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ACI 533.5R-20

Guide for Precast Concrete Tunnel Segments

Reported by ACI Committee 533



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Guide for Precast Concrete Tunnel Segments

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Guide for Precast Concrete Tunnel Segments

Reported by ACI Committee 533

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The worldwide trend in construction is toward mechanization and automation. This trend has led to continued rapid progress of mechanized tunneling. Advantages over conventional tunnel construction methods include, but are not limited to, occupational health and safety, faster advance rates, and reducing construction labor requirements. Mechanized tunneling in soft ground using tunnel boring machines is often associated with installing precast concrete segmental lining. However, very little industry-wide guidance has been provided by practice and code organizations. This document provides guidelines for precast concrete tunnel segments, including the most recent developments and practical experience, in addition to information on all aspects of design and construction. These guidelines are based on the knowledge and the experience gained on numerous precast tunnel projects in the United States, and available national and international guidelines often used as industry references.

Keywords: design; durability; fiber; gasket; joint; lining; precast; segment; tolerance; tunnel.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

Precast concrete segments are installed to support the excavation behind the tunnel boring machine (TBM) in soft ground, weak rock, and fractured hard rock applications. As shown in Fig. 1.1, the TBM advances by reacting against the completed rings of precast concrete segments that typically provide both the initial and final ground support as part of a one-pass lining system. These segments are designed to resist the permanent loads from the ground and groundwater as well as the temporary loads from production, transportation, and construction. Currently, very little guidance is provided for tunnel designers and contractors by local or international authorities, and there is an acute need for a document to clearly highlight the practical design principles, advances in construction, and the research needs in this area. Tunnel segments are generally reinforced to resist the tensile and compressive stresses at the ultimate limit states (ULS) and the serviceability limit state (SLS). Special attention is paid in this document to common methods in ULS and SLS designs of these elements. In addition, detailed design considerations are presented, such as concrete strength and reinforcement. Gasket design as sealing elements against groundwater inflow, connection devices, and fastening systems are introduced, followed by segment tolerances, measurement, and dimensional control systems.

1.2—Scope

This document provides analysis, design, and construction guidelines exclusively for one-pass precast segmental lining that is installed almost instantaneously with excavation inside TBM shields only a few yards behind the TBM cutterhead. Linings that are installed long after passing of an open-mode TBM, cast-in-place concrete linings, and segments of other materials such as steel and cast-iron segments do not fall within the scope of this guideline. Two-pass lining systems, which are no longer popular in modern tunnels, are not specifically discussed but can still benefit from the guidelines. More information about the two-pass linings can be found in ITA WG2 guidelines. This guideline provides methods of design and construction for TBM tunneling in soft ground as well as weak and fractured hard rock tunneling. The guidelines and recommendations in this document can be applied to tunnels of different types, such as road, railway, and subway tunnels; headrace, water supply, and waste water tunnels; and service, gas pipeline, and

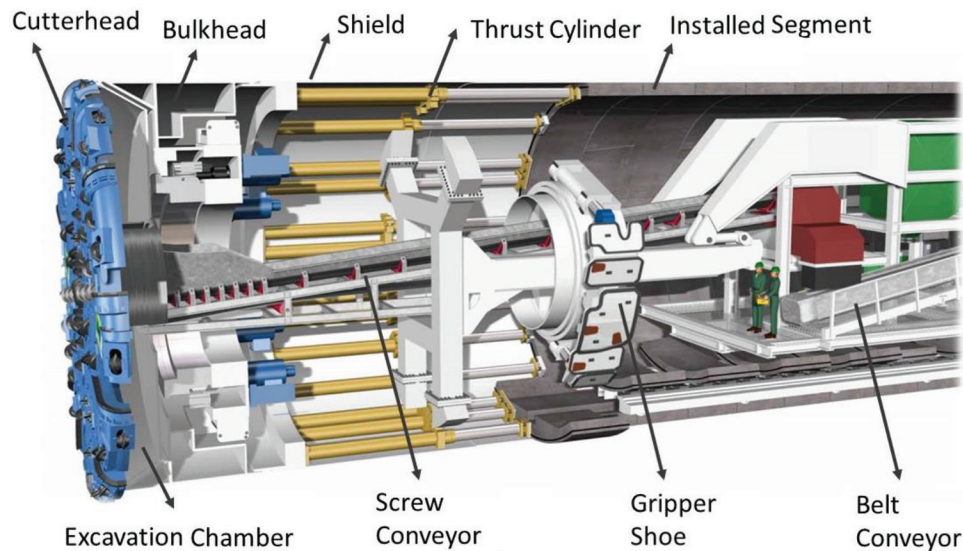


Fig. 1.1—Main parts of a typical TBM of earth-pressure balance (EPB) type, which is used for soft ground tunneling.

power cable tunnels. The structural design part of this document pertains to procedures for designing concrete tunnel segments to withstand the commonly encountered temporary and permanent load cases occurring during the production, transportation, construction, and final service phases. The procedure was developed based on global practice and review of major available design codes, standards, and guidelines related to precast segments in tunneling and concrete industries. The construction aspects presented in this guideline including segmental ring geometry and systems, gasket systems, and connection devices, and segment tolerances reflect global practice perspectives such as [ACI 544.7R](#), [AFTES:2005](#), [BS PAS 8810:2016](#), [DAUB:2013](#), [JSCE 2007](#), [LTA 2010](#), [ÖVBB 2011](#), and [STUVAtec:2005](#). This document does not address the actions of thermal variations, fire loads and explosion, or internal loads such as train loads within the tunnels. While some structural design parts of this guideline may only consider the procedures adopted by ACI, they can be extended to other structural codes such as [BS EN 1992-1-1:2004](#).

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A	= effective tension area of concrete around reinforcing bar divided by number of steel bars, in. ² (mm ²)	a_t	= transverse length of stress distribution zone at the centerline of segment under thrust jack forces, in. (mm)
A_d	= load distribution area inside segment under thrust jack forces, in. ² (mm ²)	b	= width of tunnel segment or width of tested specimen, ft (m)
A_g	= gross area of concrete section, in. ² (mm ²)	C_c	= compression force in the concrete section, lbf (N)
A_j	= area of contact zone between jack shoes and the segment face, in. ² (mm ²)	C_t	= tensile force in the section due to fiber reinforcement, lbf (N)
A_s	= area of reinforcing bars, in. ² (mm ²)	D_e	= external diameters of the tunnel segmental lining, ft (m)
a	= distance from edge of vacuum lift pad to edge of segment in the load case of stripping (demolding), or dimension of final spreading surface under thrust jack forces, in. (mm)	D_i	= internal diameter of the tunnel segmental lining, ft (m)
a_l	= transverse length of contact zone between jack shoes and the segment face, in. (mm)	d	= thickness of tested specimen, or total width of the segment cross section, in. (mm)
		d_1	= length of load transfer zone for the case of longitudinal joint bursting load, in. (mm)
		d_{burst}	= centroidal distance of bursting force from the face of section, in. (mm)
		d_c	= concrete cover over reinforcing bar, in. (mm)
		d_k	= width of the hinge joint or thickness of contact surface between segment joints for the case of longitudinal joint bursting load, in. (mm)
		d_s	= distributed width of stress block inside the segment for the case of longitudinal joint bursting, in. (mm)
		E	= modulus of elasticity of concrete, psi (MPa)
		E_r	= modulus of elasticity of surrounding ground, psi (MPa)
		E_s	= stiffness modulus of the surrounding ground determined by oedometer test, psi (MPa); or modulus of elasticity of reinforcing bar, psi (MPa)
		EH	= horizontal earth pressure, psi (MPa)
		EV	= vertical earth pressure, psi (MPa)
		e	= eccentricity, in. (mm)
		e_{anc}	= eccentricity of jack pads with respect to the centroid of cross section, or maximum total eccentricity in longitudinal joints consisting of force eccentricity and eccentricity of load transfer, in. (mm)

