joints in most cases are however smaller than the corresponding stresses in the longitudinal joints
(see section 6.4.2). The proof of the partial area compression can be carried out according to the German Standard DIN 1045, Part 1 (2001).

In addition, thrust rings which allow for the distribution of the loads from the jacks over the whole circumferential joint can be used. Thus, the loads due to partial area compression can be reduced.

**Tensile splitting reinforcement**

Because the loads are introduced into the segments by the jack shoes in limited areas and also the load transmission through the sandwich layers is local, transverse tensile forces in circumferential direction occur in the segments (Fig. 6.38). The larger the spacing of the jack shoes is, the larger the corresponding tensile forces are. At every jack shoe also transverse tensile force in radial direction occur (Fig. 6.38).

![Fig. 6.38: Proof with regards to transverse tension at the circumferential joints, transverse tensile forces](image)

The transverse tensile forces can be evaluated with the aid of finite element analyses or according to DAfStb, Issue 240 (1991). If the approximate procedure according to DAfStb, Issue 240 (1991) is used, conservative results are achieved. If finite element analyses are carried out, the transverse tensile stresses can be evaluated more accurately (see section 6.4.2).

In the following, the proof with regard to the transverse tensile stresses in the circumferential direction for the segments of the tunnels of the railroad line Genoa - Ventimiglia will be explained. The stresses are evaluated by means of the three-dimensional finite element analysis described in Section 6.3.1.
Fig. 6.39 shows the principal normal stresses within the segment in a developed view of section 1-1 (see Fig. 6.38). The compressive stresses induced by the jack shoes are redistributed in circumferential direction with increasing distance to the jack shoes.

Fig. 6.39: Three-dimensional finite element analysis, principal normal stresses (Section 1-1, developed view, see Fig. 6.38)

Fig. 6.40: Proof with regard to transverse tension at the circumferential joints, stresses in selected sections and required tensile splitting reinforcement
The stresses decrease from $\sigma_C = 23.3 \text{ MN/m}^2$ close to the areas of load introduction, to $\sigma_C = 9.6 \text{ MN/m}^2$ at the back side of the segment (Fig. 6.39). Between the jack shoes transverse tensile stresses of $\sigma_T = 4.3 \text{ MN/m}^2$ occur. At the back side of the segment the maximum transverse tensile stresses amount to $\sigma_T = 2.4 \text{ MN/m}^2$. Fig. 6.40 shows the distribution of these stresses in transverse direction.

In order to transfer the tensile stresses, a tensile splitting reinforcement is installed at the circumferential joints at the front face and the back side of the segment. The dimensioning according to the German Standard DIN 1045, Part 1 (2001) results to a required reinforcement of 7.5 cm² at the front face, in the area between the jack shoes (Fig. 6.40).

### 6.4.4 Gaskets

Single segmental linings normally are sealed by gasket frames composed of elastomer (see section 2.2.2). The proof of suitability for the gasket frames is carried out by experiments. Recommendations with regards to these tests are compiled in STUVA (2005). The tests must be carried out on test specimens made of steel or concrete. Specimens made of concrete particularly should be used for gaskets with anchoring elements (see Fig. 2.57). The shape of the groove in the specimen must correspond to that in the segment. The decisive water pressure has to be maintained at least 24 hours without leakage. At least two tightness tests with the same parameters must be carried out. The smaller of the water pressures which was withstand in both tests is decisive for the evaluation (STUVA, 2005).

The tightness tests are carried with different offsets (usually up to 20 mm) and groove basic gaps. The specifications with regards to the offsets and the groove basic gaps result from the planning for the tunneling project in question. It has to be accounted for, that the gaskets must also be effective, if the lining deforms and/or the construction tolerances are fully utilized.

Fig. 6.41 in principle shows different states of gaskets before and after installation. In the non-compressed state, before installation, the joint gap width and the groove basic gap for this example amount to $s = 24 \text{ mm}$ and $b = 46 \text{ mm}$, respectively (Fig. 6.41a). During installation of the segments, the gaskets should be
compressed by $\delta_p = 18$ mm. According to plan, the joint gap width and the groove basic gap in this state amount to $s = 6$ mm and $b = 28$ mm, respectively (Fig. 6.41b).

After installation, the segments are loaded and consequently the joints may open locally. The maximum opening according to plan can be taken from the results of the stability analyses carried out for the segmental rings. In the example represented in Fig. 6.41c, an opening of 6 mm is allowed for. Moreover, assembly inaccuracies as well as the different loading of the segmental rings lead to offsets after installation, which are usually limited to a maximum of 20 mm (Fig. 6.41c). For the example represented in Fig. 6.41 a maximum offset of $\max V = 20$ mm, a maximum joint gap width of $\max s = 6$ mm + $6$ mm = $12$ mm, and a maximum groove basic gap of $\max b = 28$ mm + $6$ mm = $34$ mm are relevant for dimensioning.

