Recommendations for the

Design, Production and Installation of Segmental Tunnel Linings

Deutscher Ausschuss für unterirdisches Bauen e. V.
German Tunnelling Committee (ITA-AITES)

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These updated recommendations replace the recommendations from 2014.

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Cover picture: Own illustration

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For reasons of readability, the simultaneous use of feminine, masculine or neutral forms of language is dispensed with in the following and the generic masculine is used. All personal terms apply equally to all genders.

Preamble

These updated recommendations build on the 2014 recommendations of the working group "Segmental Tunnel Lining" of the German Tunnelling Committee (DAUB). They provide an overview of the current best practices for the design, production and installation of segmental tunnel linings in mechanised tunnelling. The recommendations outline core design principles essential calculations, and verifications for the lining design and take into account not only the applicable standards but also relevant guidelines and experience from completed projects. Among other things, the design of the linings in fire scenarios and the incorporation of steel fibres are also considered. Beyond the detailing and structural design of the segments, the recommendations offer guidance on segment production in precast facilities, installation of segments in the tunnel, and the detailing of the lining in the area of connecting and transition structures.

1 Preliminary remarks

1.1 Purpose of the recommendations

The recommendations for the design, production and installation of segmental tunnel linings can be applied for linings made of precast reinforced concrete or steel fibre reinforced concrete segments in infrastructure and water tunnels (headrace tunnels for hydroelectric power stations, drinking water tunnels, sewage tunnels). They deal mainly with single-shell segmental linings with high demands on the individual segment and on the overall ring system.

The recommendations can, however, also be applied analogously to segmental lining systems without sealing and to double-shell linings as well as for shaft structures with segmental linings.

The recommendations are intended for standard ring systems. For special cases individual specifications may be required that deviate from the standard cases described herein.

1.2 Reason for revision of the 2014 recommendations

The recommendations for the design, production and installation of segmental tunnel linings were published by the working group "Segmental Tunnel Lining" of the *German Tunnelling Committee* (DAUB) in 2014. They reflected the state of the art at that time for the design of tunnel linings using precast concrete segments and compiled a basic principles for the design of segmental linings with regard to load-bearing capacity, serviceability properties, construction and quality assurance.

In the meantime, technical development in tunnelling has advanced. This applies in particular to the increasingly frequent use of steel fibre reinforced concrete, but also to the further development of standards for the design of concrete structures.

Due to the more frequent use of precast segmental linings in tunnelling, the production of segments has also become increasingly important. Furthermore, the issue of sustainability in tunnel structures will need to be given greater consideration during planning and execution in the future.

For the above reasons, the DAUB decided to update the recommendations and adapt them to the latest state of the art. This applies in particular to the new chapters **ch. 7** (Earthing measures), **ch. 9** (Sustainability) and **ch. 11** (Production) and the revised chapters **ch. 4** (Sealing) and **ch. 5.11** (Steel fibre reinforced concrete segments).

1.3 List of abbreviations

AGBM	Annular gap backfill material
ch.	Chapter: reference to chapters in these recommendations
GTA	General technical approval
PRA	Project-related approval
sect.	Section: reference to sections in other regulations
SFRC	Steel fibre reinforced concrete
SLS	Serviceability limit state
TBM	Tunnel boring machine
ULS	Ultimate limit state

1.4 Definitions and glossary

The following symbols, names and terms are used consistently in these recommendations.

1.4.1 Terminology

Annular gap	Space between the surrounding rock and the outer surface of the segmental lining
Centering aid	Accessory which is inserted into specially made recesses in the ring or longitudinal joints and forces guiding (centering) of the segment elements during installation. In the ring joints (circumferential joints) dowels are used as centering aids, in the longitudinal joints round plastic rods are used
Connections	Accessory that can accommodate pullout and shear forces, enabling temporary or permanent fixed connecting between two segments or rings in both longitudinal and ring joints (e.g. bolts, dowels)
Crosscut	Connecting structure between two tunnel tubes or between a tunnel tube and a shaft
Grouting	Filling of artificially created cavities under pressure with grout; within the scope of this guideline, in particular the filling of the annular gap
Injection	Filling pores, fissures or cavities under pressure with injection material. In contrast to grouting, the pressure is kept constant over a defined period of time
Inner shell	Inner structural lining that fulfils structural and/or functional requirements, it is not used for direct rock support and installed away from the excavation face
Joint	Periphery of the precast segment and contact surface between the segments
Lining	Lining system using a concrete shell. With single-shell linings, the lining shell assumes the supporting function, while with double-shell linings, an outer shell is used for primary support and an inner shell as secondary permanent lining
Logistics	The term “logistics” includes planning, provision and use for tunnel construction purposes. Logistics means the entirety of all transport, storage and handling processes of material, energy and products within and between operating sites, including the transportation of personnel on a tunnel jobsite
Longitudinal joint	Joint running parallel to the tunnel axis between the individual segments of a segment ring
Niche	A lateral widening of the tunnel cross section (without connection to other structures), usually in the wall area; the design of the cross section depends on the function of the niche (e.g. emergency call, fire alarm, rescue and extinguishing equipment niches)
Ovalisation	Deformation of an installed segment ring due to system-inherent tolerances, rock or earth pressure, grout pressure, dead weight of the segment or uplift
Pea gravel (pneumatic stowing)	Gravel with uniform particle size, typically used to fill the annular gap behind the tailskin through openings in the segments
Precast element	Concrete component produced under controlled manufacturing conditions regardless of where it is installed
Ring joint/circumferential joint	Joint running approximately perpendicular to the tunnel axis between two adjacent segment rings
Round grain	Aggregate whose surface is more than 50 % naturally rounded
Segment	Prefabricated element for the lining of tunnels, galleries and shafts made of reinforced or unreinforced concrete, steel fibre reinforced concrete, steel or in former times cast iron

Segmental lining system	Lining system for tunnels, galleries and shafts consisting of individual segments assembled to form segmental lining rings which, in conjunction with the annular gap backfill, support the cavity
Segment seal (gaskets)	Sealing system consisting of sealing strips enclosing the individual lining segments on all sides, which – after compression by joining the segments together – ensures permanent sealing of the tunnel lining against the surrounding (ground) water
Shield tunnelling	Tunnelling method by jacking a shield skin into the rock, using various loosening methods and, if necessary, face-supporting measures
Single jacks	Individually arranged jacks or pairs of jacks used to transfer the jacking forces from the TBM to the segmental lining ring
Stroke	Tunneling section in which the tunneling machine is continuously pushed forward by the extension of the tunneling jacks; usually corresponds to the width of the lining ring in longitudinal direction of the tunnel
Tailskin	Rear part of the tunnel boring machine facing the segmental lining, in which the new segment ring is assembled
Tapering (conicity) segment ring	Difference between the maximum and the minimum width of a tapered lining ring
Test ring	Complete segment ring, usually assembled lying flat, for test or demonstration purposes
Tunnel lining	Mechanical support of the cavity intrados after excavation of a tunnel
Double-shell lining	Tunnel lining consisting of two or more shells (also double-shell construction method) with different structural and constructional requirements (no composite) which are produced in independent operations and with different construction methods (e.g. outer shell shotcrete or segmental lining, inner shell in situ concrete vault)
Void	A surface irregularity resulting from the entrapment of air at the surface of formed concrete ("open pore")

1.4.2 Symbols and designations

Table 1 *Terms, symbols, designations*

Term	Description	Remarks
$\mathbf{A_{c0}}$	Load application area on the contact surface	$= d_0 \cdot l_0$ Designation analogous to EC2.
$\mathbf{A_{c1}}$	Area after load distribution in the segment	$= d_1 \cdot l_1$ Designation analogous to EC2.
$\mathbf{A_{l1}}$	Area of load distribution along the segment	$= t_1 \cdot d_1$ Previously A_{c2}.
$\mathbf{F_{Z1}}$	Splitting tensile force in the segment	At a depth of $0.4 \cdot t_s$ Splitting tensile force due to load distribution, previously F_S.
$\mathbf{F_{Z0}}$	Secondary tensile force on contact surface	At a depth of $0.1 \cdot t_s$ Tensile force directly under load application, previously $F_{S,r}$.
$\mathbf{F_{Z2}}$	Tensile force on contact surface	At a depth of $2/3 \cdot h$ (Secondary) splitting tensile force deep in the segment, previously $F_{S,2}$.

Term	Description	Remarks
F_S	Tensile force in the reinforcing steel	
M	Moment	
N	Normal force	Also when applied to partial area load of the joints (thus replacing "F" in verification for partial area load according to EC2).
b	Radian of the individual segment	Always "b" for radians.
d		Designation of load or design-dependent distances or variables generally with "d".
d_0	Height or width of the rectangular equivalent stress on the contact surface (load application) in radial direction	Replaces "b1" in EC2.
d_1	Height of the rectangular equivalent stress after load distribution in the segment as above	Replaces "b2" in EC2.
e	Eccentricity of force	Always "e" for eccentricities.
e_M	Eccentricity due to calculated bending moment M	= M/N
e_{vs}	Additional eccentricity due to offset	Offset e.g. from installation tolerance.
e_{ov}	Equivalent eccentricity due to ovalisation	Is determined from a (pre-)ovalisation as a substitute value for the verification of the partial surface load or splitting tensile forces on the longitudinal joints.
e_{KF}	Eccentricity of the contact surface relative to the segment axis	
e_{OF}	Distance of the normal force from the (nearest) segment surface	
h	Segment thickness	Is analogous to EC2. Always "h" for geometrically fixed dimensions in radial direction.
h_{KF}	Height of the contact surface (radial)	Geometrically fixed dimension.
h_{Oi}	Distance of contact surface from the inner segment surface	Geometrically fixed dimension.
h_{Oa}	Distance of contact surface from the outer segment surface	Geometrically fixed dimension.
l	Segment length	In the longitudinal direction of the tunnel. Always "l" for dimensions in the longitudinal direction of the tunnel.
l_{KF}	Length of the contact surface	Geometrically fixed dimension.
l_{Ra}	Distance of contact surface from the corners of the longitudinal joint to the ring joint	Geometrically fixed dimension.
l_0	Length of the rectangular equivalent stress (load application)	Replaces "d_1" in EC2.

Term	Description	Remarks
l_1	Length of the rectangular equivalent stress after load distribution in the segment	Replaces „d_2" in EC2.
t	Dimension for depths	Always "t" for dimensions that refer to a depth.
t_1	Depth of load distribution behind the contact surface, in the verification of partial surface pressures according to **ch. 5.7.2**.	$t_1 = d_1 - d_0$ (inclination 2:1) Indicates the distance of the area A_{c1} from the area A_{c0} on the contact surface, previously "h".
t_s	Depth of load distribution behind the contact surface, in the verification of the splitting tensile according to **ch. 5.7.3**.	$t_s = d_1$ Definition according to DAfStB issue 631 corresponding to the progression of splitting tensile stresses.
x	Local and global coordinate direction in the longitudinal direction of the tunnel	Direction along the longitudinal joints.
y	Local coordinate direction in the circumferential direction of the segment	Direction along the ring joints ("curved" direction).
z	Local coordinate direction in the radial direction of the segment	Direction normal to the segment surface.
α	Angle of twist on the longitudinal joints	
α_{ov}	Angle of twist due to ovalisation	
s_{c0}	Rectangular equivalent stress on the contact surface (load application)	
s_{c1}	Rectangular equivalent stress after load distribution	
$s_{c,\,max}$	Maximum stress on the contact surface (load application)	This definition is initially independent of the basis of the determination or basic assumption (linear progression, elastic stress, plastic stress)
σ_{SZ}	Splitting tensile stress	
$\sigma_{SZ,\,max}$	Maximum splitting tensile stress	

2 Overview of typical segmental lining systems

For segmental lining systems a distinction can be made between two principle types: single-shell and double-shell lining. Single-shell lining is the most commonly used, with only a few exceptions (see **ch. 3.2**).

Typically, the classic ring system is employed, where individual segment rings have continuous circumferential ring joints on both sides. This design ensures all segments of a ring are installed without any offset to the neighbouring segments of a ring. In the classic ring system with flat ring joints, either tapered rings, also known as "conical" rings, or parallel rings with the same width all around the circumference are used (**Figure 1**).

However, there are also segmental lining systems in which the individual segments of a ring are arranged offset to each other in the longitudinal direction of the tunnel (hexagonal or honeycomb segment rings) and therefore do not form a continuous ring joint. Ring systems with offset (or staggered) ring joints are not discussed further in these recommendations. They are very rarely used in Europe and have a number of disadvantages. For example, curve drives are difficult to realise with offset ring joints. In addi-

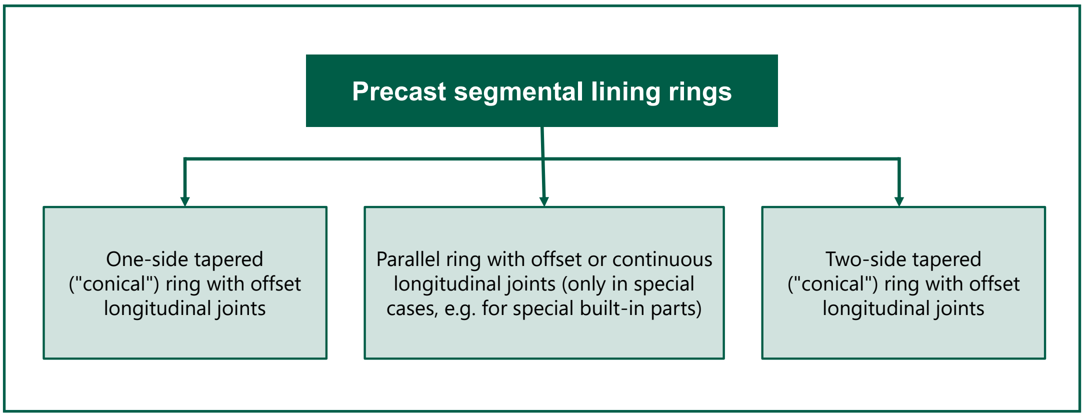

Figure 1 *Classification of concrete lining segment rings considered in this recommendation in ring types*

tion, joint sealing with joint strips is not practical with these systems.

The use of segmental linings with offset ring joints is therefore limited to special applications.

The characteristics of the different ring types are explained below.

Tapered segmental lining rings

To facilitate the installation of the segmental lining without secondary bending even with curve drives, tapered segment rings are used **Figure 2** and **Figure 3**). The rings are twisted around the tunnel's longitudinal axis following a defined ring sequence and can therefore allow a three-dimensional curve in a horizontal and vertical direction. On the outer side of a curve, the ring width (in the longitudinal direction of the tunnel) must be made somewhat greater, on the inside of the curve somewhat smaller.

Tapered rings are built either as universal rings (**Figure 2** and **Figure 3**) or as right- and left-tapered rings (**Figure 4**). Universal rings allow the key seg- ment to be installed in any ring position, accommodating the desired three-dimensional curve. It can therefore also be positioned in the invert. With right- or left-tapered rings, the key segment is always installed above the tunnel axis in the crown area. Depending on whether a left-hand or right-hand curve is to be described, the respective ring types are in- stalled in alternating order one after the other.

Tapered rings can be built as single-tapered rings or as double-tapered rings. Single-tapered rings have one tapered joint, while the other remains perpendicular to the tunnel axis. Double-tapered rings have tapered joints at both ends (see also **Figure 9**).

Parallel rings

Parallel rings , which lack a tapered shape, so-called parallel rings, have a uniform width around their entire circumference and therefore ring joints that are parallel to each other. Consequently, these rings cannot accommodate curves, making it difficult for them to follow the path of the TBM (see **Figure 5**).

This limitation can result in secondary bending and potential dam- age to the ring in the tailskin. Therefore parallel rings should only be used in exceptional cases, e.g. for special rings with inserts that have to sit in a specifically defined position.

Short or fitting rings

In order to simplify the connection of the last segment ring to the concrete target structure at the end of a tunnel drive and to avoid cutting of this ring or in case of specified positions for special rings (e.g. cross-passages), "short or fitting rings" can be used (**Figure 6**). When approaching the target structure, the ring sequence that best achieves the desired end position can be calculated in advance.

The aforementioned ring types are primarily used in tunnels with single-shell linings, where the segments form the final tunnel lining immediately after installation.

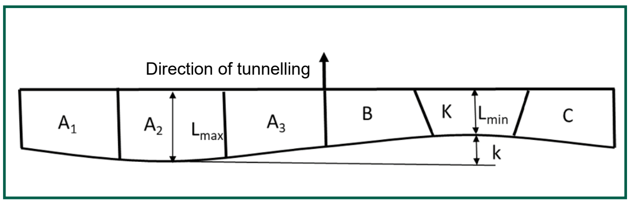

Figure 2 *Universal ring with large key segment*

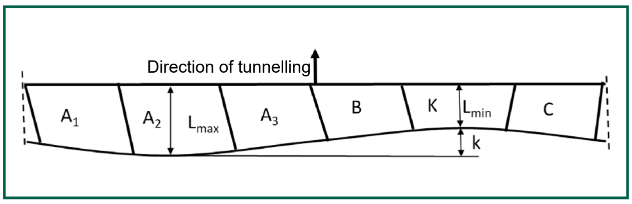

Figure 3 *Universal ring with inclined longitudinal joints between all segments*

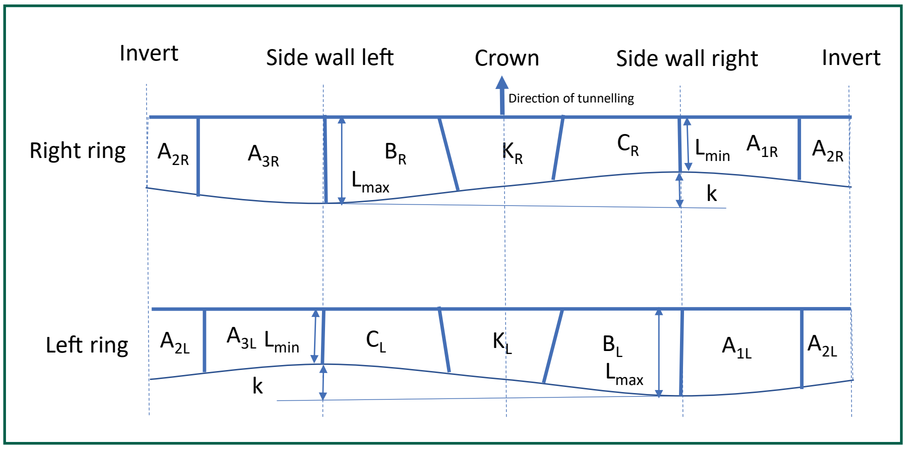

Figure 4 *Schematic diagram of right and left tapered ring*

Some tunnels, particularly ones with large diameters in solid rock, are constructed with a double-shell lining (outer shell: segmental ring; inner shell: reinforced or unreinforced cast in-situ concrete, partly featuring an intermediate flexible liner sealing membrane) (see **ch. 3.2**).

Other ring variants

The following designs are generally used for double-shell linings only.

- **Expanding segmental lining rings**
  Expanding segment rings are primarily used for simple rock stabilisation. The segment ring is pre-assembled in the simplified tailskin (without tailskin sealing) with a laterally tapered key segment shorter in width. Once the TBM has advanced and the ring has exited the tailskin, the key segment is pushed into the ring, expanding it sufficiently to make contact with surrounding rock. Filling the annular gap can thus be dispensed with. Expanding segment rings can only be employed in temporarily stable, non-water-bearing subsoil. In water-bearing rock dewatering is required during construction, since a watertight lining cannot be established with expanding segment rings.
- **Simplified segmental lining rings**
  A simplified segmental lining ring with reduced requirements can be applied in situations where the main loading will occur only after installation of the inner shell, such as during tunnelling with groundwater lowering or in squeezing/swelling rock. It is also applicable in squeezing rock conditions where deformation of the lining can help to reduce the rock pressure. Damage to the segment ring is permissible as long as stability is maintained.
  This type of ring can also be utilized in situations where the segmental ring is removed after serving a temporary function, for example to pass through unexcavated pits for shaft or station constructions.
- **Rings with integrated invert segments**
  In exceptional cases where an integrated invert segment is required, separate tapered rings must be provided for vertical and horizontal curve drives. The rings are typically constructed with continuous longitudinal joints, resulting in specific arrangement of cross joints.

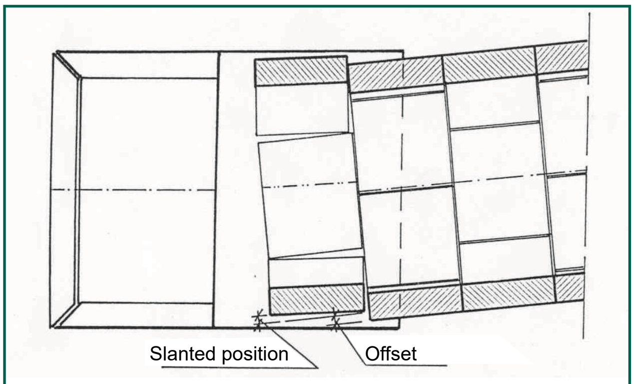

Figure 5 *Possible offsets when installing parallel rings*

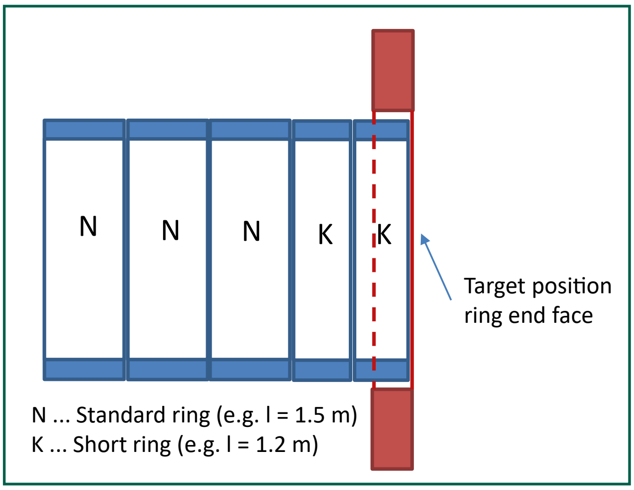

Figure 6 *Ring sequence for short or fitting rings*

3 Segment design

3.1 Description of the single-shell precast segmental ring

3.1.1 General aspects of tunnel lining

The tunnel lining must fulfil several tasks during and after completion of construction:

- Securing the rock to prevent rock falls,
- Resisting the effects of dead weight, loading, rock mass, groundwater and, where applicable, operations,
- Sealing the tunnel against groundwater and mountain water,
- Acting as mounting system for equipment components (for example catenary, lighting, fans),
- Absorbing the tunnelling and steering forces of the tunnel boring machine.

In single-shell reinforced concrete segments, the segmental lining ring must address all of these tasks.

When the segmental ring leaves the TBM, it has already largely reached its final load-bearing capacity and can immediately absorb loads once the annular gap has been filled with grout. The tunnel is also immediately sealed against the groundwater pressure if a sealing gasket system is provided. Because reinforced concrete segments are manufactured in a precast plant with rigorous quality control, the quality is generally consistently high.

The segmental lining ring forms a hinged structure that gains stability mainly through the annular gap backfilling and its bedding towards the surrounding rock. The possibility for coupling in the ring joints and a limited transfer of bending moments in the longitudinal joints also contribute to the ring load-bearing capacity of the segmental lining.

Annular gap backfilling, typically conducted through the tailskin of the TBM, is carried out continuously as the tunnel is driven. Classic annular gap grout (1C-AGBM) or two-component annular gap backfill materials (2C-AGBM), in which the activating component is added directly in the tailskin, are used as annular gap backfill material. The annular gap backfill ensures the immediate bedding of the segmental lining ring to the surrounding subsoil and thus enables that loadings effects on the segment ring are equalised.

Ring geometry

Early coordination of the interface between the segmental lining and the TBM is necessary in the design phase of the segmental lining and the TBM.

When determining the inner diameter of the ring, it must be ensured that there is sufficient tolerance for excavating the tunnel. In Europe a tolerance radius of R ±10 cm for the TBM drive is typically allowed for, i.e. the planned diameter is 20 cm greater than the internal structural boundary requires.

The segment thickness is based on structural and design factors such as sealing details or durability and results between 25 cm and approx. 75 cm, usually between 30 cm and 60 cm, depending on the tunnel diameter and the prevailing rock conditions.

Depending on the outer diameter, the ring width is chosen usually between 0.75 m and 2.25 m.

Common reference values for ring widths are for

- Small diameters (2.5 to 5 m): 75 to 125 cm
- Medium diameters (5 to 8 m): 125 to 175 cm
- Large diameters (over 8 m): 175 to 225 cm

For curve drives, tapered rings (also known colloquially as “conical” rings) must be used. The required tapering (k) can be calculated, dependent on the geometric constraints, using the following formula (see **Figure 7**):

$$k = D_a \cdot l_m / R$$

with

k	tapering (difference of maximum to minimum ring width)
D_a	outer diameter of the segment ring
l_m	mean ring width
R	minimum curve radius

When applying the above formula, it is important to consider that the tapering of a ring cannot be fully utilised with offset longitudinal joints. This is because the ring position with maximum or minimum width can only be arranged with the specified longitudinal joint offset (**Figure 8**).

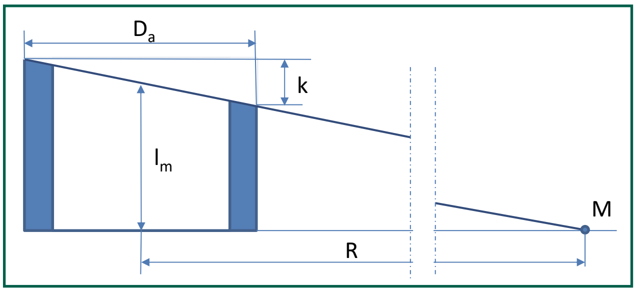

Figure 7 *Geometric relationship between tapering k and curve radius R; M corresponds to the centre of the curve*

To take this effect into account, the above formula is supplemented as follows:

$k' = D_a \cdot l_m / R \cdot 2 / (1 + \cos\alpha)$

where:

- k' usable tapering with offset longitudinal joints
- α mutual minimum angle of twist of two consecutive rings

When designing the segment ring, it is also necessary to consider a correction curve driven by the TBM to compensate for unavoidable driving tolerances. The required correction curve radius should be at least 20 % to 30 % smaller than the smallest radius of the designated tunnel alignment, both horizontally and vertically.

Because the tunnel boring machine can never follow the intended tunnel axis exactly, tapered segment rings should also be planned for straight tunnel drives. In this case, the correction curve radius should be chosen based on the control capabilities of the tunnel boring machine (for example, $R_{Correction}$ = 500 m).

Depending on whether the tapering is provided on one side of a segment joint (single-tapered ring) or on both sides (double-tapered ring), the directional correction varies (see **Figure 9**). While single-tapered rings allow a stronger directional correction, the curve correction with double-tapered rings is smoother. With double-tapered rings it is also essential to ensure that the axis of the TBM is not directed perpendicular to the reflecting surface of the ring installed last, preventing the TBM jacks from pressing onto an inclined surface. With this background, single-tapered rings have proven effective in practice in Germany, although both types are used internationally.

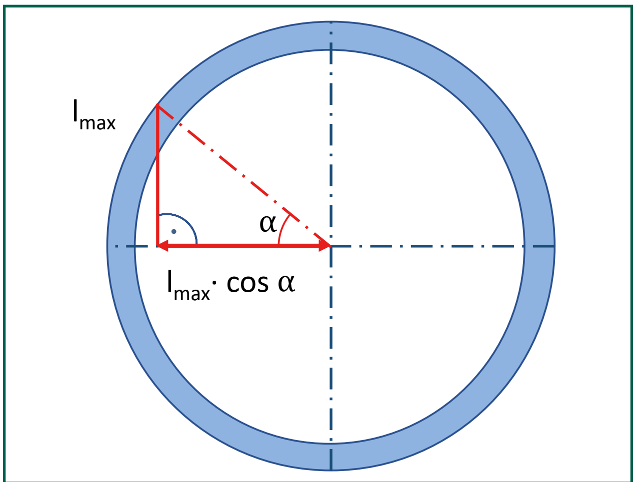

Figure 8 Tapered ring rotated by a maximum angle α from the horizontal axis

When installing a new ring, its position should be chosen to ensure that the front side of the ring is located nearly center in the tailskin and the advance differences of the thrust cylinders are as small as possible respectively reflect the required curve progression.

To avoid installing the key segment below the side wall, left/right rings can be utilised. Here the maximum/minimum ring length is arranged 90° from the key segment axis. Modern ring building erectors and thrust cylinder controls can easily install the key segment in the lower shield position and the logistics are somewhat easier for "universal ring" systems. Notwithstanding, left/right rings are not necessarily a disadvantage: if the supplied ring type does not "fit", it can be used like a universal ring with the key segment being installed in the lower ring position.

3.1.2 Ring division

The ring division (number and opening angle of the individual segments) and the arrangement of the TBM's jacks must be coordinated to each other. With offset segment rings, this means the jacks must be evenly distributed around the circumference of the TBM. The jacks should always press on the individual segment at designated positions. At these positions, tensile

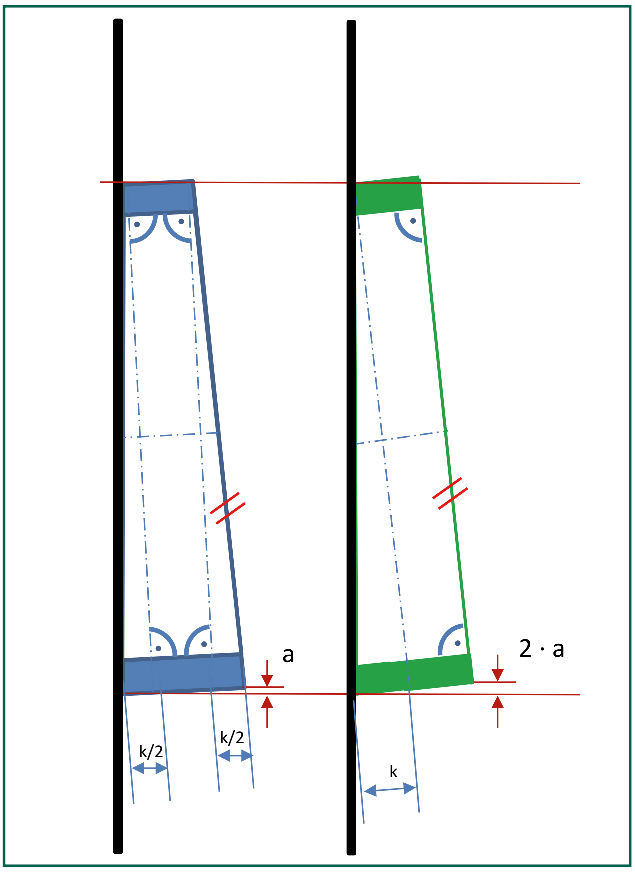

Figure 9 Two-sided (left) and one-sided (right) tapered ring

splitting reinforcement and, if necessary, packers can be provided to prevent damage to the segments.

Cross joints

Cross joints occur when the longitudinal joints of consecutive rings meet in the ring joint or are closer than approx. 10 cm apart. Cross joints have a detrimental effect on the load-bearing and sealing effect of the segmental lining. To mitigate this, each segment ring should be installed offset from the previous one by half or a third of the segment's developed length (longitudinal joint offset). If individual cross joints occur, it must be checked whether these result in a loss of quality (see also **ch. 4**).

3.2 Peculiarities of double-shell lining

With double-shell lining, the precast segmental lining takes on the function of an outer shell. The same recommendations generally apply here as for the single-shell segmental lining ring. As with a single-shell construction, the segmental lining must generally be able to absorb all impacts during construction and – depending on the project requirements – in the final state. Only the verification of water pressure can be omitted if the segments are designed without a sealing system.

Table 2 shows the functions of the segmental lining in double-shell construction depending on the function of the inner shell. Some of the advantages and disadvantages of double-shell construction method are listed below.

Advantages of of double-shell lining

- Possibility of using a sealing layer (flexible liner) between the inner shell and the segmental lining, which allows an almost completely watertight tunnel can be created (even with very high water pressures).
- The segmental lining can be designed as simplified economical ring, often without sealing gasket (for example, no design is required for future excavation and for loads from future constructions).
- The mountings in the tunnel (anchor rails, dowels, steel plates) can be made more cost-effectively since mounting points are in the inner shell.
- Fire protection is ensured by the inner shell. After a fire the inner shell can be replaced.
- The construction of crosscuts (cross passages), ventilation and rescue tunnels or shafts can be simplified.
- The inner shell can be connected directly to the concrete structures of the portals.

Disadvantages of double-shell lining

- The inner shell can usually only be made after the end of tunnelling of both tubes; leading to longer construction time and increased costs.
- If only the segmental lining must absorb the loads from rock and water pressure over a long time, the inner shell only receives the loads from its dead weight and from the equipment. This can result in larger shrinkage cracks due to the generally low normal force in the shell.
- In case of subsequent loads, for example from future excavations or future constructions, the distribution of these loads between the outer and inner shell cannot be clearly defined.
- This complicates the identification of leak locations after installation of the inner shell .
- Proofing that a shear connection exists between the outer and inner lining, or confirming that no such shear connection applies, poses significant challenges. If a shear connection between the inner and

Table 2 *Functions of in-situ concrete inner shell and segmental lining for different double-shell configurations*

Function of the inner shell	Function of the segmental lining ring
Only as fire protection lining	Permanent absorption of all actions and sealing function
With absorption of water pressure and fire loads	Permanent absorption of earth pressure
With absorption of all final actions (earth and water pressure, fire protection) and sealing	"Economical segment" to absorb the effects of rock mass during construction; where applicable, with reduced safety factors and/or sealing function during construction
With absorption of increased loads from earth and water pressure in designated areas (e.g. limited areas with high earth and/or water pressures, crosscut areas, other connecting structures)	Permanent absorption of all actions and sealing function except in zones with additional inner shell

outer lining shell is analytically required, no satisfactory structural solution to create such shear connection is known up to date.