F	= forces acting on bottom segment due to self-weight of segments positioned above when segments are piled up within one stack during storage or transportation phases, lbf (N)	k_r	= radial component of subgrade reaction modulus or stiffness of radial springs simulating ground-structure interaction, lb/ft ³ (kg/m ³)
F_{sd}	= bursting tensile forces developed close to longitudinal joints, lbf (N)	k_t	= subgrade reaction modulus in the tangential direction, lb/ft ³ (kg/m ³); or in crack width analysis, a factor depending on the duration of loading (0.6 for short-term loading and 0.4 for long-term loading)
$F_{sd,r}$	= spalling tensile forces developed close to longitudinal joints, lbf (N)	k_0	= tangential component of subgrade reaction modulus or stiffness of tangential springs simulating ground-structure interaction, lb/ft ³ (kg/m ³)
$F_{sd,2}$	= secondary tensile forces developed close to longitudinal joints, lbf (N)	L	= distance between the supports, in. (mm)
f_1	= first peak flexural strength, psi (MPa)	l_t	= full length of contact area between segments in longitudinal joints, in. (mm)
f_{bot}	= stress at the extreme bottom fiber of concrete section, psi (MPa)	$M_{distortion}$	= bending moment due to additional distortion effect, lbf.ft (N.m)
f_c'	= specified compressive strength of concrete segment, psi (MPa)	M_n	= nominal resistance bending moment, lbf.ft (N.m)
f_{co}'	= compressive strength of partially loaded concrete surface, psi (MPa)	N	= axial hoop force in segments, lbf (N)
f_{cd}	= concrete design compressive strength according to BS EN 1992-1-1:2004, psi (MPa)	N_{Ed}	= maximum normal force due to permanent ground, groundwater, and surcharge loads, lbf (N)
f_{ctd}	= fiber-reinforced concrete design tensile strength, psi (MPa)	n	= number of segments per ring excluding the key segment ($n \geq 4$); or number of layers of tensile reinforcing bar in crack with analysis
$f_{ct,eff}$	= concrete tensile strength, psi (MPa)	P_0	= surcharge load, lbf (N)
f_{150}^D	= residual flexural strength at net deflection of $L/150$, psi (MPa)	P_{e1}	= vertical earth pressure at crown of lining applied to the elastic equation method, psi (MPa)
f_{600}^D	= residual flexural strength at net deflection of $L/600$, psi (MPa)	P_{e2}	= vertical earth pressure at invert of lining applied to the elastic equation method, psi (MPa)
f_{150}^{nD}	= specified residual flexural strength at net deflection of $L/150$, psi (MPa)	P_g	= segment dead load, psi (MPa)
f_{600}^{nD}	= specified residual flexural strength at net deflection of $L/600$, psi (MPa)	P_{gr}	= radial grouting pressure, psi (MPa)
f_{150r}^D	= required average residual flexural strength at net deflection of $L/150$, psi (MPa)	P_{pu}	= factored jacking force applied on each jack pad in circumferential joints, or maximum factored normal force from the final service loads transferred in longitudinal joints, psi (MPa)
f_{600r}^D	= required average residual flexural strength at net deflection of $L/600$, psi (MPa)	P_{w1}	= vertical water pressure at crown of lining applied to the elastic equation method, psi (MPa)
f_{Ftu}	= fiber-reinforced concrete tensile strength at ultimate limit state, psi (MPa)	P_{w2}	= vertical water pressure at invert of lining applied to the elastic equation method, psi (MPa)
f_{R1}	= residual flexural strength of FRC beam corresponding to crack mouth opening displacement of 0.02 in. (0.5 mm), psi (MPa)	q_{e1}	= horizontal earth pressure at crown of lining applied to the elastic equation method, psi (MPa)
f_{R3}	= residual flexural strength of fiber-reinforced concrete beam corresponding to crack mouth opening displacement of 0.1 in. (2.5 mm), psi (MPa)	q_{e2}	= horizontal earth pressure at invert of lining applied to the elastic equation method, psi (MPa)
f_s	= stress in reinforcing bar, psi (MPa)	q_{w1}	= horizontal water pressure at crown of lining applied to the elastic equation method, psi (MPa)
f_t	= specified splitting tensile strength, psi (MPa)	q_{w2}	= horizontal water pressure at invert of lining applied to the elastic equation method, psi (MPa)
f_y	= yield stress of required reinforcing bars, psi (MPa)	R	= radius from centerline of lining, ft (m)
g	= self-weight of the segments per unit length, lbf/in. (N/mm)	r_o	= radius of excavated tunnel, ft (m)
H	= overburden depth, ft (m)	S	= distance between stack supports and free edge of segments in the load case of segment storage, ft (m)
H_w	= groundwater depth, ft (m)	s	= maximum reinforcing bar spacing, in. (mm)
h	= thickness of tunnel segment, in. (mm)	$s_{r,max}$	= maximum crack spacing, mm
h_{anc}	= length of contact zone between jack shoes and the segment face, in. (mm)	s_s	= sample standard deviations of test results
I	= moment of inertia of FRC segment, in. ⁴ (mm ⁴)	T_{burst}	= bursting force, lbf (N)
J	= tunnel boring machine thrust jack forces, kip (kN)	WA_p	= groundwater pressure, psi (MPa)
k	= coefficient of subgrade reaction or subgrade reaction modulus, lb/ft ³ (kg/m ³)		
k_{jr}	= Janssen rotational spring stiffness in longitudinal joints, lb.in./rad (N.mm/rad)		

w	= segment self-weight, lb/ft (kg/m); or maximum crack width, in. (mm)
y	= distance from extreme tension fiber to the neutral axis, in. (mm)
y_c	= distance from extreme compression fiber to centroid of equivalent compression force in the section, in. (mm)
β	= dimension of the loaded surface under thrust jack forces according to Iyengar diagram, in. (mm); or in crack width analysis ratio of the distance between neutral axis and tension face to the distance between neutral axis and centroid of reinforcing bar
$\Delta P_{g, invert}$	= vertical gradient of radial grout pressure between the crown and invert of tunnel, psi (MPa)
δ	= displacement of lining applied to the elastic equation method, in. (mm)
δ_d	= diametrical distortion, in. (mm)
ϵ'_{csd}	= compressive strain due to shrinkage and creep equal to 150×10^{-6}
ϵ_{cu}	= ultimate tensile strain
ϵ_{ru}	= ultimate compressive strain
ϕ	= strength reduction factor; or reinforcing bar diameter, in. (mm)
γ	= material safety factor
λ	= slenderness defined as the ratio between the developed segment lengths and its thickness
θ	= angle from crown in the elastic equation method, or rotation in the longitudinal Janssen joint, radians
$\rho_{concrete}$	= specific weight of concrete, lb/ft ³ (kg/m ³)
ρ_{eq}	= equivalent specific weight of grout, lb/ft ³ (kg/m ³)
$\sigma_{c,j}$	= compressive stresses developed under jack pads because of axial effects of thrust jack forces, psi (MPa)
σ_{cm}	= fully spread compressive stress in method of the Iyengar diagram, psi (MPa)
σ_{cx}	= bursting tensile stresses using the Iyengar diagram, psi (MPa)
σ_p	= specified post-crack residual tensile strength of fiber-reinforced concrete (FRC) segment, psi (MPa)
τ_{yield}	= shear yield strength of grout, psi (MPa)

2.2—Definitions

Please refer to the latest version of ACI Concrete Terminology for a comprehensive list of definitions. Definitions provided herein complement that resource.

annular gap—space between the surrounding ground and the outer surface of the segments.

circumferential joint—joint approximately perpendicular to the tunnel axis between two adjacent segment rings.

connections—devices for temporary or permanent attachment of two segments or segment rings in the longitudinal and circumferential joints.

counter key segments—two segments installed adjacent to key segment with at least one tapered joint with respect to tunnel longitudinal axis in plan view.

crosscut—connecting structure between two tunnel tubes or between a tunnel tube and the ground surface or a shaft, with special passages in the connecting area of the main tube.

crown—highest part of a tunnel in cross section.

earth-pressure balance tunnel boring machine—one type of tunnel boring machine used in soft ground tunneling; uses a screw conveyor and with controlling muck removal from the excavation chamber, the earth pressure in the chamber is maintained to balance the face pressure.

extrados—outer surface of the segment or the segment ring on the side in contact with the ground.

gasket—sealing system consisting of sealing strips placed in one or more layers around the individual segment, ensuring permanent sealing of the tunnel tube against the ingress of water from the surrounding ground.

guiding rod—segment accessories in the shape of rods, often 1 to 2 in. (25 to 50 mm) in diameter; placed in longitudinal joints along the centroid of two adjacent segments to fulfill the functions of guidance and locking adjacent segments during installation of a full ring inside tunnel boring machine shield.

ground—soil, rock, and fill into which the tunnel is placed.

intrados—inner surface of the segment or the segment ring on the tunnel side.

invert—lowest part of a tunnel in cross section.

joint misalignment—eccentricity between end of two segments at longitudinal or circumferential joints that results in limited contact areas between segment ends at joints.

key segment—last installed segment of a ring with a trapezoidal shape in plan view, which is often smaller than and accounted for as a proportion of ordinary segments such as one-third, two-thirds, or half.

longitudinal joint—joint between adjacent segments in a ring with an axis parallel to the longitudinal axis of tunnel; also known as radial joint.

one-pass lining—all static and structural requirements of the tunnel lining are handled by the segmental ring; no further internal lining is installed that contributes to load bearing or sealing.

ovalization—deformation of an initially circular segmental ring; for example, to a vertical or horizontal oval shape due to earth pressure, grout pressure, segment self-weight, or uplift.

packer—semi-rigid boards made of polyethylene or fortified asphalt core pressed between two layers of weather-proofed fiberglass plies or timber materials that are placed between tunnel segmental ring joints; they are used to relieve the stresses between segments and therefore prevent cracking and spalling. Packers are not used as often in modern segmental lining construction.

portal—entrance from the ground surface to a tunnel.

reverse key segment—first installed segment of a ring, in rectangular or trapezoidal shape, located opposite to key segment in segmental ring side view and oriented very close to tunnel invert.

ring width (ring length)—dimension of the segment ring in its center axis in the longitudinal direction of the tunnel.

segment—curved prefabricated elements that make up a ring of support or lining.

segment thickness—radial distance between the inner and outer sides of a segment.

shield—steel tube, usually cylindrical, shaped to fit the excavation line of a tunnel.

soft ground—residual soil or deteriorated rock with limited compressive strength and stand-up time.

springline—opposite ends of the horizontal centerline of tunnel.

tail void—annular space between the outside diameter of the shield and the outside of the segmental lining.

tunnel boring machine—consisting of a cutterhead, shield, and gentries used to excavate tunnels with a circular or rectangular cross section through different rock and soil strata, and to install the tunnel lining at the end of the shield.

tunnel boring machine backup—area behind tunnel boring machine shields in the shape of an equipment train that is used for providing a final staging area for feeding segments to the installation erectors as well as housing tunnel boring machine ancillary equipment such as transformers, power supply, hydraulic pumps, control room, ventilation, trail skin grouting, and spoil (muck) removal systems needed for the tunnel boring machine operation.

test ring—complete segment ring, usually assembled in horizontal orientation in segment precast plant, for test purposes.

thrust jacks—hydraulic jacks serving to transmit the thrust forces of the tunnel boring machine to the segment ring, facilitate installation, or both.

tunnel cover—perpendicular distance to nearest ground surface from the tunnel exterior.

two-pass lining—tunnel lining consisting of two shells with different structural and constructional requirements that are produced in independent operations and with different construction methods.