Fig. 6.41: Joint gap width, groove basic gap and offset of gasket: a) initial state (non-compressed state); b) compression according to plan during installation of the segments; c) admissible displacements and offset due to assembly inaccuracies and loading of segments
In Fig. 6.42 the result of a tightness test for a gasket frame is represented. The tests were carried out with offsets ranging between 0 and 20 mm. Without offset \((V = 0)\) and with a joint gap width of \(s = 16\) mm (corresponding to a groove basic gap of \(b = 2 \times 11\) mm + 16 mm = 38 mm) a water pressure of 20 bar could be maintained. To maintain the same water pressure with an offset of \(V = 20\) mm, a 4 mm greater compression, and thus a joint gap width of \(s = 12\) mm (corresponding to a groove basic gap of \(b = 2 \times 11\) mm + 12 mm = 34 mm) is required.

Fig. 6.42: Results of a tightness test and design for a gasket

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Fig. 6.43: Force-displacement line and design for a gasket
The compressive stress in the gaskets due to relaxation at room temperature, decreases within 100 years to approx. 50 % of its initial value (see section 2.2.2 and Fig. 2.58). In order to assure the tightness for the service life of a tunnel, therefore, relaxation has to be taken into account for the design of the gasket. The corresponding procedure is explained subsequently.

Fig. 6.43 shows the force-displacement line of a gasket. Without offset (V = 0) the compression up to a joint gap width of s = 6 mm (corresponding to a groove basic gap of b = 2 x 11 mm + 6 mm = 28 mm) requires a loading of approx. 95 kN/m. If a reversible, elastic stress-strain behavior and a relaxation of 50 % are assumed, a "virtual" opening of the joint gap of $\delta_{\text{Relaxation}} = 6$ mm results (Fig. 6.43). The joint gap width, which is decisive for the design of the gasket, for this case can be determined as follows:

<table>
<thead>
<tr>
<th>Joint gap width according to plan:</th>
<th>6 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>+ maximum opening due to loading and/or assembly inaccuracies:</td>
<td>+ 6 mm</td>
</tr>
<tr>
<td>+ &quot;virtual&quot; opening due to relaxation:</td>
<td>+ 6 mm</td>
</tr>
</tbody>
</table>

Joint gap width decisive for design: 18 mm

According to the results of the tightness test for a joint gap width of s = 18 mm (corresponding to a groove basic gap of b = 2 x 11 mm + 18 mm = 40 mm) a design water pressure of 11 bar can be maintained (Fig. 6.42).

The same considerations lead to a relevant joint gap width of s = 16.5 mm (Fig. 6.43) and a design water pressure of 7 bar (Fig. 6.42), if the offset amounts to V = 20 mm.

The design of the gasket based on the test results represented in Figures 6.42 and 6.43, was carried out for the following boundary conditions:

- Service life of the tunnel $\leq$ 100 years,
- maximum relaxation of the gasket of 50 % within 100 years,
- compression of the gasket during installation up to a joint gap width of s = 6 mm (corresponding to a groove basic gap of b = 2 x 11 mm + 6 mm = 28 mm),
- maximum opening of the gasket of 6 mm due to loading of the segments and/or assembly inaccuracies,
- maximum offset of 20 mm.

The relaxation behavior of the gasket must be investigated by means of separate tests carried out over 3 months at a temperature of 70° (STUVA, 2005).

**proof acc. to DAfStb, Issue 240:**

\[ \gamma_G = 1.35 \quad \gamma_c = 1.8 \text{ (unreinforced concrete)} \quad F = 95 \text{ kN/m (see Fig. 6.43)} \]

\[ Z_s = 0.25 \cdot F \left(1 - \frac{d_1}{d_s}\right) = 0.25 \cdot 95 \cdot \left(1 - \frac{0.044}{0.114}\right) = 14.6 \text{ kN/m} \]

\[ Z_{s,d} = 1.35 \cdot 14.6 = 19.7 \text{ kN/m} \]

**proof for the tensile stresses:**

\[ \sigma_{ct,d} = \frac{Z_{s,d}}{0.8 \cdot d_s} = \frac{19.7}{0.4 \cdot 0.114} = 432.0 \text{ kN/m}^2 = 0.43 \text{ MN/m}^2 \]

\[ \sigma_{Rd,ct} = \frac{f_{ctk,0.05}}{\gamma_c} = \frac{2.7}{1.8} = 1.5 \text{ MN/m}^2 \text{ (C45/55)} \]

\[ \sigma_{ct,d} = 0.43 \text{ MN/m}^2 < 1.5 \text{ MN/m}^2 = \sigma_{Rd,ct} \]

**Fig. 6.44:** Proof of groove for gasket with regards to tensile splitting, example
**System and loading:**

![Diagram showing the system and loading](image)

**Proof of groove for gasket with regards to shear loading, example**

At the edges of the groove for the gasket, compressive stresses are introduced into the segments due to the compression of the gaskets during installation of the segments. Due to the redistribution of these stresses in the segment, transverse tensile