3.3 Joint design

For segmental tunnel linings the joint design is of particular importance, as the joints between the individual segments must transfer high and sometimes very concentrated forces both in the longitudinal direction of the tunnel and in the circumferential direction without causing damage to the concrete structure. In order to meet these requirements, the joints need to be carefully considered and structurally designed.

Recommendations for the design of longitudinal and ring joints are provided below. These apply primarily to single-shell segment systems with gaskets, but can also be used for double-shell systems.

3.3.1 General requirements for joint design

The detailed design of the joints between the individual segments is of crucial importance for load transmission and watertightness. The following design details must be taken into account for the ring and longitudinal joints:

- Load-transfer in joints through centric contact areas with effective reinforcement. Segment areas such as corners and edges that cannot be adequately secured by reinforcement must be relieved from loading by appropriate geometric layouts (recesses, chamfers).
- Avoid notches and notch effects in the load-transmitting areas of joints.
- Arrangement of the compression gasket frames with sufficient edge distance to ensure a safe transfer of the prestress forces from the gasket (see **ch. 4** and **ch. 5**)
- In case of making use of a local increase in concrete compressive strength in accordance with the concept of partially loaded areas, any splitting tensile forces must be verified and covered by reinforcement (see **ch. 5.7** and **ch. 5.11**).

The impact of manufacturing and installation tolerances must be taken into account in the structural verification.

3.3.2 Longitudinal joint

The internal forces must be transmitted between two segments of a ring via the longitudinal joints. These are essentially normal forces and, depending on these, the moments and shear forces that can be transmitted in the joints. In addition, the restoring forces resulting from compression of the gasket frames act in the longitudinal joints.

The longitudinal joint must be designed in such a way that the occasually high normal ring forces can be transmitted without damage, taking into account the simultaneously acting bending moment. To ensure a safe load transfer the contact surface is designed with sufficient edge distance from the top and side surfaces and is slightly raised from the surrounding edges (**Figure 10**). Furthermore, the distance to the sealing groove should be sufficient to avoid stress concentrations in the area of the sealing profile. Generally the contact surface of the longitudinal joints is designed flat, although convex or concave designs can also be considered for special cases.

The contact surface should be arranged approximately centric to the joint axis in order to avoid undesirable bending moments from eccentricity of the normal force. The height of the contact zone has a decisive impact on the load transfer and deformation behaviour of the ring and must therefore be carefully adjusted to the internal forces that are to be transmitted through the joint (see **ch. 5.7**). In particular the edge distance of the contact surface must be selected in such a way that edge spalling of the segment concrete is ruled out. This is particularly important for high normal ring forces.

All edges of the longitudinal joint should generally be chamfered to prevent spalling. A dovetailed groove can also be arranged on the inside of the longitudinal joints to facilitate the subsequent insertion of seals (**Figure 10**). Plastic guiding rods are often employed to minimise offsets in the longitudinal joint. The main function of these centring aids is to facilitate the installation of the segments, minimise offsets in the longitudinal joints during ring building and prevent the key segment from sliding inwards.

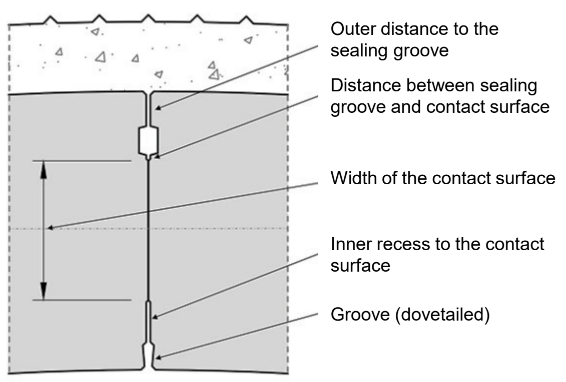

***Figure 10** Typical configuration of the longitudinal joint*

A tongue-and-groove design of the longitudinal joints can quickly lead to damage to the joint's face if installation is inaccurate and should therefore not be used.

To facilitate the installation of the segments, the longitudinal joints are inclined by an angle ψ to the longitudinal axis of the tunnel (longitudinal joint conicity, **Figure 11**). The angle of the longitudinal joint conicity has a decisive influence on the required installation gap length for installing the key segment and thus the required stroke of the thrust cylinders.

In the standard system, only the longitudinal joints of the key segment and the two adjacent counter segments are designed with an inclination to the longitudinal axis, while the longitudinal joints of the other segments (standard segments) are arranged perpendicular to the ring axis. In the so-called trapezoidal system, however, all the segments in a ring have a longitudinal joint conicity.

In addition to the angle ψ, the longitudinal joints of the key segment and the two counter segments may be inclined by an angle φ to the radial axis in order to facilitate the insertion of the key segment from below and further reduce the installation gap length (longitudinal joint angle, see **Figure 11**).

Both angles ψ and φ not only have a considerable influence on the ring errection, but also on the structural system and the load-bearing behaviour of the ring system. They must therefore be carefully tailored to the selected segment system and, if necessary, considered in the structural analysis.

For small key segments, it is recommended that the longitudinal joints are designed radially to prevent the key segment from sliding inward.

3.3.3 Ring joint

The jacking forces of the TBM and the coupling forces between the rings are transmitted in the ring joints. The non uniform loading caused by steering forces must be taken into account in the design. The following joint shapes are commonly used for the ring joints:

- Flat joint design
- Cam and pocket configuration
- Tongue and groove joint design

For single-shell lining, the flat joint configuration is preferred for both the ring joints and the longitudinal joints. Additional centring and coupling devices may be provided for. This can be dowels, centring cones or reinforced or unreinforced cam and pocket toothings, whereby dowels or centring cones are meanwhile the rule. Based on past experience with spalling and chipping, cam and pocket toothing should only be used if a sufficiently large construction tolerance (usually > 15 mm) is provided for (see also **ch. 5.7**).

A tongue and groove layout for the ring joint increases the demands on ring building because even minor ring offsets can result in significant spalling during the TBM advance. For this reason, a tongue and groove design is not recommended for a single-shell segmental lining.

The flatness of the entire ring joint, referred to as ring face, is significantly influenced by the ring construction. Packers of constant thickness made of hard fibre material or plywood are often provided in the ring joints to allow loads to pass through at defined locations and to compensate for unevenness. Ductile packers (for example those made of bituminous material) should not be employed for this purpose, as they deform plastically under load and thus loose their function. In terms of dimensional accuracy, hard plastic sheets are advantageous.

The packers and their dimensional tolerances must always be taken into account for the design and verifying of the sealing profiles and connections devices (e.g. plug-in dowels).

It is also possible to design the lining without packers in the ring joints; however in this case, compensating for ring-building tolerances and constraints

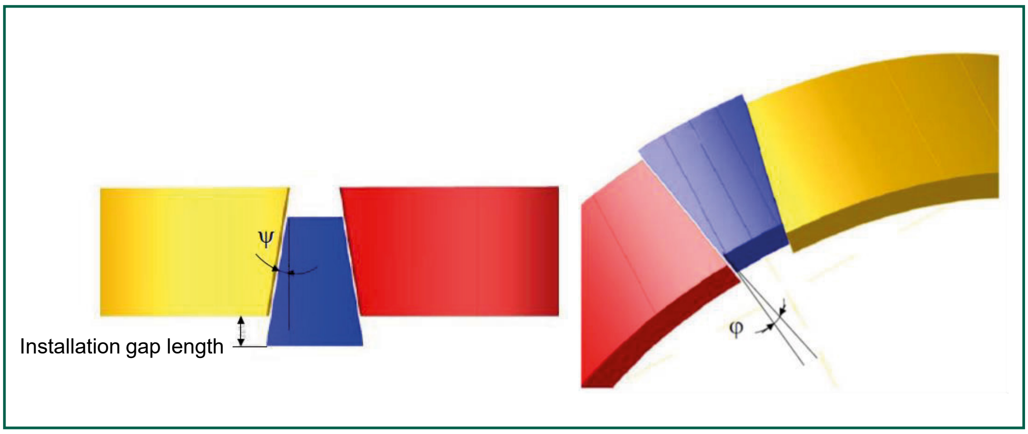

Figure 11
Longitudinal joint angle ψ and angle of longitudinal joint conicity φ

becomes more challenging. When transmitting the longitudinal forces without intermediate layers, ring joint flatness requirements are higher. In this scenario, load transfer surfaces should be confined to the jack shoe areas , with consideration of rolling effects if necessary.

For the design of the ring joints, special attention should be given to the following:

- Consideration of packers and their compressibility for the ring geometry in the joint facing away from the TBM.
- Interface with TBM: number, position and dimensions of the jacks or jack shoes.
- To avoid bending moments in the jacks and eccentric force on the segment (which can create "carding moments" in the segment ring), the radius of the jacks should be aligned with the the axis radius of the ring joint.
- In case that thrust cylinders are located close to the longitudinal joints it must be ensured that the forces are introduced with sufficient distance to the joints (e.g. by provision of chamfered load transmission zones).

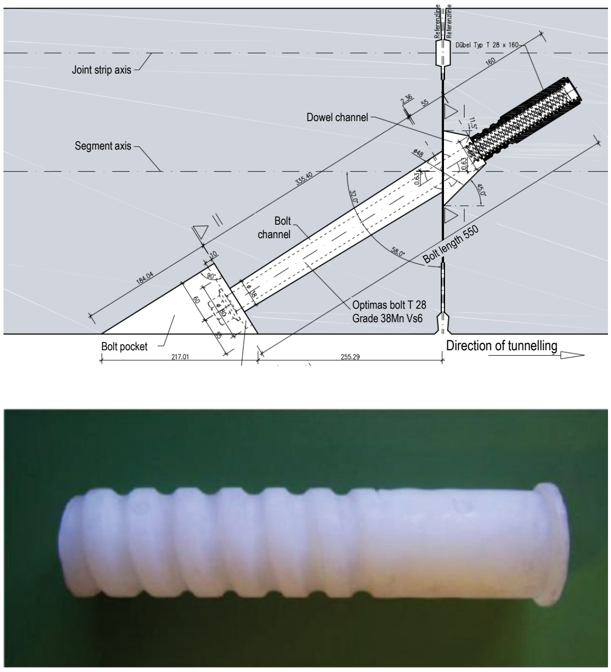

Figure 12 *Bolted connections with dowels*

3.3.4 Connections

Angled bolts with plastic dowels (**Figure 12**) have proven effective as temporary connections in segmental lining joints. They provide a force-fit connection, are easy to install and can be easily removed.

In ring joints plug-in dowels are increasingly replacing these bolted connections (**Figure 13**).

Such plug-in dowel connections can significantly reduce the offsets between the rings, shorten installation times with precise ring building, and simplify reinforcement guidance inside the segment by eliminating the bolt-pockets.

When plug-in dowels are used, they must be designed to be tension-resistant to absorb the restoring force of the compression joint seals. Note that pre-tensioning of this connection is not possible, which may result in slight opening of the joint ("slippage"). This slippage in the millimeter-range must be considered for verification.

To reduce the slippage of plug-in dowel connections, special dowel types with a one-sided click connection and a screw attachment on the other side can be used.

Joint connections are typically needed only during ring building, particularly for pre-tensioning of the compression gaskets, and for advancing the tunnel boring machine. Once the annular gap backfill has set, generally after approximately 10 to 15 rings, this function is generally no longer needed, allowing for

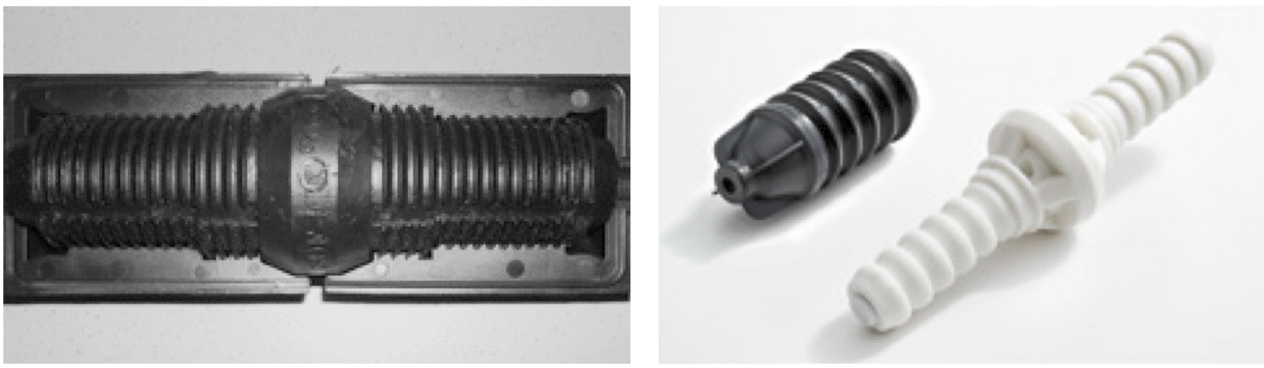

Figure 13 *Plug-in dowel as an alternative for bolting the ring joint*

removal of the bolts. In traffic tunnels, temporary bolts above the track or roadway must be removed mandatory to prevent them from falling down during later operation. Permanent bolts, such as bolts at transitions to in situ concrete and at cross passages, must be secured against loosening and protected from corrosion.

In certain cases, dowels for shear force transfer are installed in the ring joints (**Figure 14**), to improve the ring stability especially during ring erection for rings with uncured annular gap material.

Plastic inserts generally do not impact the durability of the segment ring negatively. However, they must be suitable for use under load in their specific application.

3.4 Mountings in the segmental ring

With double-shell tunnel lining, any mountings that may be required (e.g. for the catenary or signals) are installed in the in situ concrete inner shell. Generally, no special inserts are required for the segment ring in this setup.

For single-shell lining, future mounting options should be considered during the planning phase already. The main mounting options include:

- Installation of anchor rails (embedded in the segment mould),
- Installing steel mounting plates (embedded in the segment mould),
- Post-installation of the mountings by doweling.

When installing anchor rails or mounting plates it is essential to make sure that their intended position in the tunnel can be safely achieved. For tapered, rotatable rings, the anchoring must either be installed all around the entire ring or positioned for a pre-determined location during ring building. The required edge distances from the ring and longitudinal joints must be observed.

During planning, the spacing of the mountings must align with the ring width. If mounted components change after segmental lining installation, the existing components must be taken into account. The position of the anchor rails or mounting plates generally influences the design of the erector's suction plate.

Figure 14 Examples of dowels for shear force transmission in the ring joints

Post-installation dowelling of mountings allows for greater flexibly in fitters' requirements for fixing components. To avoid damaging segment reinforcement during dowel drilling, it is advisable to specify dowel lanes and coordinate them with reinforcement. Dowel lanes should be marked on the inner segment surface, or dowels should be drilled using jigs.

Auditability of all mountings must be considered for future site inspections. For road tunnels, chloride contamination should be considered in terms of durability and corresponding material specifications must be followed. For heavy loads, for example fans, direct mounting onto the rock can also be considered. However, since this design requires perforating the segmental lining ("buttonhole"), it is only feasible under compatible hydrogeological conditions.

3.5 Design tolerances

3.5.1 Fundamental considerations

Design tolerances mean tolerances from segment production and from erection of the segments in the tunnel. Driving tolerances of the tunnelling machine, installation tolerances of the segmental lining ring in the machine, and segment ring deformations during and after tunnel excavation should be considered separately (cf. **ch. 5.8**).

Tolerance specifications are intended to control geometrical deviations from the planned segment geometry. Deviations are only permissible to the extent that resulting damage from stress peaks in the segments and leaks due to insufficient compression of sealing profiles remain unlikely. In practice, the permissible degree of deviations depends on several different factors:

- Stress level for the planned geometry,
- Utilisation and deformation characteristics of the sealing profile,
- Primary segment dimensions (length, width, thickness, radius).

Tolerance defaults must be defined in the project specifications to specify the acceptable deviations of the nominal geometry. The reasons for strict tolerance requirements are:

- Sensitivity of the segmental lining to geometric inaccuracies and deformation of individual segments,
- High internal forces in the segment ring from earth, water and grouting pressure,

- High jacking forces during TBM advance,
- Load transfer occurs only in limited areas (partial loaded area),
- Damage is often only detected after water ingress (e.g. damage to the ring exterior),
- Repairing damaged segments is costly and time-consuming and may not be technically or operationally feasible.

3.5.2 Tolerance requirements

Deviations form the nominal position typically occur when segments are installed in the tunnelling machine. In addition to avoidable installation errors, ring ovalisation can also result from load actions, floating of the ring in the annular gap backfill material and the formation of offsets between the segments due to restraint forces in the sealing of the tailskin.

These installation tolerances must be considered and depend on the tunnel's intended use (required clearance profile) and the segment design. They must be specified individually for each project by the designer in consultation with the client.

For permissible joint offsets (load eccentricities), refer to **ch. 5.7**. The allowable ovalisation of the ring largely depends on its diameter and the number of individual segments per ring, but should generally be less than 0.5 % of the internal diameter.

All tolerance dimensions, including access tolerance, must be restricted in such a way that the required clearance gauge is always maintained.

The permissible manufacturing tolerances provided in **Table 3** (**Figure 15**) apply to reference conditions described in **ch. 11**. They are specified for inner ring diameters ≤ 8.0 m and ≥ 11.0 m. For diameters between 8.0 and 11.0 m, the values can be interpolated linearly and rounded up to full 1/10 mm. ZTV-ING **[2]** specifies slightly different tolerance values.

An alternative approach for determining the permissible tolerance values is offered by the fact sheet "Segment Tolerances – Derivation and Application" developed by the Austrian Society for Construction Technology (ÖBV) **[29]**. This leaflet determines tolerance classes through a mathematical risk analysis. The approach considers the likelihood of occurrence and the impact of damage or risk-related scenarios. The objective of this approach is to allow smaller geometrical deviations in cases of higher damage risk and to accept larger values when the risk of damage is lower.

For determining the permissible tolerance values the geometric dimensions of the segments and the material properties are considered. A mathematical formula was defined for each permissible deviation, enabling the calculation of the respective tolerance value. A corresponding calculation tool was developed to facilitate tolerance determination. This tool can calculate tolerances for any segmental lining system based on the main segment dimensions and the concrete strength class.

The client must specify permissible dimensional deviations on a project-specific basis.

3.5.3 Exceeding the manufacturing tolerances

If segments have been produced outside the tolerance ranges, they can only be used if compatibility can be demonstrated. In this regard, manufacturing tolerances can be better accommodated and verified retrospectively if the planned degree of utilisation of a segment and the gasket is lower. Possible measures when permissible tolerances have been exceeded are described below (see also **ch. 11** or also in **[29]**). Coordination with the project owner about this is strongly recommended.

- **Exceeding the segment width tolerance**
  If the ring reflecting surface does not result in a correct plane because the segment width tolerance has been exceeded, the segments can be used for example in areas where the projected jacking forces are low. A structural analysis must be carried out for this.
- **Exceeding the angle of longitudinal joint conicity/ longitudinal joint rectangularity deviation**
  If the load transfer surfaces in the longitudinal joints open up as a result of exceeding the tolerance value for the longitudinal joint rectangularity, the load transfer surface will be compressed by the ring nor-

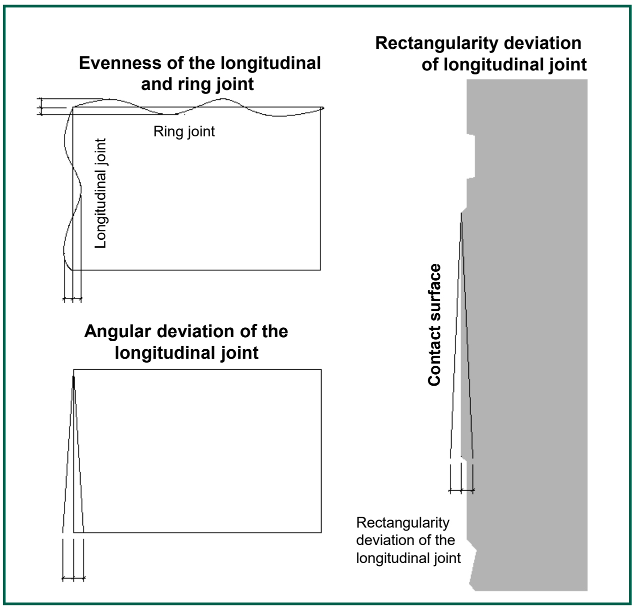

Figure 15** Manufacturing tolerances on manufactured segment of single-shell structures **[2]

Table 3 *Manufacturing tolerances for the individual segment and the entire lining ring (single-shell lining ring with packers in the ring joints)*

Manufacturing tolerance		Ring size (internal diameter)	
		≤ 8.0 m	≥ 11.0 m
1	**Longitudinal joints tolerance based on the load transfer surface**		
1.1	Longitudinal joint rectangularity deviation	± 0.5 mm	± 0.7 mm
1.2	Angular deviation of the longitudinal joint	± 0.5 mm	± 0.7 mm
1.3	Sum for 1.1 and 1.2	± 0.6 mm	± 0.9 mm
2	**Overall segment deviations (related to the median plane)**		
2.1	Segment width	± 0.5 mm	± 0.7 mm
2.2	Segment thickness	± 3.0 mm	± 4.0 mm
2.3	Segment arch length	± 0.6 mm	± 0.7 mm
2.4	Inner radius of each segment	± 1.5 mm	± 2.5 mm
2.5	Diagonal length deviation of a segment	± 1.0 mm	± 2.0 mm
2.6	Vertical out-of-plane distance of fourth segment edge from the plane spanned by the other three edges	± 5 mm	± 8 mm
3	**Gasket groove**		
3.1	Gasket groove width	± 0.2 mm	± 0.2 mm
3.2	Gasket groove depth	± 0.2 mm	± 0.2 mm
3.3	Position of gasket groove axis	± 1.0 mm	± 1.0 mm
4	**Evenness of the contact zones**		
4.1	Longitudinal and ring joint	± 0.3 mm	± 0.5 mm
5	**Tolerances on the entire segment ring**		
5.1	Outer diameter	± 10 mm	± 15 mm
5.2	Inner diameter	± 10 mm	± 15 mm
5.3	Outer circumference (measured at three levels)	± 30 mm	± 45 mm
6	**Position of the fixing components**		
6.1	Erector cones	± 2 mm	± 2 mm
6.2	Bolt pockets and channels	± 1 mm	± 1 mm

mal force until an equilibrium is reached. The ring normal force must then be transferred over a reduced area, which is therefore higher loaded. Again, it is advisable to install the defective segments in areas of lower loading. The splitting tensile reinforcement must then be verified for the stress at this location.

- **Proximity relationships**
  One way to compensate exceeded tolerances is to sort the segments in such a way that exceeded tolerances of adjacent segments balance each other. This requires considerable effort to sort the segments in the segment storing area and for transport into the TBM. This option greatly restricts the ability to combine different segments and should therefore be approved by the client.
  However, if the contractor can demonstrate an adequate supply chain, this combination option is possible.
  A data recording system for the stored segments is absolutely necessary for this.

4 Sealing of the segment joints

4.1 General

The following statements apply to single-shell linings, in which the sealing function is performed solely by the segmental lining.

The water tightness of tunnel linings made of precast reinforced concrete elements is determined almost exclusively by the water tightness of the segment joints. The standard solution for sealing the joints are sealing frames made of sealing gaskets that are arranged around the segments outer edges in their ring and longitudinal joints. During ring building, the sealing frames of adjacent segments are compressed towards each other and towards the concrete. Since the sealing capacity is based solely on the compression of the sealing gaskets, sufficient compression must be provided along the entire length of the profiles to resist the expected water pressure. This applies to both, the surfaces of the sealing gaskets that are in contact with the concrete and to the joint between the sealing gaskets of adjoining segments, taking possible construction tolerances into account.

More detailed information on the application of gasket frames can be found in the STUVA recommendations **[56]**. In Germany, the Technical Guidelines for Delivery and Technical Test Specifications of sealing profiles (TL/TPDP) provide further guidance in this regard **[28]**.

4.2 Requirements for watertightness, sealing gaskets and dewatering

The water pressure that is to be considered in the design must be specified by the client, taking the project-specific water levels into account.

The demands for water tightness of the tunnel lining must be specified in the contract. In addition, the contract must also include post-sealing measures, permissible residual leakage values and their possible discharge. With regard to the residual leakages a distinction should be made between individual leakages and the total leakage that is accumulated over longer tunnel sections. Where applicable the location of the water ingress should also be considered in the specifications.

Running or dripping leakages are generally not permitted in traffic tunnels. In railway tunnels, dripping on conducting installations including the rails must be prevented. In case of supply tunnels or headrace tunnels, lower watertightness demands may be permissible depending on the specific boundary conditions.

The general impermeability requirements for rail-tunnels can be found in RiL 853. 4101 **[1]** and for traffic tunnels in ZTV-ING, Part 7, section 3, chapter 8 **[2]**. However, both guidelines are not directly applicable for segmental lining shells, as their specifications relate to the concrete surface and not to the joints.

In accordance with TL/TP DP **[28]**, gasket frames must at least meet the following requirements:

- Functionality,
- Ageing resistance,
- Long-term impermeability,
- Resistance to environmental influences, including those from operation of the tunnel,
- Material compatibility with the surrounding media.

One measure to control dripping water ingress, is the post-installation of additional sealing profiles that are inserted in the segment joints to drain the water away from the tunnel interior. To ensure that these sealing profiles can be properly fixed and stay in place, the segment joints must be geometrically designed accordingly. Usually additional dovetailed grooves are provided for this purpose.

Such post-sealing of the joints is generally considered as a temporary measure before final sealing measures are carried out (see **ch. 11.3**)

In addition to the residual water from leakages, water in traffic tunnels is carried in from vehicles and firefighting water has to be removed in the event of fire. Further, significant condensation water can occur in longer tunnels during summer. Therefore, a backflow-free drainage of the accumulated water should be foreseen, at least in traffic tunnels. A continuous drainage pipe with inspection shafts and, if necessary, pumping systems is recommended for this purpose..

4.3 Sealing frames

Sealing frames are usually made of EPDM synthetic rubber. The sealing profiles are extruded and then vulcanised. They are assembled to sealing frames by cutting the straight sealing profiles to length and vulcanising the corners **[56]**.

Both glued-in and anchored sealing gaskets are used.

Glued-in gaskets are attached to the precast segments by gluing them into a groove that is cast into the surrounding edges. This is to ensure that the sealing frame remains in the intended position during ring erection and afterwards to establish the required sealing level.

Anchored sealing gaskets provide anchoring devices such as feet or ribs on their rear side. Occasionally fibres are subsequently glued to the profile as further technical innovation. Anchored profiles are installed into the moulds before concreting a segment and cast-in directly. To accommodate the sealing frame with anchoring, the moulds must be equipped with corresponding retaining grooves. With regard to the reinforcement cages, it is important to ensure that they can be lifted into the mould without interfering with the pre-installed sealing frames.

One advantage of anchored sealing gaskets is that the gluing-works can be omitted. Loosening of the gaskets during transportation and storage can also be largely ruled out. In addition, anchored profiles reduce the risk of a reduced sealing capacity due to water creep. This is offset by the disadvantage that any repair works, should they be required, are more complex for anchored profiles than for glued-in sealing frames. Accordingly, anchored gaskets must be protected against damage at all times from production until erection of the segment in the tunnel.

Early coordination between the designer, mould manufacturer and sealing gasket supplier is always recommended in order to synchronise the geometries of the retaining grooves, the contact surface and the edge distance of the gaskets.

During ring erection, the compression of the adjacent sealing frames generates a restoring force in the sealing profiles. The resulting stresses in the sealing gasket act on the groove bottom and the groove flanks and thus ensure the watertightness of the joints. The restoring force of the sealing gaskets decreases over time; initially quickly, later very slowly. This process, known as stress relaxation, reduces the sealing capacity. To ensure that watertightness is maintained over time, two conditions must be met:

- Material persistence of the sealing gasket,
- Maintaining the minimum required compression for watertightness over the entire service life time.

For the design of the sealing gasket, the groove bottom distance, the groove depth and the offset after installation of the segments are the relevant variables (see **Figure 16**).

Designs with a double sealing system provide two sealing barriers, one on the inside and one on the outside of the precast lining. However, a lasting effect of this system associated with a higher watertightness level could not be confirmed.

4.4 Corners of the sealing frame

The corners of the segments are usually the critical point in a precast segmental lining. To avoid concrete spalling, the sealing gasket supplier must design the shape of the sealing frame corners in such a way that the volume concentration of elastomer in the corners is minimized.

To avoid discontinuities in the sealing profile, it must be ensured that the gasket frames are installed evenly and in the correct order when gluing them into the grooves or clipping anchored gasket frames into the formwork. According to the STUVA recommendations **[56]**, any indentation or elevation of the corner tip relative to the profiles' contact surface, observed on the installed sealing frame, should not exceed 5 mm. Further requirements can be found in the STUVA recommendation **[56]**.

During ring erection and installation of the segments forces from friction between adjacent gaskets arise in the longitudinal direction of the frames. Such

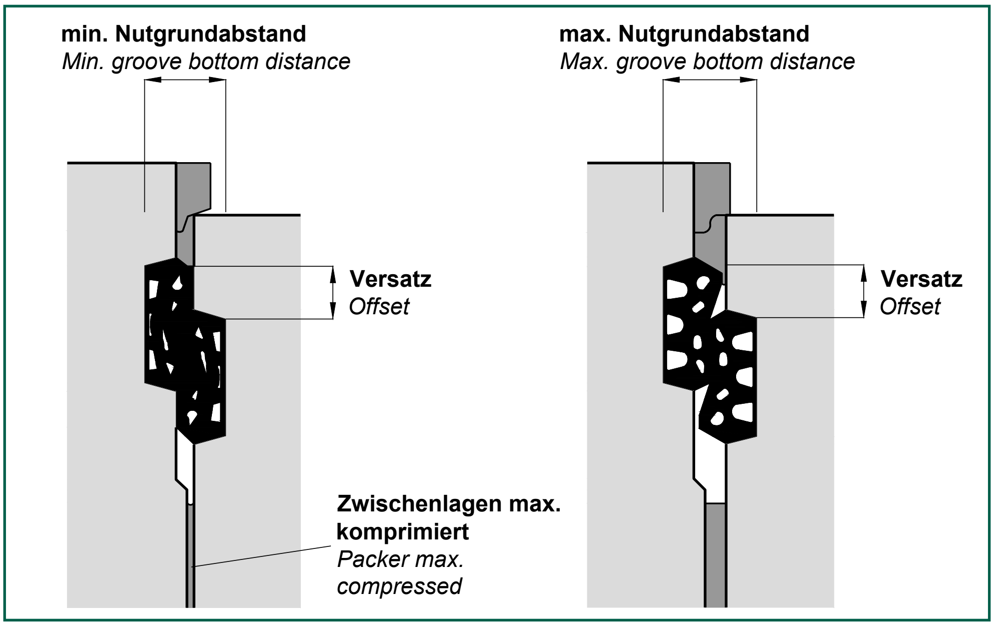

Figure 16
Groove bottom distance and offset according to ***[56]***

forces may lead to significant stresses in the sealing frames and to related longitudinal displacements. The risk associated with this effect is particularly high when pushing in the key segment. For this reason, care should be taken to ensure that the sealing profile is securely fixed in the frame corners to prevent longitudinal displacement.

4.5 Effects to the sealing capacity

The sealing capacity of a gasket can be significantly influenced by the following aspects that are related to the production and installation process of the segments:

1) Dimensional tolerances of the segments,
2) Rock pockets and air bubbles or voids at the concrete surface of the groove,
3) Ring building tolerances and ring deformations,
4) Dimensional tolerances in the sealing frames including mass or rigidity tolerances of the sealing profile
5) Tolerances of the geometry and mechanical behaviour in the corners of the sealing frame,
6) Discontinuity from a level offset resulting from longitudinal offset of adjacent corners due to frame assembly or ring erection,
7) Unplanned expansions and compressions as well as loosening of the gasket from the groove during ring erection,
8) Offset of the sealing frame,
9) Insufficient compression of the sealing gaskets, i.e. higher groove bottom distance than designed for,
10) for glued-in sealing frames: dimensional tolerances of the preformed sealing groove.

In the watertightness tests the offset and the groove bottom distance are varied in accordance with the STUVA recommendations **[56]** or TL/TP DP **[28]**. Accordingly, for a service life time of 100 years, the influence of the relaxation of the sealing gaskets and other deviations are covered by increasing the test pressure by a factor of 2 compared to the design water pressure .

4.6 Concrete spalling of the groove

Excessively high restoring forces in the sealing frames can cause concrete spalling, particularly in the corner area of the segments. Spalling or cracking in the vicinity of the groove leads to a reduction or even loss of watertightness and must therefore be avoided at all costs.