CHAPTER 3—DESIGN PHILOSOPHY AND SEGMENTAL RING GEOMETRY

3.1—Load and resistance factor design

The design engineer should use load and resistance factor design (LRFD) method to design concrete precast tunnel segments. LRFD is a design philosophy that takes into account the variability in the prediction of loads and the variability in the properties of structural elements. LRFD employs specified limit states to achieve its objectives of constructability, safety, and serviceability. In **BS EN 1992-1-1:2004**, this is defined as limit state design.

Even though force effects may often be determined using elastic analyses, the resistance of elements using LRFD design methods is determined on the basis of inelastic behavior. Concrete precast tunnel segments should be designed using load factors and strength reduction factors specified in concrete design codes such as **ACI 318**. For load cases not covered in these codes, load factors, load combi-

nations, and strength reduction factors from other resources such as **ACI 544.7R** or **AASHTO DCRT-1** can be used.

3.2—Governing load cases and load factors

The current practice in the tunnel industry is to design segmental tunnel linings for the following load cases, which occur during segment manufacturing, transportation, installation, and service conditions:

- a) Production and transient stages
 - i. Segment stripping (demolding)
 - ii. Segment storage
 - iii. Segment transportation
 - iv. Segment handling
- b) Construction stages
 - i. Tunnel boring machine (TBM) thrust jack forces
 - ii. Tail skin back grouting pressure
 - iii. Localized back grouting (secondary grouting) pressure
- c) Service stages
 - i. Ground pressure, groundwater pressure, and surcharge loads
 - ii. Longitudinal joint bursting load
 - iii. Loads induced due to additional distortion
 - iv. Other loads (for example, earthquake, fire and explosion, TBM load of upper tunnel to lower tunnel in case of stacked arrangement of tunnels, aerodynamic loads, mechanical and electrical loads, railway loads, temperature load, and loads during segmental ring erection)

In the strength design procedure, the required strength (U), also known as required design strength, is expressed in terms of factored loads such as the ones shown in Table 3.2 for presented governing load cases. Note that this table provides comprehensive factored load combinations for a specific case of tunnel segments. If different load factors are provided by the local codes, they should be used in place of the factors in this table. In this document, aforementioned load cases are divided into three categories: production and transient loads, construction loads, and service loads. The resulting axial forces, bending moments, and shear forces are used to design concrete and reinforcement.

3.3—Design approach

A common design approach for concrete tunnel segments starts with selecting an appropriate geometry, including thickness, width, and length of segments with respect to the size and loadings of the tunnel. Considering specified compressive strength (f'_c) and type and amount of reinforcement, the design strength of segments is compared with required strength against all critical load cases. Methods of calculation for required strength against these load cases will be explained in the following chapters. The geometry, compressive strength, and reinforcement of segments should be specified to provide sufficient design strength against all load cases as well as satisfying all service conditions. The design procedure starts with initial considerations for a segmental ring system and geometry that is discussed in the following section and further checked against the different loadings.

Table 3.2—Required strength (U) for governing load cases (ACI 544.7R)

Load case	Required strength (U)
Load Case 1: stripping (demolding)	$U = 1.4w$
Load Case 2: storage	$U = 1.4(w \pm F)$
Load Case 3: transportation	$U = 1.4(w \pm F)$
Load Case 4: handling	$U = 1.4w$
Load Case 5: thrust jack forces	$U = 1.0J$ (1.2 if maximum machine thrust is unknown)
Load Case 6: tail skin grouting	$U = 1.25(w \pm P_{gr})$
Load Case 7: secondary grouting	$U = 1.25(w \pm P_{gr})$
Load Case 8: earth pressure and groundwater load	$U = 1.25(w \pm WA_p) \pm 1.35(EH + EV) \pm 1.5P_0$
Load Case 9: longitudinal joint bursting	$U = 1.25(w \pm WA_p) \pm 1.35(EH + EV) \pm 1.5P_0$
Load Case 10: additional distortion	$U = 1.4M_{distortion}$

3.4—Segmental ring geometry and systems

Segmental tunnel linings installed in the rear of the TBM shield are generally in the shape of circular rings. The size of the ring is defined by the internal diameter, thickness, and length of the ring. Other important design considerations include ring systems, ring configurations in terms of number of segments that form a complete ring, geometries of individual segments, and geometry and tapering of key segments (Bakhshi and Nasri 2018b).

3.4.1 Internal diameter of the bored tunnel—The dimensions of the tunnel inner section should be determined considering the internal space required during the service, which depends on the intended use of the tunnel. For the railroad and subway tunnels, the inner dimensions of tunnels in a single-track case are generally governed by the train clearance envelope (clearance gauge), track structure, drainage trough, structure of the overhead catenary contact line stays, and emergency evacuation corridor (egress space). In a double track and twin tunnel cases, tunnel inner dimensions are additionally governed by distance between the centers of tracks and the cross passageway. The internal diameter of the tunnel is first set by obtaining a circle that satisfies these conditions. Then, the electrical equipment, water pipes, and other equipment are installed in the unoccupied space inside this circle. Sufficient ventilation space is generally provided if egress space and cross passageways are allocated (RTRI 2008), but this needs to be verified. For the road tunnels, the geometrical configuration of the tunnel cross section should satisfy the required horizontal and vertical traffic clearances; shoulders or sidewalks/curbs; barriers; fans and suitable spaces for ventilation, lights, traffic control system, and fire life safety systems including water supply pipes for firefighting, cabinets for hose reels, fire extinguishers, and emergency telephones. As shown in Fig. 3.4.1, the smallest tunnel encircling these clearances and elements are considered as the minimum internal tunnel diameter. The available spaces in a circular cross section can be used to house other required elements for road tunnels including tunnel drainage, tunnel utilities and power, signals and signs above roadway lanes, CCTV surveillance cameras, communication antenna and equipment, and monitoring equipment of noxious emissions and visibility (AASHTO DCRT-1). If the

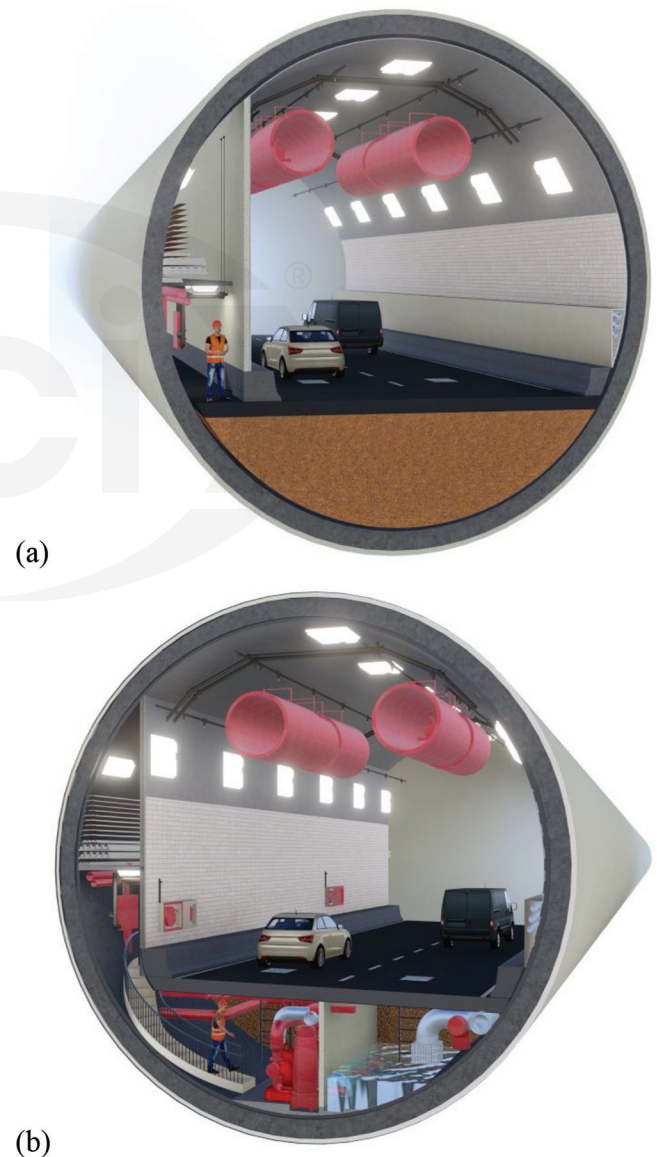


Fig. 3.4.1—Schematics of interior space of TBM-bored road tunnels: (a) typical section; and (b) section at low-point pump station.

space is not sufficient for all these elements, the tunnel needs to be enlarged accordingly. Note that for the spaceproofing of road and railway tunnels, it is crucial to consider impact of maximum super elevation of the tightest curve on the alignment on the rotation of clearance envelopes. In determining the ring internal diameter, sufficient construction tolerance should also be provided. DAUB:2013 recommends a radius tolerance of $R = \pm 4$ in. (100 mm) for TBM-bored tunnels. Therefore, the internal diameter of the tunnel needs to be made 8 in. (200 mm) greater than the required internal structural boundary. The internal size of the wastewater or combined sewer overflow tunnels is designed based on the storage volume and sometimes volumetric flow rate of design storm (for example, 25-year, 50-year, and 100-year) specified by local authorities and updated collection system modeling. The internal diameter of the water tunnel is sized based on the volumetric designed flow rate.

3.4.2 Thickness of ring and outside diameter—The thickness of segmental lining ring as a structural member is determined in accordance with the result of design calculation. The segment design is an iterative procedure that starts with an assumption of a reasonable thickness and is later optimized during detailed design calculation. Therefore, it is crucial to consider a reasonable thickness for segmental rings in the beginning of the design process. A review of more than 100 projects published in ACI 544.7R, AFTES:2005, Groeneweg (2007), and Blom (2002) shows that the ratio of internal tunnel diameter (ID) to the lining thickness falls in a specific range of 18 to 25 for tunnels with an ID of more than 18 ft (5.5 m), and 15 to 25 for tunnels with an ID of 13 to 18 ft (4 to 5.5 m). JSCE 2007 recommends that the ring thickness be less than 4 percent of the outer diameter of segmental ring, which translates into an ID to thickness ratio of 23. Most of tunnels are larger than 13 ft (4 m) in internal diameter and therefore it is suggested to consider one-twentieth of the ID as the initial lining thickness. For tunnels under 13 ft (4 m) in diameter, no correlation could be identified between the lining thickness and the tunnel diameter, which ranges between 5.9 and 11 in. (150 and 280 mm).