**Load bearing capacity of unreinforced concrete with regards to shear force:**

\[
V_{Rd,ct} = \sqrt{(f_{ck;0.05}/\gamma_c)^2 - \sigma_{cd} \cdot f_{ck;0.05}/\gamma_c} \cdot \frac{l \cdot b_w}{S} \quad \text{(acc. to DIN 1045, Part 1)}
\]

\[f_{ck;0.05} = 2.7 \text{ MN/m}^2 \quad \text{(C45/55)}\]

\[l/S = 0.023 \text{ m} \quad \gamma_c = 1.8 \quad \text{(unreinforced concrete)}\]

\[\sigma_{cd} = 0.3 \text{ MN/m}^2 \quad b_w = 1 \text{ m}\]

\[V_{Rd,ct} = 0.031 \text{ MN/m}\]

**Proof:**

\[V_{sd} = 1.35 \cdot 0.0217 = 0.029 \text{ MN/m}\]

\[\Rightarrow V_{sd} = 0.029 \text{ MN/m} < 0.031 \text{ MN/m} = V_{Rd,ct}\]

**Fig. 6.45:** Proof of groove for gasket with regards to shear loading, example

At the edges of the groove for the gasket, compressive stresses are introduced into the segments due to the compression of the gaskets during installation of the segments. Due to the redistribution of these stresses in the segment, transverse tensile
stresses occur. Because of the small size of the groove for gasket, and the small spacing between the groove and the outside edge of the segment the installation of a tensile splitting reinforcement in this zone usually is not possible. Therefore, the tensile strength of the unreinforced concrete of the segments is accounted for the proof of the transverse tensile stresses. Fig. 6.44 shows, as an example, the proof for tensile splitting for a gasket with a restoring force of F = 95 kN/m.

Furthermore, it needs to be proven that the shear loading of the unreinforced concrete of the segments does not exceed the admissible values. Fig. 6.45 exemplarily shows the proof for shear loading for a gasket with a restoring force of F = 95 kN/m.

Regardless of these proofs, spalling tests should be carried out before production of the segments, in order to check if the restoring forces of the selected gasket frames can be carried by the concrete of the segments (STUVA, 2005).

In order to prevent penetration of the grout into the longitudinal joints, pre-gaskets should be foreseen at the outside edges of the joints (DB Netz, 2003). At the inner edges grooves for gasket must be foreseen, in order to allow for a subsequent installation of gaskets, if the outer gasket frame becomes leaky.

6.4.5 Screwing of segments

The segmental rings are screwed together during installation. The screwing is not allowed to be released until the restoring forces of the gaskets can be transferred into the rock mass by means of the hardened mortar in the annular gap (Fig. 6.46a). The screwing must be durable maintained in areas where the segmental rings may displace in longitudinal direction, e. g. in portal areas. Therefore, corrosion-resistant screws must be used in these areas (see section 2.2.2).

The purpose of the screwing is to prevent an opening of the joints after compression of the gaskets. The restoring force of the gaskets are decisive for the dimensioning of the screwing. In the case of railroad tunnels with a single lining, the pre-stressing force of the screws should exceed the restoring force of the gasket frame, determined in the laboratory, by at least 50 % (Fig. 6.46a; DB Netz, 2003).
Fig. 6.46: Proof for the screwing: a) Requirements; b) developed view of screwing (example); c) evaluation of the required pre-stressing force (= 1.5-fold restoring force)

Fig. 6.46b shows an example for the screwing of a segmental ring. The circumferential joints here are connected with 36 screws, which are distributed uniformly along the circumference. In the longitudinal joints 2 screws per segment are foreseen.

The evaluation of the required pre-stressing force is given in Fig. 6.46c. The required pre-stressing force of the screwing for the circumferential joints can be calculated using the following formula:

\[ P \geq 1.5 \cdot \frac{C \cdot R}{n \cdot \cos \alpha} \]

where:
- \( C \): circumference of the joint,
- \( R \): restoring force of the gasket,
- \( n \): number of screws in the circumferential joint,
- \( \alpha \): angle between segment axis and screws (Fig. 6.46a),
1.5: factor of safety for railroad tunnels according to Guideline 853 (DB Netz, 2003),
P: pre-stressing force per screw.

The required pre-stressing force for the screwing of the longitudinal joints can be evaluated by

\[ P \geq 1.5 \cdot \frac{W \text{ [m]} \cdot R \text{ [kN/m]}}{n \cdot \cos \alpha} \]

where:
- \( W \): width of segment,
- \( R \): restoring force of the gasket,
- \( n \): number of screws in the longitudinal joint per segment,
- \( \alpha \): angle between segment axis and screws (Fig. 6.46a),
- 1.5: factor of safety for railroad tunnels according to Guideline 853 (DB Netz, 2003),
- \( P \): pre-stressing force per screw.

For the design of the screwing of the longitudinal joints between the keystone and the boundary stones, the double sided inclination of the screws with respect to the segment axis must be considered in addition.

The screw forces are induced concentrated into the segment at the screw heads using shims. The partial area compression should be designed for by calculations or experiments. Due to the redistribution of the stresses in the segment, transverse tensile stresses occur. These stresses should also be accounted for by means of calculations. Possibly a tensile splitting reinforcement may become necessary. An example for a corresponding reinforcement is shown in section 6.5.