The edge distance, i.e. the distance from the outer edge of the groove to the outer surface of the segment, has a decisive influence on the spalling be haviour in the vicinity of the groove. This distance should be sufficient large to prevent the concrete from spalling in the corners. Due to multiple factors of influence, however, it is difficult to give a simple value as recommendation for the edge distance. As an initial guiding value, an edge distance greater than or equal to the groove bottom width, but at least 30 mm, is recommended for glued-in sealing gaskets. Greater edge distances of 40 mm to 50 mm are preferable if possible.

Compared to glued-in gaskets, seals with anchoring devices lead to a deeper transmission of the restoring loads into the concrete and thus to a increased notching effect. Sealing gaskets with anchoring devices can therefore have a higher risk of spalling. New developments with rounded frame corners or fibre anchors attempt to address this.

Adjacent segments should have a gap of at least 3 mm, preferably 5 mm, measured at the segments edge and with maximum compression of the sealing gasket, in order to prevent contact between the concrete surfaces even in the event of undesigned distorsion. Larger gap values, provided for structural reasons, are also possible.

The groove filling ratio is the proportion of solid sealing material in the total groove cross-section area. The air-filled chambers make up the remainder compared to 100 %. The groove filling ratio increases when the sealing profile is compressed, whereby the stiffness increases steadily. When 100 % filling is reached, a sudden increase in stiffness appears, because from this point only the elasticity of the solid material of the gasket is decisive for the stiffness.

To avoid high restoring forces and resulting concrete spalling, the groove filling ratio of the sealing profile should be less than 90 % for the design compression level. In any case, even with an unfavorable combination of various tolerances, the filling ratio should be less than 100 %, i.e. the groove is not completely filled.

In addition, exposure of the sealing gaskets to low temperatures during installation should be avoided, since this causes the material to stiffen and the restoring forces to increase accordingly. If necessary, appropriate countermeasures should be taken.

4.7 Verification and testing

4.7.1 Basic and suitability tests

Regarding the requirements for the sealing frame and the sealing gasket the ZTV-ING, Part 7 **[2]**, applicable to road tunnels, refers in section 3, clause 8.2.2, to the "Technical Guidelines for Delivery and Technical Test

Specifications of sealing profiles" (TL/TP DP) **[28]**. The directive RiL 853 **[1]** governing railway tunnels adapts this provision in module 4005, clause (14). Extensive information on tests and their implementation can be found in the STUVA recommendations **[56]**, which is also referred to in the TL/TP DP.

The tests are divided into basic and suitability tests. The basic tests examine the general properties of a sealing profile, primarily the material properties and general properties, which are explained in more detail in **[1]** and **[28]**.

The suitability tests are supplementary project-specific tests that must be carried out with consideration of the project-specific boundary conditions such as groove geometry, water pressure and concrete of the segments.

Where applicable, these tests can be omitted if sufficient comparable experience is available from previous projects.

The following minimum numbers of project-related tests should be carried out:

- Restoring force of the sealing gasket and the corners of the sealing frame, at least 3 individual tests each,
- Watertightness, at least 2 individual tests,
- Spalling behaviour of the concrete, at least 2 individual tests.

The suitability tests are usually carried out in a suitable research centre.

Since sealing gaskets have a pronounced temperature-dependent behaviour, project-specific considerations may be required in the event of unusual temperature conditions.

4.7.2 Tests to determine the restoring forces

The restoring force of a sealing profile is required in particular for the design of the connecting devices in the ring joints and the longitudinal joints. In addition, the restoring force is used to analyse the concrete stresses in the vicinity of the groove to evaluate the risk of spalling in these zones. The test setup and procedure as described in the STUVA recommendations **[56]** has proven adequate to determine the restoring forces. A distinction must be made between tests for straight profile sections and for the corners of a sealing frame. Due to the stiffer behaviour of the corners, the restoring force is locally increased at these locations compared to the straight profile section. It must therefore be determined separately and considered in the verification. The restoring forces received from the tests are generally shown in a force-displacement and a force-time diagram.

4.7.3 Tests for verification of watertightness

The tests to verify the watertightness are carried out for a T-joint, a cross joint or for both types, depending on the project requirements. In these tests, the material-dependent stress relaxation of the sealing gasket is, depending on the intended service life time, reflected by applying a test water pressure that is higher than the design water pressure.

The test setup and test procedure for verifying watertightness are described in both, the STUVA recommendations and in the TL/TP DP, whereas the test setup is the same in both guidelines. However, differences occur for the test procedure, which is why the applicable guidelines or test instructions must be specified individually for each project

4.7.4 Tests on spalling behaviour

Depending on the applicable guidelines, spalling tests must be carried out on concrete specimen to evaluate whether the restoring forces of the sealing frames can be absorbed by the segments. Related requirements must be provided in the project specifications.

According to TL/TP DP **[28]**, spalling tests must be carried out, whereas, RiL 853, on the other hand, does not provide for this. The STUVA recommendations **[56]** allow spalling tests to be omitted if sufficient practical experience is available to evaluate the spalling behaviour of the selected sealing frame, whereas, such experience and additional analytical investigations or test results must be obtained under comparable project boundary conditions. Spalling tests should be carried out before the series production of the segments starts.

The test procedure is described in both the TL/TP DP and the STUVA recommendations, although the specifications for the test setup differ with regard to the dimensions of the sealing frames and the level of joint compression.

4.8 Selection of the sealing gasket

When selecting the sealing gasket, the required sealing capacity must be achieved, taking into account the offset and the expected joint opening in the longitudinal joints that result from the deformation of the segmental lining ring. A higher sealing capacity of the gasket generally results in a higher restoring force, which consequently requires a higher edge distances to prevent concrete spalling.

If possible, proven and sufficiently tested sealing gaskets should be used, as they provide for a higher planning reliability. The development of new gasket types is the duty of the gasket supplier or manufacturer respectively. This usually involves an iteration process, which can on a project-specific basis be realized with involvement of the relevant parties, such as the contractor, the designer, the client and its consultants, the independent engineer, the test institutes and the material test institutes. The approval of a sealing gasket is based on the successful verification of the gasket by testing. The tests are to be specified on a project-specific basis.

Figure 17 can be used as an initial tool to check whether a sealing gasket can fundamentally meet the requirements. It shows the relation between the restoring force F, the test water pressure p and the gasket crosssection A for the following boundary conditions.

- Restoring force F [kN/m] is measured after 5 minutes of relaxation.
- The watertightness is determined at a test water pressure p [bar] over a test period of 20 to 24 hours, an offset of 10 mm and a groove bottom distance that is 5 mm bigger than the one of the associated restoring force.

The diagram can be applied, for example, to determine the required restoring forces or the size of a sealing gasket in order to provide sufficient watertightness for a certain test water pressure. If the manufacturer's specifications of a sealing gasket for watertightness, restoring force and profile cross-section are recorded in the diagram outside the grey shaded application areas, considering the above-mentioned boundary conditions, it is recommended to carry out additional investigations or project-specific tests in order to verify the specified values.

It is to be noted that the use of this tool does not replace any required testing or other verifications.

It should be noted that higher test pressures can be applied if offset values of less than 10 mm or smaller groove bottom distances are provided. This can be achieved, for example, by appropriate structural measures in the segment design.

4.9 Effect of connection devices on the watertightness

When applying sealing gaskets, temporary bolts or dowels are usually required in the ring and longitudinal joints, to hold adjacent segments in place and withstand the restoring force until the annular gap backfill material has hardened. Please refer to **ch. 5** for the structural verification of such connection devices.

The load-deformation behaviour of the connections has an influence on the compression level and thus on the watertightness of a sealing gasket and must therefore be included in the overall consideration of the system.

Close to crosscuts, at transitions to shafts or stations and at the tunnel portals, special measures must generally be taken to prevent deformations and associated de-compression of the gaskets. Further details can be found in **ch. 10**.

4.10 Verification of watertightness

For the verification of the watertightness project-specific specifications are required for the test pressure. If no other information is provided, a factor of 2 to the in-situ water pressure is recommended (see **ch. 5**). Further, the relevant offset value and the change of

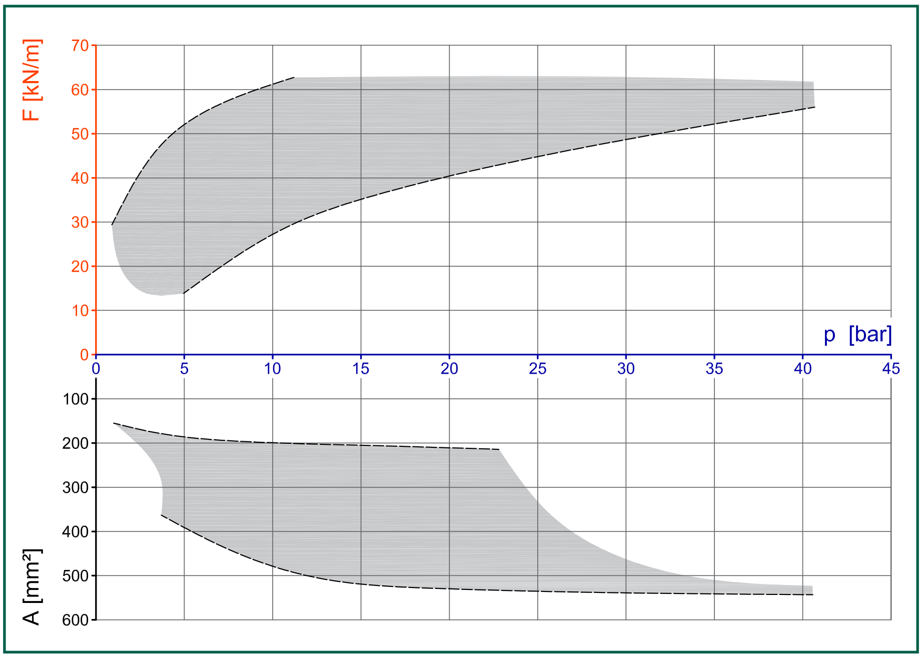

Figure 17
Force/watertightness cross-section diagram according to STUVA recommendation ***[56]***

the groove bottom distance must be taken into account in the design. In the past, a joint opening of 5 mm and a maximum offset of 20 mm have proven adequate. With the use of centering devices an improved ring building quality could be realized in the recent years justifying a reduction of the offset value to 10 mm.

The value for the joint opening depends on the segment geometry and the expected deformations.

When applying dowel connections, especially if dowels with low stiffness are used, further considerations may be required especially in the ring joints to account for increased joint openings resulting from the load-deformation behaviour of the connection devices. For verification of the watertightness the test method described in **ch. 4.7.3** can be used. Existing test results that are represented in the form of watertightness-diagrams can also be applied. The acceptable water pressure is shown in these diagrams as a function of the groove bottom distance and the groove bottom depth respectively the joint opening (gap value) (**Figure 18**).

4.11 Dimensional and weight tolerances of the sealing profile

Compliance with the standard manufacturing tolerances (see **ch. 4.5**), which only refer to the external dimensions such as height and width, is often not sufficient to ensure a consistent component behaviour. Deviations of the total rubber volume can also be critical as such deviations result into a modified deformation behaviour and thus to different restoring forces. Weighing of the sealing gasket has been found to be an effective test-method to control the rubber volume. This should be done during the production process as part of the regular quality checks and when carrying out the watertightness and force-displacement tests.

4.12 Post-sealing

For new projects the provision to repair leakages is usually contractually regulated. From a technical point of view, post-sealing or repair also depends on the type of tunnel and the possible impact of a leakage on the required properties of the structure. The success of sealing or repair works depends on several factors, in particular the cause of the damage and when it occurred.

A distinction must be made between the following causes:

- Installation inaccuracies, including insufficient compression of the sealing profiles on undamaged segments,
- Age-related reduction in compression,
- Concrete spalling originating from a sealing groove, which locally completely prevents or destroys the sealing gasket compression.

Leakages of the gaskets on undamaged segments can generally be eliminated by re-injection. Various grouting materials and different methods of injection are suitable for this application. In regard of the material cement suspensions and chemical substances, in particular acrylate gels, can be used. The flow properties and the properties in the final state as well as

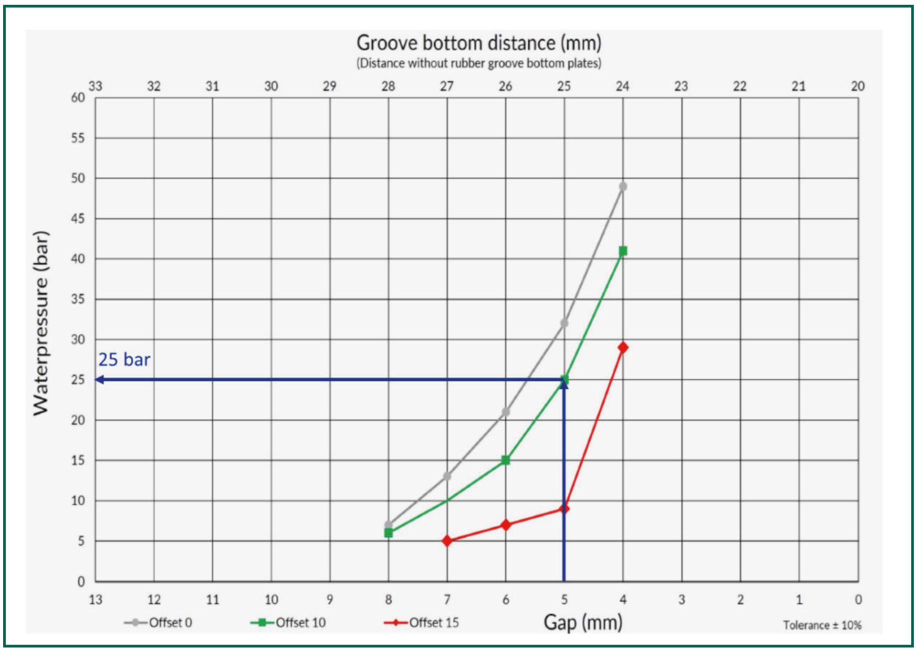

Figure 18
Example of a watertightness diagram

the costs are decisive for the final choice. Tests are usually necessary to determine the procedure. Further attention must be paid to the material compatibility of the different products applied.

4.12.1 Conventional injection

The guidelines **[28]** and **[56]** require that for the purpose of resealing holes are used that are drilled in the concrete to reach the exterior of a segment. To reach the joint gap behind the sealing gasket precise drilling of an inclined hole is necessary. This is very time-consuming and involves the risk of damaging the gasket; further, several holes may be required to ensure that the area to be sealed is sufficiently filled with grout. Drilling perpendicular to the segment surface and thus parallel to the joint is much easier and less risky. However, during the injection process the injection material first reaches the backfilled annular gap before it finds its way to the joint and to the leaking area. A large proportion of the injection material – if it reaches the "right" place at all – is lost on its way to the leakage. This can result in very high costs, especially when chemical injection agents are applied.

4.12.2 Injection via preinstalled hoses

In order to reach the outer joint gap directly, systems have been developed that use injection hoses that are cast into the segments and lead to the joints (**Figure 19**). The hoses are either attached to the grouting nozzles or to the recesses for the erector cones and are accessed by drilling. The permanent sealing of the hoses that lead from the outside of the segment to the inside of the tunnel can pose a problem. If a non-hardening acrylic gel is used, it is pushed out of the hose by the groundwater pressure. In this case, the sealing elements on the inside of the segment must be designed to withstand the full water pressure.

4.12.3 Needle injection

As a new development, a resealing injection was carAs a new development, a post-sealing injection was carried out directly through the gasket itself in Deutsche Bahn's Finnetunnel project for the first time. For this method a hollow injection needle is driven through the sealing gasket without removing any material from the profile. The needle is equipped with a non-return valve to prevent the injection material from flowing back.

To allow for postsealing of the gasket by needle injection the insertion point must be accessible. This usually requires drilling of a hole in the concrete in the segment joint, particularly for the standard case of gaskets arranged on the outside. For joints without a gap, such as longitudinal joints or ring joints without packers, a risk prevails, that the bore will deviate and miss its target. The drilling process can be simplified if groove-like recesses are provided at the joint surface at regular intervals to guide the bore. This can be easily achieved by applying an appropriate coating to the segment formwork and is particularly recommended for segments with gaskets arranged to the outside and high demands on the watertightness.

Figure 19 *Injection hoses for subsequent joint sealing*

5 Structural design

5.1 Subsoil properties (geology, hydrology)

The geotechnical investigations must be carried out on the basis of DIN 4020 **[13]**. Determining characteristic values, presenting and evaluating the results of the geotechnical investigations, as well as formulating conclusions, recommendations, and instructions, should align with the (anticipated) future lining system early in the design process.

Since the segmental lining ring already has reached its final stiffness shortly after installation, deformations of the ground are mostly confined to the area of the tunnel face and the TBM. Load redistribution in the ground around the tunnel can be applied for segmental linings only to a small extent compared to shotcrete linings.

For tunnels in hard rock with high rock loads and for systems that use pea gravel blowing, specific approaches must be considered for each individual case.

In mechanised tunnelling with segmental linings, a comprehensive subsoil investigation before detailed design is crucial, as the segment thickness cannot be altered during tunnelling in response to changing conditions.

Table 4 outlines key geotechnical parameters needed for the calculations. These parameters must be determined for all expected subsoil types and, if necessary supplemented according to the specific project requirements.

It is beneficial to depict the anticipated subsoil conditions in a geotechnical longitudinal section and assign them to corresponding tunnelling/lining categories.

If the annular gap is grouted with suitable annular gap backfill material (AGBM), the segment ring is immediately bedded against the rock with the applied grouting pressure. In the final state, the annular gap grouting has a positive effect and can possibly be considered as improved annular bedding. When compressible annular gap backfill is used, rock loads can be mitigated in line with the deformability of the ring or the backfill material.

5.2 Influence of annular gap backfilling on calculations

In many cases, the influence of the mechanical parameters of the annular gap backfill material on the structural design is negligible. An exception to this is an AGBM with significantly lower stiffness than the rock mass (a difference of at least an order of magnitude). If the influence of the annular gap backfill is considered in the modelling, the model must reflect the properties, meaning the behaviour of the AGBM and, if applicable, its setting process over time must be known.

When fully modelling the AGBM, it is essential to ensure that this leads to realistic and safe results, preventing an unintended loadbearing effect occurring in the annular gap. For example, a high stiffness and strength of the AGBM can result in a proportional transfer of normal forces and therewith in a load relief of the segmental lining.

The shear strength development of the AGBM must be considered for the tangential bedding. If a fresh AGBM does not have sufficient shear strength, a limited value between 1 % and 5 % of the radial subgrade modulus is usually considered in the structural calculations to ensure numerical stability. For an AGBM with a fast increase in shear strength and for later load cases, such as future construction work or one-sided loads, the tangential ground-lining interaction may need to be considered. The tangential bedding should always be limited to the maximum transmissible friction.

If the segment ring is backfilled with pea gravel, special considerations must be made regarding mathematical modelling, taking into account project-specific boundary conditions, such as subgrade-free

Table 4 *Required geotechnical parameters for the structural calculations*

Graphical calculation		Slab calculation (Mohr-Coulomb)		Slab calculation (Hardening-Soil)	
Young's Modulus	E_s	Modulus of elasticity	E	Reference Young's Modulus	$E_{s,ref}$
Density	γ/γ'	Poisson's ratio	n	Reference triaxial modulus	$E_{50,ref}$
Lateral pressure coefficient	k_0	Density	γ/γ'	Un-/reloading modulus	E_{ur}
Water pressure	p_w	Friction angle	φ	Reference tension	σ_{ref}
		Cohesion	c	Stiffness exponent	m
		Dilatancy angle	Ψ	Poisson's ratio	n
		Tensile strength	f_t	Density	γ/γ'
		Water pressure	p_w	Friction angle	φ
		Lateral pressure coefficient	k_0	Cohesion	c
				Dilatancy angle	Ψ
				Tensile strength	f_t
				Water pressure	p_w
				Lateral pressure coefficient	k_0
				Shear factor	R_f

areas – especially in the crown – and possible load patterns.

For the functionality of the AGBM, see Gehwolf et al. **[37]** and DB AG's RiL 853.4006 **[1]**.

5.3 Actions and load combinations

5.3.1 Permanent actions

Permanent actions include:

- Dead weight of the segmental lining and all other incorporated components,
- Load from the ground/rock including squeezing and swelling,
- Water pressure,
- Loads from tunnel installations,
- Internal pressure, usually as internal water pressure,
- Loads from the annular gap grouting, if unfavorable,
- Permanently acting loads on the ground surface and influences from neighboring structures underground,
- Loads from possible subrosion and suffusion phenomenons (for example karst, sinkholes).

The jacking forces of the tunnel boring machine (TBM) acting on the lining during TBM advance are a special case. These are temporary actions that result from the construction period but remain in place long enough to be regarded as permanent loads. A distinction is usually made between the "jacking forces from regular TBM advance" and, as a limit case, the maximum "installed jacking forces" that can be utilised for tunnelling. In the structural analysis, it should be assumed as limit case that the jacking forces will completely dissipate over time due to creep effects, particularly in the intermediate layers (packers), or due to shrinkage effects of the concrete.

Loads from shrinkage can generally be neglected for segmental lining rings, as the segments are almost fully hardened before installation.

As creep processes in the segments generally reduce bending and would thus lead to an associated lower overall loading, such creep effects do usually not need to be considered.

When dimensioning the segment ring, potential future development should also be considered. In particular, excavations above and to the side of the tunnel tube as well as loads from future buildings can lead to significant load conditions. This problem increases for tunnels with low overburden (shallow tunnels). Therefore, appropriate specifications should be provided on a project-specific basis and addressed during the tender process. Different concepts can be pursued:

A: The loads and possible subsequent excavations are already known and are explicitly specified in the tender documents.

B: The tunnel is designed without accounting for influences from future development. Any subsequent development must therefore be planned in such a way that it does not lead to any unacceptable load conditions for the tunnel lining.

C: A "protection zone" or area of influence around the tunnel is defined, within which – without detailed mathematical investigations – no subsequent construction, load application or excavation may take place. The protection zone must be dimensioned in such a way that external influences cannot lead to significant load conditions for the tunnel lining.

Figure 20 shows an area of influence within which it can be assumed that excavations or loads may significantly impact the tunnel tube. The angle of load distribution Θ is usually assumed in the range from 45° to 45° + φ/2. **Figure 21** shows an example of a protection zone, within which no load transfer and/or excavation should take place in the future.

The relevant parameters such as the angle of load distribution or the size of a protection zone must always be agreed at an early stage with the client and, if necessary, other experts and described in the tender documents, as no generally recognised regulations are available for this load action.

5.3.2 Particularities of actions from squeezing and swelling

Under certain circumstances, the squeezing of anhydrite rock or the swelling of some clay soils, both caused by water ingress, can represent an extremely unfavorable load action for segments. The squeezing and swelling pressures are generally dependent on the mineralogical composition of the subsoil and the stiffness of the segment ring or its ability to alleviate the load through displacement. It should also be noted that the pressures determined in squeezing and swelling tests are usually significantly higher than the pressures measured in situ.

The combination of squeezing and swelling pressures with the other actions must be specified in the project specifications and illustrated in load diagrams. Subsequent load changes, such as those from building development or excavation, with potentially different load actions must be considered. The water pressure must be treated as an independent load.

For the combined pressures from the subsoil/rock mass and squeezing or swelling pressures, superposition shown in **Figure 22** is recommended.

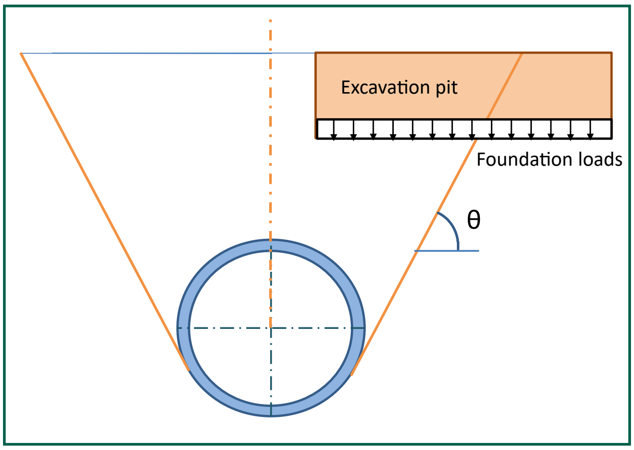

Figure 20 *Area of influence of a shallow tunnel for lateral excavations and foundation loads*

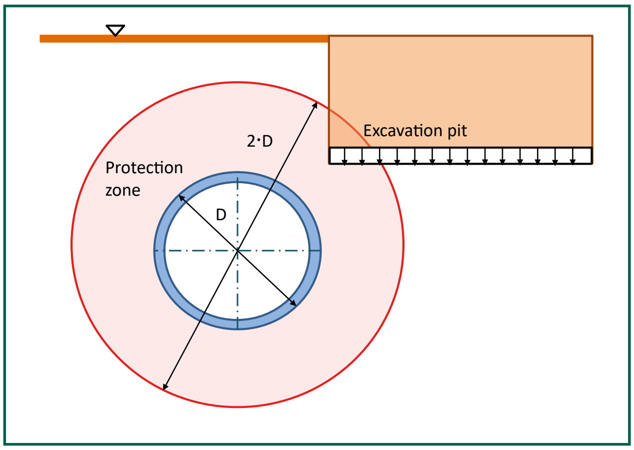

Figure 21 *Example of a protection zone around a deep tunnel*

5.3.3 Regularly occurring variable actions

- Temperature variations: these generate constrain forces and should be applied in the final state with their full range as limit case.
- Traffic loads on the ground surface, in neighboring underground structures and on traffic routes in the tunnel's zone of influence.
- Characteristic actions for traffic inside the tunnel.
- Aerodynamic actions caused by the traffic inside the tunnel: they can generally be neglected for the design of the segments.

5.3.4 Rarely occurring actions and transient actions

Rarely occurring traffic loads include impacts from vehicles on the ground surface away from transport routes. Due to their usually small size and the load distribution in the subsoil, they can generally be neglected. In ground with low stiffness and in tunnels with low overburden, they can potentially cause significant loadings due to the relative stiff and thus load-attracting tunnel tube.

Transient actions during construction include:

- Temporarily acting loads of tunnelling machines, construction equipment, scaffolding, construction materials, and construction components. This includes, for example, loads from the back-up bridge, the back-ups and the supply vehicles for the TBM.
- Secondary grouting, such as post injection or compensation grouting; the grouting pressures and their range must be determined on a project-specific basis.
- Actions from compressed air, where they act unfavorably.

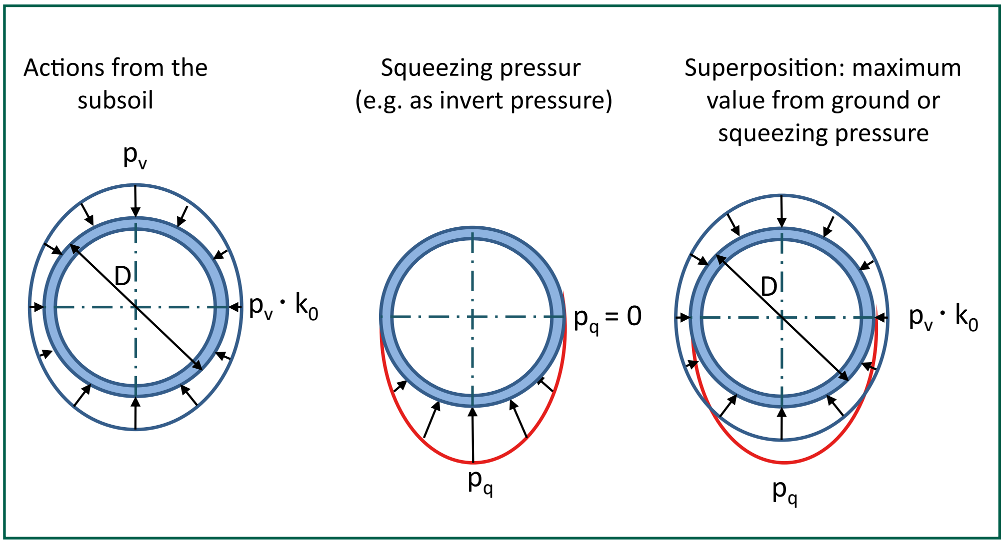

Figure 22 *Recommendation for superimposing squeezing pressure and rock pressure (the water pressure must be superimposed)*

Temporarily acting actions from changes to the ground surface (e.g., excavations or embankments) and from adjacent cavities (e.g., subsequent tunnelling) are generally to be classified as permanent actions. Only in the case of very short-term load conditions (e.g. support pressure from passing by of a TBM) they can be regarded as rarely occurring actions.

5.3.5 Accidental actions

Accidental actions include:

- Temperature loads in the event of fire,
- Earthquake/seismic actions: in Germany, these can generally be neglected due to their minor impact on the segmental lining,
- Accidental actions from tunnel installations (e.g., failure of the catenary in railway tunnels),
- Flooding of the tunnel via the portals: for tunnel projects near bodies of water, where flooding of the tunnel tube is possible during exceptional flooding events, the segment tube must be checked for internal water pressure,
- Impact loads of vehicles on the tunnel lining. These must be specified on a project-specific basis, considering any existing protective measures such as impact walls,
- Explosions in traffic tunnel tubes: generally, these actions need not to be considered. Load cases from explosions are to be specified by the client or the authorities only in special cases.

5.4 Concepts for verification in the ultimate limit state and serviceability limit state

5.4.1 General Remarks on concepts for structural veririfcation

The following sections describe the verification and design concept for tunnels with segmental lining as it is usually applied in Germany. For countries outside Germany, the regulations may be applied analogously taking, where applicable, national regulations into account.

The recommendations listed below should be considered individually for each project. It is recommended that the regulations that are to be applied and the relevant actions are provided in the tender documents in a “design specification sheet”. It is also recommended that the project participants agree on the basic principles of the verification process and summarise them in a design basis document before commencing the structural calculations.

The European standards DIN EN 1990, DIN EN 1991, DIN EN 1992 and DIN EN 1997 cited in this chapter always apply in conjunction with the associated National Annexes and the regulations cited therein (e.g. DIN 1054) (**[3]** to **[11]**).

The concepts listed below relate to the structural analysis of the finished segmental lining system. According to DIN EN 1990, the ultimate and serviceability limit states for the permanent, temporary and exceptional design situations BS-P, BS-T and BS-A must be verified. The verifications are to be carried out in conjunction with DIN EN 1997-1 and DIN 1054.

In the ultimate limit state (ULS) these are generally the STR (structural failure - internal failure or major deformation of the structure) and GEO-2 (geotechnical failure - failure or major deformation of the ground) verifications. In accordance with the specifications of the National Annex, the STR verification is carried out according to Method 2 applying the partial safety factors in accordance with DIN EN 1997-1/ NA and DIN 1054, Table A.2.1. For shallow tunnels in particular, UPL verification of buoyancy safety must also be provided. As vibration loads in tunnels are usually low compared to external actions, FAT verification of fatigue failure of the segments is generally not required.

Please refer to **ch. 5.4.3** for verifications of the serviceability limit state (SLS).

The structural analyses are usually non-linear due to the subsoil and subgrade behaviour as well as the couplings in the ring and longitudinal joints. If the non-linear material behaviour of the segments is also considered, the degree of non-linearity may be excessive. Regardless of the extent of their non-linearity, the calculation results are subject to the fundamental problem that the internal forces cannot be calculated individually for the load components of permanent, temporary, or exceptional actions and then superimposed. This makes it difficult to consider the different partial safety and combination factors when determining nominal internal forces. Possible approaches for solution are described in **ch. 5.4.2**.

The material parameters and partial safety factors specified in DIN EN 1992-1-1 and DIN EN 1992-2 apply to the resistance of reinforced concrete segments. The design is usually carried out according to DIN EN 1992-1-1 and for railway tunnels, following the specifications of RiL 853, according to DIN EN 1992-2.

➲ ***Note:*** *The reinforced concrete standard DIN EN 1992-1-1 refers to buildings and DIN EN 1992-2 to concrete bridges. There is no reinforced concrete standard specifically for tunnel structures. Internationally, it is common practice to design tunnel lining according to EN 1992-2 or in line with regulations for concrete bridges, among others due to the more stringent requirements for the design lifetime (durability). For the design of segments, however, the differences in the practical application of DIN EN 1992-1-1 compared to DIN EN 1992-2 are minor.*

Apart from special cases, for example for tunnels in unstable slopes, verifications of the limit state GEO-3 (slope or embankment failure) are not relevant for tunnels.

In addition to the verification of the overall system, further detailed design verifications must be carried out:

- Tangential splitting tensile due to jack loading,
- Partial surface pressure and splitting tensile in ring and longitudinal joints,
- Watertightness of the segment joints, i.e. verification of the sealing gaskets.

5.4.2 Verifications of load-bearing capacity (verifications in the ULS)

Verification of the limit states STR and GEO -2

Verification of the limit states STR and GEO-2 is carried out according to DIN 1054, in addition to the specifications of DIN EN 1997-1 with the partial safety factors provided in these standards. Tunnels with segmental lining in unconsolidated or fissured rock are usually assigned to geotechnical category GK 3, and segmental lining tunnels in solid, less fissured rock to geotechnical category GK 2.

For the verifications according to DIN 1054 or DIN EN 1997-1, the fundamental problem is that these require non-linear calculations for which the superposition principle for the internal forces in the tunnel is not applicable. The following principle options are available for determining the internal forces in line with the STR or GEO-2 verification method:

1) For linear elastic analyses, the method described in DIN EN 1997-1 / DIN 1054, section 2.4.7.3.4.2 (1) with determination of the internal forces according to equation A (2.6c) can be applied. This requires the application of the load factors to the actions, i.e. to the individual loads.