During the analysis and design stage, the capacity of the lining section with the selected thickness should be sufficient when transverse reinforcement ratio is less than 1 percent and close to the minimum reinforcement (AFTES:2005). The minimum segment wall thickness should satisfy the conditions imposed by the contact joints such as sufficient bearing surface area and sufficient space and clear distance for segment recesses. The minimum segment wall thickness should be compatible with the bearing surface area of TBM longitudinal thrust cylinders (AFTES:2005). To achieve a robust design, the segment thickness should be adequate to all requirements with an extra amount for unforeseen loads, particularly if sealing gaskets are installed. In addition to structural factors, the lining thickness is also designed based on durability, and DAUB:2013 recommends a minimum thickness of 12 in. (300 mm) for one-pass tunnel linings. Note that in combined sewer overflow tunnels, if a sacrificial layer was considered for design life of the tunnel (for

example, 125 years), the sacrificed layer thickness should be added to required structural thickness.

The outer diameter of a tunnel is determined by adding the lining thickness to the inner dimension. The shield outer diameter is determined by adding the tail clearance and shield skin plate thickness to the tunnel outer diameter (RTRI 2008). Shield outer diameter also limits the minimum curve radius of the alignment. A review of more than 100 tunnel projects with different sizes (JSCE 2007) shows that when shield outer diameter is less than 20 ft (6 m), between 20 and 32 ft (6 and 10 m), and more than 40 ft (12 m), the minimum curve radius can be limited to 260 ft (80 m), 520 ft (160 m), and 990 ft (300 m), respectively. In practice, larger radii are being considered in projects and mentioned limits can be used as lower bound limits for the curve radius. It is noteworthy to mention that minimum curve radius limitation is a function of ring geometry (taper and ring width), overcut, shield design (articulated or not), and radial gap between segment and tail skin rather than just the shield outer diameter. All these parameters should be taken into account for determining minimum curve radius. In addition, curves can be both horizontal and vertical but usually horizontal curves are the tightest curves on tunnel alignments. For transportation tunnels, the minimum radii are generally governed by the policy documents published by the transportation agency, which include considerations of various safety-related factors such as the speed of vehicle, line of sight, length of curve, vehicle characteristics, and drainage considerations.

3.4.3 Length of the ring—Depending on the diameter, the ring length can range between 2.5 ft and 8 ft (0.75 and 2.50 m) (DAUB:2013). On one hand, it is desirable that the ring length be narrow for transportation and erection simplicity, easy construction of curved sections, and to reduce length of the shield tail. On the other hand, it is desirable for the ring length to be larger to reduce production cost, numbers of joints, total perimeter of segments, gasket length, and the number of bolt pockets where leakage can occur, as well as increasing the construction speed (JSCE 2007). Therefore, the ring length needs to be optimized for the efficiency of tunnel works. Analysis of data from more than 60 projects presented in JSCE 2007 demonstrates that although in some cases increasing the diameter results in an increased ring length, there is no consistent relationship between the ring length and outer diameter of the segmental lining. This is mainly because, for the smaller diameters, the available space for segment supply and handling defines the limitation of the ring length, whereas for larger diameter, segment weight and production are the limiting factors. The current practice is to use a ring length of 5 ft (1.5 m) for TBM tunnel diameters of 19 to 23 ft (6 to 7 m), and a ring length of 6 ft (1.8 m) for tunnels of 23 to 30 ft (7 to 9 m) diameter. When a TBM larger than 30 ft (9 m) is used for excavation, the most common ring length is 6.5 ft (2 m). Recent large-diameter projects have taken advantage of 7.2 ft (2.2 m) long rings by optimizing the segmental lining thickness. Depending on the project conditions, in many cases, weight limit and rigging

by transportation to the site on public roads are the main considerations.

3.4.4 Segmental ring systems—Parallel rings, parallel rings with corrective rings, right/left rings, and universal ring systems are among different systems used for tunnel segmental rings. Parallel ring systems, as shown in Fig. 3.4.4(a), comprising rings with parallel end faces and with circumferential faces perpendicular to the tunnel axis are not suitable for curved alignments. Practically all tunnel alignments have curves and, therefore, for directional corrections (even in case of curves with large radius), packers are the only solution that can be adopted to be placed in circumferential joints. Such a segmental ring system cannot always be properly sealed because the packing reduces the compression in the gasket. This system is not inherently suitable for curves. It is not suitable either for straight alignments without packers as rings can never be built perfectly straight; and methods to restore line and grade are always needed with this system. Parallel rings with corrective ring systems are similar to parallel ring systems, with corrective rings (up, down, left, right ring) replacing packers for directional corrections. With this system, the requirements for different types of formwork set is the main disadvantage.

The right/left ring is a type of ring that tapered in a way that when the key segment is above the springline, the ring turns in the right/left direction. The most common taper for this system is when the ring has a slightly longer length on the right/left side, respectively. Nonetheless, there are right/left systems with double taper. As shown in Fig. 3.4.4(b), right/left systems are often assembled from rings with one circumferential face perpendicular to the tunnel axis and the other one inclined to the tunnel axis. The difference between maximum and minimum ring length is called taper. Right/left rings have been also made of rings with tapers on both faces of the rings, with half of the required taper on each end face of the rings. The sequence of right-tapered and left-tapered rings produces a straight alignment or a tangent alignment whereas a sequence of right/right ring, as shown in Fig. 3.4.4(b), results in a curve to the right, and left/left rings produces a curve to the left with a minimum system radius. Upward and downward directional corrections are achieved through rotation of the tapered segment ring by 90 degrees (ÖVBB 2011). This ring system provides a proper sealing performance for an impermeable tunnel, with the only disadvantage being requiring different types of formwork set.

The universal ring system is a system in which the key segment can be located anywhere in the tunnel including below the springline. This system can turn the ring into any desired direction: up, down, left, right, and their combinations. The most conventional universal system is the one with the circumferential surface of the ring inclined to the tunnel axis on both sides. However, universal rings with one tapered side are also used. The required ring taper is divided in one or both circumferential ends of the universal rings. As shown in Fig. 3.4.4(c), all curves and directional corrections can be negotiated through the rotation of the segmental ring. The main advantage of this system is the requirement of only

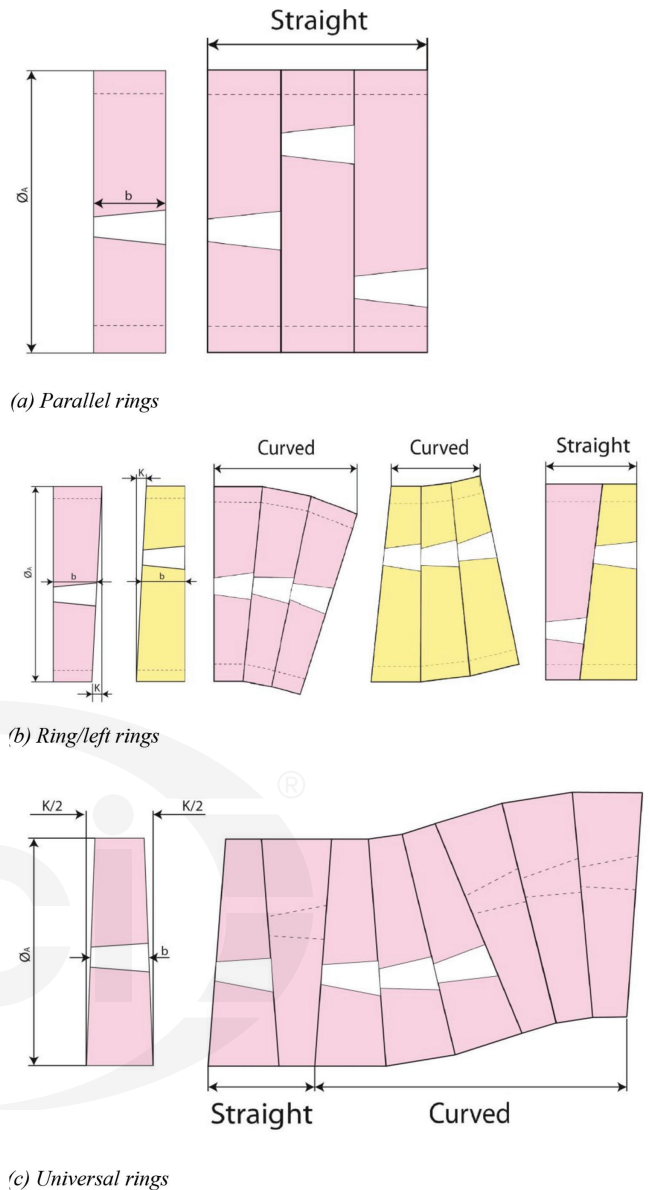


Fig. 3.4.4—Different ring systems plan views and tapering and curve negotiation schematics: (a) parallel rings; (b) right/left tapered rings; and (c) universal rings.

one type of formwork set (ÖVBB 2011). The required ring taper (k) can be calculated with the following formula.

$$k = \frac{\phi_A \cdot b_m}{R} \quad (3.4.4)$$

where ϕ_A is outer diameter of the segment ring; b_m is the average ring length; and R is the minimum curve radius. Note that a correction curve drive that in the event of deviation returns the TBM back into the designed tunnel alignment should be taken into consideration. The correction curve radius should be at least 20 percent less than the smallest desired curve radius horizontally and vertically (DAUB:2013).