The implementation of the pre-stressing forces in the segments by means of dowels should be verified by experiments. The tests should be carried out preferably on segments, at least however on concrete with the same strength category. Load drops due to creep, which appear comparatively fast after pre-stressing and then fade away asymptotically, should be considered for the dimensioning.
The factor of safety of 1.5, specified for railroad tunnels in Guideline 853 (DB Netz, 2003), can be reduced in special cases if uncertainties with regards to

- the restoring forces of the gaskets,
- the torque required for the prestressing of the screws,
- the devices and the required energy for the application of the preload force, and
- the creep behavior of the dowels

are eliminated by means of particular investigations and if a company internal approval as well as an agreement of the regulatory authority is available (DB Netz, 2003).

6.5 Reinforcement layout

6.5.1 General

The reinforcement of a lining segment consists of the load bearing reinforcement in circumferential and longitudinal direction, the tensile splitting reinforcement adjacent to the longitudinal and circumferential joints, the boundary reinforcement as well as the reinforcement for block outs and built-in units. In addition a shear reinforcement or a reinforcement for the time of installation may become necessary. The different types of reinforcement are treated in the following sections.

During the design of the reinforcement, the general rules regarding reinforcement included in the German Standard DIN 1045, Part 1 (2001) need to be observed. These include, amongst others, information on the spacing of bars, lengths of overlap and anchoring as well as on the design of stirrups.

The required concrete cover is regulated in the Guideline 853 (DB Netz, 2003) and in the ZTV-ING, Part 5, Section 1 (ZTV-ING, 2003). According to Guideline 853 (DB Netz, 2003) a minimum concrete cover of \(c = 40\, \text{mm}\) must be guaranteed for the circumferential reinforcement on the inner and outer side of the segments made of reinforced concrete.
For structural fire protection a larger concrete cover is favorable. Therefore, in the Guideline 853 (DB Netz, 2003) and the ZTV-ING, Part 5, Section 1 (ZTV-ING, 2003) a minimum concrete cover of \( \text{min } c = 50 \text{ mm} \) is demanded for cast-in-place concrete if a proof of stability is not carried out for the load case fire (see section 6.2.7). Therefore, also for lining segments a minimum concrete cover of \( \text{min } c = 50 \text{ mm} \) for the load bearing reinforcement is to be recommended with regards to fire protection. There is however the risk, that the larger concrete cover in case of unforeseen impact loads during storage, transportation and ring erection may lead to larger spalling.

The concrete cover at the longitudinal and circumferential joints as well as at the block outs and built-in units can be selected smaller than at the inside and outside of the segment. In these areas normally a minimum concrete cover of \( \text{min } c = 20 \text{ mm} \) is considered sufficient.

In order to take into account unforeseen deviations, the minimum concrete cover \( \text{min } c \) needs to be increased to \( \text{nom } c \) by adding a forward allowance of \( \Delta c \). The forward allowance in general is specified as \( \Delta c = 10 \text{ mm} \).

6.5.2 Circumferential and Longitudinal load bearing reinforcement

The load bearing reinforcement in circumferential and longitudinal direction is exemplarily represented in Fig. 6.47. The main load bearing reinforcement carries the tensile and compressive loads in circumferential direction. For this purpose rebars which are bent in circumferential direction (1 and 2 in Fig. 6.47) are provided at the inside and outside of the segment and anchored at the longitudinal joints using stirrups (3 in Fig. 6.47). These stirrups also serve as radial boundary reinforcement. The load bearing reinforcement, installed in longitudinal direction of the tunnel, results from the requirements on the minimum reinforcement (see section 6.4.1). For this, two stirrups (4 and 5 in Fig. 6.47) can be used, which are stuck together and overlap at the inside and the outside of the segment. The length of the overlapping varies, in order to allow for an adaption of the stirrups mostly conical shape of the segments (see section 2.2.2). The overlappings should be staggered. For this purpose the stirrups 4 and 5 are to be installed alternating from both sides (Fig. 6.47).
At the block outs at the inside of the segment a persistent installation of the load bearing reinforcement often is not possible. For this reason replacements are necessary. Rebars which cannot be installed continuously and therefore must be shortened have to be replaced by additional rebars with the same cross-section (6 in Fig. 6.47), which must be installed next to the block outs.

Fig. 6.47: Load bearing reinforcement in circumferential and longitudinal direction
Fig. 6.48: Tensile splitting reinforcement and boundary reinforcement
6.5.3  **Tensile splitting reinforcement**

Tensile splitting reinforcement must be installed at the longitudinal joints and the circumferential joints (see sections 6.4.2 and 6.4.3). Fig. 6.48 illustrates an example for the tensile splitting reinforcement of a segment.

When using this arrangement of reinforcement it is advantageous for production-related reasons, to install the reinforcement in longitudinal direction of the tunnel inside, and the main load bearing reinforcement in circumferential direction outside (see Fig. 6.47). The curved shape of the main load bearing reinforcement however leads to deflection forces, directed towards the boundary. These deflection forces have to be carried by the concrete cover, since the outside main load bearing reinforcement cannot be enclosed by stirrups and cannot be held by the transverse reinforcement respectively. The required proofs can be carried out for instance according to Leonhardt (1977).