   This method is suitable for calculations where the actions are known or specified as external loads. In tunnelling, this method is therefore generally limited to systems with bedded beams or shells. For detailed information, refer to the information provided in RiL 853.2001 **[1]**, section (19).

2) Internationally, a modified method to 1), using DIN EN 1990, section 6.3.2(2), is also applied for systems with non-linear behaviour. For this method all loads are factored with the associated load safety factors according to DIN EN 1990 and, where applicable, partial safety factors applied to the soil parameters. In the subsequent design, the resulting internal forces are not further factored.

   Since design results on the unsafe side cannot be ruled out, this procedure should not be used for structural analyses of the entire system.

3) For segmental linings, the method according to DIN 1054, section 2.4.7.3.4.3 A(4), can be used. This requires the analysis with non-factored loads, i.e. under characteristic or representative load conditions. The combination factors and partial safety factors are applied only afterwards to the internal forces.

   To separate the internal forces into permanent and to variable actions, the internal forces of the variable actions must be determined by subtraction. For detailed information, refer to the information provided in RiL 853.2001 **[1]**, section (17). This procedure is time-consuming and can be confusing, especially if many subtractions are required due to different actions and/or if the combination coefficients are applied for an economical design. Therefore, the method of DIN 1045 , section 2.4.7.3.4.3 A(4) is generally only practical to a limited extent.

4) The internal forces for the segments can be determined in accordance with DIN 1054, section 2.4.7.3.4.3 A(5). For the verification of tunnel segmental linings, where variable actions in the overall system are generally subordinate, this method is suitable regardless of the size, distribution and type of the variable or exceptional actions. The calculation of the internal forces is carried out with the permanent loads applied with their characteristic value and the variable loads partially factorised with the factor $f_q = \gamma_Q / \gamma_G$. The resulting internal forces are then uniformly multiplied by the partial safety factor gG to determine the internal design forces.

   This procedure essentially corresponds to that described in DIN EN 1990, section 6.3.2(4b). As this method leads to plausible internal forces even with a non-linear system behaviour and is easy to implement, see also **[51]** and **[58]**, it is recommended for the design of segments in accordance with DIN 1054.

Verification of the bearing capacity of the ground is usually not required for segmental lining tunnels with sufficient depth (overburden ≥ 2 ∅). In special cases, such as shallow tunnels, excavations near tunnels or tunnels in slopes (especially in unstable slopes), it may be necessary to verify the load-bearing capacity of the ground. The verification is then carried out according to the GEO-3 method, using the nominal resistances according to DIN EN 1997-1, section 2.4.7.3.3 (equation 2.7a), i.e. reduced ground bearing capacities.

Experience has shown that when considering temperature variations in the verification of the STR and GEO-2 limit states, this would lead to implausible, unfavourable results due to the partial safety factor γ_Q = 1.5 that is to be applied in BS-P as per DIN 1054. Therefore, special specifications are recommended for the coefficients for temperature variation in line with RiL 853.2001 (19):

- γ_Q = 1.5: when the internal forces are determined with the stiffness (EI) of the tunnel shell in state II , as an approximation EI_{II} = 0.6 EI_I may be assumed,
- γ_Q = 1.0: when the internal forces are determined with the stiffness (EI) of the tunnel shell in state I.

Verifications of the limit states UPL / verification of buoyancy safety

Verification of buoyancy safety (UPL limit state) is usually only required for shallow tunnels and in the case of excavations near the tunnel. It must be carried out according to DIN EN 1997-1 supplemented by DIN 1054, for the design situations BS-P, BS-T and BS- A. The verification can be performed using analytical or numerical methods.

Analytical procedure: in simplified terms, the buoyancy on the tunnel cross-section and any required additional destabilising actions are usually regarded as unfavourable permanent actions. The dead weight of the tunnel and the subsoil above the tunnel, located within the vertical boundary lines from the outer edge of the tunnel, are considered favourable permanent actions. If additional ground resistance is considered that results from a spreading of loads in the subsoil, suitable approaches must be selected and justified from geotechnical point of view.

When using numerical methods on the continuum, the partial safety factors must be fully incorporated into the analysis. External loads and dead weights of structural elements must generally be adjusted accordingly by increasing or decreasing them. The soil parameters should be applied in their reduced magnitude, as should retaining effects from piles and anchors. This procedure corresponds in principle to the verification procedure according to DIN 1054 and DIN EN 1997-1, equation (2.6a). The verification is provided by convergence of the analysis with tolerable displacements of the tunnel. It should be noted that this is a design verification allowing higher displacements than those permissible for the serviceability limit state verification.

For the verification of buoyancy safety in tunnel tubes that are not completely located within groundwater, the highest expected water pressure (design water level) during the lifetime of the tunnel should be considered for the final state, and the highest expected water pressure during construction (construction water level) for the construction state. For a later replacement of the roadway in traffic tunnels the partial safety factors for the design situation BS-A may be used for the verification "without roadway", unless deviating project-related specifications apply.

5.4.3 Serviceability verifications (SLS)

As part of structural design, the serviceability verifications include the crack width design and a limitation of the displacements of the lining. If necessary, distortion in the longitudinal joints must also be verified or limited to ensure watertightness of the sealing gaskets.

Crack width limitation is generally verified according to DIN EN 1992-1-1, for railway tunnels pursuant to ZTV-ING according to DIN EN 1992-2. The associated determination of the internal forces is carried out analogously to the procedures listed in **ch. 5.3.2** to **5.3.5**. According to RiL 853, the internal forces must be determined for railway tunnels considering linear material behaviour of the segments. The internal forces should be determined for the frequent load combination. For BMDV traffic structures (road tunnels of the German Federal Ministry for Digital and Transport), the requirements of ZTV-ING apply. Deformations of the segment shells should be determined for the frequent load combination in accordance with DIN EN 1990. Permissible deformations must be determined on a project-specific basis. In any case, the calculated deformations must not impair the functionality of the tunnel.

5.5 Determination of internal forces – calculation method

The structural behaviour of a tunnel lining is primarily determined by the supporting effect of the surrounding rock and the interaction between ground and structure. Consequently, structural design in tunnelling has a number of specific characteristics:

- The surrounding rock acts simultaneously as a load and as support (resistance). It is generally not possible to separate these two effects, or only with very simplified modelling of the real load-bearing behaviour.
- Segment rings are hinged chains that essentially derive their stability from the interaction with the surrounding rock (including the annular gap backfill) and, if necessary, from the coupling of individual rings.
- The structural systems of tunnel shells are highly indeterminate.
- Tunnel shells are typically stressed by a combination of normal forces and bending moments. It is often

unclear a priori whether the maximum or minimum actions or soil properties are decisive for the design. Limit case considerations or sensitivity studies are therefore often necessary in tunnel design.

- The rock mass parameters and design parameters generally show significant variability.
- Construction methods and construction conditions have considerable influence on the loads on the segment rings and must generally be considered in the analytical verifications.

The calculation method and safety concept must be adapted to these conditions.

5.5.1 Overview of the calculation models

The internal forces for the segment ring are determined using numerical models in 2D and 3D. 2D models assuming a plane strain condition are common. If more complex boundary conditions or interactions between the individual rings have to be considered, 3D models are used.

The interaction of the individual segments or rings must be taken into account. When selecting the model, a distinction must be made as to whether the rings can be considered as individual rings or whether a coupling between the rings must be taken into account. When coupling is applied in the ring joints, the deformation behaviour of the individual rings is equalised (**Figure 23**), whereby the bridging of the concrete hinges by the adjacent segments leads to an increase in system stiffness. If the ring coupling cannot be guaranteed over the entire service life, a single ring system without coupling is to be considered in the final state as limiting case.

Generally the applied analytical models differ with regard to the discretisation of the ground and the lining (**Figure 24**).

A key feature is the modelling of the ground, realized either with bedded beam spring models or continuum models. Bedded beam spring models model the ground by means of elastic springs. For more complex ground conditions, continuum models with explicit modelling of the ground have become established.

The lining is discretised either in 2D with bedded beams, in 3D with shell elements or in special cases with continuum elements in 2D or 3D. The interaction of the segments in the longitudinal joints and ring joints is simulated using coupling elements.

The various options for modelling the segmental lining can be combined with those for simulating the ground. The resulting models are suitable for structural analysis in different ways, depending on the boundary conditions and objectives. The results allow evaluations that range from first estimates to comprehensive recording of the load-bearing and deformation behaviour in the lining and the ground. An overview of the combinations and areas of application can be found in **Table 5**.

Models of soil-structure interaction with bedded spring models

In bedded spring models, the ground and thus the segmental lining's bedding is modelled using linear-elastic springs with radial tension cutoff. The radial bedding stiffness K_r is determined from the stiffness modulus of the existing ground E_s and the system radius R_{sys}, see also explanations in Behnen et al. **[36]**:

$$K_r = E_s / R_{sys}$$

For later load cases, e.g. future construction work or unilateral loads, the tangential bedding must generally be taken into account, but this should be limited to the maximum transmissible friction. For the tangential bedding approach, see also the comments in **ch. 5.2**.

Due to the limited soil-structure interaction, spring models cannot directly take into account any preliminary deformations of the ground.

The load is applied using suitable and accepted load models, e.g. according to EBT/Duddeck **[31]** or Behnen et. al **[36]**. The application of tangential loads must be coordinated with the total tangential bedding value.

Tunnel lining as beam elements

For 2D calculations with bedded beam spring models, beam elements have become established for the discretisation of the lining. Here, the cross-section properties of the segmental lining are assigned to the beam elements.

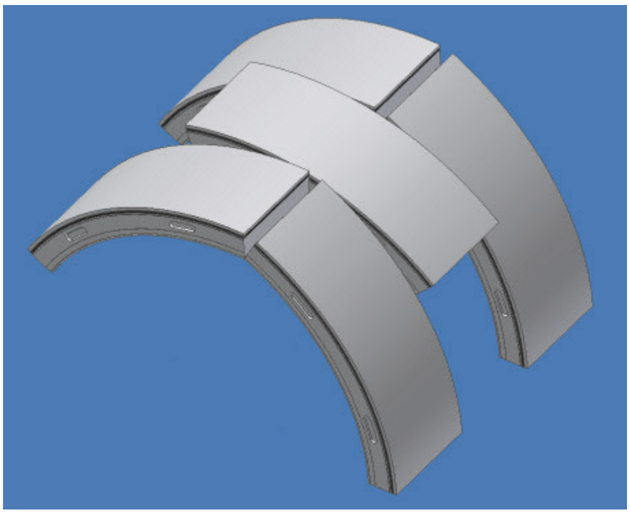

Figure 23 *Different deformation of individual segments (highly exaggerated)*

Table 5 *Simplified overview of calculation models and areas of application for segmental lining analysis. Other analytical models, combinations of models and evaluations are possible and are not listed here.*

			Use case								
Model	Modelling of		Realistic recording of			Determination of					Remarks
	Segmental lining	Ground	Characteristics of the ground	Loads on segment ring	Construction process	Internal forces	Longitudinal joint rotation	Deformation of the ring	Ground settlement	Actions from ground	
2D	Bedded beams (rigid)	Bedded springs	○	○	–	○	–	–	–	–	"Muir-Wood" classic
2D	Bedded beams, coupled (hinges in longitudinal joint)	Bedded springs	○	○	–	+	+	–	–	–	Single or double continuous member
2D	Bedded beams (rigid)	Continuum 2D	+	+○	–	○	–	○	○	○	"Muir-Wood" in continuum
2D	Bedded beams, coupled (hinges in longitudinal joint)	Continuum 2D	+	+	–	+	+	+	+	+	Single or double continuous member
2D	Continuum elements (2D)	Continuum 2D	+	+	–	+	+	+	+	+	
Pseudo-3D	Shells or volume elements, coupled	Bedded springs/springs modulus	○	○	–	+	+	○	–	–	For 3D effects in the segments
Pseudo-3D	Shells or volume elements, coupled	Continuum 3D	+	+	–	+	+	+	+	+	For 3D effects in the segments
3D	Uncoupled shells (rigid)	Bedded springs/springs modulus	○	○	○	○	–	–	–	–	For special areas, e.g. with stresses in the longitudinal direction of the tunnel (e.g. crosscuts)
3D	Coupled shells	Bedded springs/springs modulus	○	○	○	+	+	○	–	–	For special areas, e.g. with stresses in the longitudinal direction of the tunnel (e.g. crosscuts)
3D	Volume elements	Bedded springs/springs modulus	○	○	○	○	+	–	–	–	For special areas, e.g. with stresses in the longitudinal direction of the tunnel (e.g. crosscuts)
3D	Uncoupled shells (rigid)	Continuum 3D	+	+	+	○	–	○	○	○	High calculation effort; therefore mainly for special
3D	Coupled shells	Continuum 3D	+	+	+	+	+	+	+	+	High calculation effort; therefore mainly for special
3D	Volume elements	Continuum 3D	+	+	+	○	+	+	+	+	High calculation effort; therefore mainly for special

+ suitable/recommended
○ less suitable/recommended
– not suitable/recommended

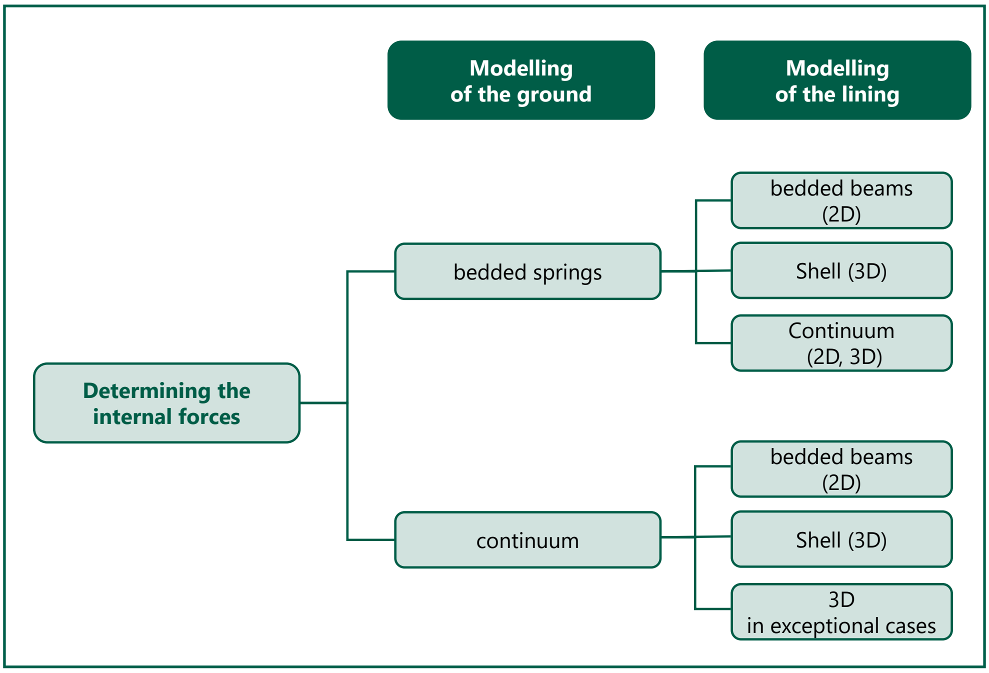

Figure 24
Modelling options for the ground and segmental lining

The coupling of adjacent rings can also be carried out in 2D models by selecting the appropriate beam elements dimensions and implementation of suitable coupling elements.

Tunnel lining as shell elements
Bedded shell elements are often used for the discretisation of segmental lining segments in pseudo-3D and 3D models. The load and bedding assumptions here correspond to those of the beam element models. The main advantage of shell elements over beam elements is that the force flow is also represented in the longitudinal direction of the tunnel. This is particularly useful for more complex interactions between individual segments, such as those resulting from a pronounced coupling effect in the joints or in the area of crosscut openings.

Tunnel lining using continuum models
In exceptional cases, continuum elements with bedding can also be used to model the tunnel segments. Advantages and application are analogous to the shell models.

5.5.2 Continuum models for the ground

Continuum models for the ground have become established in recent years. In these models the lining is modelled in the same way as in the bedding models, using either beam, shell or continuum elements.

Due to the discrete modelling of the subsoil, its loading effects on the lining are "automatically" determined as a result of the calculation and it is no longer necessary to apply them as external loads as in the bedding models. For a realistic determination of the resulting effects it is required that all individual components, such as the tunnel lining, interactions between the ground, the AGBM and the segmental lining, as well as their stiffnesses and their interaction are realistically modelled. Continuum models for the ground provided advantages especially in the following situations:

- Consideration of inhomogeneous stress states in the ground,
- Consolidated and over-consolidated ground,
- The consideration of relief states,
- Inhomogeneities in the ground, including structures and construction components,
- Crosscuts, especially in the presence of auxiliary construction measures such as icing or grouting,
- Determination of settlement and stabilising, settlement reducing measures,
- Pronounced plastic subsoil conditions,
- Deep tunnels,
- Tunnels in rock with extremely stratified ground conditions.

Due to the computing time, these models are often limited to 2D calculations in the plane strain state. 3D models are used, for example, to realistically capture construction processes and the resulting 3-dimensional stress redistributions.

5.5.3 Couplings

The individual segments and rings interact in their joints. Generally, this effect has to be taken into account for determination of internal forces by modelling the coupling conditions. Couplings are present due to geometric-mechanical boundary conditions and due to special coupling devices. For considering the coupling effects a distinction must be made between longitudinal and ring joints, see also **ch. 5.7.11.**

Longitudinal joints

Due to the concrete hinges in the longitudinal joints bending moments can only be transferred to a limited extent. Usually contact surface elements or torsion springs are used to simulate the reduced, non-linear torsional spring stiffness. The established approaches according to **[14]** and **[34]** can be used to derive the torsional spring stiffnesses (**Figure 25**). According to these approaches, the torsional spring stiffness depends on the magnitude of the normal force and the geometric joint layout as well as the concrete stiffness.

Depending on the software options, it may be necessary to simplify the characteristic curves to a bilinear approximation. When using this approach it must be ensured that the constant branch of the approximation curve is applied at a level of joint rotation that conforms to the calculation results.

Under certain circumstances, the maximum shear forces that can be transferred in the longitudinal joints must also be taken into account. This can done for example by using normal force shear springs with consideration of friction. Otherwise, excessive shear forces may be calculated in the longitudinal joints, which cannot be transferred with the given normal forces. This is particularly important in the case of a significant three-dimensional load transfer, e.g. at crosscuts, or in case of low normal ring forces.

Ring joints

The individual rings can be coupled in the ring joints on the one hand by friction in the contact surfaces and on the other hand by built-in devices, such as shear dowels or tension dowels with shear capacity. Since the coupling between the rings increases the overall stiffness of the system the deformations of the individual segment rings are equalised. Depending on the coupling device different force-displacement curves and, if necessary, slippage can be applied.

To consider the coupling effect of friction in the ring joint it is important to know the forces acting in the longitudinal direction of the tunnel in the respective state. If longitudinal forces are present, these are generally caused by the residual jacking forces from the TBM advance. Longitudinal forces can be subject to load-related changes, such as load redistribution at crosscuts, thermal constraints, e.g. in the event of fire, or high additional loads. When considering the coupling from friction, the layout of the ring joints and their contact surfaces as well as any packers and their friction behaviour must be taken into account. These determine both the normal and shear force stiffness as well as the magnitude of the transmittable friction. Due to time and temperature-dependent effects, the presence of coupling from friction can neither be guaranteed nor ruled out for the entire service life time of a tunnel. Therefore it is reasonable to consider "friction" and "no friction" in the ring joints as limiting cases.

The shear force behaviour of accessories in the ring joint should be based on the force-displacement

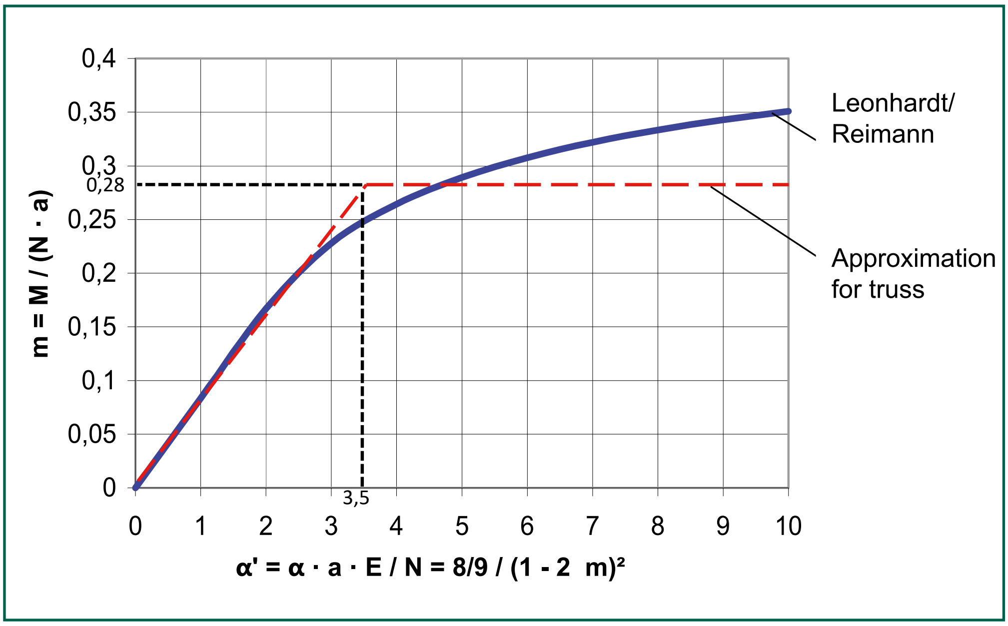

Figure 25
*m-α' relationship (according to **[14]** or **[34]** as well as its bilinear approximation)*

characteristics determined in tests. Tolerances in the segments for the accessories are usually considered in the design. These installation tolerances may not be considered in the tests and result in slippage or a lower initial stiffness, which may need to be allowed for when applying the force-displacement charts.

The coupling is often modelled using non-linear coupling elements, usually springs. It should be taken into account here that the couplings can act in both radial and tangential directions.

5.6 Determination of the internal forces and stresses from the construction process

5.6.1 Tensile splitting resulting from jacking forces

During tunnelling the TBM supports itself with the advance force with the jacks on the ring joints of the last segment rings already installed. This creates partially loaded areas under the jack shoes, which depend amongst others on the geometry of the ring joint, the geometry of the jack shoes and the type of jacks. The partially loading and the resulting splitting tensile forces must be verified in accordance with DIN EN 1992-1-1 **[7]** in conjunction with **ch. 5.7.7**.

In accordance with ZTV-ING, section 3.2.3(2) **[2]**, a distinction can be made in the verifications between an operating jacking force for standard tunnel advance and the maximum installed jacking force. For further details, please refer to **ch. 5.3.1** and **ch. 5.7.7**.

5.6.2 Plane stress due to offsets or gaps in the ring joint

"Plane stress" refers to stresses in the curved segment plane; the terms "shell stress" or "stress in the segment plane" are also used.

According to **[30]**, the typical arrangement of the jacks leads to plane stresses and edge tensile stresses at the segment ring joints, which must be structurally verified and covered by reinforcement. Table values for the edge tensile stresses for segments can be found in **[38]**. In general, the calculation of the splitting tensile stresses according to **[58]** and **[59]** is carried out numerically with plane or shell models of a curved single segment.

During ring installation, assembly inaccuracies can lead to offsets between adjacent segments, an uneven ring surface (ring joint surface) or gaps in the ring joint, which may be exacerbated by manufacturing tolerances. Depending on the extent of the unevenness and the type of joint design, with joint inserts or without inserts, i.e. with pure concrete-to-concrete contact, the coupling in the ring joint may be very stiff and unable to compensate for the unplanned deviations. The resulting additional plane tensile stresses may cause damage to the segments (**Figure 26**).

It is difficult to specify realistic values for the allowable gap Dx for the design, as these depend primarily on the quality of the ring assembly, but also on other boundary conditions. Assumptions that are too unfavourable may lead to uneconomical reinforcement cross-sections, while assumptions that are too favourable can lead to increased formation of cracks. In practice, it has therefore proved useful to define project-specific limit values in consultation between the client, the designer and the contractor.

5.6.3 Stresses from jack eccentricities, misalignments and rolling effects

In practice, it can happen that the jacks do not act perpendicular to the face of the ring joint or eccentrically to the ring axis. This results into downward forces that can lead to a "trumpet-shaped" expansion of the rings and to increased cracking.

Rolling of the ring can also lead to increased tensile stresses in the segments due to the deviating load application. Rolling can usually be limited by specifications in the TBM control system.

The additional stresses from imperfections listed in this paragraph are generally not analysed mathematically, but are taken into account by means of a robust segment design and systematic tolerance allowances. Mathematical estimates of the influences mentioned could be useful to verify the robust design of the segments.

5.6.4 Stresses from transport, storage and installation processes

Storage

Storage of the segments is described in **ch. 11.1.9**. From a structural point of view, care must be taken to ensure that the supports are arranged precisely with

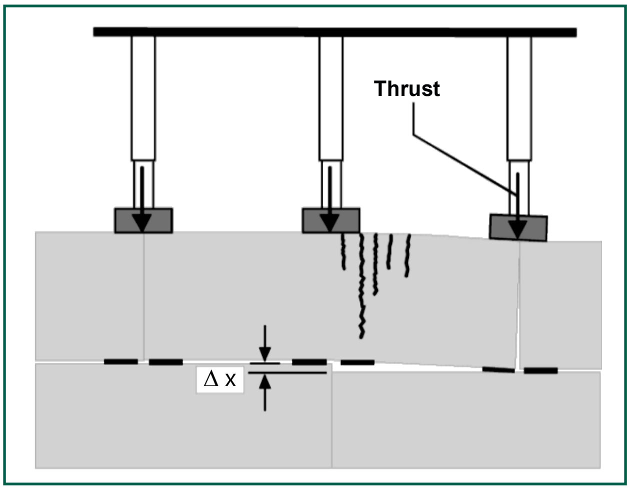

Figure 26 *Cracking from offset in the ring joint*

regard to their evenness and in accordance with the structural specifications. Further it must be ensured that the spacers reliably prevent contact between the segments themselves or with the base surface. The segments must be placed exactly in the intended position. If the supports or intermediate spacers are arranged inclined a risk remains that the segment experiences internal torsion from creep and can no longer be installed in a precise position.

The supports of the segment stack must be structurally verified or the supports must be specified based on a structural analysis. Based on this the tolerances for the positions of the spacers must be determined. Furthermore, the concrete strength at time of storage begin must be taken into account in the verifications.

Transport

During transport, risks due to accidental contact or dynamic actions must be prevented. If damage to the segments is found after delivery to the TBM, it must be investigated how this damage occurred and remedial action must be taken. All damage from transport must be documented.

Stresses from installation processes

Stresses resulting from the installation processes must already be taken into account during design of the segmental lining. As a rule, no mathematical verification is required when using vacuum erectors for the segments. If mechanical erector pick-up units are used that grasp the segments at single points, the segments must be designed for the resulting bending moments.

5.6.5 Splitting tensile stresses in the gasket groove from the restoring force

From tests performed on the 4th tube of the River Elbe Tunnel in Hamburg it is known that the restoring forces of the sealing gaskets have a considerable impact on the watertightness and on the damage-free state of the segment joints **[39]**.

As a rule, the existing reinforcement cannot be considered to absorb splitting tensile forces due to the required concrete cover to the segment surface and to the joints. A verification of the edge concrete against spalling in the area of the gasket groove can therefore only be carried out using the tensile strength of the concrete. There is no reliable analytical verification method for this load; instead, verification should preferably be carried out by means of spalling tests in accordance with **ch. 4.7.4** and/or by means of structural measures in accordance with **ch. 4.6**.

If a mathematical verification is to be carried out, this can only be done in a meaningful way using numerical analyses, for the corner areas in 3d, taking the load concentration in this area into account. However, this procedure is very time-consuming and should be limited to exceptional cases.

5.7 Specific verifications of the segment joints

5.7.1 General remarks on partially loaded areas and splitting tensile reinforcement

Due to the typical joint design with flat contact surfaces, concentrated loads act in both the ring joint and the longitudinal joint. These are usually verified using analytical verification methods (stress block method) as "partially loaded areas". The locally increased concrete compressive stresses and the splitting tensile stresses resulting from the load distribution must be verified respectively splitting tensile reinforcement must be determined.

All verifications in **ch. 5.7** assume that the actions of normal force N and bending moment M acting in the joint are known. They are determined using the calculation methods described in **ch. 5.5**.

The verifications described in this chapter apply exclusively to conventionally reinforced concrete segments. Specific verifications for steel fibre reinforced concrete segments are dealt with in **ch. 5.11**. Analytical joint verifications are dealt with here first. For joint verifications using numerical non-linear calculations, see **ch. 5.7.9**.

The geometric variables and loads used in the verifications are shown in **Figure 27**.

Stress block method

The verification concepts for the design of longitudinal joints mentioned in **ch. 5.7.2** to **5.7.8** are based on the analytical "stress block method".

Here, the stresses N and M acting in the contact face on the width h_{KF} are assumed as a block load with a constant stress level. The width d_0 of the stress block only covers a partial area of the contact face, which derives from the resulting total eccentricity e of the applied force. The width d1 of the load distribution area is calculated in the same way.

For symmetrical joint geometry (without offsets), the widths d_0 and d_1 are calculated as follows:

$$d_0 = 2 \cdot [h_{KF} / 2 - e] = h_{KF} - 2 \cdot e$$

$$d_1 = 2 \cdot [h / 2 - e] = h - 2 \cdot e = 2 \cdot e_{OF}$$

Verification with the stress block requires appropriate local plasticisation in the contact surfaces, which has been verified by experimental investigations and has been used as the basis for determining the maximum mean contact pressure occurring in the design case. The acceptable stresses resulting from the partial area loading in accordance with DIN EN 1992-1-1 and **ch. 5.7.2** therefore only apply to a stress block, but not to local stress peaks, such as those resulting e.g. from FEM analyses with modelling of the longitudinal joints using linear-elastic contact elements (**Figure 28**).

If the segments are modelled in the numerical analyses with continuum elements stresses, but not internal forces, are determined as output for the longitudinal joints. In order to apply the stress block method the resulting stresses in the joint must then be integrated over the contact area and the resulting normal force N and the bending moment M must be determined from this. The stress block can then be back calculated and the verifications of the partial area loading can be carried out in accordance with **ch. 5.7.2** (see **Figure 28**).

h	Segment thickness in radial direction
L	Segment length in the longitudinal direction of the tunnel, ring width
h_{KF}	Height of the contact surface, hinge joint width
d_0	Height of the analytical load transfer surface (radial) or load application width or load width
d_1	Height of the analytical load distribution surface (radial)
l_0	Length of the analytical load transfer surface
l_1	Length of the analytical load distribution surface (longitudinal)
$A_{c0} = d_0 \cdot l_0$	Analytical load transfer surface on the contact surface
$A_{c1} = d_1 \cdot l_1$	Analytical load distribution surface (after load distribution)
t_1	Depth of the analytical load distribution figure (in the design check for partial area loading according to **ch. 5.7.2**)
σ_{c0}	Pressure acting on the joint (as rectangular equivalent stress block)
σ_{c1}	Rectangular equivalent stress from load distribution
e	Total eccentricity/eccentricity of the load
e_{OF}	Distance of the normal force from the nearest segment surface, edge distance of the resultant load ($e_{OF} = h/2 - e$)

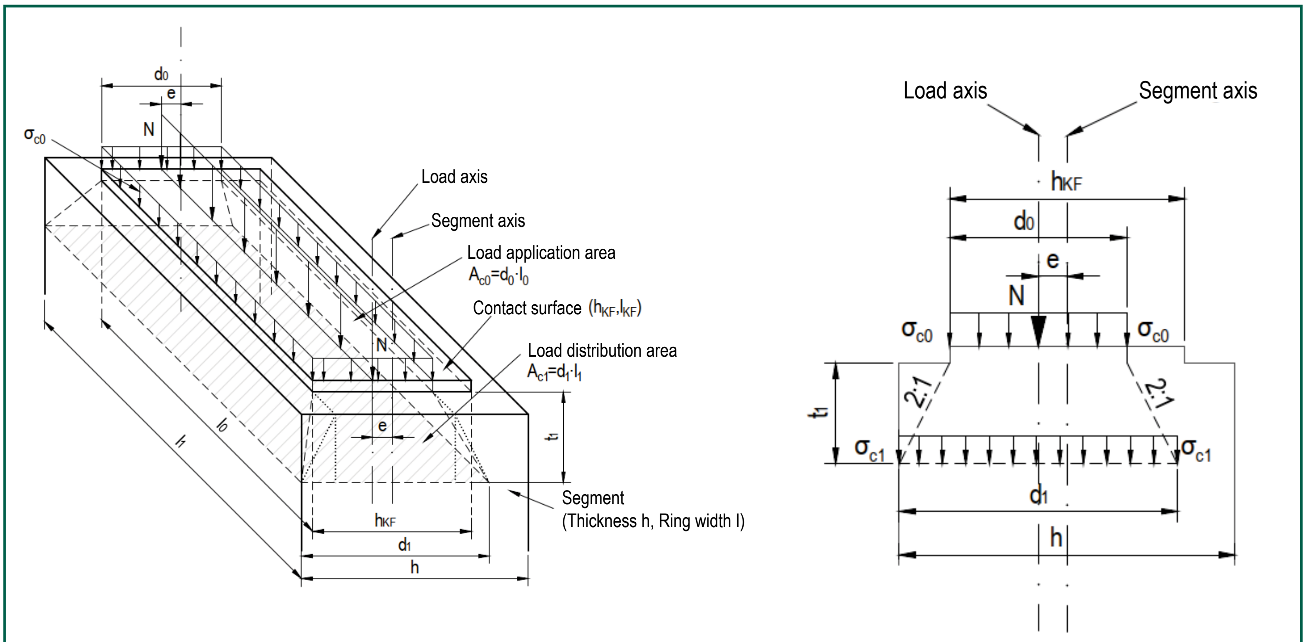

__Figure 27__ Schematic diagram of the load distribution across the segment thickness resulting from partial area loading with definition of the analytical distribution areas

Additional eccentricities from production and assembly

In addition to the load eccentricity e_M, additional eccentricities e_{vs} from production and assembly tolerances must generally be taken into account in all individual verifications of segment joints:

$$e = e_M + e_{vs} = M/N + e_{vs}$$

In accordance with RiL 853.4005 **[1]**, the additional eccentricities can generally be assumed to be e_{vs} = 10 mm, unless smaller tolerances can be justified by quality assurance measures on the jobsite or centring devices in the joints.