To have a straight drive using universal rings, it is necessary to turn each ring by 180 degrees in reference to the

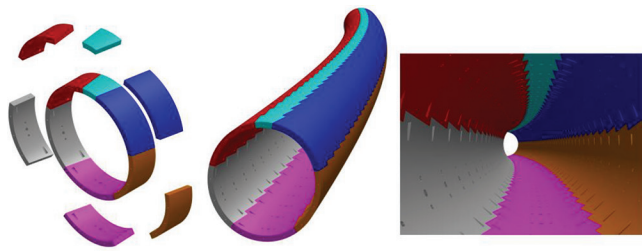


Fig. 3.4.5—Most common ring configuration (5+1) used for mid-size tunnels under 20 ft (6 m) diameter and virtual build of tunnel when negotiating the tightest curve.

previous one, having the key segment both on the top and the bottom or left and right. Taper can be provided alternatively on the right and left sides of the rings and therefore key segment can be placed on the left and right sidewalls to yield a straight drive. Using the right ring and the left ring, it is possible to always have the key segment on the top and, therefore, to be able to construct the ring from the bottom upward. Nonetheless, in recent years and using advanced software for guiding TBM, universal rings can negotiate the straight drives with keys above springlines through adjusting the drive error of less than a few millimeters in two or three rings.

3.4.5 Ring configuration—One of the main parameters for segmental lining design is the number of segments comprising a ring. Similar to the ring length, the shorter the curved length of each segment, the easier the transportation and erection process. Attention should be paid to the fact that ring length and curved segment length are not measured in the same plane. From the handling perspective, it is good to divide a ring to as many segments as possible. However, longer segments and less joints results in a much stiffer segmental ring, reduced production cost, less hardware for segment connection, shorter gasket length, and fewer number of bolt pockets where leakage can occur. More importantly, the construction speed can increase significantly. Normally the space available in the backup to turn the segments is a major factor in setting limits for maximum length of segments. In very large diameter tunnels, however, the segment weight is a decisive factor in selection of maximum length of segments rather than the available space for handling segments inside TBM shields and in the backup gantries. The slenderness of the tunnel segment (λ), defined as the ratio between the breadth or curved length of segment along its centroid and its thickness, is a key parameter for segment length. Review of tunnel projects show that rings are divided into as many segments that yield a segment slenderness of 8 to 13, with fiber-reinforced concrete (FRC) segments around the lower boundary of this range. However, using the latest fiber technologies and high-strength concrete materials, FRC segments with slenderness ratios of 10 are frequently adopted. The latest developments show a record of successful use of FRC segment with slenderness of more than 10 and up to 12 to 13 in some recent projects (ACI 544.7R; Bakhshi and Nasri 2017a; Beño and Hilar 2013; Harding and Francis 2013; ITA Working Group 2 2016). In general, and for midsize tunnels, it is suggested to divide

the ring into as many segments as necessary for a slenderness ratio of at least 10 to be obtained. Segment thickness depends on several factors that are mostly project-specific, such as related design criteria and load cases, resulting in slenderness values outside of the mentioned ranges in some specific cases.

A review of various tunnel projects (Bakhshi and Nasri 2018b) shows that for tunnels with a diameter of 20 ft (6 m) and below, a ring division into six segments is typical. Often, an even number of segments with an odd number of ordinary segments and one key segment are preferred. This type of design is more compatible with configurations of TBM thrust jack forces pushing on the segments. The most common configuration is five ordinary segments and one smaller key segment, also known as 5+1 ring (Fig. 3.4.5). A 4+2 configuration with four ordinary segments and two key segments alternating above and below the springline is also a common configuration. In tunnels under 13 ft (4 m) in diameter, the ring can be easily divided into a fewer number of segments but a division of rings into six segments is still more common. Other than the 5+1 and 4+2 configurations for a six-segment ring, other configurations are sometimes adopted: a 3+2+1 configuration with three ordinary segments (each covering 72 degrees on tunnel perimeter), two counter key segments (each covering 56.5 degrees), and one key segment (covering 31 degrees); or a 6 configuration with all six segments having the same size (each covering 60 degrees on tunnel perimeter) as every other segment can be the key segment in a trapezoidal shape. Usually these configurations are used when the key segment turns out to be too small in a 5+1 configuration. For tunnels of 26 to 36 ft (8 to 11 m) in diameter, a 7+1 configuration is the most common configuration and 6+2 configuration is also common. However, when tunnel diameter ranges between 20 and 26 ft (6 and 8 m), a 5+1 configuration may result in excessively long segments, and a 7+1 configuration in too short segments. In some cases, the size of key segments can be increased to reduce the size of ordinary segments, or a 6+1 ring can be adopted. Similar complexities are encountered when tunnel diameter is between 36 and 46 ft (11 and 14 m) as segmentation of a ring into an 8+1 configuration is not preferred. Special solutions are required, such as dividing the ring into eight segments (each covering 45 degrees on tunnel perimeter) with also dividing one of the ordinary segments into key and counter-key segments (covering 15 and 30 degrees). Using such configurations, excessively large key segments can be avoided, especially for such large diameter tunnel. At the same time, the configuration is compatible with TBM thrust jacking pattern of an eight-segment ring. For tunnels larger than 46 ft (14 m), a 9+1 configuration is the most common configuration with both small key and large key sizes. A 11+1 configuration has been also considered for this size of tunnel.

The common ring configurations explained previously are summarized in Table 3.4.5 according to the category of tunnel sizes. Note that presented ring configurations are the most common cases in practice and there will be exceptions.

Table 3.4.5—Most common ring configuration/segmentation for tunnel segmental lining systems

Range of tunnel diameter	Most common ring configuration/segmentation
Tunnel diameter < 20 ft (6 m)	5+1; 4+2; 3+2+1*
20 ft (6 m) < tunnel diameter < 26 ft (8 m)	6+1
26 ft (8 m) < tunnel diameter < 36 ft (11 m)	7+1; 6+2
36 ft (11 m) < tunnel diameter < 46 ft (14 m)	7+1 large key; 7+1+1†
46 ft (14 m) < tunnel diameter	9+1; 9+1 large key; 11+1

*This configuration consists of three ordinary segments, two slightly smaller counter key segments, and one small key segment.

†In this configuration, key can be one-third of ordinary segments and one of counter keys can be as small as two-thirds of ordinary segments.

lining configurations that fall outside of the presented configurations.

3.4.6 Segment geometry—Segment systems from the perspective of individual segment geometry can be divided into four main categories: hexagonal system, rectangular system, trapezoidal system, and rhomboidal system.

Hexagonal systems are assembled continuously from hexagonal elements, alternating bottom/top and left/right, with each element serving as a key segment by means of closing of a ring (Fig. 3.4.6a). The geometry of this system does not allow effective use of gaskets, and because it compromises the watertightness of the segments, it is not often used. If it is used, it allows very rapid advance rates. In hard rock tunnels without watertightness requirement, sometimes this system is adopted as part of a two-pass lining system and in combination with double-shield TBMs.

Rectangular systems are assembled in rings of rectangular or slightly tapered segments with a wedge-shaped key segment assembled from bottom to top, alternating between left and right (Fig. 3.4.6b). This system can provide proper sealing performance and has some advantages such as simple longitudinal joint geometry. However, staggered longitudinal joints are not always guaranteed and star or crucifix joints may be present that may cause leakage. One issue with this system is that it is difficult to place rectangular segments without impacting the gasket on the adjacent segment. This may also prevent use of fast-connecting dowels and enforce use of time-consuming bolt systems. The main reason is that unlike bolts that are fastened after complete segment insertion into the ring, dowels are preinserted into segments, which limits the segment approach path into a very narrow path and may cause more friction between gaskets of adjacent segments. This system is still in use for large-diameter tunnels where shear capacity of the dowel system connection between the circumferential joints may not be sufficient.

Trapezoidal systems are assembled from an even number of trapezoidal segments in a ring often with the same length at centerline, with half the segments as counter key segments (wider on the side of the previously placed ring) and the other half are key segments (narrower on the side of the previously placed ring). As shown in Fig. 3.4.6c, after installation of all counter key segments as the first part of the ring like an open-tooth shape, the second part is inserted in the gaps to form a complete ring. Advantages of this system are staggered longitudinal joints without encountering star joints and the fact that every other segment can be used as a key segment. The main disadvantage of this system is that the ring is not

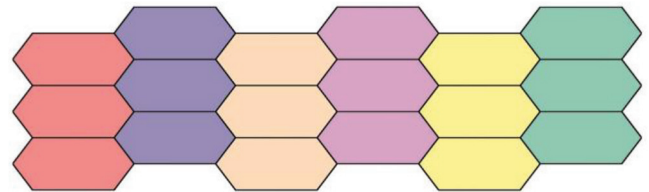


Fig. 3.4.6a—Hexagonal systems (ring developed plan view)

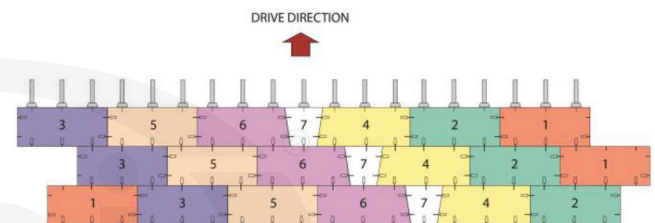


Fig. 3.4.6b—Rectangular systems developed plan view.

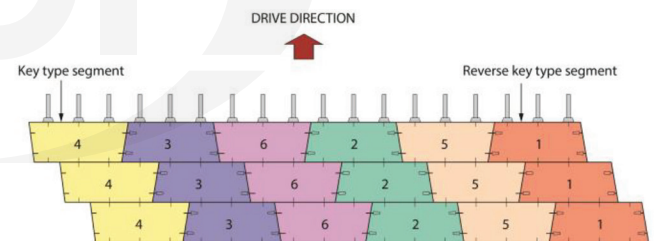


Fig. 3.4.6c—Trapezoidal systems developed plan.