For the tensile splitting reinforcement at the longitudinal joints often so-called "ladders" are used (7 and 8 in Fig. 6.48). Here, several vertical rebars, which have to carry the tensile splitting forces, are connected with longitudinal rebars on top and at the bottom. The longitudinal rebars in such a case must take over the function of anchoring.

As tensile splitting reinforcement in radial direction at the circumferential joints, stirrups can be used (9 in Fig. 6.48). These stirrups must be installed at the front face of the segment, in the areas which are loaded by the jack shoes. At the backside, the stirrups have to be provided in the area of the load transition. The tensile splitting reinforcement in circumferential direction consists of several curved rebars (10 in Fig. 6.48), running parallel to the circumferential joints. These can be installed within the stirrups and at the vertical leg of the longitudinal reinforcement, respectively.

6.5.4  **Boundary reinforcement**

As boundary reinforcement along the longitudinal and the circumferential joints, radial rebars in longitudinal and circumferential direction respectively have to be foreseen. Rebars which are required for load bearing reinforcement and tensile splitting reinforcement respectively, can also serve as boundary reinforce-
ment. In the example represented in Fig. 6.48 therefore, additional rebars are required only along the longitudinal joints (11 in Fig. 6.48).

Fig. 6.49: Reinforcement for block outs and built-in units
6.5.5 Reinforcement for block outs and built-in units

If necessary, separate reinforcement must be installed for block outs and built-in units (Fig. 6.49). In the area of the openings for the cones for centering usually a Helical reinforcement is installed (12 in Fig. 6.49). To carry the tensile forces in the area of the screw heads (see Section 6.4.5) and the dowels, it may become necessary to install tensile splitting reinforcement (13 and 14 in Fig. 6.49).

6.5.6 Shear reinforcement

In most cases a shear reinforcement is not required, because the segments are loaded mainly by normal thrusts (see Section 6.4.1). If necessary however, the shear reinforcement can be installed in the form of stirrups or additional, ladder-shaped or S-shaped rebars respectively.

Fig. 6.50: Reinforcement cages
6.5.7 Installation

The various reinforcements of the segment are combined to a reinforcement cage, which can be placed into the formwork completely (Fig. 6.50). For this, the several rebars are welded together at single points. If required, a mounting reinforcement has to be installed to fix the position of single bars.

The required concrete cover is ensured by means of reinforcement bar spacers, which have to be mounted along all formwork surfaces.

6.6 Concrete

6.6.1 Requirements

The concrete for a single segmental lining normally must meet demands with regards to workability, strength, durability and fire protection. Furthermore, a watertight concrete is required. Besides these requirements resulting from structural engineering, also economical considerations must be made regarding the raw materials and the composition of the concrete.

Workability

During placement into the formwork, the concrete must be evenly distributed, and during vibration a uniform compaction has to be achieved. The spread of the concrete and the compaction is not allowed to be hindered by the reinforcement, by block outs, e. g. for screwing or the cone for centering, and by built-in units such as dowels. Therefore, concrete for a segmental lining should have a good workability and a corresponding consistency during placement.

Strength

The segments for the single lining method must be produced from concrete of a strength class of at least C35/45 (DB Netz, 2003; ZTV-ING, 2004; Balthaus et al., 2005). The static proof in particular cases may lead to higher requirements for the final strength. Striking, storage and transportation of the segments normally takes place already, when the concrete has an age of approx. 8 to 12 hours. Depending on the dimensions of the segments, and the support conditions (punctual, linear or laminar
supports) early strengths ranging from 15 to 25 MPa may become necessary. The installation of the segments normally takes place, when the concrete has an age of 10 to 14 days. Thus the final strength required for statical reasons (C35/45 or higher) has to be available comparatively early.

**Durability**

The surfaces of the segments in the tunnel are subjected to air, humidity, water and aqueous solutions, which are present in the tunnel. It has to be assumed, that the external surfaces of the segments are in contact with the ground and the groundwater, because the mortar in the annular gap generally cannot be considered to be a permanent sealing. The exposures on these surfaces according to the German Standard DIN 1045, Part 2 (2001) are specified by exposition classes. Each tunnel can be classified into an exposition class, depending on the environmental conditions. These classes are listed in Table 6.2. The selection of the exposition class for an individual tunneling project has to be made in accordance with the relevant boundary conditions.