5.7.2 Verification of the partial area loading in the longitudinal joint

The analytical verification concept for partial area loading is based on the "stress block method", see also **ch. 5.7.1**, and follows DIN EN 1992-1-1 **[7]** section. 6.7 "Partial area loading". Accordingly, the acceptable partial area laoding in the longitudinal joint is determined as follows:

$$\sigma_{c0,Rd} = f_{cd} \cdot \sqrt{A_{c1} / A_{c0}} \leq 3 \cdot f_{cd}$$

with:

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$$

A large number of segment-specific tests have already been carried out for longitudinal segment joints with partial area loading. Examples include the tests for the 4th tube of the Elbe Tunnel in Hamburg **[33]**, for various DB AG railroad tunnels ("Bossler Tunnel", "Filder Tunnel") and in research projects (see **[41]**, **[43]**). All tests show high stress resistance capacities that, with suitable joint reinforcement and larger related eccentricities e/h_{kf} are in some cases significantly higher than the previous normatively permissible load-bearing capacities. For this reason, it is recommended that the following adjustments be made for the partial area loading on segment joints that are designed according to DIN EN 1992-1-1, together with meeting the related conditions:

- For the depth t_1 of the load distribution in the direction of loading, the conditions of **Figure 27** apply with a load distribution with an inclination 2:1, which is based on Figure 6.29 of DIN EN 1992-1-1.
- The limitation of the load distribution width d_1 to three times the load transfer width d_0 ($d_1 \leq 3 \cdot d_0$) is not applicable for longitudinal segment joints, provided the concrete strength class is not higher than C50/60. For strengths higher than C50/60, however, at least the absorbable partial area loading may be applied that results from the above regulations for a C50/60 concrete.
- The centre of gravity of surface A_{c1} must coincide with the centre of gravity of the load transfer surface A_{c0} in the direction of loading. The curvature of the segment may be neglected.
- The criterion of "geometric similarity" between the urfaces A_{c1} and A_{c0} does not apply to segments.
- If several compressive forces act on the concrete cross section, the calculated distribution areas must not overlap within the depth t_1. This does not apply to the usual recesses of guiding rods.
- The calculated loads must be almost constant over the length l_0, whereby recesses can be disregarded. If the progression in the longitudinal direction of the segment is not constant, the further details described in **ch. 5.7.8** must be taken into account.
- The transverse tensile forces (splitting tensile force) resulting from the load distribution due to the partial area loading must be covered by sufficient reinforcement in accordance with **ch. 5.7.3**.

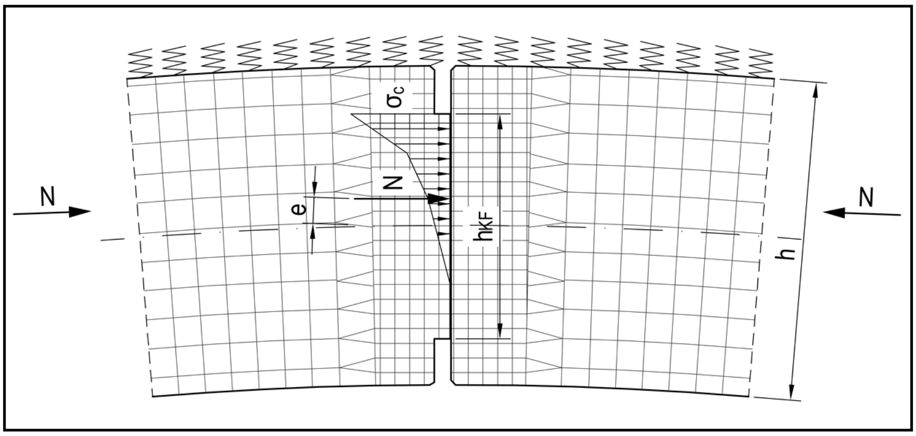

Figure 28
Exemplary representation of the joint stresses in an FE model with linear-elastic concrete behaviour

- In addition to the load eccentricity e_M = N/M, eccentricities from production and assembly tolerance (e_{vs}) must be taken into account in the analytical verifications.

- With large relative eccentricities e/h_{kf}, tests have shown that even partial area loads higher than $3 \cdot f_{cd}$ can be absorbed. In individual cases, it may therefore make sense to verify the increased load-bearing capacity by means of project-related tests as part of a test-based design.

5.7.3 Verification of splitting tensile reinforcement in the longitudinal joint

The splitting tensile stress in the longitudinal joints must be verified analytically. To this end, the splitting tensile reinforcement is determined for the most unfavourable combination of normal force and associated bending moment, or the eccentricity e_M = M/N, and with additional consideration of production and assembly tolerances.

For an eccentrically applied load with e > h/6, tensile forces F_{z0} and secondary splitting tensile forces F_{z2} occur in addition to the splitting tensile forces F_{z1}, see **Figure 29** and DAfStB, Issue 631 **[18]**. Edge tensile forces and secondary splitting tensile forces are generally subordinate for segment design. If required, the edge tensile forces can be verified against the permissible concrete tensile strength. They are not dealt with further here.

The calculation of the forces and the verification of the reinforcement are carried out by applying the force transfer using the stress block method based on DAfStB Issue 631 **[18]** and DIN EN 1992-1-1-1, section 6.5.3.

The following applies to the resulting forces of the splitting tensile stresses:

$$F_{z1} = 0.25 \cdot N \cdot (1 - d_0 / d_1)$$

with:

$$d_1 = 2 \cdot (h/2 - e) = 2 \cdot e_{OF} \leq h \text{ (Figure 29)}$$

The required splitting tensile reinforcement req. $A_{S,Z1}$ is to be determined as follows for the verification in the ultimate limit state (ULS):

$$\text{req. } A_{S,Z1} \text{ [cm}^2\text{]} = F_{z1,d} / (f_{yk} / \gamma_S)$$

For the splitting tensile reinforcement no crack width verifications need to be performed in the serviceability limit state (SLS).

The following is recommended for the structural design of the splitting tensile reinforcement:

- The position of the splitting tensile reinforcement should correspond to the distribution of the splitting tensile stresses or the position of the resulting forces according to **Figure 29**; details can be found in DAfStB Issue 631 **[18]**, section 5.2.1.

- The joint surface reinforcement has the function of a secondary edge reinforcement.

➲ ***Note:** The analytical depth of the load distribution figure is determined differently in the verifications of the splitting tensile stress in accordance with **ch. 5.7.3** (“t_s”) and in the verifications of the partial area loading in accordance with **ch. 5.7.2** (“t_1”). It is designated differently here (“t_s” or “t_1”) to avoid confusion.*

- The splitting tensile reinforcement should be anchored outside the load application area. If the anchoring is carried out using welded cross bars, the anchoring length must be determined in accordance with DIN EN 1992-1-1, section 8.6. The regulations of NA, NDP to 8.6(2) and NCI to 8.6.(5) (F_{btd} = 0) are not applicable.

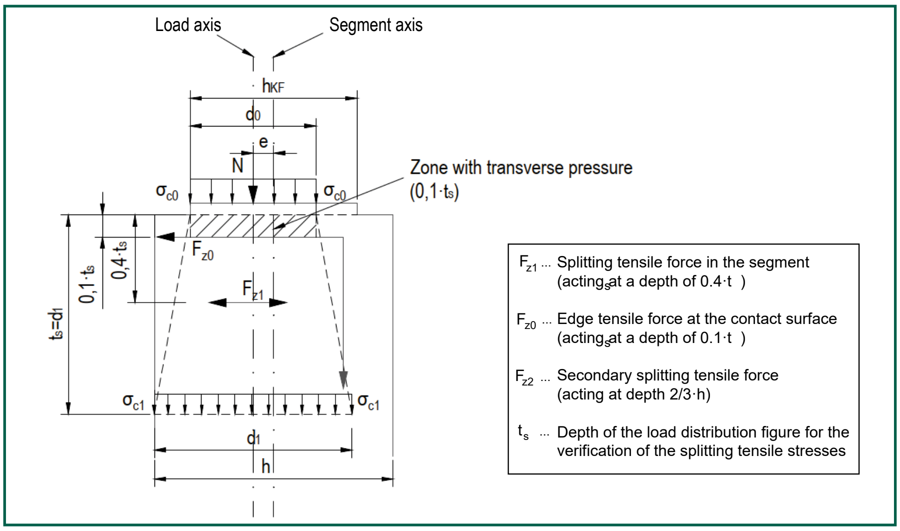

Figure 29
*Designations of the splitting tensile stresses for partial area loading according to the stress block method. For designations not mentioned here, see **Fig. 28***

- If shear factors according to EN ISO 17660-1 are specified in the design for the anchoring of welded reinforcement elements, these must be verified in the course of QA-inspections of the reinforcement production. Shear factors for structural connections are usually between 0.5 and 0.7.
- The joint surface reinforcement is designed to protect the corners from major spalling. The detailing must be executed with particular care. It is recommended that the minimum permissible concrete cover in accordance with **ch. 5.9.1** is utilised. From design point of view, it may be advisable to specify the tolerance allowances not only for a shortfall, but also for certain overruns of the nominal concrete cover dimensions.
- The joint surface reinforcement is usually located in the zone of the transverse compressive stresses that occur at edge distances $\leq 0.1 \cdot t_s$, see **Figure 29**. It should therefore not be considered in the calculation of the splitting tensile reinforcement. However, tests have shown the positive effect of the surface reinforcement on the introduction of surface pressures; in terms of a robust structural design, surface reinforcement should therefore always be provided.

5.7.4 Maximum hinge rotation in the longitudinal joint

In addition to the structural verification of the longitudinal joints, the compatibility of the calculated hinge rotation with the joint geometry and the design of the sealing gasket must be checked. Particular attention must be paid to the adequate sealing effect of the gasket (increase of the groove bottom distance when the joint is opened on the outside) and the absorption of the restoring force of the sealing profile by the concrete in the area of the groove (increase of compression and spalling effect when the joint is opened on the inside). The axis of hinge rotation can be assumed in a simplified manner according to Janssen **[34]**, section 4.2, unless more detailed investigations are carried out. For verification of the groove filling ratio, see also **ch. 4.6**.

5.7.5 Verification of shear force transfer in the longitudinal joint

For segment hinges, the ratio of shear force V to compressive force N can be evaluated using a friction model.

Based on DIN EN 1992-1-1, section 6.2.5 **[7]**, the coefficient of friction between concrete and concrete can regularly be assumed to be $\mu = 0.5$ (even for smooth joints). Separate considerations are required for the longitudinal joint of the key segment, see **ch. 5.7.6.** For the most unfavourable ratio of the characteristic values V and N in the longitudinal joint, the minimum global safety against sliding is thus obtained:

$$\min.\ \eta \geq (0.5 \cdot N_k) / V_k$$

In regard of the safety level a value of $\eta \geq 2$ is recommended for the BS-P and BS-T design situations and a value of $\eta \geq 1.5$ for the BS-A design situation.

5.7.6 Verification of the longitudinal joint on the key segment

The longitudinal joints of standard lining segments are formed radially. This ensures that the normal ring force always acts perpendicular to the longitudinal joints and that no downward components have to be considered. Therewith it is possible to design the longitudinal joints flat and without toothing or dowelling.

Due to the usual steeper layout of the longitudinal joints of the key segment as mentioned in **ch. 3.3.2**, however, additional downward forces arise in the key segment joints, which may need to be verified analytically by more detailed investigations. Guiding rods and bolts are generally not suitable for the transfer of downward forces.

It is known from practice that simplified design checks for the position stability of the key segment in ring building may lead to results on the uncertain side, as, for example, in the case of lubrication to facilitate installation (**ch. 11.2.2**), the application of the coefficient of friction $\mu = 0.5$ in accordance with **ch. 5.7.5** may be too favourable. Back analysis from projects with key segment displacements indicate that the coefficient of friction can reduce to around $\mu = 0.25$.

5.7.7 Verification of partial area loading in the ring joint

As a result of the jacking forces from TBM tunnelling, the ring joints, including the load-releasing side, must be verified. For the verification of the partial area loading and the determination of the splitting tensile stresses, an additional eccentricity e_{vs} of the ram shoe from construction deviations must be taken into account, see also **ch. 5.6.3**.

Verification of the partial area loading is carried out in accordance with DIN EN 1992-1-1/NA, para. 6.7. The curvature of the segments can be neglected and the “similarity criterion” does not need to be taken into account (see **ch. 5.7.2**). The recommendations for splitting tensile reinforcement according to **ch. 5.7.3** can be applied analogously. In practice, it is common to base the verifications on

- The “assumed operating jacking force for standard tunnel advance”, taking into account the normative partial safety factors on the material side, and

- the "maximum installed jacking force (or tolerated value)", without taking into account partial safety factors on the material side.

The load increases in the jacking forces (partial safety factors γE on the action side) are handled very differently in practice. Values in the range γ_E = 1.35 to γ_E = 1.0 (no increase) are common, depending, among other things, on how the jacking forces are applied. Due to the great influence on the reinforcement content of the segments, the possible risk of longitudinal cracking, the flexibility in the TBM drive and any contractual boundary conditions, no generally valid recommendation can be given. Rather, if possible, a project-related specification should be included in the tender specifications. In any case, the analytically verified jacking forces must be specified on the execution drawings and strictly adhered to during tunnelling.

5.7.8 Longitudinal joint stresses in the longitudinal direction of the tunnel

In the longitudinal direction of the tunnel, a constant stress curve can generally be assumed as an approximation. The longitudinal joint verifications are based on one linear metre (running metre) of tunnel or the ring width L.

In special areas, for example next to crosscut openings, usually a highly variable stress distribution in the longitudinal direction of the longitudinal joints occurs due to the load transfer around the opening (**Figure 30**). In practice, it has proven useful to average the joint pressures to form stress blocks over subsections in the longitudinal direction. The subsections can be defined from engineering judgement and the verifications can be performed in accordance with **ch. 5.7.2** and **ch. 5.7.3** for each individual section. The following recommendations apply to this procedure:

- Averaging should only be carried out over stress ranges of the same algebraic sign. Averaging over areas with joint gaps or analytical tensile stresses is not permitted.
- The maximum partial length for averaging should not exceed half the ring width, but no more than 1.0 m.
- The averaged stress value of a partial length should not be less than 50 % of the maximum value in this area.
- The normal forces N and the bending moments M should always be averaged over the same partial lengths.
- If the verifications are carried out for partial sections in the longitudinal direction, it must be ensured that the load distribution zones do not overlap over the depth t_1 in the longitudinal direction of the tunnel.

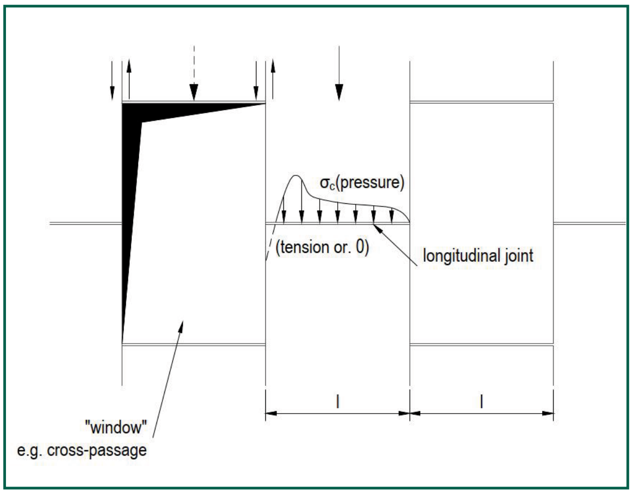

***Figure 30** Examples of longitudinal joint stresses next to a crosscut opening, for a stress distribution that is not constant in the longitudinal direction*

5.7.9 Numerical verification of partial area loads

Generally the partial area load and radial splitting tensile stresses should be verified using the analytical methods according to DIN EN 1992-1-1 or DAfStB, Issue 631 **[18]**. Additionally, or alternatively, these verifications can also be performed using numerical verifications. However, as these have not yet been widely tested and cannot yet be regarded as established, the admissibility and procedure for verification should be determined by the client as part of the tendering process. The procedure should also be agreed between the designer, the client and the checking engineer as part of a design manual with regard to the models, loads, acceptable resistances, partial safety factors, the simulated material behaviour and other boundary conditions before the verifications are carried out.

The numerical calculations can be used to perform the following verifications

- Partial area loading, especially in the case of high compressive stresses, locally or in total;
- Absorption of radial splitting tensile stresses inside the segment;
- Absorption of edge tensile stresses in the case of highly eccentric loads;
- Absorption of tensile stresses that occur outside of reinforced areas and therefore cannot be covered by reinforcement.

With regard to numerical modelling, no specific model has yet established itself or proven to be in particular suitable. The models for longitudinal joints shown in **Figure 31** are listed as examples and have individual advantages and disadvantages. Other suitable models can also be used.

The models shown in **Figure 31** have in common that the internal forces determined from the calculation on the overall model are applied to the longitudinal joint as external loads and can therefore be factored with partial safety factors. For non-linear analyses, it is recommended to apply all partial safety factors – i.e. the action and material coefficients or the system coefficient – to the external loads (γ_E) as a whole. All building material characteristics (streas a whole. All construction component characteristics (stressstrain relationships) can then be applied with their characteristic values as "actual values". This fulfils the requirement of DIN EN 1992-1-1, section 5.7 for the application of realistic stiffnesses. Depending on the objective of the verification, the loads can also be applied as characteristic values or as design loads.

In general, non-linear material models should be used, as with linear material models only limited conclusions are possible for the load bearing behaviour of the joints . Due to the multi-axial stress states in the joint area, the applied material model must be suitable for the selected numerical modelling with continuum or solid elements and the approach with characteristic or design values must correspond to the scope of work. This applies in particular to the approach for the concrete tensile strength. Tensile loads in the segment should in the ULS be assigned as far as possible to a respective splitting tensile reinforcement.

In the numerical models, the radial reinforcement and the circumferential reinforcement can be represented by tension members, e.g. with beam elements. For the bond behaviour between reinforcement and concrete, the bond strength f_{bd} can be used in accordance with DIN EN 1992-1-1, section 8.4.2.

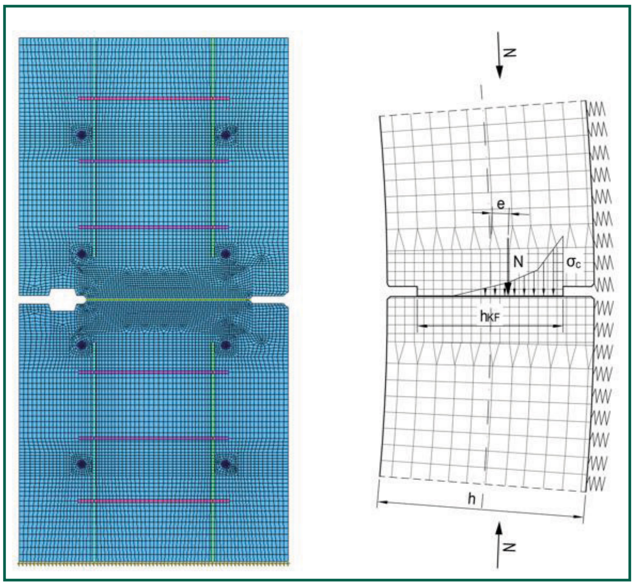

Figure 31 *Detailed numerical models for the verification of the close-up range of longitudinal joints: double-sided block model (left) and curved double-sided model with bedding (right)*

5.7.10 Verification of segment joint connections

In the assembly state during ring erection, temporary bolting in the longitudinal joints is applied to counteract the restoring forces of the sealing gasket until the surrounding ground, the annular gap backfill or form closure in the longitudinal direction prevent joint opening, see **ch. 3.3.4**.

The bolts or dowels in the joints are designed to ensure that the sealing gasket is not relaxed beyond the permissible release value respectively the acceptable joint gap. The stiffness of the bolt or dowel connection, the restoring force/deformation behaviour of the selected sealing gasket and the geometric boundary conditions (e.g. inclination of the bolted connections, opening angle of the longitudinal joints) must be taken into account. This results in the maximum calculated force in axial direction of the connection.

An overall increase factor of 2.0 is usually applied to this calculated force. The design of temporary bolts or dowels is then carried out considering the yield strength without any additional material-related safety factors. For a robust design, it is recommended to provide an additional material-related partial safety factor of 1.1 for steel bolts and 1.25 for plastic dowels. RiL 853.4005 (11) **[1]** provides for an increase of 150 % (factor 1.5) and a maximum analytical utilisation of 80 %.

If the load-bearing capacities of the connections are not specified by the manufacturer/supplier – usually as average values based on tests – they can be determined on the basis of the relevant regulations.

The safety coefficients for permanent bolted connections must be determined on a project-specific basis; for plastic dowels creep effects may need to be considered.

The use of permanent plastic dowels and permanent plug-in dowel connections (in the longitudinal direction of the tunnel) is not yet common practice in Germany.

5.7.11 Transfer of coupling forces in the ring joints

Coupling elements in the ring joint are usually deCoupling elements in the ring joint are usually designed as a dowel connection, occasionally also as a cam and pocket construction. The previously common tongue and groove constructions are no longer used today.

In terms of the load-bearing behaviour, a distinction must be made between:

- **Structural, centring coupling elements:** Such elements are used as structural elements to asist assembly or to secure the position in construction stages. Plug-in dowel connections in the ring joints often also serve to absorb the restoring forces of the sealing gasket. Cam and pocket connections only create a coupling in the radial direction. Coupling elements with a low to moderate load-bearing capacity and ultimate loads per coupling point of up to around 300 kN are used, e.g. cam and pocket connections or plastic dowels.

- **Load-bearing coupling elements:** These are elements that are structurally effective, in particular for load transfer of the ring compression forces to neighbouring rings (for example when opening crosscuts), or for control of deformations as a result of the increase in stiffness due to ring coupling.

  For the transmission of the coupling forces structural verifications must be provided. The load-bearing capacity per coupling element results from structural requirements and can be very high (up to over 1000 kN); depending on the load, mainly steel dowels (possibly plastic dowels with a steel core) are used.

To avoid constraints on installed coupling elements during ring erection, the coupling elements should be provided with sufficient clearance. The manufacturer of the coupling element usually provides respective recommendations. The clearance should be taken into account in the structural calculations accordingly. However, since the in-situ clearance is not know, since this depends on the local boundary conditions for installation, the structural calculations have an unavoidable “inaccuarcy” in this respect.

The coupling elements are usually simulated analytically as coupling springs with non-linear spring characteristics. Details on the modelling of coupling elements with slippage can be found in **[45]**.

If a clearance in the coupling connection is not permitted or should be limited for structural reasons (e.g. in the case of crosscut openings), special solutions are usually required to allow for higher static loads. A comparison of the effect of different coupling elements on crosscuts is carried out in **[57]**.

The absorption of the coupling forces in the ring joint must be verified for both, the installed coupling devices and for the segments. The approaches for this must be agreed early enough. Particularly in the case of larger coupling forces, the absorbable coupling force may have to be determined as a “testbased design” using component tests.

General approaches for the verification of the load transfer in segments are given in **[31]** and **[32]**. With regard to estimating the load-bearing capacity of dowel connections (especially with smaller dimensions), reference is made to the planned STUVA recommendation **[44]** and the analogy of shear bolts according to **[30]** and **[40]**. Dowels for shear force transfer are verified in accordance with **[44]** using punching shear or strut models. The load-bearing capacity of cam and pocket connections can be estimated according to **[42]**. The aforesaid literature and in **[44]** also information on a suitable reinforcement layout are provided. It is essential that the coupling forces transmitted in the radial direction are covered by suitable reinforcement (**Figure 32**).

The load-bearing capacity of coupling elements and detailed input values for their deformation behaviour (force-deformation curve) can be estimated from detailed analytical or numerical analyses or determined from manufacturers’ specifications or tests.

5.8 Post-hardening of concrete for tunnel segments

Measurements on completed lining segments have shown that considerable post-hardening occurs in the high-quality concrete types for segments compared to the nominal 28d strength.

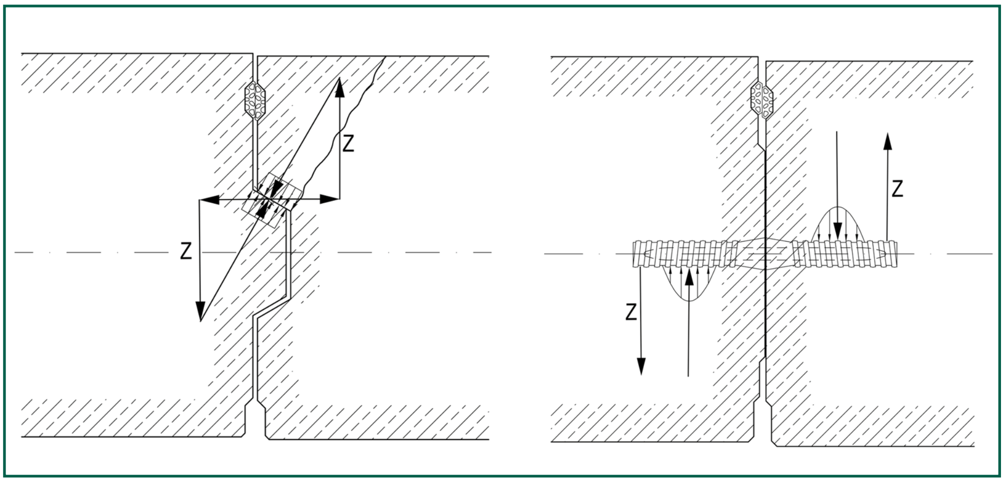

Figure 32
Schematic diagram of the load transfer of a cam and pocket connection (left) and a dowel connection (right)

➲ ***Note:*** *A distinction must be made between "post-hardening" and intended excess strength, for example to enable rapid demoulding. Excess strength is not the subject of this* ***ch. 5.8****.*

Post-hardening is already taken into account in the application of the fatigue strength coefficient $\alpha_{cc,EC2}$ of DIN EN 1992-1-1 as compensation for the drop in load-bearing capacity with longer-lasting loads (see fib Model Code MC-2010 **[19]**). In analogy to DIN EN 1992-1-1, section 3.1.2 (6) **[7]**, a analytical consideration of post-hardening is therefore generally not required.

The fatigue strength coefficient should also generally be taken into account for temporary loads, as segment tests have shown that long-term effects can influence the load-bearing capacity even with short-term load durations of just a few hours.

➲ ***Note:*** *DB-RiL 853.4005* ***[1]*** *contains information on the use of 90d strengths in late construction stages. These regulations are consistent with this* ***ch. 5.8****.*

5.9 Additional notes for the structural configuration

5.9.1 Concrete cover

On the surfaces a minimum concrete cover of c_{min} = 40 mm must be maintained, on the end faces and in local areas (for example recesses and built-in parts) the minimum concrete cover is $c_{min,red}$ = 20 mm.

The tolerance allowance of the concrete cover Δc is dependent on the production conditions and the quality control. As a guiding value, Δc = 5 mm can be assumed with appropriate quality assurance during segment production and in accordance with ZTV-ING Part 7, section 3-7.2.3.(2). Particular attention must be paid to the outside surface of the segments that lies on top during the production; the tolerance allowance for this may need to be increased.

It may be necessary to increase the concrete cover for reasons of durability (exposure classes in accordance with DIN EN 1992) or due to structural fire protection requirements. Because of the associated structural disadvantages (including reduced structurally effective height, poorer containment of the splitting tensile forces), increased concrete cover should be carefully evaluated by the designer and alternative measures examined.

5.9.2 Spacing of reinforcement

Typical reinforcement grids are in the range of between 100 mm and 150 mm. Larger bar spacings are possible, but have a negative effect on the calculated crack widths. After the reinforcing cage has been lifted into the mould, it must be possible to locally reach through the upper layers of steel to fix built-in parts inside the mould. Clear openings in the reinforcing cage of at least 90 mm to 120 mm must be provided for this purpose.

5.9.3 Minimum reinforcement

A grid reinforcement of Ø 10 mm, a = 150 mm is generally recommended as the minimum reinforcement for the segments. For road tunnels, ZTV-ING Part 7 **[2]**, section 3-7.2.3.(1) requires a minimum reinforcement of Ø 10 mm, a = 100 mm on all segment surfaces. Local adjustments on a small scale are possible in compliance with section 3-7.2.3.(3). RiL 853.4005 **[1]** requires a surface reinforcement of at least 0.15 % of the concrete cross-section per direction on the inside and outside for rail tunnels. For segment thicknesses of 35 cm, this corresponds to the ZTV-ING requirement mentioned in the first paragraph before.

The minimum reinforcement required in accordance with DIN EN 1992 is generally not applicable for segments due to the redistribution capability after initial crack formation and the low tensile stresses from restraint.

5.10 Verifications in the serviceability limit state (SLS)

5.10.1 Analytical service life time

These recommendations are based on the "analytical service life time" of 100 years which is common in Germany for tunnel segmental linings. Analytical service life analyses are not common practice in Germany. Such calculations require additional input values that are not determined as standard (e.g. chloride or carbonation resistance of the concrete). In many cases, service life analyses also lead to analytically increased concrete cover requirements, which can have a detrimental effect on the design (see **ch. 5.9.1**).

5.10.2 Calculated crack widths

Crack widths for durability reasons must be determined in accordance with DIN EN 1992-1-1/ NA, Table NA 7.1. In general, an analytical crack width for permanent loads of w_{cal} = 0.3 mm can be assumed for all exposure classes.

For reasons of water impermeability or other functional requirements, the client may specify more extensive crack width or impermeability requirements. For example, a calculated crack width of w_{cal} = 0.2 mm or 0.15 mm (outside) is required for railway and road tunnels located under water. For further details, please refer to ZTV-ING, Part 3, sections 3-8 **[2]**, RiL 853.4005 and RiL 853.2001 **[1]**.

Crack width verifications must be carried out in accordance with DIN EN 1992-2. The determination of internal forces in SLS load cases is described in **ch. 5.5**.

Even if the calculated crack widths are met, individual cracks with larger crack widths cannot be ruled out. Project-related procedures should be defined for dealing with such cracks.

5.11 Steel fibre reinforced concrete segments

5.11.1 General

To date, segments with reinforcement made exclusively of steel fibres or with "hybrid reinforcement", i.e. in combination with conventional reinforcement, have only been used in few cases in Germany; for example, for temporary segments (Wehrhahn Line Düsseldorf) or for double-shell construction methods (Mangfall Valley drinking water tunnel – Munich).

This situation is a result in particular of formal aspects, as the DAfStB Guideline for Steel Fibre Reinforced Concrete **[15]** introduced in Germany by the building authorities – referred to in the following as DAfStB-RiL SFB – considerably restricts the application of steel fibre reinforced concrete (SFRC) in tunnelling, among other things by limiting the application of SFRC to certain exposure classes only. National regulations that go beyond this also limit the use of steel fibre reinforced concrete in tunnelling. For example, according to DB Guideline RiL 853.4005 **[1]**, segments made of steel fibre reinforced concrete are defined as a non-regulated construction element in the DB AG' area odf responsibility and therefore require internal company approval (ICA) and project-related approval (PRA).

In international projects, on the other hand, steel fibre reinforced segments have often proven to be a robust and economical construction method under suitable boundary conditions, see for example **[49]** and **[50]**. These DAUB recommendations close the gap in the application guidelines in Germany for steel fibre reinforced concrete segments. The recommendations are essentially based on DAf-StB-RiL SFB **[15]**, but also take into account the special boundary conditions of the segmental lining construction method in tunnelling, the experience made with the predecessor code of practice, the DBV-Merkblatt SFB **[17]**, and the experience gained in international projects.

At European level, steel fibre reinforced concrete is to be adopted by the building authorities through the 2nd generation of Eurocode 2. According to the current status, this will regulate the design of steel fibre reinforced concrete based on post-cracking strength tests in accordance with EN 14651 **[12]** and an assessment of performance in accordance with the RILEM guideline **[20]** or fib Model Code 2010 **[19]**. However, details on the planned consistent use of steel fibre reinforced concrete for all design and construction phases, including quality assurance, are not yet available. The DAUB has therefore decided to refer exclusively to the DAfStB-RiL SFB **[15]** in these recommendations. It is noted that "mixing or combining" with other regulations is not allowed.