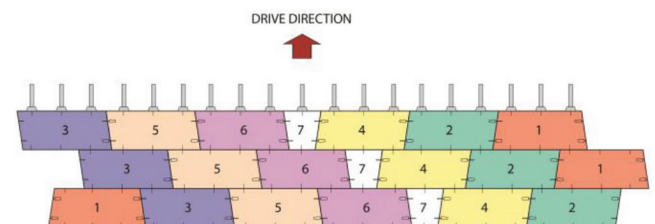


Fig. 3.4.6d—Rhomboidal or parallelogrammatical and trapezoidal systems developed plan.

built continuously, which makes it difficult to place several key segments between the counter key segments.

Rhomboidal or parallelogrammatical-trapezoidal systems are assembled from ordinary segments in the shape of parallelogram and a key and a counter key segment in the shape of trapezoid. As shown in Fig. 3.4.6d, the tunnel is built continuously ring by ring and often from bottom to top. The assembly procedure starts with installing a

ezoidal reverse key segment and placing parallelogrammatical segments next to the counter key first and then next to previous parallelogrammatical segments alternating left and right. Ring assembly is completed with often smaller trapezoidal key segment. This system is now the most common system because of preventing crucifix joints and improved sealing performance, continuous ring build from bottom to top, and compatibility with dowel connection system. Note that angled segment joints provide another major advantage, which is avoiding early rubbing of the gaskets during segment insertion in the ring assembly phase, and facilitates use of fast connecting dowels in circumferential joints. Regardless of the shape of segments, the bolted connections are usually needed for longitudinal joints but not needed for circumferential joints in this configuration. In certain cases, longitudinal bolts are replaced by guiding rods, which will be explained in the following chapters.

3.4.7 Key segment geometry—The key segment is the last installed segment of a ring, always with a trapezoidal shape in plan view to provide an easy path for sliding into the ring. The key segment is often smaller than ordinary segments and is a proportion of ordinary segments such as one-third, two-thirds, or one-half, but it can also be of a different ratio with respect to ordinary segments. **ITA WG2** and **JSCE 2007** present two different key segment tapering geometries according to methods historically used for the ring assembly. One method that is not typically used is to insert the segments from the inside of the tunnel, in which the longitudinal side faces of the key segment are tapered in the direction of the tunnel radius. ITA WG2 provides a geometry formulation that is specified towards key segment insertion in the radial direction. The other method, which is common practice, is to insert the segments from the cut face side in the longitudinal direction of the tunnel in which the longitudinal side faces of the key segment are tapered in the longitudinal direction of the tunnel. Using this method, the key segment tapering is defined by the angle of the side faces with respect to joint centerline or longitudinal tunnel axis. Depending on designer and contractor experience, this tapering can be selected differently. A key segment taper angle of 8 to 12 degrees is recommended based on a review of recent projects. Note that when adopting a rhomboidal or parallelogrammatical-trapezoidal system, the same joint angle for a key segment should be used for defining the geometry of other segments in the ring.

CHAPTER 4—DESIGN FOR PRODUCTION AND TRANSIENT STAGES

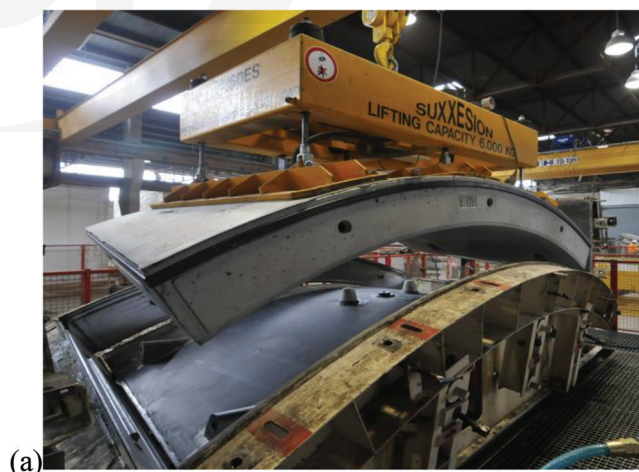
Production and transient loadings include the loading stages starting from the time of segment casting up to the time of the segment erection within the tunnel boring machine (TBM) shield. During these phases, the internal forces and stresses from stripping (demolding), storage, transportation, and handling are used for the design of precast concrete segments. Production and transient loads during these stages result in significant bending moment with no axial forces.

Figure 4a(a) shows the stripping (demolding) phase that is modeled by two cantilever beams loaded under their own

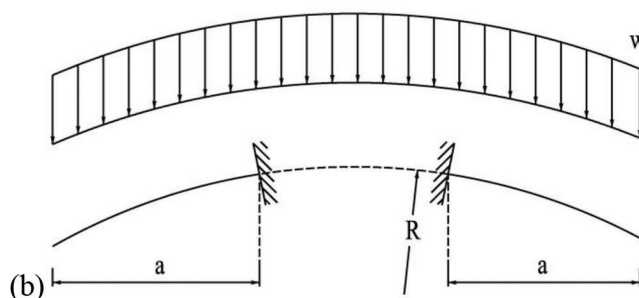
self-weights (w). The design is performed with regard to the specified strengths when segments are stripped or demolded (that is, 6 hours after casting). As shown in Fig. 4a(b), the self-weight (w) is the only force acting on the segment and, therefore, the applied load factor in ultimate limit state (ULS) can be taken as 1.4 (**ACI 544.7R**).

Segment stripping (demolding) is followed by the segment storage phase in the storage yard at the precast plant where segments are stored to gain specified strength before transportation to the construction site. As shown in Fig. 4b(a), in the most common scheme, all segments comprising a full ring are piled up within one stack. Designers, in coordination with the segment manufacturers, provide the distance between the stack supports considering an eccentricity of $e = 4$ in. (100 mm) between the locations of the stack support for the bottom segment and the supports of above segments. This load case can be represented by a simply supported beam loaded under its self-weight as shown in Fig. 4b(b). The dead weight of segments positioned above (F) acts on the designed segment as a concentrated load in addition to its self-weight (w). Therefore, corresponding load combination can be considered as $1.4w + 1.4F$ (**ACI 544.7R**).

During the segment transportation phase, stored precast segments in the storage yard are transported to the construction site and TBM trailing gear. Segments may encounter dynamic shock loads during this phase and, as shown in Fig. 4b(a), half or all of the segments of each ring are transported on one dolly. Wood blocks provide supports for the segments. An eccentricity of 4 in. (100 mm) is recommended



(a)



(b)

Fig. 4a—(a) Stripping (demolding) segments from forms, and (b) forces acting on segments.

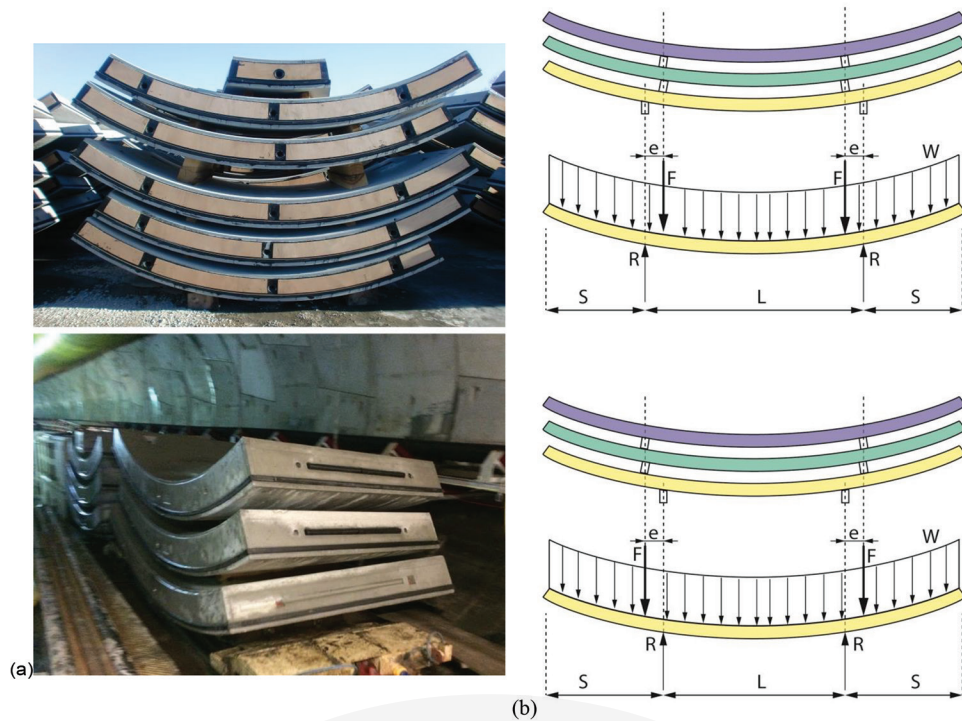


Fig. 4b—(a) Segments stacking for storage and transportation; and (b) schematics of forces acting on bottom segment.

Table 4—Summary of required design checks and factors for production and transient stages

Load case number	Phase	Dynamic impact factor	Maximum unfactored bending moment	Maximum unfactored shear force
1	Stripping (demolding)	—	$wa^2/2$	wa
2	Storage	—	$w(L^2/8 - S^2/2) + F_1e w(S^2/2) + F_1e$	$\frac{wS}{wL/2 + F_1}$
3	Transportation	2.0	$w(L^2/8 - S^2/2) + F_2e w(S^2/2) + F_2e$	$\frac{wS}{wL/2 + F_2}$
4	Handling (forklift)	2.0	$w(L^2/8 - S^2/2) + F_2e w(S^2/2) + F_2e$	$\frac{wS}{wL/2 + F_2}$
	Handling (other methods)		$wa^2/2$	wa

Note: F_1 is the self-weight of all segments completing a ring, excluding the bottom segment; F_2 is the self-weight of all segments carried by forklifts or placed in one carriage for transportation phase, excluding bottom segment.

for design. Note that the wood blocks should be installed nearly parallel to the segment axis. Similar to segment storage phase, simply supported beams represent the load case of transportation with dead weight of segments positioned above (F) and self-weight (w) as the acting loads on designed segment. In addition to load combination of $1.4w + 1.4F$ per ACI 544.7R, a dynamic impact factor of 2.0 is recommended to be applied to the F force for the transportation phase.