<table>
<thead>
<tr>
<th>exposition classes</th>
<th>attack on concrete</th>
<th>exposures</th>
</tr>
</thead>
<tbody>
<tr>
<td>X0</td>
<td>no risk of attack</td>
<td>-</td>
</tr>
<tr>
<td>XC1 to XC4</td>
<td>corrosion of reinforcement caused by carbonating</td>
<td>air and humidity</td>
</tr>
<tr>
<td>XD1 to XD3</td>
<td>corrosion of reinforcement caused by chlorides</td>
<td>water containing chloride including de-icing salt, no seawater</td>
</tr>
<tr>
<td>XS1 to XS3</td>
<td>corrosion of reinforcement caused by chlorides</td>
<td>seawater containing chloride</td>
</tr>
<tr>
<td>XF1 to XF4</td>
<td>attack by frost (combined-with or without de-icing agents)</td>
<td>freezing and thawing cycle</td>
</tr>
<tr>
<td>XA1 to XA3</td>
<td>chemical attack</td>
<td>ground and groundwater</td>
</tr>
<tr>
<td>XM1 to XM3</td>
<td>attack by wear</td>
<td>e. g. vehicles on concrete</td>
</tr>
</tbody>
</table>

Table 6.2: Exposition classes (from: DIN 1045, Part 1 and Part 2, 2001)
Clayey soils with a permeability of less than $10^{-5}$ m/s may be classified into a lower class.

The testing method describes the leaching of $\text{SO}_4^{2-}$ by hydrochloric acid; leaching by water may be used instead, if adequate experience is available at the location of the application of the concrete.

The limiting value must be reduced from 3000 mg/kg to 2000 mg/kg, if there is a risk of accumulation of $\text{SO}_4^{2-}$ ions in the concrete - caused by alternating drying and wetting or capillary suction.

The most unfavorable value of each individual chemical parameter defines the class. If two or more parameters lead to the same class, the environment must be classified into the next higher class, unless it is verified by means of a special investigation that this is not necessary for the given case.

If the content of $\text{SO}_4^{2-}$ ions within the groundwater is $\geq 1500$ mg/l, a cement with a high sulfate resistance (HS cement) has to be used.

<table>
<thead>
<tr>
<th>ground-water</th>
<th>chemical parameter</th>
<th>reference testing method</th>
<th>XA1 slightly aggressive</th>
<th>XA2 moderately aggressive</th>
<th>XA3 strongly aggressive</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\text{SO}_4^{2-}$ [mg/l]</td>
<td>EN 196-2</td>
<td>$\geq 200$ and $\leq 600$</td>
<td>$&gt; 600$ and $\leq 3000$</td>
<td>$&gt; 3000$ and $\leq 6000$</td>
<td></td>
</tr>
<tr>
<td>pH-value</td>
<td>ISO 4316</td>
<td>$\leq 6.5$ and $\geq 5.5$</td>
<td>$&lt; 5.5$ and $\geq 4.5$</td>
<td>$&lt; 4.5$ and $\geq 4.0$</td>
<td></td>
</tr>
<tr>
<td>$\text{CO}_2$ [mg/l]</td>
<td>prEN 13577: 1999</td>
<td>$\geq 15$ and $\leq 40$</td>
<td>$&gt; 40$ and $\leq 100$</td>
<td>$&gt; 100$ up to saturation</td>
<td></td>
</tr>
<tr>
<td>$\text{NH}_4^+$ [mg/l]</td>
<td>ISO 7150-1 or ISO 7150-2</td>
<td>$\geq 15$ and $\leq 30$</td>
<td>$&gt; 30$ and $\leq 60$</td>
<td>$&gt; 60$ and $\leq 100$</td>
<td></td>
</tr>
<tr>
<td>$\text{Mg}^{2+}$ [mg/l]</td>
<td>ISO 7980</td>
<td>$\geq 300$ and $\leq 1000$</td>
<td>$&gt; 1000$ and $\leq 3000$</td>
<td>$&gt; 3000$ up to saturation</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.3: Limit values for exposition classes XA1 to XA3, for a chemical attack by the ground and the groundwater, valid for water and soil temperatures between 5 °C and 25 °C and approx. hydrostatic conditions (from: DIN 1045, Part 2, 2001)

The limit values, which are decisive for the selection of the exposition classes XA1 to XA3 in case of chemical attack from the ground and the groundwater, are compiled in table 6.3. The decisive parameters for the degree of attack of the groundwater are the pH-value and the contents of $\text{SO}_4^{2-}$ ions, $\text{CO}_2$, $\text{NH}_4^+$ ions and $\text{Mg}^{2+}$ ions. For the chemical attack from the ground, the acidity and the content of $\text{SO}_4^{2-}$ ions are of importance (DIN 1045, Part 2, 2001). According to the German Standard DIN 4030 (1991) the contents of $\text{S}^{2-}$ and $\text{Cl}^{-}$ ions in mg/kg of the air-dried soil should be deter-
mined in addition. Sulfides for instance occur in pyrite containing soils or rocks respectively (pyrite = FeS₂). If the sulfide content is > 100 mg/kg, a separate assessment of an expert is required (DIN 4030, 1991).

The criterion of durability is fulfilled, if the tunnel maintains its load bearing capacity and serviceability during the planned service life-time. In DIN 1045, part 1 (2001) a minimum strength class of the concrete is given for each exposition class. According to DIN 1045, Part 1 (2001) minimum concrete covers of 10 mm to 50 mm and minimum forward allowances of 10 mm to 15 mm are required, for the exposition classes XC1 to XC4, XD1 to XD3 and XS1 to XS3, in order to ensure the protection of the reinforcement against corrosion and the transmission of bond forces within the concrete. An adequate durability of the structure is deemed to be ensured, if the requirements with regards to the raw materials and the composition of the concrete according to DIN 1045, Part 2 (2001) and DIN EN 206, Part 1 (2000), as well as the requirements regarding the execution of the construction work according to DIN 1045, Part 3 (2001), are fulfilled. Amongst others, these requirements include minimum cement content, maximum W/C-ratios and the need of a cement with a high sulfate resistance (HS cement), if the content of SO₄²⁻ ions in the groundwater is > 1500 mg/l.