The technology and function of steel fibre reinforced concrete is assumed to be known and is not described in the context of these recommendations; please refer to the extensive relevant technical literature; such as, for example, the articles in the Betonkalender ("Concrete Calendar") 2011 **[46]** with further references.

5.11.2 Application of segments made of steel fibre reinforced concrete and hybrid segments

At the start of the project, the technical and contractual feasibility of applying steel fibres should be checked and, if necessary, the layout of the individual segments and the overall ring, for example the ring division, be adapted.

In particular, "extremely slim" segments are generally not suitable for the use of steel fibres. The following radian/thickness ratios b/h of the segment can be used as reference values:

- b/h < 10 usually suitable for steel fibres
- b/h = 10 – 14 conditionally suitable for steel fibres
- b/h > 14 rather unsuitable for steel fibres

In addition to the verification of the longitudinal joints in the final state, the verifications for the construction stages are relevant in many cases for the technical feasibility of steel fibre reinforced concrete segments. These should therefore be checked and verified with particular care, in particular

- Demoulding and turning,
- Storage and stacking,
- Transport to the TBM and the erector,
- Moving the segment with the erector,
- Annular gap backfilling and
- the introduction and transfer of the TBM jacking forces in the longitudinal direction of the tunnel.

If required, a hybrid reinforcement, consisting of a conventional steel bar reinforcement to cover the splitting tensile forces at the segment joints, combined with a pure steel fibre reinforcement in the segment's body to absorb normal force/bending stresses (N-M stresses) can be used.

The structural requirements for steel fibre reinforced concrete, denoted as "performance", are defined in accordance with DAfStB-RiL SFB, section R.3.6.1. In addition to the strength and exposure class, the performance classes L1/L2 required for the concrete must therefore be specified on the construction drawings. The performance classes L1/L2 are determined by the structural engineer as part of the structural verification of the segments. It is then the sole responsibility of the concrete supplier to select a suitable concrete mix design, the fibre type and the required minimum dosage of steel fibres.

Pure SFRC segments are usually produced with a fibre dosage of around 30 to 50 kg/m^3 and, for hybrid reinforcement, between 25 and 40 kg/ m^3. These are generally subcritical fibre contents, which means that the centric post-cracking tensile strength f^f_{ct0} of the steel fibre concrete is lower than the centric tensile strength f_{ctm} of the unreinforced concrete. Despite the subcritical fibre content, there is generally no loss of load-bearing capacity during initial crack formation under bending load, as stress redistribution can activate load-bearing reserves for the post-crack range. In recent years, with the development of new high-performance fibres, a tendency towards supercritical fibre contents has been observed **[50]**. This means that the application range of SFRC segments can be expanded in future.

The preliminary trials and any tests to verify the performance classes and to monitor the concrete must be carried out in accordance with DAfStB-RiL SFB, Part 2, Annex O on 150 x 150 x 700 mm beams in a four-point bending test. Accordingly, tests in accordance with DIN EN 14651 (CMOD tests) are not considered in Germany. The requirements for the test specimen dimensions according to DIN EN 12390-1 must be complied with.

Further information on the production of steel fibre reinforced concrete and SFRC segments can be found in **ch. 11.1.4** of these recommendations.

5.11.3 Structural verifications of segments with steel fibre reinforcement

➲ ***Note:*** *The recommendations in this chapter have been agreed with the DAfStB-Sub-Committee SFB.*

Basic information, regulations

The introduction of the DAfStB-RiL SFB **[15]** by the building authorities provides a consistent set of rules for the design of pure steel fibre reinforced concrete or steel fibre reinforced concrete in combination with reinforced concrete. The DAfStB-RiL SFB refers to DIN EN 1992-1-1.

The DAfStB-RiL SFB follows an approach determined by the material behaviour: the post-cracking material behaviour of steel fibre reinforced concrete is evaluated using small-scale experiments from which basic parameters of post-cracking tensile strength are derived. From these basic parameters the analytical and design parameters for defined stress-strain curves are determined. These stress-strain curves are incorporated into the standard design of reinforced concrete structures according to DIN EN 1992-1-1; in order to improve load-bearing capacity and serviceability.

As a result of his structural calculations the design engineer determines the required construction material properties of the steel fibre reinforced concrete by specifying the concrete strength class and the performance classes L1 and L2, where applicable in combination with conventional reinforcement for hybrid segments.

The DAfStB-RiL SFB is primarily focusing to the needs of the standard building and industrial construction sector. Below, the DAfStB-RiL SFB is detailed and supplemented for the special requirements of precast segments, taking into account the highly structurally indeterminate systems in tunnel construction. The recommendations refer in particular to the currently common steel fibre reinforced concrete segments with a subcritical fibre content.

A major reason for the widespread use of pure steel fibre reinforced segments abroad is also the acceptance of a test-based design in conjunction with high demands on the performance class of the steel fibre reinforced concrete. The possibility of a "test-based design" is therefore explicitly mentioned as an alternative verification method for segments.

5.11.3.1 Supplementary recommendations for steel fibre reinforced concrete segments

For the analysis of segments with structurally effective steel fibre reinforced concrete (SFRC) or a hybrid reinforcement made of steel fibres in combination with conventional reinforcement, the application of the approved guidelines DAfStb-RiL SFB **[15]** is recommended as a matter of principle. The following specifications and amendments must, however, be taken into account.

The following alternative test-based verification options refer to a "test-based design" in accordance with EN 1990 **[3]**, **[4]**, Annex D, usually within the scope of a project-related approval (PRA) or according to separate requirements of the supervisory authorities.

- **For DAfStB-RiL SFB R.1.1.2, section (1)P – Area of application (Requirements):** The stated requirements for the use of structurally effective steel fibre reinforced concrete, i.e. the possibility of internal or external redistribution, can generally be regarded as fulfilled under normal boundary conditions in tunnelling. In exceptional cases, in which an external, redistribution possibility on the load-side is not given or only given to a insignificant extent (for example in the case of predominantly water pressure loading with a low bedding reaction of the ground), the uncritical impact of deformations, plasticisation or cracking associated with redistributions in the ultimate limit state (ULS) must be verified separately.

- **For DAfStB-RiL SFB, R1.1.2, section (4)P – Area of application (Exposure classes):** In the exposure classes XS2, XD2, XS3, XD3, the analytical effect of the steel fibres may only be taken into account in the verifications of the ULS if it is verified for the SLS that no limitation of the service life time from excessive cracking (w_{max} = 0.2 mm) is to be expected for the lining. These verifications can be carried out in accordance with the recommendations below "For DAfStB-RiL SFB, R.7.3.1 to R.7.3.4 – Verifications in the SLS – Limitation of crack widths".
- **For DAfStB-RiL SFB, R.1.1.2, section (4)P – Area of application (Compressive strength classes):** Steel fibre reinforced concrete may only be used for high-strength concrete of compressive strength classes of C55/67 and more with project-specific approval.
- **For DAfStB-RiL SFB, R.3.6.3, section (5)P – Strengths (Factor κ^f_G of the component size):** For all verifications in longitudinal tunnel sections (e.g. N-M stresses, splitting tensile verifications in the longitudinal joint, but not for splitting tensile verifications in the ring joint), the ring width "l" can be assumed as the reference length in the longitudinal direction of the tunnel to determine the cracked cross-section area A^f_{ct}. To avoid an iteration to determine the factor κ^f_G, an approximation for the tensile zone height of $0.75 \cdot h$ (h = segment thickness) is recommended as an empirical value of the DAUB for N-M stresses. This value must be checked after the design and adjusted if necessary.
- **For DAfStB-RiL SFB, R.3.6.3, section (5)P – Strengths (Factor of the fibre orientation):** For bending verifications (N-M stresses), segments are to be regarded as flat, horizontally manufactured planar components within the meaning of DAfStB-RiL SFB. For the splitting tensile verifications in the ring and longitudinal joints, $\kappa^f_F = 0.50$ is to be assumed. As an alternative to determining the κ^f_F values, "test-based verification" can be carried out in accordance with the preliminary remarks.
- **For DAfStB-RiL SFB, R.3.6.3, Table R.3 – Performance classes L1 and L2 (Footnote 2):** The conditions for precast segments are generally more favourable than for building construction:
  - Production in the precast plant with continuous monitoring,
  - Use of high performance fibres,
  - High standard of quality monitoring, e.g. high number of tests,
  - Higher concrete strengths.

  It is therefore justified that the requirement of the DAfStB-RiL SFB in accordance with Footnote 2 (additional GTA or PRA) can be waived for the application of performance classes 2.7 and 3.0. These performance classes must always be verified by tests in accordance with the DAfStB-RiL, Annex O; in deviation from the general regulation, at least n = 12 tests must be carried out.
- **For DAfStB-RiL SFB, R.3.6.3, Table R.3 – Performance classes L1 and L2:** The use of intermediate values of the performance classes specified in Table R.3 in steps of 0.1 N/mm² is permissible. The tabulated basic values must then be interpolated accordingly.
- **For DAfStB-RiL SFB, R.6.5.1, section (R.2) P – Strut-and-tie models (General):** Verifications using strut-and-tie models are only carried out for segmental lining design only in few special cases, for example when a load is applied through concrete brackets. As a rule, it must then be verified that the tensile stresses occurring in the tension struts of the model are less than $f^f_{ctd,u}$. in the ultimate limit state.

➲ ***Note:*** *The splitting tensile stress under partial area loading in accordance with R.6.7 is not to be understood as a strut-and-tie model within the meaning of the above paragraph.*

- **For DAfStB-RiL SFB, R.6.7, section (4) – Partial area loading:** Section R.6.7, (4) is supplemented as follows:
  a) If the splitting tensile reinforcement of the longitudinal joints consists exclusively of steel fibres, the elimination of the limitation of load distribution $d_1 \leq 3 \cdot d_o$ in accordance with **ch. 5.7.2** may not be applied for partial area loading.
  b) According to R.6.7, Figure R.5, the idealised splitting tensile stress for partial area loading is a stress block with $\sigma_{Sz,d} = F_{z1,d} / 0{,}8 \cdot t_s$. Here, $F_{z1,d}$ represents the design value of the resulting splitting tensile force F_{z1} and t_s the depth of the load distribution figure in accordance with **ch. 5.7.3**.

  As an alternative to the verification method b), verification of the splitting tensile stresses for partial area loading may be carried out using one of the following methods c) to f):

c) According to the method by Mark-Schnütgen **[35]**, the two transverse strains in the load application zone must be limited to a maximum of 0.1 ‰/γ_c. Analytically, a uniform, linear-elastic material behaviour and a Poisson's ratio μ = 0.2 must be assumed for the steel fibre reinforced concrete.

d) It must be verified that the design values of the maximum main tensile stresses occurring in the load application zone can be absorbed. This can be done by means of a linear-elastic calculation in which the maximum values of the splitting tensile stresses in the ULS are covered by the tensile strength of the unreinforced concrete f_{ctd} without consideration of the steel fibres. In accordance with DIN EN 1992-1-1, section 3.1.6, the design value of the concrete tensile strength is to be defined as $f_{ctd} = \alpha_{ct} \cdot f_{ctk;0,05} / \gamma_{c,uB}$. Here, α_{ct} = 0.70 should be used and the partial safety factor for unreinforced concrete should be set to $\gamma_{c,uB}$ = 1.8.

e) The splitting tensile stresses in the ULS can be verified using non-linear numerical models according to **ch. 5.7.9**. The verification must then be carried out in line with DAfStb-RiL SFB, section R.6.7 applying DAfStb-RiL, section R.3.6.5 (stress-strain curves of steel fibre reinforced concrete). For simplification, $f^f_{ctd,u}$ may be used. In the numerical model, the struts must not be steeper than 2:1 (for an explanation of the angle, see also **Figure 27** in **ch. 5.7.1**).

f) The verification of the partial area loading at the segment joints, including the verification of the splitting tensile stresses, can be carried out by means of a test-based design in accordance with the preliminary remarks.

In all other cases, conventional reinforcement in accordance with **ch. 5.6.1** must be provided (so-called "hybrid segments") to cover the splitting tensile stresses at the segment joints.

- **For DAfStB-RiL SFB, R.7.3.1 to R.7.3.4 – Verifications in the SLS – Verifications of crack width limitation:** For pure steel fibre reinforced components with subcritical fibre content, calculations do not show multiple cracking. The verification concepts for crack width limitation in the SLS mentioned in DAfStB-RiL SFB, sections R.7.3.1 to R.7.3.4 are therefore not suitable for standard steel fibre reinforced concrete segments and cannot be applied.

  Due to the small dimensions of the segments and as a result of prestressing by external loads and possibly by the thrust cylinders, no tensile stresses can generally build up from restraint actions. Crack width verification in the SLS from restraint stresses therefore does not generally need to be carried out for segments. However, practical experience shows that with pure steel fibre reinforced segments, the propagation of cracks that have occurred at the segment's edge further into the centre of the segment cannot be fully ruled out. For further discussion of this aspect, please refer to the comments in **[38]**.

  In the ring direction, it may be necessary to limit the crack width in the SLS under the N-M stresses arising from external loads, for example to verify durability in accordance with section R.1.1.2 (area of application with regard to exposure classes) or water impermeability in the case of waterproof concrete requirements. There are currently no suitable verification methods available for pure steel fibre reinforced components with a subcritical steel fibre content. It is therefore recommended that, if necessary, one of the following alternative verification methods be used to verify the crack width limitation in the SLS:

a) If the zero stress line of the N-M stress in the SLS is mathematically located in the cross section, the calculated crack width w_k may be simplified in accordance with DAfStB-RiL SFB, Eq. (R.7.11) as $w_k = s^f_w \cdot \varepsilon^f_{ct}$. The tensile zone height (h-x) should be used for s^f_w and the edge tensile strain of the steel fibre reinforced concrete from the analytical verifications of the SLS for ε^f_{ct}.

   ➲ ***Note:*** *The value s^f_w is not to be understood as a crack spacing for determining crack widths from strains determined in numerical calculations (FEM).*

b) Compliance with the crack width requirements must be verified using suitable numerical, non-linear calculation models that include the mathematical determination of crack widths.

c) It must be verified that under the SLS loads the lower fractile value of the concrete tensile strength $f_{ctk,0.05}$ of the unreinforced concrete according to DIN EN 1992-1-1, Table 3.1, is not exceeded.

   ➲ ***Note:*** *It is common practice abroad to perform this verification against the mean concrete tensile strength f_{ctm}. This verification method does not constitute a reliable crack width limitation, but may be suitable depending on the respective project requirements.*

d) The crack width verifications can be carried out as a "test-based design" in accordance with the preliminary remarks.

- **For DAfStB-RiL SFB, R.9.2.1.1 – Minimum reinforcement for ductile behaviour:** Tunnels with segmental lining are considered to be systems capable of redistribution, which develop an increased inherent load-bearing capacity after initial cracking. Therefore, no minimum reinforcement for ductile load-bearing behaviour is required for tunnel segments.

5.11.3.2 Determining properties of the concrete in tests

The classification of steel fibre reinforced concrete in the performance classes L1 and L2 is based on the results of a 4-point bending test of beam-shaped specimens according to DAfStB-RiL SFB, Part 2, Annex O – Test to determine performance classes.

To determine the post-cracking tensile strength of steel fibre reinforced concrete, so-called CMOD tests in accordance with EN 14651 **[12]** and RILEM TC 162 **[20]** are now standard practice internationally. In addition, extensive test results are available, both nationally and internationally, from tests in accordance with the DBV Code of Practice for Steel Fibre Reinforced Concrete **[17]**. However, due to different test and evaluation conditions, the calculated values obtained for the post-cracking strength are not comparable with the performance classes of the DAfStB-RiL SFB. Consequently, the DAfStB Guideline SFB does not allow "mixing or combining of regulations" in section R.3.6.3.

➲ ***Note:*** *The DAfStB-RiL SFB is currently being revised and will probably take three-point bending tests into account in the next revision.*

If, in exceptional cases, the post-cracking tensile strengths of steel fibre reinforced concrete cannot be determined using the DAfStB-RiL SFB tests – for example, because the testing laboratories are not equipped for this – special considerations regarding applicability and verification methods are required; separate calibration tests may then be useful. In this case, the involvement of an expert assessor is recommended. Information on converting the post-cracking tensile strengths of 3-point and 4-point bending tests is given in **[48]**.

5.11.3.3 Production Control

For steel fibre reinforced concrete segments, the regulations for production control in accordance with DAfStB-RiL SFB **[15]**, Part 2, Annex Q and Part 3 of DIN 1045-3, Annex NB apply.

If the acceptance criteria for the result of the post-cracking bending tensile test in accordance with Table NB.4 are not met, the relevant segments must be rejected and not installed. Alternatively, further comparative tests of the post-cracking bending tensile strength on additional retention samples or specimens obtained from drill cores can be used before the segments are installed. Alternatively , analytical verifications for the performance class reduced in accordance with the test results may need to be carried out.

6 Structural fire protection

6.1 Introduction

The requirement, type and extent of structural fire protection for a tunnel lining are always to be considered and defined accordingly in association with the operational protection measures (e.g. traffic management and control, possibilities to drive into safety/evacuation zones, escape routes design, fire detection, smoke removal, ventilation, cooperation with emergency services), and the risk to the tunnel structure in the event of fire in accordance with the relevant regulations.

Following the relevant regulations DIN EN 1992-1-2 **[8]**, DIN EN 1363 **[21]**, RiL 853 **[1]** and ZTV-ING **[2]**, ensuring adequate stability and, if necessary, also serviceability (watertightness, limitation of permanent deformations) during and after a fire are named as the key protection goals of structural fire protection in tunnels. If necessary, sufficient load-bearing capacity for the status of a subsequent refurbishment must also be taken into account.

6.2 Actions

Measurements from fire tests in tunnels and the evaluation of real tunnel fires show that tunnel fires differ from fires in buildings both by the level of the reached maximum temperatures and on the other hand by the extremely rapid temperature rise at the beginning of the full fire.

For the description of the thermal actions, usually temperature-time curves are defined for the fumes mixture. **Figure 33** shows the temperature-time fire exposure curves of relevant regulations.

The ZTV-ING curve (1) is usually applied for German road tunnels. For German railway tunnels, the EBA curve (5) is generally used, which corresponds to the internationally more common EUREKA curve.

The RWS curve (2) from the Dutch regulations used for both road and rail tunnels.

Curves that are open to the rear side (e.g. RWS) must be defined for the analytical consideration of the cooling phase.

This must be specified on a project-specific basis. In the absence of specifications, the cooling should not be considered faster than described by curves (1) and (5).

6.3 Possibilities for structural fire protection

Concrete spalls when heated to high temperatures; the reinforcement and also the concrete lose their strength and stiffness due to excessive heating. According to DIN EN 1992-1-2 **[8]**, Table 3.2a, a reduction in strength occurs in cold-formed structural steels from a temperature of 300 °C and in hot-rolled structural steels from a temperature of 400 °C. If an increase in temperature in the regular reinforcement layer above these values can be ruled out, sufficient load-bearing capacity can generally be assumed even after the fire. If the steel reaches higher temperatures, the reduced properties of the steel may need to be considered. Fire tests have shown that the segments can retain sufficient load-bearing capacity after the fire even at higher temperatures in the reinforcing steel.

The following measures are used for structural fire protection:

- Cladding with fire protection panels or fire protection mortar, which have a heat-insulating effect, can prevent the reinforcement layers from heating up intolerably or the concrete from being seriously damaged.
- By using concretes with high fire resistance and correspondingly low spalling, the remaining concrete cover has a heat-insulating effect on the reinforcement and the surrounding concrete. Increased fire resistance of the concrete can be achieved, for example, with an optimised concrete mix with the addition of PP fibres.

6.4 Fire protective claddings and fire tests

6.4.1 Fire protection panels

For fire protection panels the mounting is part of the protective system, as it must not lose its functionality during the fire to prevent the panels falling off. Fire tests are necessary to verify the heat-insulating effect of the cladding and the load bearing capacity of the mounting. Special cladding panels must be provided in the vicinity of tunnel installations.

The service lifetime of cladding systems is limited, which is why the panels usually have to be replaced within the service life of the tunnel. Studies in **[55]** suggest that this may even have to be done multiple times. Particular attention should be paid to the attachment of the fire protection panels in regard of corrosion resistance. Since fire protection panels hide the concrete surfaces underneath from inspection or structural examination, their application can certainly be considered critically.

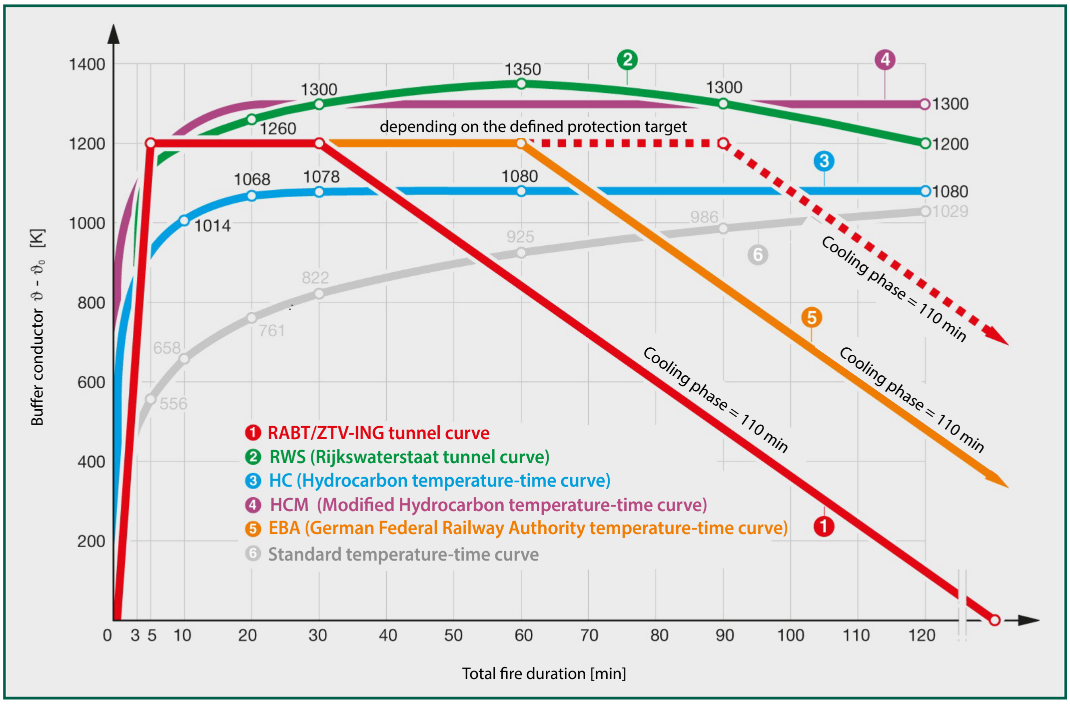

Figure 33
*Temperature-time curves of relevant regulations **[54]***

6.4.2 Fire protection mortar

Because of the fibre content, fire protection mortar is relatively soft and elastic and therefore have a crack-bridging effect. Harmful cracks in the load-bearing concrete are therefore not transferred into the mortar and consequently cannot be detected during the mandatory regular inspections of the structure. As mounting elements for signage, tunnel guidance systems, etc. must penetrate the protective mortar layer, special designs are necessary for fire protection and for load transfer in these areas.

To ensure the protective effect of fire protection mortar, sufficient adhesion to the structure by means of sprayed application, adhesives or a mortar base layer must be ensured.

As for fire protection panels, the service life time of protection mortar is limited. This is one of the reasons fire why protection mortar has not yet become a standard application in practice and is primarily used as a special measure.

6.4.3 Concrete with high fire resistance

In recent years, polypropylene fibres (PP fibres) have been added to concrete to increase its fire resistance and thus provide adequate structural fire protection. This addition can significantly reduce spalling in the event of fire and thus maintain the insulating effect of the concrete cover. Monoor multifilament PP fibres with a round cross-section and a smooth surface should be used.

Fibre contents in the order of 1.5 to 2.0 kg/m^3 are common.

Current experience with optimised fibres has shown that lower fibre contents can also be used with the same fire protection effect. One way of demonstrating the equivalence of the lower fibre contents is to carry out comparative tests on concrete specimen with conventional PP fibres at a fibre content of 2 kg/m^3.

Verification of adequate fire resistance must be provided by large-scale fire tests on a representative section of the tunnel lining, by analytical stability verifications taking into account the spalling that is expected in the event of fire – usually determined from fire tests with the selected concrete mix design and addition of PP-fibres – or a combination of both options.

If structural fire protection can be achieved by means of the load-bearing structure, there are no restrictions with regard to inspections of the structure and the detection of cracks. Structural fire protection is effective for an unlimited period of time. Structural fire protection with PP fibres therefore currently represents the standard case.

6.4.4 Fire tests

To ensure adequate structural fire protection and to determine the in-situ spalling depths under real conditions, preliminary fire tests can be carried out on representative sections of the tunnel lining.

As a rule, segments with the same dimensions and concrete composition as in the final tunnel must be used as test specimens. In addition, the predicted loading level in the segment is approximately applied by means of horizontal and radial or vertical jacks. In a fire chamber the specimens are exposed to the required temperature-time curve (see **ch. 6.2**). With smaller test specimens and test specimens without loading, significantly less spalling than to be expected in reality can occur in the tests.

Otherwise the tests are carried out in accordance with DIN EN 1363 **[21]**. Parts 1 and 2 and according to Schuck/Städing **[53]**.

If the load-bearing capacity is maintained in the fullscale tests, this can generally be regarded as a criterion for adequate structural fire protection. For a combined verification of tests and analysis, the spalling depth that is determined in the fire-test can be adopted as input value for the analytical verification.

It is recommended to carry out tests in advance, at least as small-scale fire tests, to check the suitability of the concrete mix design and to determine an indicative actual spalling depth. The above instructions must be considered.

Project-specific specifications are recommended. The necessary lead times for segment design on the one hand and the time required to prepare, carry out and evaluate the fire tests on the other hand must also be taken into account.

6.4.5 Joints and watertightness

Generally, it can be assumed that the hot gases generated during a fire will, due to the limited gap width, in the joints not lead to critical temperatures for sealing gaskets on the outer side **[47]**, **[51]**. If, on the other hand, major spalling should occur in the joint area, the watertightness of the structure could still be jeopardised. In real fires, e.g. in the Channel-, Great Belt- and Follo Line-tunnels, it has been shown that severe spalling occurred more frequently in the centre of the segment, whereas at the segments edges, usually webs remained or the visible damage only occasionally extended up to the segment joints **[52]**, **[51]**. It can therefore be assumed that the sealing gaskets in the segment joints are also adequately protected if the spalling depth is limited.

6.5 Analytical studies

The segments can be analytically verified using the general verification method in accordance with DIN EN-1992-1-2 **[8]**. The following aspects must be taken into account.

A spalling depth that is determined, estimated or specified through fire tests must be taken into account in the thermal and structural analyses. Reference values for spalling depths are given in RiL 853 **[1]**. To be on the safe side and due to the fact that spalling usually occurs very early, it is assumed analytically that spalling occurs at time "zero" in the analyses, i.e. immediately at start of the fire.

The temperature distribution in the segment cross-section is determined with a prior thermal analysis for the relevant fire temperature curve or alternatively, if available, by evaluation of temperature measurements in fire tests. Due to their highly non-linear character, this temperature distribution must be evaluated for several points in time, which should also consider the decay period of the fire up to a point in time after the fire. The temperature-dependent parameters of DIN EN 1992-1-2 **[8]** can be used for the thermal analyses.

For the structural verification, the non-linear and temperature-dependent material behaviour according to DIN EN 1992-1-2 **[8]** can be applied. Different points in time during the fire, for the decay of the temperature load in the segment and for the condition after the fire are examined and in each case the previously determined temperature distribution in the cross-section is applied as a restraint load.

Due to the temperature-dependent stiffness and strength over time and the cross-section height, the loads vary greatly, which is why the analysis must be carried out with several consecutive calculation steps.

The restrained loads arise in the segmental lining rings on the one hand in circumferential direction, due to the restrained deformations caused by the tunnel bedding, and on the other hand, in the longitudinal direction of the tunnel due to the restrained expansion of the segments caused by the adjacent rings. To determine the constraints in the longitudinal direction, calculations using shell models are recommended for the analytical verification of a fire event.

7 Earthing measures in tunnels

Due to the complexity of the earthing measures, an adequately qualified and experienced electrical engineer should be involved at an early stage in the construction and design phase. The client need to specify earthing requirements in the project or tender specifications.

7.1 Rail tunnel

7.1.1 Current return circuit and earthing for rail tunnel

EN 50122 **[22]** can be seen as the normative basis for the rail sector.

Basic principle (normal operation): In the substation, the electric current is fed into the overhead contact line and drawn by the consumer through the current collector. The traction return current must then be returned to the feed-in source by means of a return conductor. This extended return conductor in accordance with EN 50122 **[22]** is part of the traction current circuit and should prevent the currents from flowing back through the ground as much as possible. Elements of the return conductor can be running rails, return conductor rails, return conductor wires, etc. Hereafter only conventional systems with overhead contact lines or conductor rails for supplying traction energy are discussed.

The return conductor is also characterised by the fact that the circuit breaker is tripped upon contact with an active part. This means that in the event of a fault, for example if the overhead contact line breaks, a short circuit will occur in order to switch off the overhead contact line due to the high current flow through the low-resistance connection.

Traction current return via the rails results in a potential difference between the rails and the surrounding area (usually "earth", i.e. zero potential), which can pose a risk to people, animals and other objects from an electric shock. To reduce the rail potential, rail earthing is used with connecting to as many tracks and return conductors as possible. Earthing is the electrical connection between conductive parts and a conductive medium ("earth electrode") that is in contact with the "earth" (zero potential).

At the same time, potential differences can occur in any operating state – for example as a result of induced voltage in metallic components – which can be intolerable for people or other components. This requires equipotential bonding (conductive connection) or earthing of the track.

Electrically conductive components in the overhead contact line area – i.e. the area whose limits are generally not exceeded by a broken contact wire or a broken catenary wire, see DIN 50122-1 **[22]** – and in the pantograph area – the area whose limits are generally not exceeded by a live current collector, even in the event of breakage or derailment – must be earthed.

In accordance with DIN 50122-1 **[22]**, partially conductive structures (reinforced concrete) must have protective measures in the form of a connection to the return conductor. Structures without electrically continuous reinforcement can, for example, have the following precautions:

- Electrical connection between the reinforcements of all parts of the structure and the return conductor
- IInstallation of conductors or meshes to connect all parts of the structure
- Connection of a few parts of the structure to the return conductor
- Installation of conductors between the parts of the structure that are connected to the return conductor. However, attention must be paid to the stray current problem for direct current (DC) railways in accordance with DIN 50122-2 **[22]**.

If there are conductive or partially conductive components in the overhead contact line area that cannot carry the short-circuit current for the duration of the protective reaction, buffer conductors (as a special form of return conductors) must be provided in accordance with RiL 997.0223 **[23]**.

7.1.2 AC railways

Alternating current (AC) railways are all 16.7 Hz/15 kV or 50 Hz/25 kV railways. In Germany, the first-mentioned system is common in long-distance transport and, with a few exceptions (Hamburg and Berlin S-Bahn rapid transit systems), in local public transport. Subways, e.g. the Berlin subway, are operated with a DC voltage of 750 V.

Earthing for AC railways is regulated in RiL 997.0223 **[23]**, for example. For single-shell sealed tunnels, usually tunnels with precast segmental lining,

- Earthing for equipotential bonding with the rock and
- Buffer conductors to allow short circuits in the event of contact between the tunnel structure and the broken overhead contact line

must be taken into account. In addition, a return conductor wire can be used for each catenary near the overhead contact line for partial compensation of the magnetic field of the overhead contact line and as an earthing bus for easy connection to parts of the overhead contact line to be earthed.

The conventional reinforcement in reinforced concrete structures must always be connected to a return conductor for equipotential bonding.

A distinction must be made between the following applications for buffer conductors:

- Buffer conductors at rail level to create short circuits in the event of contact wire breaks. These are used for equipotential bonding for escape and rescue routes and to protect installations in the track bench. If buffer conductors are cast in concrete in the track bench, a maximum concrete cover of 10 cm is permitted. Running rails are also included in this type of buffer conductor.
- Buffer conductor halfway up between the imaginary intersection of the overhead contact line area/side wall and the escape or rescue route level.

➲ *Note: If available, this can also be accomplished using a suitable handrail.*

The return conductors mentioned must be connected across block joints and cross-connected approx. every 250 m and also connected to the rails.

Particular features of segmental tunnels

Due to the large number of individual segments in segmental lining, connecting the entire reinforcement and therefore all individual segments for equipotential bonding has proven to be impractical during construction and maintenance. In addition, due to the direct contact of the segments with the ground via the annular gap material, the spreading resistance is significantly lower compared to sealing with plastic liners; therewith the earth sensitivity (quality of the earthing) is higher and thus a direct discharge takes place.

This also means that the installation of dispersing earth electrodes (earth electrodes with longer lengths in the ground), e.g. in the area of connecting structures, is often not necessary for segmental linings. However, this must be agreed with the designer of the overhead contact line before the start of the project, taking into account the boundary conditions of the individual case. Tests to determine the conductivity of the subsoil should also be carried out as part of the geotechnical investigation. Because it is not possible to have buffer conductors running in the concrete in the crown area, an internal return conductor wire is installed above each overhead contact line in a zigzag arrangement (± 1.5 to 2 m).