Segment handling inside the precast plant and from storage yard to trucks or rail cars are carried out by specially designed lifting devices such as vacuum lifters, forklifts, mechanical clamping, or a combination of them (Fig. 4c). For handling by vacuum lifters and mechanical clamping, the same analysis and design procedure used for segment stripping (demolding) should be followed. However, when segments are handled by forklifts as schematically shown in Fig. 4c(d), a loading scheme similar to segment stacking for

storage and transportation (Fig. 4b(b)) should be adopted. In this case, the total eccentricity is equal to sum of spacing of forklift contact point from the wood blocks axis and the minimum design eccentricity of $e = 4$ in. (100 mm). A dead load factor of 1.4 in ULS and a dynamic impact factor of 2.0 are recommended for this load case (ACI 544.7R).

The maximum bending moment and shear forces developed during the aforementioned stages are used for design checks. In addition, as shown on Fig. 4c(e), pullout capacity of lifting inserts and concrete should be calculated. The plastic insert used to lift the segment from its center will be subject to the full self-weight of the segment under dynamic loading conditions. Nominal concrete breakout strength of a single bolt in tension, as shown in diagram of Fig. 4c(e), is calculated following concrete design codes such as ACI 318 and considering an appropriate strength reduction factor. The calculated capacity of concrete, as well as the pullout capacity of lifting inserts, should be greater than the

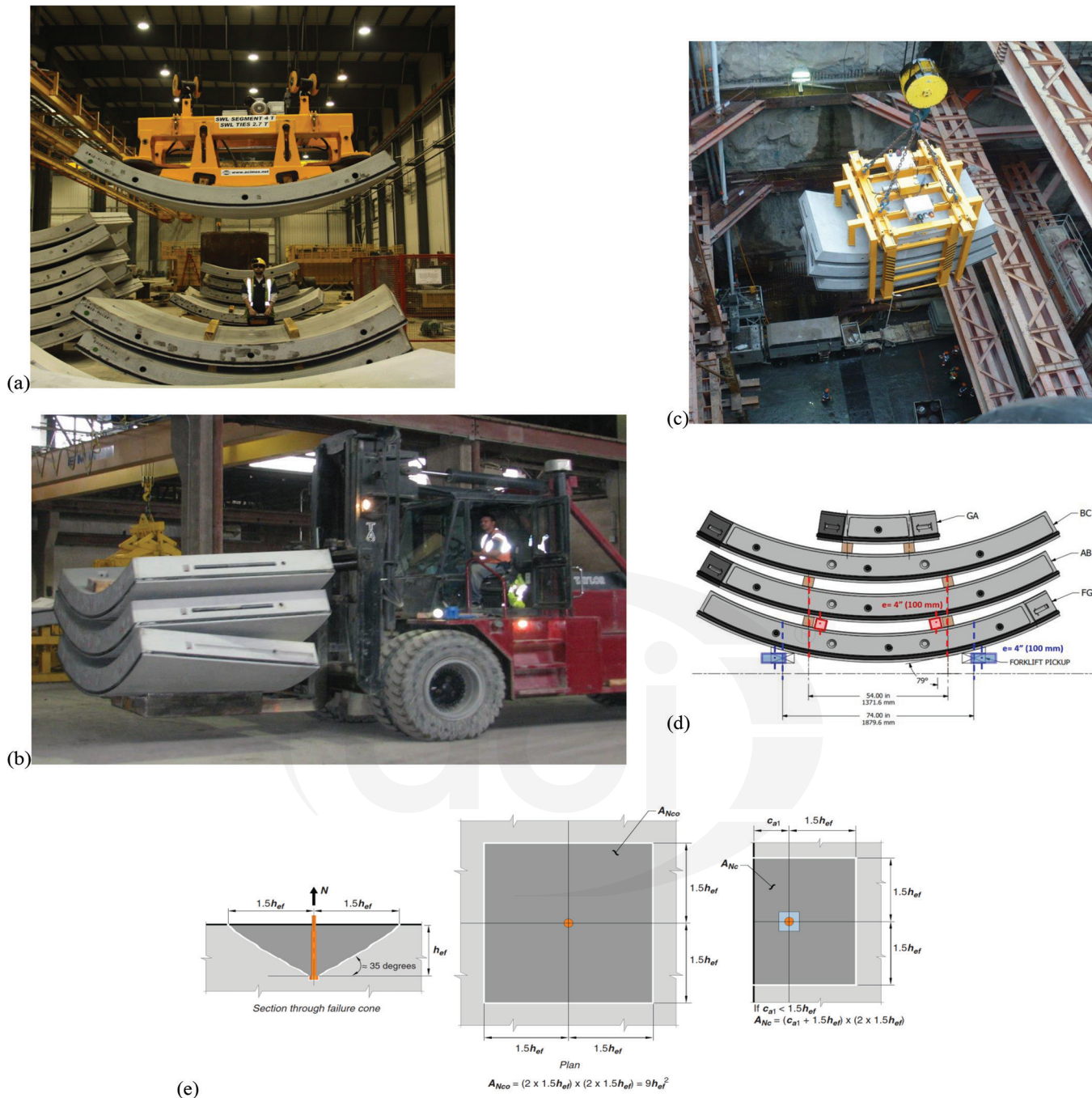


Fig. 4c—Segment handling: (a) using vacuum lifters; (b) forklifts; (c) mechanical clamping; (d) schematics of design eccentricities for segment handling with the forklift; and (e) diagram of pullout capacity of concrete during handling (ACI 318-19, Fig. R17.6.2.1).

demand, which is maximum pullout force applied on the lifting socket due to self-weight of segments and a dynamic impact factor.

Table 4 presents a summary of load cases; applied dynamic impact factors, if any; and maximum developed bending moments and shear forces for the various manufacturing, transportation, and handling stages. Precast concrete segments should be sufficiently reinforced to withstand developed bending moments due to these actions at an early age. Designers should follow structural codes and guides such as ACI 318 and ACI 544.7R for calculating the bending

strength (M_n) and check the design strength versus developed bending moments and shear forces.

CHAPTER 5—DESIGN FOR CONSTRUCTION STAGES

Construction loads include the tunnel boring machine (TBM) jacking thrust loads on the circumferential ring joints and the pressures during the grouting operation exerted against the exterior of the completed rings. Precast concrete segments are designed to resist significant bursting and spalling tensile stresses that develop along the circumferential joints due to the advancing of the TBM. The segments

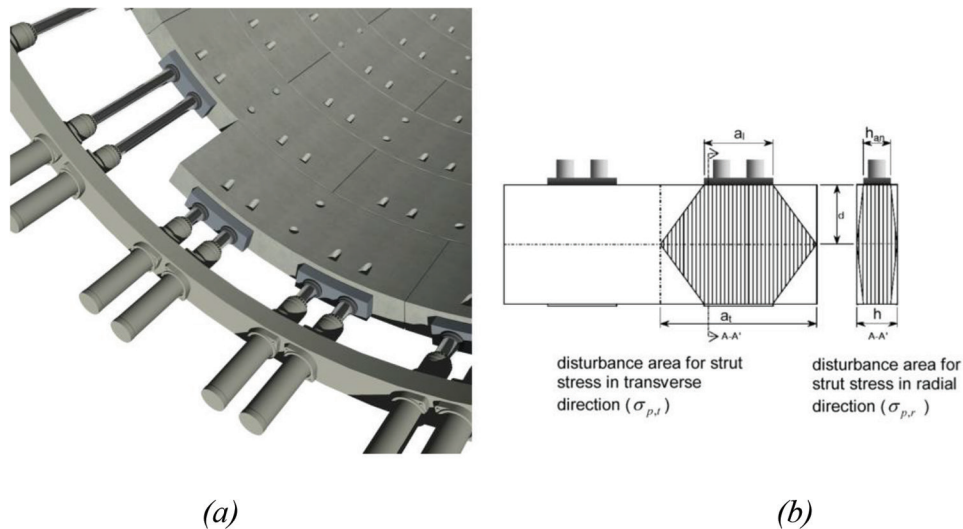


Fig. 5.1—(a) Thrust jacks pushing on circumferential joints; and (b) schematics of a simplified disturbance area of strut under TBM jack shoes (Groeneweg 2007).

should also be able to resist the axial forces and bending moments that develop when the annular space between the segments and the ground is pressure-filled with grout. Included is the primary backfilling of the tail skin void and the secondary grouting that is needed in case a complete contact of the lining with the ground has not been achieved through the primary grouting.

5.1—Tunnel boring machine thrust jack forces

After assembly of a complete ring, the TBM advances by thrusting against the most recent assembled ring, as shown in Fig. 5.1(a). As part of this process, the TBM jacks bear against the jacking pads placed along the exposed circumferential joint. Schematics of a simplified disturbance area of strut under TBM jack shoes are presented in Fig. 5.1(b). High compression stresses develop under the jacking pads and result in the formation of significant bursting tensile stresses deep within the segment. Furthermore, spalling tensile forces also act between adjacent jack pads along the circumferential joint.