In individual cases it may be necessary or economically advantageous, reasons to deviate from the requirements documented in DIN 1045, Part 2 (2001) and DIN EN 206, Part 1 (2000) respectively. With regards to the early strength of the concrete for instance, it may become necessary to use a Portland cement with high strength instead of the HS cement, although the sulfate content of the groundwater is > 1500 mg/l. Economically it may be advantageous to replace a high fraction of the cement by fly ash, and thus not to reach the prescribed minimum cement content. In such cases at least the following properties should be verified:

- workability,
- strength and strength development,
- permeability,
- hydration heat,
- durability (if relevant influences are present):
  - Permeability of the fly ash,
  - corrosion caused by chlorides,
  - alkali reserve of the cement lime,
- carbonating,
- resistance against frost and de-icing agents,
- alkali silica reaction,
- sulfate attack.

With regard to the proof of resistance against sulfate attack, three types of reaction have to be considered:

- Sulfate ions in the ground (e. g. in pyrite containing rocks) or in the groundwater, react with the cement lime (sulfate expansion).

- Ettringite is transformed into monosulfate and the latter reacts with the cement lime (sulfate expansion). This transformation can take place at temperatures ranging from 65 °C to 70 °C.

- Sulfate ions in the ground (e. g. in pyrite containing rocks) or in the groundwater react with fine-grained carbonatic concrete additives (e. g. limestone powder, limestone aggregate etc.) and with the cement lime. This reaction leads to a disaggregation of the matrix of the cement lime, and the concrete is transformed into a pulpy mass (DAfStb, 2001).

The resistance of concrete against sulfate attack consists of a chemical and a physical part. The chemical resistance is mainly determined by the used type of cement and the used cement fly ash ratio. The physical resistance mainly depends on the compactness of the fabric (DAfStb, 2003).

**Fire protection**

The following approaches for structural fire protection are currently pursued (see also section 2.2.4):

- Selection of non-inflammable construction materials (EBA, 201) of building material class A (DB Netz, 2003) and fire protection class ≥ F90,

- sufficient concrete covers,

- addition of synthetic fibers (Winkler, 2003; Dorgarten et. al., 2003),
- selection of suitable aggregates (Balthaus et. al., 2005; Dor-garten et. al., 2003),
- fire protection plates (Schlüter, 2005),
- fire protection plaster, fire protection reinforcement.

The selection of non-inflammable construction materials and, if required, the addition of synthetic fibers or the selection of suitable aggregates have to be considered during the selection of the raw materials and the composition of the concrete.

Watertight concrete

Specifications regarding the minimum thickness of the structure, as well as requirements with regards to the W/C-value, the minimum cement content, the consistency during installation and the admissible crack width can be found in DB Netz (2003), DIN 1045, Part 2 (2001), DAfStb (2004) and ZTV-ING (2003). The temperature of the green concrete, the development of hydration heat and the after-treatment can influence the occurrence of restraint, and are to be taken into account during the specification of the composition of the concrete.

6.6.2 Composition of the concrete

The following raw materials normally are used for the production of concrete for a single segmental lining:

- cement,
- fly ash,
- superplasticizer,
- aggregates,
- water.

The strength of the concrete is determined by the type of cement and the cement content. The replacement of cement by fly ash reduces the strength and the heat development during hardening. Furthermore, a more dense fabric of the concrete and thus a greater resistance against chemical attack is achieved. The fly ash also improves the workability and reduces the risk of segregation. Moreover, the fly ash normally is cheaper than cement. The avail-
ability of the concrete in the specified composition must be ensured for the complete duration of construction of the segmental lining.

The addition of superplasticizers leads to the desired workability of the concrete during placement.

The aggregates normally consist of sand (grainsize 0/1 mm and 0/2 mm) and gravel (grain size 2/8 mm, 8/16 mm and 16/32 mm). The maximum size of aggregate is limited to 8, 16 or 32 mm, depending on the concentration of reinforcement as well as the spacing between the reinforcement to block outs and built-in units.

For the evaluation of the amount of water to be added, the water content of the aggregates must be taken into account.

The selection of the raw materials should be made under economical considerations. The distances from the particular pre-casting plants to the construction site, and thus the costs for transportation, have to be accounted for.