Due to the high voltages in existing installations, it is not possible to avoid buffer conductors in the track bench area despite the better discharge.

7.1.3 Earthing and equipotential bonding of other electrical energy systems

For AC railway tunnels, the entire earthing and equipotential bonding system must be designed in addition to the railway earthing; see for example RiL 954.0101 **[24]** and RiL 954.0107 (specifically RiL 954.0107A01) **[24]**. Further regulations on electrical installations for emergency management (e.g. tunnel safety lighting and electrants) are listed in particular in RiL 954.9107

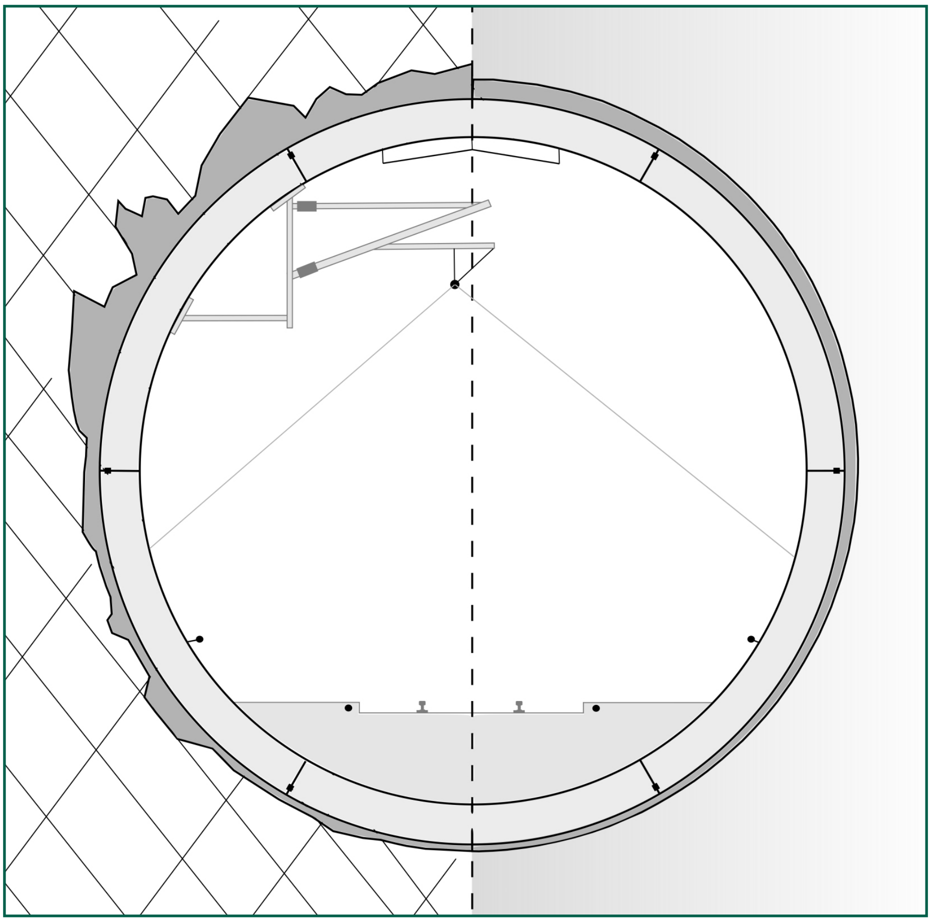

Figure 34
Buffer conductors in the rupture area of the overhead line (shoulder, side wall, track) and in the crown in the pantograph area in form of a rope

[24]. An example of a complete earthing concept can be found in the earthing manual **[25]**.

7.1.4 DC railways

DC railways (usually 600 V or 750 V) include most subways, tramways and, in special cases, rapid transit systems (e.g. in Hamburg and Berlin). These differ significantly from AC railways in terms of return current circuit and earthing.

Only the track or a busbar should be used as return current conductor for traction energy. The reason for this is that stray currents are more critical in the area of DC railways than with AC railways. These can lead to corrosion of metallic components. Regulations on this are contained in EN 50122-2 **[22]**. However, in all component protection considerations, protection against electric shock in accordance with EN 50122-1 **[22]** always takes precedence.

Due to the stray currents, the return conductor must be insulated from the rest of the tunnel. The required cross-connection of all conductors (equipotential bonding) is not permitted here in accordance with VDV 501-1, 2 **[26]** and DIN EN 50122-1, -2 **[22]**.

Conductive components in electrical contact with the return conductor must not have a conductive connection to components that are not earth-insulated. If a connection to the return conductor is un- avoidable to protect against electric shock, measures must be taken in accordance with EN 50122-2 **[22]**. These range from voltage limiting devices to insulat- ing the structure's reinforcement against earth.

DIN 50122-2 **[22]** contains calculation examples for estimating the effects of stray current corrosion, but due to the complexity of the measures, early involvement of an electrical planner is recommended.

7.2 Road tunnels

For road tunnels, EABT-80/100 **[27]** applies with regard to lightning and surge protection, whereby the equipotential bonding to be carried out within the tunnel tube is also described.

In tunnels with segmental linings, a continuous earth connection between the individual segments cannot generally be derived from these regulations and is usually not implemented.

Earthing for the connection of electrotechnical installations must be interconnected.

8 Durability

8.1 Requirements

The durability of the tunnel structure assumes its intended load bearing capacity and serviceability – taking appropriate maintenance into account – over the entire service life. Generally, a service life time of 100 years is specified by the client for the load-bearing system of segmental lining tunnels.

Inspection and maintenance options are generally only available to a limited extent. In particular, changes to the carrier's operating concept may change the requirements for the clearance profile and must be taken into account in the event of repairs. Based on life-cycle considerations, how ever, maintenance strategies are increasingly being developed in accordance with DIN 31051 in order to extend the service life of the structure. The so called "structurally usable space" of 30 cm, added to the radius, which is specified for railway tunnels, is a good contingency for later repairs. The more difficult areas for inspection and repair apply in particular to the surfaces that are in contact with the ground, to the sealing gaskets and to the joints that are no longer accessible after installation. The same applies in principle to surfaces that are covered by fire protection cladding, for example. The following requirements must always be met:

- Load bearing capacity of the ring and its individual components,
- Watertightness against water under pressure,
- Avoidance of spalling, which can limit serviceability (e.g. road safety), but can also impact the load bearing capacity,
- Avoidance of excessive degradations such as corrosion and ageing effects in the materials concrete, reinforcement steel (conventional or fibre reinforcement) or sealing and built-in components.

The following sections cover only segmental linings with reinforced concrete segments. With regard to the special characteristics of steel segments for installation, corrosion protection, transitions, etc., please refer to **ch. 10.2** and additional literature. The durability of the sealing gaskets is discussed in **ch. 4**. With regard to analytical verifications for durability, reference is also made to **ch. 5.10.1**.

8.2 Ageing mechanisms

Segments are exposed to ongoing ageing from processes attacking concrete and steel. The individual mechanisms occur mostly in combination and may enforce each other. They include:

- Carbonation of the outer surfaces that are in contact with the ground or the air side surfaces respectively, which causes the reinforcement to corrode,
- Sulphate attack due to sulphate-containing water or subsoil with dissolving or driving effect, which leads to sulphate crystallisation and sanding of the surface, cracking and spalling at the surface, cracks in the interior of the concrete structure and/or destruction of the concrete structure,
- Chloride attack from de-icing salts (for example, in portal vicinity), saline water environment, possibly the effects of fire causing corrosion of the reinforcing steel,
- Mechanically abrasive wear of the inner surfaces and edges, including from unplanned events (e.g. also from tunnel cleaning),
- Accidents with or without the development of fire, which can lead to partial or localised damage to the lining,
- Degradation from thermal stress due to climatic changes, such as frost-thaw cycles in the portal area,
- Contact corrosion of reinforcing steel with stainless steel components, which is for example possible with permanent longitudinal bolting, although usually not significant.

Unplanned cracking and spalling due to constraints or mechanical action have an unfavourable effect on ageing processes and should be avoided in favour of a closed concrete structure with low porosity. Such defects can occur at corners, edges, the ring joints toothing or at the longitudinal joints. Possible repair measures for spalling include, for example, surface protection systems and, in the case of cracks, impregnation and injections using epoxy resin, polyurethane, cement paste or cement suspension, depending on the crack width and type of loading.

With the usual concrete strengths for precast lining segments the depassivation front of carbonation (pH values of <9) usually remains within the standard concrete cover values over the life time.

On the other hand, in case of unfavourable conditions, the penetration depths of chloride-containing waters (with chloride ion concentration that is relevant to reinforcement corrosion) can be significantly higher than standard concrete cover values, which then needs to be considered in the lining design.

8.3 Recommendations to improve durability

The following boundary conditions are recommended to improve the durability for tunnel lining segments made of reinforced concrete or steel fibre reinforced concrete:

- A robust design that does not reach the limits of utilisation. This includes e.g.:
  - Adequate segment thickness,
  - Adequate width of the sealing gasket,
  - Adequate reinforcement content,
  - Use of a normal strength concrete at the upper limit of the strength spectrum. This must on the lower side provide adequate strength for the expected loading, but should at the same time have sufficient deformation or working capacity. High-strength concrete mixtures are more prone to brittle edge and corner spalling in the event of unplanned edge pressures during installation or transport, as well as from thermal actions. Repaired damages from spalling or cracks with larger crack widths are often weak points in terms of durability.
- A dense concrete structure with low porosity is desirable particularly in the areas of concrete cover. This must be ensured by:
  - concrete technology (low w/c-ratios, balanced grading curves, etc.),
  - adequate production at the plant, usually with closed steel formwork (compaction, adequate curing, protected storage). The curing of the outer side of the segment is of special importance because air pockets can gather here as a result of the concreting process and top-closing moulds (absorbent formwork systems on the tops or manual curing and removal of segregated surface concrete).
- Adhering to demoulding times
- Transport, storage and installation that avoid damage (avoidance of cracking and necessary repair works)
- Adequate concrete cover as specified in **ch. 5.9.1**
- Use of a securely fixed reinforcement cage
- Avoidance of unplanned mechanical loads (overloading, local damage) and unplanned exposures (e.g. chloride or sulphate-containing backwater or leakage water) by continuous structure inspections and maintenance during operation
- Limitation of the analytical crack width in accordance with **ch. 5.10.2**.
- Consideration of additional installation components and anchorage devices (anchoring of pipes, fittings, electrification, etc.) in the durability design.
- Use of other special constructions such as HDPE concrete protection panels, polymer-modified protective layers, foils, etc.

8.4 Special considerations for the use of steel fibres

Steel fibres can corrode when exposed to a relevant aggressive environment; corrosion is, however, limited to the near-surface region. Due to the small fibre di- ameter, the corrosion products do not lead to spalling effects of the concrete matrix and the corrosion does not reach very deep.

The corrosion of the steel fibres on the concrete surface can potentially be an aesthetic problem, but is otherwise acceptable.

For the use of steel fibre reinforced concrete segments in aggressive environments (exposure classes XS2, XD2, XS3, XD3), refer to **ch. 5.11.3**.

9 Sustainability, carbon footprint

9.1 Introduction

In the fight against climate change, the aim is to achieve climate neutrality in all areas and, as an interim goal, to significantly reduce greenhouse gas emissions. Tunnelling can be an important tool in achieving this goal, as sustainable construction methods focus in particular on issues such as recycling and protection of natural resources. Underground structures protect the environment, as they help to reduce land sealing, for example, and thus protect the soil and groundwater, serve as heat reservoirs or as shelters for natural disasters or as storage facilities for waste.

Only in recent years has the issue of sustainability beeOnly in recent years has the issue of sustainability been increasingly taken up and addressed with regard to tunnel structures. Different approaches and assessments shape the discussion. In particular, when assessing the sustainability of tunnel structures, a large number of different performance indicators describing the ecological, economic, socio-cultural and functional quality of the structure must be taken into account. A defined and uniform assessment parameter or an assessment scheme for tunnel structures does not yet exist in practice apart from the public approval procedures. For this reason, current discussions generally focus on the "CO_2 emissions" during construction of the structures that dominate public perception. However, the use of resources is no less important. Primary resources must be used multiple times and protected. Tunnelling has significant reserves in this regard, as it already reuses excavated material as far as possible, for example.

Due to complex soil-structure interactions and high demands on the durability of the structures, tunnelling consumes considerable quantities of material, the production of which is associated with significant CO_2 emissions and resource consumption. Added to this are CO_2 emissions from material logistics, which inevitably occur during the construction of running structures. Despite these enormous environmental impacts, resource consumption-optimised and low CO_2 construction methods in tunnelling – and in particular in mechanised tunnelling – have so far only received sporadic attention.

As the topic of sustainability is becoming increasingly important, it is addressed for the first time in this recommendation. The recommendation is limited to the minimisation of CO_2 emissions. However, it is urgently necessary to assess sustainability on the basis of additional criteria, taking into account the functional equivalent (functional equivalent = techni- cal properties of a structure that must fulfill all examined sustainability criteria, e.g. cross-section size, distance between emergency tunnels, crack widths, etc.). In this context, CO_2 emissions from materials, logistics and production are relevant for precast tunnel segments.

Sustainability is assessed for the entire civil engineering structure throughout its lifecycle. This ensures a comprehensive assessment. In justified cases, however, functional units or components may be separated from the overall project, provided that a comparability between different construction variants is ensured. The recommendations given here must therefore always be scrutinised in relation to the specific project and adapted if necessary. Socio-cultural aspects, in particular, are becoming increasingly important in society and are included in the early stages of the assessment of infrastructure schemes. These include, for example, the aspect of land sealing, which is stopped in urban areas and replaced by green zones. Tunnels are often the only way to combine the desired mobility and quality of life.

Mechanised tunnelling in particular, with its high potential for automation solutions with regard to segment prefabrication and production optimisation, is a key technology for modern transport projects and their sustainable implementation.

9.2 Material

Worldwide, segments with CO_2 emissions of approx. 700 to 1000 kg CO_2/(m^3 segment) based on Portland cement concrete mixes are identified as the standard. Projects in the UK and elsewhere are pursuing alternative approaches to increasingly replace Portland cements with blast furnace cements. Substituting reinforcement with fibres is another way of reducing CO_2 emissions.

However, a significant reduction in CO_2 through material changes in the area of cement also leads to changes in manufacturing processes as a result of longer demoulding times and modified material properties. A precise quantification of these effects based on reliable empirical values is currently not yet possible. However, it is undisputed that a significant reduction in CO_2 can only be achieved through drastic changes to the materials used as concrete aggregates and their production. The reduction or substitution of the Portland cements is one possibility that can already be considered in current projects.

It should be noted that the reuse of excavated material in particular, e.g. in concrete production, has considerable potential for a sustainable project and should be more frequently used in the future as part of the recycling process.

9.3 Logistics

The necessary transport processes on tunnel jobsites result in CO_2 emissions, which should be limited if possible. Local segment production using tunnel excavation material as an aggregate would, for example, contribute to a reduction in transport volume.

The consideration of other emissions from the construction process could be relevant in order to identify the most sustainable construction method from a variety of options: mobilisation, demobilisation, equipment operation, equipment wear.

9.4 Final considerations

There is currently no real alternative to the use of concrete in tunnelling. To increase the sustainability of tunnel structures not only a more resource-efficient and CO_2 emission-minimising use of materials can contribute. Concrete is highly durable. A design that is adapted to the material and using the structure for as long as possible (also considering a change of use) also contribute to improving the sustainability of tunnel structures. Current material developments show the potential of new types of concrete.

Tunnel structures address many other aspects of sustainability and should always be considered holistically when comparing structures. The DAUB has decided to give the subject greater attention and to develop recommendations for tunnelling as part of a "Sustainability" working group. A white paper **[60]** has recently been adopted that addresses all key aspects and as part of the future DAUB recommendation sets out the framework.

10 Special designs – cross passages, steel segments, transition to cut & cover sections

10.1 Cross passages

10.1.1 Introduction

Cross passages in tunnel tubes with segmental lining serve as escape tunnels between two tubes or to emergency exits, as connection to stairways or as service rooms. The cross passages are usually constructed with conventional tunnelling methods, less commonly with the pipe jacking method. The layout of the cross passages itself has few peculiarities. In contrast, the transition between the segmental lining and the cross passage is more critical in regard of design and structural aspects and will therefore be exclusively discussed hereafter. Similar challenges also often occur with shaft connections, niches, emergency or breakdown bays and pump sumps.

10.1.2 Placement of the cross passage opening

The optimum formation of a cross passage opening greatly depends on the specific boundary conditions (cross-section of the main tunnel and the cross passage, geometry of the segmental lining and location of longitudinal joints, construction method and lining type of the cross passage, required clearance profile, pipelines and cables, rock type, water pressure, sealing concept for the cross passage, number of cross passages, construction schedule, requirements for corrosion and fire protection, etc.). The upper and lower edges of the cross passages and their transitions should be within the segment tunnel if possible. It is advantageous if the height of the cross-passage axis coincides approximately with the height of the tunnel axis of the main tunnel.

Generally it is possible to design special segment rings with steel segments in the cross passage area or to install reinforced concrete segments with standard geometry, the latter, however, often with increased reinforcement and additional bolted connections/ dowels. The cross passage opening is taken or sawn out of these special segment rings.

If special segmental rings are foreseen, it is advantageous to arrange parallel rings of uniform width in the area of the cross passage. This is particularly the case if built-in parts are provided in the area of the cross passages and the ring position is therefore predetermined. With tapered rings the location and order of the rings must be determined in advance or be limited. The key segment has special geometric proportions and is for simplification often placed on the side opposite the cross passage opening. To reduce the number of special segments, a mirror-image arrangement of the segments in the cross passage area is desirable in both tunnel tubes. It should be noted that this means that in both tubes the possibility of TBM alignment correction is lost or limited in the area of the special segments.

If ground freezing is to be executed from the segmental lining tunnel, the structural analyses must take into account the arrangement and location of the freezing lances around the cross passage opening as well as any freezing pressures acting on the seg- mental lining.

10.1.3 Support of the Segments

The segmental lining rings are permanently interrupted in the area of the cross passage opening. In the final state the normal forces are generally transferred around the opening via in-situ concrete frames or frames made of steel segments. The cast iron segments used in the past are no longer in use.

Because an in-situ concrete frame can only be constructed after the breakout of the cross passage opening, this solution additionally requires temporary support of the segments during construction. This can be done with steel rings, steel girder trestles or frames transverse to the tunnel axis. It is also possible to bolt or dowel the segments in the ring joints using special connecting devices in the concrete segments. Combinations of multiple measures are also possible, whereby in this case particular atten- tion must be paid to the realistic distribution of the loads on the individual components proportional to their respective stiffnesses.

If segment frames or dowelling, respectively bolting, are used as support in the construction and the final state, the bedding of the load-bearing rings in the side wall must be ensured at all stages of construction. Connecting devices between the segment rings that are to remain permanently effective require the corresponding approvals.

In case that the entire junction area is stabilized all round by ground freezing, jet grouting or ground injection, the stabilization my potentially replace the temporary support or the load that needs to be transferred can be reduced to the dead weight of the segments plus a loosening pressure.

The analytical approach must take into consideration that the loading and the bedding reaction are no longer effective in the area of the opening and possibly beyond. This implicates that the load-bearing system of the segment ring is disrupted, exhibits asymmetries and transverse displacements between the rings may occur. These must be reduced by appropriate measures to a tolerable level. It is recommended that any existing water pressures on both sides of the tunnel tube be reduced as much as possible during the construction phase.

10.1.4 Steel segment frames

In the past, in areas around cross passages steel segments were often used, which were bolted together to form a rigid frame around the opening (**Figure 37**). However, as this solution is comparatively expensive and not very sustainable, alternative solutions have recently established themselves.

Special steel segments must have the same ring width as the standard segments; if necessary smaller segments must be supplemented with additional special elements to make up the full ring width. Bolting of the ring and longitudinal joints must correspond to the concrete segments. The longitudinal joints must also be matched to the concrete segments. In extension of the TBM jacks the steel segments must be stiffened accordingly. Because of the tailskin seal, the outer diameter of the special segments must correspond to that of the standard rings, whereas the inner diameter may be the same or larger.

When defining the inner diameter of the special segments the requirements for corrosion and fire protection must be clarified in advance, see **ch. 6**. For corrosion protection steel coatings, an increase in the steel plate thickness or concrete coatings are commonly used. For fire protection, fire resistant plasters, fire protection panels or PP fibre-reinforced concrete can be used. In order not to restrict the clearance profile, the inner diameter of the special segments is usually increased when concrete coatings and/or fire protection panels are applied (see **ch. 6**).

For railway tunnels with electrification, details for earthing and electrical through-connection of the steel segments must be clarified in advance.

Steel segments usually also use circumferential elastomer profiles as sealing gasket. Therefore a sealing groove must be foreseen in the same way as for concrete segments. Recessed corners are not permitted. It is also important that the sealing gasket is not damaged when the filling segments are removed.

The steel segment frames are also used to support the segmental lining ring during construction. This requires sufficient lateral bedding reaction of the rings. The required width for the bedding depends on the load, the cross-section of the main tunnel, the width of the transition frame and the geology.

At the latest before removing the filling segments from the opening, the individual steel segments are bolted together with high-tensile prestressed bolt connections to form the rigid frame. This requires appropriate mounting holes.

10.1.5 Bolting or dowelling in the ring joints

When using special segments, the segments can be force-locked bolted or dowelled in the ring joints above, below and next to the cross passage opening (**Figure 35**). The coupling allows the normal forces of the interrupted segment rings to be transmitted to the adjacent rings.

Such structures can be designed both solely for the construction stage and for the final state. If they are merely for load transfer during construction, the ring forces of the segments must in the final state be absorbed by an in-situ concrete collar that is arranged within the opening (bracket). If the connections are considered for the final state, special requirements must be considered for fire protection and corrosion protection.

Since the load transfer may results in high local stresses in the ring joints between adjacent rings, usually special precautions are taken for the load introduction into the concrete segments and the reinforcement is designed accordingly. The load on these special segments usually requires a considerable high reinforcement content.

Depending on the geotechnical and geometric boundary conditions, forces in radial direction (i.e. transverse forces acting in direction of the segment thickness) may occur at the coupling points in the ring joint in addition to the forces acting in the tangential direction (i.e. normal forces that act in the ring direction). These forces in radial direction often determine the load bearing capacity and thus the possibility of using such type of construction.

For further information on coupling force transmission, please refer to **ch. 5.7.11**.

10.1.6 Inward facing steel structures

Steel frames around a cross passage opening can either be arranged inside the main tunnel cross-section (**Figure 36**) or in the central area of the segmental lining (see **ch. 10.1.4**).

Arrangement to the inside of the cross-section requires the transmission of forces from the segmental lining into the support structure via brackets, steel mandrels or similar. In addition, the support structure must be installed to allow full force transfer (potentially with pre-tensioning of the struts), the deflection of the bars must be taken into account and, in the case of curved struts, their bedding must be ensured. If the main tube consists of tapered rings, the tunnel describes a three-dimensional curve. This must be

Figure 35 *Special segments with a bicone dowel (left) and a bolted solution (right)*

considered in the design of the frame and the brackets.

10.1.7 In situ concrete frames

In situ concrete frames between the segmental lining tunnel and the cross passage are often used as the final support of the tunnel lining. The level and the segment division of the main tube and the level of the cross passage are decisive for the design. In addition to the clearance height of the opening, a sufficient height of the bracket above and below the opening is required for supporting the lining rings. Sometimes problems are caused by necessary cable conduits or drainage pipes in the invert of the cross passage that may reduce the cross section of the lower bracket.

The in situ concrete frame serves together with the inner concrete lining of the cross passage as stiff bedding element for the steel segment frames.

In case of a double-shell lining construction the in situ concrete inner lining, that is installed around the opening in advance, can often act as temporary and potentially permanent support for the segmental lining in the cross passage area.

10.1.8 Creating the cross passage opening

In most cases, the opening in the segmental lining is realised using secant core drills and/or saw cuts along the opening contour. However, filling elements can also be used which close the later cross passage opening during tunnelling and are subsequently removed. The removal of such filling elements needs to be considered already in the design of the cross passage opening.

10.1.9 Sealing of junctions

When designing the sealing of the junction between the cross passage and the segmental lining the water pressure head and the sealing type of the cross passage are decisive. Possible solutions are an in-situ concrete lining made of impermeable concrete with or without an additional waterproofing geomembrane.

Figure 36 *Steel frame construction Katzenbergtunnel*

Figure 37 *Steel segments as part of the cross passage support*

A technical possibility for sealing the junction is gluing the geosynthetic membrane of the cross passage onto the appropriately prepared outer face of the segmental lining outside the cross passage opening. Fastenings with fix-loose flange constructions are also used.

The disadvantage of these solutions is that the outer face of the segment is usually rough and curved in one direction and the sealing system crosses the segment joints a number of times. These joints must be cleaned and filled up to the depth of the sealing profile to avoid water creeping at the clamp construction. Backfilling with special grout is recommended only at low water pressures. In any case, an additional grouting hose should be foreseen. For higher water pressures, a PU resin filling can be considered. In this case, however, considerable problems with joint filling overhead can occur. PU also does not adhere to the elastomeric seal. For further details and possible solutions reference is made to the technical literature on the subject (see e.g. **[61]**).

If steel segments are arranged around the cross passage opening, a loose flange construction with a joint sealing tape or a protruding steel flange can also be subsequently mounted on the outer face or end face of the segment.

Fix-loose flange constructions can generally be used only for water pressure up to about 3.5 bar.

Differential settlement is generally possible between the main tunnel and the cross passage, but this is often very small. When a reinforced concrete frame is formed the expansion joint between the main tunnel and cross passage is advantageously arranged as a compression joint between this concrete frame and the standard cross-section of the cross passage

10.1.10 Emergency locks

Emergency locks at the cross passage openings, that are sometimes required by clients to prevent flooding during construction are very expensive. The locks must be designed for the full water pressure and are therefore very heavy. On the other hand, they must be quick and easy to close in the event of danger. The pre-tensioning force for the lock is usually generated by the water pressure itself. Emergency bulkheads that can be slid in from one side or drop gates respectively flaps placed above the opening have been used.

10.2 Steel segments

10.2.1 Introduction

In the past, “steel” segments were mostly produced as cast iron segments made from nodular cast iron. Despite the steel-like properties of nodular cast iron, these are not steel components. Today, steel segments are made from rolled steel sheets which are welded to form a segment. Steel segments are generally used where, due to the loading or the geological conditions, the tunnel lining is exposed to extreme loads which can no longer be absorbed by reinforced concrete segments.

10.2.2 Formation

Steel segments welded from rolled sheet metal usually consist of an outer shell plate, which forms the watertight shell of the tunnel lining. On the inside web plates are arranged in the circumferential and the longitudinal direction of the tunnel, so that a kind of casket segment is created. Together with the shell plate the web plates arranged circumferentially absorb the ring forces consisting of normal force, bending moment and shear force. The web plates arranged in the longitudinal direction of the tunnel are used to transfer the normal forces in the longitudinal direction of the tunnel, for example, from the TBM jacking forces. In some areas, the caskets can also be filled with non-shrinking concrete ex works to improve the transmission of compressive forces.

In both, the longitudinal joints and the ring joints, the steel segments are connected by means of bolts. Depending on the number and the preloading of the bolts the connections can be made rigid and shear resistant.

In a segmental lining ring either only single reinforced concrete segments can be replaced by steel segments (see **ch. 10.1.4**) or the entire ring can be formed from steel segments.

10.2.3 Production

For the production of the steel segments process related boundary conditions result that must be taken into account already in the design phase.

The individual plates are first cut and then welded together to form segments. Welding results into stresses and distortion in the sheets and the entire segments. For this reason the final outer contour line is formed only after welding by milling. The segments are aligned in such a way that as little material as possible is milled. The holes for the bolts are also drilled in this step. This ensures that the segments are sufficiently dimensionally accurate and exactly fit together.

When determining the plate thicknesses the amount for the milling plus a certain tolerance value must be taken into account. It must be ensured that minimum thickness of the plates, that is statically required, is maintained after milling.

10.2.4 Installation

The installation of steel segments is much more complicated than the installation of reinforced concrete segments. This results from the low tolerances for bolting, which requires a very much exact ring erection. The installation of the numerous bolt connections is very time consuming. All bolts must be tightened to their required preload which reduces the tunnelling performance considerably. Usually special adapter plates are required for the installation of the steel segments with the erector.

Due to the production of the individual segments offsets can occur on the inner side between adjacent segments, although the bolt connections and thus also the sealing profile perfectly fit together.

10.2.5 Fire protection

In order to ensure the fire protection in the final state, it is possible to design the steel segments slightly thinner than the reinforced concrete segments and to provide a cladding with fire protection panels or fire protection mortar. See also **ch. 6** (Fire protection).

10.3 Transition to cut & cover sections (e.g. at stations or portals)

10.3.1 Introduction

Stations, portals and other special structures are usually constructed as in-situ concrete structures using the cut-and-cover construction method. To provide for a tight connection between the segmental lining and the in-situ concrete, a special transition structure is required. This is usually ensured by means of joint tape sealing systems. Fundamentally it must be differentiated whether the sealing system should be permanently accessible and replaceable from the outside, or whether it should be integrated into the concrete structure and no longer accessible.

10.3.2 Replaceable sealing systems

Permanently accessible and replaceable sealing systems can be accomplished only with the help of a clamp construction and an omega-type sealing profile. For this purpose, complex fix-loose flange constructions are required. Appropriate fixed flanges must be arranged for this solution on both sides of the segment lining and the in-situ concrete structure. An elastic joint tape is attached to it at both ends by means of a loose flange.

For corrosion protection, the flange constructions usually have to be of stainless steel. Because such constructions are technically very complex and usually no deformations are expected between the in-situ concrete structure and segmental lining, they are usually avoided.

For fire safety reasons, the joint tape must usually be protected against the effects of fire my means of appropriate fire safety equipment (fire protection mats).

10.3.3 Non-replaceable sealing systems

Constructions in which a joint sealing tape is embedded directly into the situ concrete are less complex (**Figure 38**). On the segmental lining side the seal construction should be flanged onto the sealing gasket frames of the adjacent segmental lining ring. It has proven useful to provide an additional circumferential insert made of raw rubber between the sealing gasket frame and the sealing flange. The free end of the sealing tape generally ties into the in situ concrete construction made of waterproof concrete. Alternatively, the joint sealing tape can also be connected to an external sealing membrane.

To accommodate possible movements between the in-situ concrete construction and the segmental lining, the joints between the segmental lining and the in-situ concrete structure should be designed as expansion joints. Appropriate compressible materials and joint sealing tapes that can accommodate the movements should be used for this purpose.

Special fire safety precautions are not generally required because the joint tape is adequately protected by the in situ concrete.

As valid for any joint construction, appropriate grouting hoses should be provided for any resealing that might be required at later stage.

11 Segment production and installation

11.1 Notes on the production of segments in a precast plant

11.1.1 Carousel/stationary production

Two types of production have become established in the manufacture of segments stationary production and carousel systems.

For stationary production, the segment moulds are permanently positioned at a defined location in the production hall. It is recommended to separate the formwork from the concrete floor slab using an elastomer bearing so that the vibration energy of the vibrators can be fully transferred to the fresh concrete via the mould base. Clamps fixed to the foundation prevent the formwork from lifting off when the segment is lifted out. With this production method, all consumables (reinforcement, built-in parts, release agents, concrete) are brought to the formwork. The concrete can be poured using buckets or directly from the truck mixer. Heating pipes are placed under the formwork to optimise the hardening of the concrete. This production method should be selected if, for example

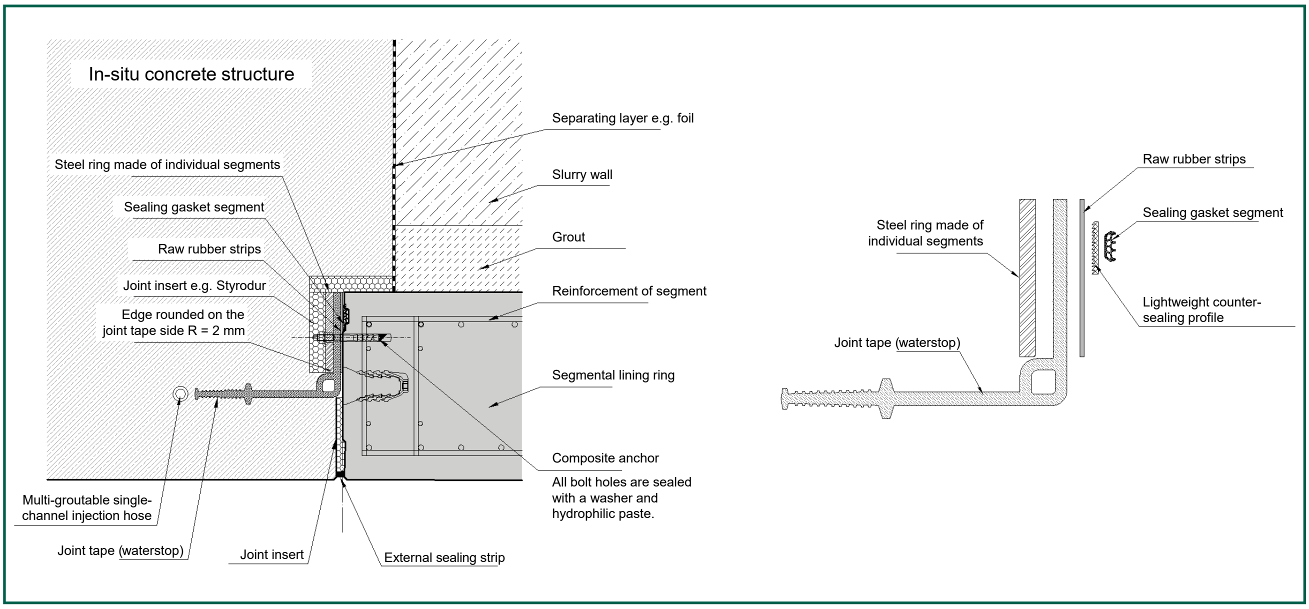

Figure 38 *Transition construction with an L-shaped joint tape and raw rubber layer as well as sequence of sealing elements*

- A quick start to production is required,
- The production period is short (1 – 1.5 years),
- Segments only need to be produced for a single project,
- The existing hall layout only permits this type of production.