6.6.3 Production

The production of a lining segment is carried out in the following sequence:

1. Cleaning of the formwork
2. Spraying with form release agent
3. Closing of the sides of the formwork
4. Mounting of the built-in units
5. Installation of the reinforcement
6. Assembly of the dowels and screw cones
7. Closing of the formwork (top cover) and check
8. Placing of concrete and compaction
   - temperature of green concrete: 25 to 35 °C
9. Aftertreatment of surface/smoothing
10. Hardening and aftertreatment
    - 8 to 12 hours within the formwork, storage in a steam vapor curing chamber, temperature 30 to 45 °C
11. Striking and labeling
12. Temporary storage
    - 2 to 3 days storage at room temperature covered with foil
13. Dismantling
14. Shipment
The development of strength is significantly influenced by the heat treatment during concreting and hardening. The green concrete is placed into the formwork with a temperature between 25 and 35 °C. Afterwards the concrete in the formwork is stored in a steam vapor curing chamber, at an air temperature between 30 and 45 °C. The air humidity and the temperatures have to be selected according to the requirements on the hydration and the strength development. After leaving the steam vapor curing chamber, the segments are struck, covered with foil and temporarily stored at room temperature for approx. 2 to 3 days, in order to avoid cracking due to differences in temperature.

The temperature in the concrete should amount to ≤ 60 to 65 °C (DAfStb, 2001), in order to avoid a transfer of ettringite into mono-sulfate and thus, not to obtain an unfavorable effect on the durability, caused by heat treatment (DAfStb, 2003). The temperature difference within the concrete, from the core to the surface, should amount to ≤ 30 to 35 °C, in order to avoid inadmissible cracking.

6.6.4 Quality assurance

The workability and development of strength as well as the temperatures within the concrete, must be verified at test segments. The temperatures should be monitored continuously during hydration of the test segments.

Segments made of reinforced concrete provided for the single segmental lining, are subjected to comparatively high demands with respect to their production tolerances (DB Netz, 2003 and section 2.2.2). The dimensional accuracy of the steel formworks, therefore, should be checked by a precision measurement before production. During production the dimensional accuracy of the formworks should be checked at regular intervals. The dimensions of the segments should be checked on an installed segmental ring. Each segment should be approved before installation (DB Netz, 2003).

The supervision of the concrete quality should be carried out at least according to the corresponding standards for reinforced concrete (DIN 1048, Part 2, 1991).
### 6.6.5 Examples for concrete mix compositions

In Table 6.4 exemplarily 6 mix compositions are listed. Furthermore, the strengths for concrete ages of 12 hours and 14 days are given, which were achieved for these mixtures, by production using heat treatment.

<table>
<thead>
<tr>
<th>cement</th>
<th>CEM I (Portland cement)</th>
<th>52.5 N (strength class)</th>
<th>42.5 R (strength class)</th>
</tr>
</thead>
<tbody>
<tr>
<td>fly ash</td>
<td>[kg/m³] 190 200 210</td>
<td>190 200 210</td>
<td>190 200 210</td>
</tr>
<tr>
<td>water</td>
<td>[kg/m³] 132 135 138</td>
<td>132 135 138</td>
<td>132 135 138</td>
</tr>
<tr>
<td>superplasticizer</td>
<td>[kg/m³] 1.9 2 2.1</td>
<td>1.9 2 2.1</td>
<td>1.9 2 2.1</td>
</tr>
<tr>
<td>aggregates</td>
<td>[kg/m³] 1943 1938 1933</td>
<td>1943 1938 1933</td>
<td>1943 1938 1933</td>
</tr>
<tr>
<td>sand 0/1 mm</td>
<td>[5 %] 97 97 97</td>
<td>97 97 97</td>
<td>97 97 97</td>
</tr>
<tr>
<td>sand 0/2 mm</td>
<td>[25 %] 486 485 483</td>
<td>486 485 483</td>
<td>486 485 483</td>
</tr>
<tr>
<td>gravel 2/8 mm</td>
<td>[18 %] 350 349 348</td>
<td>350 349 348</td>
<td>350 349 348</td>
</tr>
<tr>
<td>gravel 8/16 mm</td>
<td>[24 %] 466 465 464</td>
<td>466 465 464</td>
<td>466 465 464</td>
</tr>
<tr>
<td>gravel 16/32 mm</td>
<td>[28 %] 544 543 541</td>
<td>544 543 541</td>
<td>544 543 541</td>
</tr>
<tr>
<td>density</td>
<td>[kg/m³] 2397 2395 2393</td>
<td>2397 2395 2393</td>
<td>2397 2395 2393</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>temperature of green concrete</th>
<th>approx. 25 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>hardening in a steam vapor curing chamber</td>
<td>8 hours at approx. 30 °C</td>
</tr>
<tr>
<td>$\sigma_c$ (12 hours)</td>
<td>[MPa] 19.4 - 19 - 19 - 15.7 - 16.8 - 15.9 -</td>
</tr>
<tr>
<td></td>
<td>20.1 20.6 21.1 17.2 17.6 18.7</td>
</tr>
<tr>
<td>$\sigma_c$ (14 days)</td>
<td>[MPa] 50.2 - 50.3 - 52.4 - 49.9 - 50.3 - 48.7 -</td>
</tr>
<tr>
<td></td>
<td>51.6 53.7 54.9 51.6 52.3 52.8</td>
</tr>
</tbody>
</table>

Table 6.4: Examples for mix compositions of concrete for the single segmental lining method and compressive strengths achieved by using heat treatment