Stationary production is generally more labour-intensive than carousel production.

In the more investment-intensive carousel system, the moulds are moved on tracks by a drive system. This should move the formwork as smoothly as possible using frequency-controlled drive technology. The following areas/work steps are typical for carousel plant production:

a) Production line – all the steps required for demoulding, cleaning, reinforcing and concreting are carried out here

b) Lateral shift – the formwork is moved alternately on a trolley to the lines of the curing tunnel

c) Curing tunnel (often also called heat tunnel) – the formwork is pushed into the curing tunnel by the shift trolley. There are usually 3 zones with different temperatures in the curing tunnel

d) Lateral shift – the formwork is alternately pulled out of the curing tunnel onto the shift trolley and returned to the production line.

With carousel plants, the term cycle time is usually used. This describes the time required per work station, including the shifting time to reach the next work position.

A carousel plant usually allows two complete cycles per working day. This means that each mould is filled with concrete twice a day. The production output is usually determined by the number of available formwork sets and the curing time.

With stationary production, two fillings per working day are also possible. However, in contrast to the carousel system, the number of formwork sets can be increased at any time, thereby achieving a higher production output.

Automation/use of robots

Both forms of production have automation options that have already proven themselves in practice. These include:

- Hydraulically opening and closing formwork: this can significantly reduce the time and manpower required. The plant technology also prevents incorrect operation of the formwork.
- Robots for cleaning the formwork: stationary robots or robots mounted on linear units clean the formwork quickly and safely and prepare it for its next use. These robots also blow out residual concrete and oil the formwork.
- Robots for surface finishing: stationary robots or robots mounted on linear units smooth the surface of the freshly poured segment.

Other automation options (fully automated demoulding gantries, semi-automated concreting equipment, use of image-processing camera systems, etc.) are also possible and already in use.

11.1.2 Concrete: production and monitoring

The strength class of the concrete must be defined in advance in accordance with DIN EN 206 and verified by a specified date at the latest. According to ZTV-ING, Part 7 Tunnelling, section 3, clause 7.2.2.1, a minimum compressive strength of 15 N/mm^2 is required for lifting. For anchored gasket frames, a minimum compressive strength of 18 N/mm^2 is recommended to prevent the sealing gasket from being pulled out. If demoulding is planned with a lower compressive strength, a structural analysis must also be carried out for this. The conditions of segment production (in particular the type of lifting equipment) must be taken into account. As general rule, the early strength can only be determined on the demoulded segment in maturing storage. Determination of the early strength, which is recommended once per production shift, is carried out on the segment surface. These values are on the safe side compared to the strengths in the core.

Due to heat treatment, segments often experience a deviation from the standard-compliant storage of the test specimens in the form of an increased temperature during curing. The strength should therefore also be verified on heat-treated test specimens from steel moulds. The core temperature of the segment should not exceed a concrete temperature of 55 °C. A fresh concrete temperature between 18 °C and 25 °C is recommended. Fresh concrete with temperatures above 30 °C should not be used.

If necessary, the strength of the finished segment can be verified using non-destructive testing (e.g. rebound hammer) on the basis of DIN EN 12504-2.

The tests for monitoring the concrete should generally be carried out in accordance with DIN EN 206 and DIN EN 13369.

For fibre-reinforced concrete with increased fire protection, DIN EN 14889-2 must be used for the PP-fibres. To determine the fibre content, please refer to the ÖBV guideline “Increased structural fire protection for underground traffic structures made of concrete” and the ZTV-ING, Part 7 Tunnelling, section 1, appendix B. Principles of structural fire protection can be found in **ch. 6**.

11.1.3 Reinforcement: production, installation, positional stability

It is recommended to use welded cages for segment production, as this reduces the overlap and anchoring lengths, leading to a reduction in the reinforcement content. It also results in increased dimensional accuracy and a low-deformation storage option for the cages. Partially welded cages are also possible.

Welding robots are increasingly being used today for the production of welded cages. These offer the advantage of high availability in terms of time and enable traceability of the individual welds (welding time, current consumption) with regard to quality assurance. The high demands on the position of the intersection points to be welded must be taken into account, especially when welding on the entire cage, so that welding is possible at all. Furthermore, the requirements for minimum distances and production tolerances resulting from automated production must be coordinated in advance during reinforcement design and taken into account accordingly.

When producing the reinforcement, particular attention must also be paid to the dimensional accuracy with regard to the durability of the segments. In the case of structurally required force transmission of welded joints, the respective shear factor (e.g. SF 50) must be specified for the relevant connections. Compliance with the specifications must be verified at regular intervals. Only welders with appropriate certificates of competence in accordance with EN ISO 17660 should be employed.

It is recommended to maintain a maximum of one diameter increment for welds between load-bearing and structurally welded bars.

Weldable reinforcing steel B500 B in accordance with DIN EN 10080 must be used for the reinforcement.

EN ISO 17660 -1, -2 (Welding of reinforcing steel) must be used for welded reinforcement.

The number and position of the concrete spacers must be selected depending on the geometry of the reinforcement cage. For light and unstable cages, the number of spacers must be increased (also in deviation from the standard).

In tunnel segment production, point-type spacers made of cast concrete or fibre-reinforced concrete, which are mounted on the intersection points of the meshes, have proven effective. Plastic spacers are often not approved by the client.

In the course of quality assurance, particular attention must be paid to checking of the concrete cover. Non-destructive measuring methods have proven effective for this purpose. It is advisable to agree the measuring method that is to be used between the contracting parties in advance.

11.1.4 Reinforcement: use of steel fibres

In addition to using conventional reinforment for the segments, steel fibres are also applied. The steel fibres must be certified in accordance with DIN EN 14889.

In accordance with the DAfStB Guideline for Steel Fibre Reinforced Concrete **[15]** the designer determines the performance class of the steel fibre reinforced concrete as part of the design, see also **ch.**

5.11. The composition of the concrete, including the type and quantity of fibres, is determined by the manufacturer of the steel fibre reinforced concrete. The required equivalent bending tensile strength is confirmed by means of preliminary tests, in accordance with the requirements of DIN EN 12390-1, with the specified concrete mix design and different steel fibre contents. Aggregates with a maximum grain size of 16 mm for round grain and 22 mm for crushed grain are recommended. The length of the steel fibres should not be less than 1.5 times the maximum grain size. The higher the desired fibre content, the lower the proportion of coarse aggregate should be.

The steel fibres are usually added to the fresh con crete mix using a dosing system with a vibrating device for separating the fibres. This is filled with steel

fibres from big bags. To prevent moisture-induced sticking of the fiber bundles ahead of production, these systems should be enclosed and protected from damp weather. The desired quantity to be added per batch in kg is set on the weighing instrument or in the mixing computer. The fibres are added to the aggregates on the discharge conveyors, in the inclined elevator bucket, in the weighing hopper or directly into the mixer. Mixing the fibres with aggregates before adding the binder facilitates the breaking up of the fibre packages and helps to reduce fibre balling in the fresh concrete.

Maintaining a minimum mixing time in the mixing plant for each batch after adding all materials is recommended as follows:

- One minute for particularly good mixing action (twin shaft compulsory mixer, pan mixer with agitator),
- Two minutes for all other concrete mixers.

Addition of the fibres into the truck mixer is also possible. If the fibres are added as continuously as possible and the drum rotates at maximum speed, a mixing time of at least five minutes after adding the steel fibres is recommended. The mixing time per cubic metre of fresh concrete in the drum should not be less than one minute.

The exact quantity (± 1-2 % deviation) of steel fibres is determined by negative weighing.

The following tests should be carried out at regular intervals during production:

- Strength determination on the bending beam,
- Washout test,
- Test specimens to determine fibre distribution in the cross-section,
- Retention samples.

The influence of the addition of steel fibres on the compaction behaviour of the segment concrete during vibration is negligible. Excessive compaction should be avoided, as this can lead to an uneven distribution of the steel fibres.

The "Inductive Method" **[20]** was developed as a non-destructive test to determine not only the fibre content but also the distribution and orientation of the fibres.

11.1.5 Concreting: vibration, compaction, curing

Two processes must be taken into account when concreting and compacting tunnel segments:

- Placement and distribution of the concrete: due to the given shape and the mould covers, the concrete usually has to be placed centrally in the formwork and then transported to all sides, especially the corners, by the vibration energy.
- Filling the formwork and compacting (removing air) the concrete: filling the concrete in layers at high compaction energy and air release from concrete.

External vibrators are used in most segment formworks. These are attached to the underside of the formwork base. Attaching the external vibrators to the lateral formwork is not recommended, as this would lead to the tolerances being exceeded in the segment width. Compressed air vibrators are usually used. It has been shown that these vibrators are sufficiently suitable for the two above-mentioned processes and are gentle on the construction of the steel formwork. Electric vibrators are often used for segments with larger lengths and thicknesses (>0.5 m), as the controllable compaction frequency enables a purposeful distribution and compaction even with these large dimensions.

The compaction vibrators should always be controllable in groups so that optimum compaction of the concrete is possible in all areas and filling states.

Vibration time and intensity should always be recorded by the concreting supervisor.

The use of vibrating tables is also possible, but has so far only been carried out very rarely.

Curing

The concrete for tunnel segments must be cured at least until the cube compressive strength of the concrete is at least 60 % of the average cube compressive strength of the conformity test or exceeds a cube compressive strength of 30 N/mm², whichever is higher.

If curing agents are used, a sufficiently high sealing coefficient must be ensured for effectiveness. Compatibility with subsequent coating systems must be investigated.

The young segment must be protected against rapid cooling; the cooling rate at the surface should be max. 10 K/h. The temperature difference between the surface and the ambient temperature should be a maximum of 25 K when moving to the outdoor storage area.

11.1.6 Demoulding

Once the concrete has reached its demoulding strength, the segment can be lifted out of the formwork. The use of a vacuum suction plate is preferred. This enables gentle and safe demoulding and transport.

The crane and vacuum suction plate must be designed in such a way that the segment can only be lifted out once the vacuum has been fully established. In addition to the design safety factors, the suction plate must also pass a test that simulates a power failure and thus a stop of the vacuum pump. According to DIN EN 16191, Tunnelling machines, it must be demonstrated that the vacuum is maintained for at least 20 minutes after the power supply or the vacuum pump fail.

Additional safety is provided by attaching belts or using safety arms that reach under the segment after demoulding to prevent it from falling.

When dimensioning the vacuum suction plate and the demoulding crane, the lifting force should always be added. This additional force is usually caused by the segment sticking to the formwork. As general rule, at least 30% of the total weight of the segment and lifting device should be added. According to DIN EN 13155:2020, vacuum lifters must be dimensioned in such a way that they can still hold at least twice the load capacity at a distance to the end of the working area or at the beginning of the danger zone.

When using a vacuum suction plate, the location of use must already be taken into account in the design phase of the device. For example, when used in geographically higher production facilities, additional safety measures must be taken into account.

A reduced suction area must be considered in the design if the suction area is restricted by the covering of installation parts, e.g. anchor rails.

Attention should also be paid to the evenness of the outer surface (rock side). Larger local offsets prevent the buildup of a vacuum, thereby impairing functionality and/or reducing safety.

Mechanical gripper systems

Mechanical grippers can also be used for smaller segments, usually with a segment weight of less than 4 t. For this purpose, the lateral formwork is opened slightly wider and the gripper clamps the segment between rubber or plastic claws. The dowel openings in the ring joint are often used to centre the gripper system. Lifting, on the other hand, takes place purely by friction and is therefore limited by the available surface area and the compressive strength of the concrete.

The advantage of this system is the integration of a simple turning device. This means that, if necessary, the segment can be turned into the final installation position immediately after demoulding.

11.1.7 Equipment

The tolerance specifications must be observed and continually checked for all built-in parts of the segment.

Particularly high demands must be placed on the quality of the concrete structure for the main equipment parts, such as the gasket frame. Care must be taken to ensure that the sealing groove is dimensionally accurate and free of voids before the seal is installed. Cracks crossing grooves, if permitted, must be repaired beforehand.

For reliable bonding of the sealing gasket in the groove, the groove must be brushed out with a hard brush and blown out with oil-free compressed air before applying the adhesive. The use of a jacking frame is recommended to achieve an even fixation of the gasket frame.

When using anchored gasket frames, it is recommended that sections of the gasket be removed at regular intervals from the rejected segments in order to obtain information on the condition (especially the presence of voids) of the concealed areas.

If packers are foreseen in the ring joints, it must be ensured that the adhesive is applied evenly; the use of self-adhesive packers is recommended. The materials used must be resistant to weathering for a sufficiently long storage period of the segments or should otherwise only be applied at a later date. Self-adhesive plastic packers are a good alternative to conventional packers that are usually made of swellable wood-based materials.

Currently, the proportion of jobsites that completely avoid the use of packers for segmental linings is increasing. An increase in defects attributable to the lack of packers can only be observed to a small extent, if at all.

When using guiding rods in the longitudinal joints, it is preferable to install them at the precast plant using pins instead of adhesive. Pins are small anchor nails that are attached to the end formwork and then cast into the segment. After demoulding, the guiding rod, which already has holes for the pins, can then be attached to them. Gluing is not required.

In the case of anchor rails cast in concrete on the inside, appropriate precautions must be taken for vacuum lifting devices, e.g. covering the anchor rail inside the suction plate with a foam pad to ensure a safe vacuum build-up.

11.1.8 Survey

To determine compliance with manufacturing tolerances, in addition to the necessary survey on individual segments, surveys on the complete segmental lining ring are also useful. The respective locations for survey should be specified in the construction contract.

Test ring, virtual test ring and virtual ring building

A test cast is often made after the first formwork set has been produced and a single-layer or double test ring is built from this. The test ring should be built without the EPDM gasket, as the gasket cannot be compressed by bolting or tensioning of the ring. After assembly, the ring is measured (inner diameter, outer circumference). Other dimensions can only be checked to a limited extent or not at all.

The tolerance values (see **ch. 3.5.2**) apply to measurements on the young segment. During the control measurement, the component and ambient temperature must be recorded and taken into account when evaluating the survey results.

An alternative and much more meaningful option is to create a virtual test ring. Here, the survey data of the formwork or a segment can be used and virtually assembled to a ring. Deviations on the ring can be precisely identified with this method. The survey results from several sets of formwork can also be used and assembled to several rings. It is also possible to record and evaluate the position of the built-in parts.

Virtual ring building can also be used to assemble manufactured and measured segments with tolerance deviations to acceptable lining rings.

Formwork before first use

When producing the formwork, the formwork manufacturer creates a 3D certificate for factory acceptance. After transport and assembly on the jobsite or in the precast plant, a 3D measurement should be carried out again. In principle, the formwork manufacturer should constructively provide a possibility for readjustment.

Frequencies

After the initial survey on the jobsite or in the precast plant, the segments should be measured at regular intervals. A new formwork measurement should only be carried out if significant deviations are repeatedly detected during segment survey. The measurement intervals after 1, 5, 50 and then after every further 100-250 shuttering operations have proven to be practicable. If manufacturing tolerances are exceeded during a measurement, the affected segments should be randomly remeasured in a backward sequence up to the last measurement where the tolerances were met. Segments exceeding the tolerance limits must be examined separately.

Type of measurement

Various survey service providers offer 3D laser tracker measurements. The type of survey (devices, number of survey points, etc.) and evaluation software is not standardised. Different survey and evaluation techniques have an influence on the measurement results. A survey regime that is accepted by all parties should be established before formwork and segment production begins.

Rod micrometers, templates

During production, a regular quick check of the segment length in the formwork using a rod micrometer is advisable. Diagonals can be checked using a precision tape measure. These measuring methods do not meet the requirements of a 3D measurement, but any deviations can be detected quickly. Impact marks in the joint area of the side formwork indicate whether the formwork is properly closed.

In addition, 3D templates can be used to locally determine deviations of the side and end formwork using sensors. However, since these results do not provide information in geometric space, modern segment producers no longer use these templates and instead prefer the 3D laser tracker method.

11.1.9 Transport and storage

During the production process, segments are moved several times at an early concrete age. Gentle methods such as the use of vacuum lifters have proven effective in the past. Grippers equipped with rubber buffers are also able to move the young precast elements without causing damage.

When using mechanical segment clamps or forklift tines, precautions must be taken to prevent spalling. For this purpose, resilient supports or polyamide pads should be used.

The edges and corners of the segments to be moved must remain free of load. Appropriate edge distances must be maintained for the load-bearing points of the lifting equipment.

To avoid damaging the gasket, segments with sealing frames should generally be stored in a horizontal position. Segments without sealing frames can also be stored upright on the ring joint.

For stacked segments, in addition to ensuring a sufficiently load-bearing surface, it is important that the stacking blocks are aligned as precisely as possible. A placement template or markings on the segment can be helpful for this purpose. For larger segments and higher stacking, a three-point support system is recommended.

11.1.10 Documentation during production

To clearly identify a segment, a negative imprint is embedded in the formwork, usually on the formwork base. Typically, the formwork type and set number are stamped into the concrete. However, interchangeable matrices can also be used, to include additional information such as the production date and the segment's serial number.

Central database

In general, a data management system is recommended. The core of the system is a central computer/ server where all production data can be collected and analysed. Modern management systems operate with real-time data.

Documentation of production steps

The documentation of individual process steps is carried out using a scanner with a user interface. The responsible operator logs in with their name or ID and selects the process step to be documented, such as the placement of the reinforcement cage. First the formwork is scanned (using a barcode or QR code) followed by the code on the reinforcement cage. The system verifies whether the correct cage has been selected and records the installation. This links the formwork or the emerging concrete segment to the reinforcement cage.. Another example is the documentation of the "demoulding" process step. During demoulding, the formwork is scanned recording the demoulding time and thus the concrete age at the time of demoulding. After demoulding, the segment receives one or more barcode labels containing the serial number and the production date.

Documentation of acceptance processes

After demoulding, the young segments undergo a quality inspection. Using the attached barcode, any damages can be documented, along with the corresponding repair process. Related photos are also attached to this data record.

Measurement results from routine inspection can be clearly assigned to the segment via the serial number contained in the barcode.

If segments do not meet the quality requirements, they can be systematically blocked from being used in tunnel installation.

Documentation of concrete batches with component assignment

For clear traceability, the concrete batches used must be assigned to the respective segments. For this purpose, the data of the mixing plant or the respective delivery batch should be assigned to the segment via the serial number. In some cases, it is also helpful to include an individual barcode or QR code on the concrete delivery note. This code can be scanned at the installation site to associate the concrete batch with the formwork and, consequently, with the segment's serial number.

Storage and retrieval timing

By collecting production data and using a segment identification number, the removal of the segment from the production hall and its storage in the external storage yard can be documented. This also allows for verification of the concrete age ,ensuring that only segments that have reached the required curing time are approved for shipment.

A simple storage management system can be implemented with a black-box storage approach, where only the entry and exit of segments are recorded.

However, more advanced warehousing management programs are available, that document not only the storage and retrieval times but also the exact storage location and stacking order. In such systems, the crane operator receives information on which stack should be retrieved next, either following the **First-In/ First-Out** principle or based on crane positioning to minimize unnecessary crane movements.

These warehouse management programs are essential when dealing with a large variety of segment or ring types, such as rings with different reinforcement configurations or concrete grades.

Tunnel Installation with ring number, ring position and location assignment

The segment ID on the barcode can also be used in conjunction with a scanner during ring assembly in the tunnel. The ring number, ring position etc. can be combined with geological, hydrological and tunnelling machine data and recorded in the digital construction log-book.

This data records can be crucial for future damage analysis.

The data collection process can also be automated using readers attached to the erector suction plate. The same applies to all transport equipment such as clamps, cranes, and multi-service vehicles (MSVs).

RFID tags Tracking

In addition to adhesive barcodes, RFID tags (usually embedded in plastic plates) can be permanently installed in the concrete and visibly placed on the inner surface of the segment. This allows for easy segment identification using a reader, even at a later stage. The RFID system also enables a seamless connection to the database and the segment or tunnel history.

However, identifying individual segment IDs via RFID in the tunnel is generally only possible when in close proximity to the respective segment. For reading RFID tags on segments located in the tunnel crown, the use of a working platform may be necessary.

11.2 Tunnel excavation and ring assembly

A tunnel excavation manual as specified in ZTV-ING, Part 7, section 3 describes the excavation process including ring assembly. The following execution recommendations are provided for inclusion in this guideline document.

Typically, ring assembly is carried out after the completion of excavation and once the required shield extension for ring assembly has been reached. However, recent developments show that with appropriate TBM modifications, including an extended shield tail and additional thrust cylinders, segment installation during excavation is also possible. In this method, individual thrust cylinders are retracted during excavation, allowing for the installation of a new segment. The adjacent segments next to the newly installed one must be designed to withstand higher thrust forces and the absence of lateral bedding during this process.

11.2.1 Ring type and assembly position

The selection of the ring type to be installed (e.g. standard ring, left/right ring, special ring with built-in parts, etc.) is based on the tunnel design and the current advance position of the TBM. The TBM initiates an order to the intermediate storage facility, from where the corresponding segments are loaded and delivered.

During lifting operations and loading of transport vehicles, special attention must be paid to the correct stacking sequence of the segments. When transporting segments inside the tunnel and unloading them onto the TBM's segment feeder, they must be placed in the required assembly order. Due to limited space conditions, extra caution is necessary. The permissible ring assembly sequences and ring orientations must be defined as part of project work preparation.

The tunnel lining generally follows the TBM, which in turn follows the designated tunnel alignment advance.

To ensure that the segmental lining follows the tunnel alignment and compensates for steering movements, rings are typically designed conically and installed with an angular offset relative to each other. During construction, the optimum ring or key segment position is usually suggested by the automatic ring sequence calculation of the TBM navigation software. This calculation considers parameters such as target tunnel alignment, actual TBM position and deviation, differences in thrust cylinder stroke lengths and clearances between the last installed ring and the shield tail. The final decision regarding the final ring position is usually made by the responsible shift engineer.

11.2.2 Ring assembly

Prior to ring assembly the tailskin must be cleaned to ensure a safe and flawless assembly process. It is essential to make sure that no dirt or contamination is present in particular in the contact areas between the segments, on the sealing gaskets and in the contact points between the segment and the erector. When using vacuum suction plates, the erector's access holes must be free of water and ice. Additionally, it is necessary to verify that the vacuum effect is sufficient in relation to the altitude of the jobsite.

During ring assembly, the segments must be installed floating with sufficient clearance from the shield tail, ensuring that they do not rest against the shield tail or the deflection strip. Segments should be installed without misalignment at both the longitudinal and circumferential joints, and collisions with adjacent segments must be avoided.

Markings on the inner side of the segments near the ring joints, e.g. in the form of notches, have proven useful. Plastic centring devices in the longitudi

nal joints and in particular in the ring joints have been beneficial in practice (see **ch. 3.3**).

The ring face at the thrust side must be even to prevent localized stress concentrations, which could lead to cracking or concrete spalling. A reliable indicator for this evenness is the stroke measurement of the thrust cylinders, from which the surface flatness can be determined through cross-comparisons.

To prevent displacement of the sealing gasket frame in the longitudinal joints, the gaskets may be lubricated with a compatible material before installation. However, it is crucial to prevent lubricant from reaching concrete contact surfaces, especially at the key segment. The segments should be moved together in the longitudinal joint as much as possible without the gaskets making contact.

Compression in the longitudinal joint

The erector must exert sufficient force in any ring position to overcome the restoring force of the sealing gasket. The bolting through the longitudinal joint must prevent the sealing gasket from relaxation.

The temporary bolting in the longitudinal joint can only be removed once the surrounding annular gap backfill material is able to transfer the minimum surface stress that is required to compress the gasket profile into the rock.

Therefore, the earliest time for removing the bolting is dependent on the strength development of the annular gap filling material and not on the advance rate.

Compression in the ring joint

Typically, the pressure of the thrust cylinders exceeds the pressure necessary for ring assembly. Therefore, when using segment bolts, it is necessary to retighten them during the next thrust stroke. When using plug-in dowels, it must be ensured that the applied forces are sufficiently high for the dowel to engage properly.

The ring face of the most recently installed ring should be regularly measured, at least every 10th ring, to determine deviations from the ideal plane. Deviations from the ideal plane of approx. 1-2 mm can often be compensated for during ring assembly – especially if sufficiently soft packers are applied; However, larger deviations require special measures to create an ideal joint plane. The previously common practice of inserting intermediate layers has proven to be only partially effective, as the increased joint gap can lead to sealing problems if the gasket profiles are not designed for such gap. Therefore, this measure should only be used as a last option and should not be considered standard practice.

Key segment installation

From the beginning of ring assembly, it is essential to ensure a dimensionally accurate installation. Depending on the size of the key segment, the use of templates is recommended.

The key segment must be installed without constraint to avoid damage, yet the installation tolerance should not be too large. The use of lubricating soap on the gasket frame to facilitate a stress-free installation should be carefully considered if there is a higher risk for the key segment slipping inwards. A decisive factor is the radial inclination of the longitudinal joint. The risk of the tapered key stone slipping out towards the backside without bolting in the ring joint increases as the key stone becomes smaller and the angle of the longitudinal joint's conicity increases.

11.2.3 TBM advance

Position of the thrust cylinders

When positioning the thrust cylinders on the segments, particular attention must be paid to the position and direction of the jack shoes in both tangential and radial direction. The predefined permissible limits for any rolling between the segment ring and the shield or jack shoe must be observed. The same applies to the radial eccentricities of the jack shoes that have been considered in the segment design. Correcting measures must be initiated in a timely manner before reaching the maximum permissible value for rolling in order to ensure that the thrust cylinder forces are introduced in the designated contact surface areas of the ring joints.

The maximum value for rolling is usually limited to around 2°. With increased effort during tunneling, a limit of around 1° can also be achieved.

For single jacks, care must be taken to ensure that the jack shoe is not mounted in a twisted position on the segment joints. For standard ring assembly, it is essential that the thrust presses acting on a single segment are never released simultaneously.

TBM advance and annular gap grouting

When tunnelling resumes after ring assembly, the last rings built are first loaded by the thrust cylinder forces in the longitudinal tunnel direction and subsequently by the outside pressure from grouting of the annular gap acting in radial direction. The annular gap grouting is part of the annular gap backfilling (definition from RiL853.4006). The annular gap grouting must be carried out continuously during TBM advance and must be operated to ensure an early bedding of the ring that is leaving the tailskin.

When grouting the annular gap, care must be taken to avoid excessive ring deformation and floating of the tunnel tube. Consequently, both the mix of the annular gap backfill material and the machine control system for the annular gap grouting must be designed accordingly.

Experience shows that most deformations of the segmental lining occur in the early phase after ring erection until setting of the annular gap backfill and activation of the segments' bedding. Since large ring deformations can lead to constraint stresses, it is particularly advisable at the beginning of construction to measure the area of the most recently installed rings to document system behavior. In addition to convergence measurements, additional joint offset and joint opening measurements have proven effective during the first 3 to 5 thrust strokes after ring assembly. Furthermore, crown leveling measurements are useful for quickly detecting potential floating effects.

11.3 Segment damages and repairs

11.3.1 Possible segment damages

Many causes of possible segment damage are well known and should therefore be taken into account in the design phase and afterwards in the production and installation/execution phase.

In the design phase, a robust segment design should be ensured, including:

- with regard to the segment geometry (e.g. acute corners), the arrangement of built-in parts (e.g. gasket framea too close to the segments outer face), the loading (e.g. significant high restoring forces of the gasket frame), the reinforcement layout (e.g. in areas of bolt pockets or embedded components).

During segment production, including:

- Dimensional accuracy must be checked regularly, (e.g. to avoid load peaks in the segments due to exceeding tolerances),
- Care must be taken to prevent voids,
- The groove for the gasket frame must be reworked if necessary,
- Defects must be reworked if the limiting values are exceeded,
- Care must be taken not to overstretch the gasket-frame in the corners.

During the installation/execution phase, including:

- By appropriate monitoring (grout pressure, grout volume, control drillings) insufficient bedding and the resulting ring deformation/ovalization, as well as cracks, spalling, leakages, and misalignments, can be prevented.
- By regular inspections of the dimensions of packers, the risk of segment damage can be reduced.
- By routinely checking the ring face, crack formation due to due to hollow or saddle shaped positioning of adjacent segments can be avoided.

11.3.2 Damage register

For the production of segments, as stipulated in ZTV-ING, Part 7, Section 3, a comprehensive damage catalog must be developed. This catalog shall systematically include various damage categories and scenarios, along with the corresponding remedial procedures. It must include precise definitions for permissible damages as well as for those damages that are repairable and those that are not. In the event of damage, at a minimum, distinctions should be made between nesting, shrinkage voids, cracking, spalling, damage to concrete contact surfaces and/or the gasket groove, and damage to built-in parts.

Furthermore, repair options, procedures and related materials in accordance with the requirements detailed in ZTV-ING, Part 3, Sections 4 and 5 must be specified.

In addition, it should include guidelines for dealing with possible damage from transport, ring assembly and TBM advance.

11.3.3 Root cause analysis

If damage occurs to the segment lining, a root cause analysis must determine whether it is caused by an isolated incident or a systematic issue. A root cause analysis can only be successful if it is based on sufficient data. Therefore, maintaining thorough documentation of the segmental lining process is strongly recommended.

In addition to the production history, the analysis should particularly focus on evaluating the load and deformation history in detail for any anomalies.

TBM data management systems allow to correlate TBM operational data with information on the segment lining, thereby supporting an engineering-based root cause analysis of segmental lining damage.

11.3.4 Acceptance and repairs

In addition to acceptance of the segments upon arrival at the jobsite, they must also be checked for any damage and accepted before and after installation. On the jobsite and in the tunnel, this inspection particularly focuses on the concrete contact surfaces, segment corners, and gasket areas.

To track segments throughout all phases and document any repair measures, including inspections and approvals, the use of scanners and tracking software is recommended. This allows direct integration of data into the management system; see also **ch. 11.1.10**.

If damage is discovered before installation, it must be checked whether the affected segments can be installed as is, require repair or whether they need to be replaced. Segments that do not meet the performance requirements must not be installed.

If damage to a segment is identified during tunnelling or ring assembly, a decision must be made - based on construction logistics, technical feasibility, and predefined damage criteria from the repair concept - whether the segment should be dismantled and replaced. Any repair must comply with the specifications and requirements outlined in the repair concept, with proper documentation of its execution. Before resuming excavation, an inspection must be conducted. Whenever possible, replacing a damaged segment should be prioritized over on-site repair.

Once installed, the segments undergo a damage assessment, with any identified damage classified into predefined categories. This assessment primarily focuses on leakages, spalling, and crack widths, in line with the repair concept.

Typically, crack injections are carried out for water-bearing cracks and for all cracks exceeding 0.3 mm width. Leakages in the gasket area are sealed using curtain injections, inclined borehole injections in the joint area, or direct injections through the gasket frame (see **ch. 4.12**).

Offsets between adjacent segments are generally not addressed for single-shell segmental linings.

Depending on the size and location of spalling, repairs are carried out according to a predefined repair concept.

12 Regulations, standards and publications

12.1 Regulations, standards and guidelines

[1] Deutsche Bahn AG: Richtlinie Eisenbahntunnel planen, bauen und in Stand halten, RiL 853, 9. Aktualisierung, gültig ab 01.09.2018

[2] Zusätzliche Technische Vertragsbedingungen und Richtlinien für den Bau von Straßentunneln (ZTV-ING), Teil 7, Tunnelbau, Abschnitt 3 Maschinelle Schildvortriebsverfahren, Hrsg.: Bundesministerium für Digitales und Verkehr, 2022

[3] DIN-EN-1990 (Eurocode 0): Grundlagen der Tragwerksplanung: 2010-12

[4] Deutsche Bahn AG: Richtlinie Eisenbahntunnel planen, bauen und in Stand halten, RiL 853, 9. Aktualisierung, gültig ab 01.09.2018

[5] Zusätzliche Technische Vertragsbedingungen und Richtlinien für den Bau von Straßentunneln (ZTV-ING), Teil 7, Tunnelbau, Abschnitt 3 Maschinelle Schildvortriebsverfahren, Hrsg.: Bundesministerium für Digitales und Verkehr, 2022

[6] DIN-EN-1990 (Eurocode 0): Grundlagen der Tragwerksplanung: 2010-12

[7] DIN EN 1990/NA, Nationaler Anhang – National festgelegte Parameter – Eurocode: Grundlagen der Tragwerksplanung, 2010-12

[8] DIN EN 1990/NA, Nationaler Anhang – National festgelegte Parameter – Eurocode: Grundlagen der Tragwerksplanung, 2010-12

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