Different methods are used for estimating TBM thrust due to the various geologic materials that can be encountered. For tunnel excavation through rock, TBM thrust can be estimated by the summing forces required to advance the machine. These forces include the forces necessary for boring through the rock, countering the friction between the surface of the shield and the ground, and the hauling of trailing gears. Methods found in Fukui and Okubo (2003) and Rostami (2008) can be used to evaluate the rock thrust based on rock strength, tunnel diameter, and cutter characteristics (Bakhshi and Nasri 2013a). For soft ground tunneling applications, the method presented by JSCE 2007 can be used to calculate penetration resistance based on earth or slurry pressure that acts at the cutting face.

Once the required machine thrust has been estimated for the ground conditions, the average thrust force per jack pair is determined simply by dividing the required machine thrust by the number of jack pairs. On sharp curves, the machine

thrust is higher on the convex side of the curve than on the concave side. The higher thrust should be accounted for in the design. A simple technique can be used to account for the increased loading on the convex side by doubling the jacking loads. The load factor for this case is taken to be 1.2, which is in agreement with suggested load factor in ACI 544.7R. Note that this factor is only considered when the machine characteristics are unknown, which is the case before the completion of the final design. Whenever the TBM's total thrust is available, maximum thrust jack forces are known and therefore cannot be exceeded. Accordingly, the corresponding load factor should be selected as 1.0. Furthermore, different analysis and design methods are available for this load case, including ACI 318 equations for bursting forces, DAUB:2013 formulas, Iyengar (1962) diagram, and two- and three-dimensional finite element simulations.

5.1.1 Simplified equations—For post-tensioned anchorage zones of prestressed concrete sections, structural concrete codes such as ACI 318 allow the use of simplified equations (Eq. (5.1.1a)) to determine the bursting force, T_{burst} , and the centroidal distance from the face of the section, d_{burst} . These simplified equations are used to obtain the forces and stresses developed in the circumferential joints due to TBM advancement. DAUB:2013 recommends similar equations (Eq. (5.1.1b)) specifically for the design of tunnel segments.

$$ACI\ 318\ T_{burst} = 0.25P_{pu} \left(1 - \frac{h_{anc}}{h} \right); d_{burst} = 0.5(h - 2e_{anc}) \quad (5.1.1a)$$

$$DAUB:2013 \\ T_{burst} = 0.25P_{pu} \left(1 - \frac{h_{anc}}{h - 2e_{anc}} \right); d_{burst} = 0.4(h - 2e_{anc}) \quad (5.1.1b)$$

This load case and corresponding parameter are schematically shown in Fig. 5.1.1. For specific value has been

provided for e_{anc} , then the eccentricity of the jacking forces is generally considered to be 1.2 in. (30 mm). Figure 5.1.1 and Eq. (5.1.1a) and (5.1.1b) represent the radial bursting stresses in the circumferential joints. These equations are also applicable to tangential bursting stresses developed in the circumferential joints.

Reinforcing bar or fiber reinforcement is designed to resist the bursting stresses developed by jacking forces. Equations (5.1.1c) and (5.1.1d) have been adopted to determine the required area (A_s) of reinforcing bars with a yield stress of f_y for a reinforced concrete (RC) segment.

$$T_{burst} = \phi f_y A_{s_radial} \text{ for radial direction} \quad (5.1.1c)$$

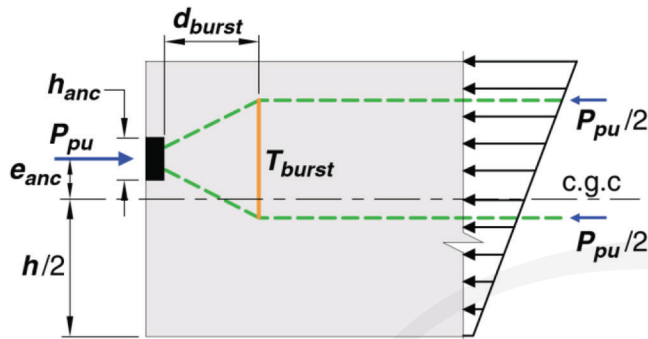


Fig. 5.1.1—Bursting tensile forces and corresponding parameters recommended by ACI 318.

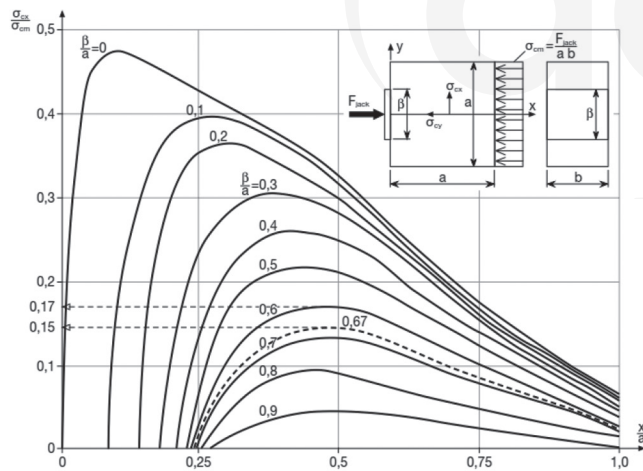


Fig. 5.1.2—Iyengar (1962) diagram for determining bursting tensile stresses.

$$T_{burst} = \phi f_y A_{s_tangential} \text{ for tangential direction} \quad (5.1.1d)$$

High compressive stresses can be developed under the jacking pads due to the TBM thrust jacking forces. These compressive stresses, $\sigma_{c,j}$, can be estimated using Eq. (5.1.1e).

$$\sigma_{c,j} = \frac{P_{pu}}{A_j} = \frac{P_{pu}}{a_l h_{anc}} \quad (5.1.1e)$$

Because only part of the circumferential segment face is actually in contact with the pads, the allowable compressive stresses (f'_c) can be factored to account for the strength of a partially pressurized surface. ACI 318 specifies the formula used for designing the bearing strength of concrete (Eq. (5.1.1f)) with a partially loaded segment face. DAUB:2013 recommends a similar formula for designing tunnel segment faces.

$$f'_{co} = 0.85 f'_c \sqrt{\frac{A_d}{A_j}} = 0.85 f'_c \sqrt{\frac{a_l (h - 2e_{anc})}{a_l h_{anc}}} \quad (5.1.1f)$$

5.1.2 Iyengar diagram—The analytical method of the Iyengar (1962) diagram has also been used for calculating the bursting tensile stresses for the design of tunnel segments (Groeneweg 2007). Similar to previous methods, the extent of load spreading and the resulting magnitude of tensile stresses depend on the dimensions of the loaded surfaces, β , and the final spreading surfaces, a , shown in Fig. 5.1.2. Using this approach, the bursting tensile stresses (σ_{cx}), which vary significantly from the face that TBM jacks bear against toward the centerline of segment, are determined as a fraction of the fully spread compressive stress ($\sigma_{cm} = F/ab$). Reinforcing bars are designed for the bursting tensile stresses shown in the diagram. Total bursting force can be obtained as the integration of stresses or the area under the curve, and required reinforcing bar area is determined by Eq. (5.1.1c) and (5.1.1d).

5.1.3 Finite element method simulations—As shown in Fig. 5.1.3a, in addition to the bursting stresses under the jacking pads, spalling stresses develop in the areas between the jacking pads and in the areas between the jacking pads and the end faces of segments due to the concentration of the jacking forces. Analytical design methods for spalling

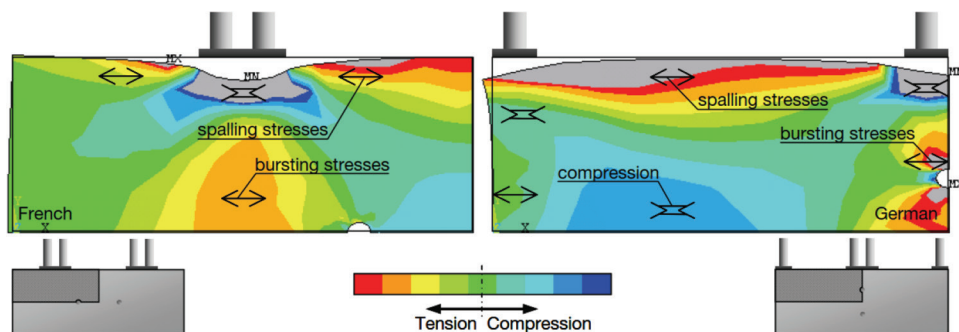


Fig. 5.1.3a—Spalling and bursting stresses in segment joints due to jack thrust forces (Groeneweg 2007).

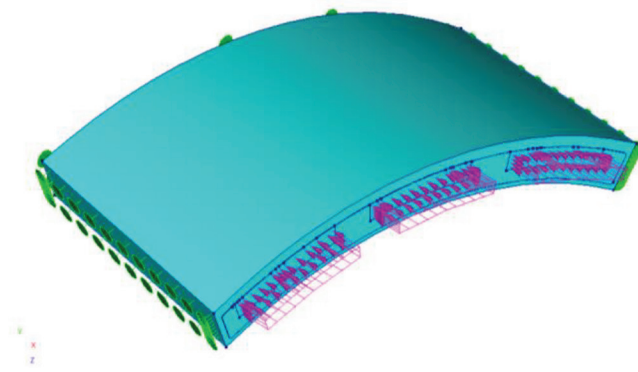


Fig. 5.1.3b—Three-dimensional FEM model for load case of TBM thrust jack forces.

stresses have not been provided in any of the previously reviewed approaches; however, analyzing the problem with a three-dimensional finite element method (FEM) is appropriate. While both linear elastic and nonlinear FEM simulations can be performed, the latter is considered suitable for the service design as nonlinear analyses capture the nonlinear response of the materials after cracking with respect to crack opening. As shown in Fig. 5.1.3b, this load case is simulated by modeling typical segments of two adjoining rings. The factored jacking forces are applied along the contact area between the jacking pads and the segment face. Recesses due to the gasket and stress relief grooves are modeled between two segments to simulate the transfer of force through a reduced cross section. With

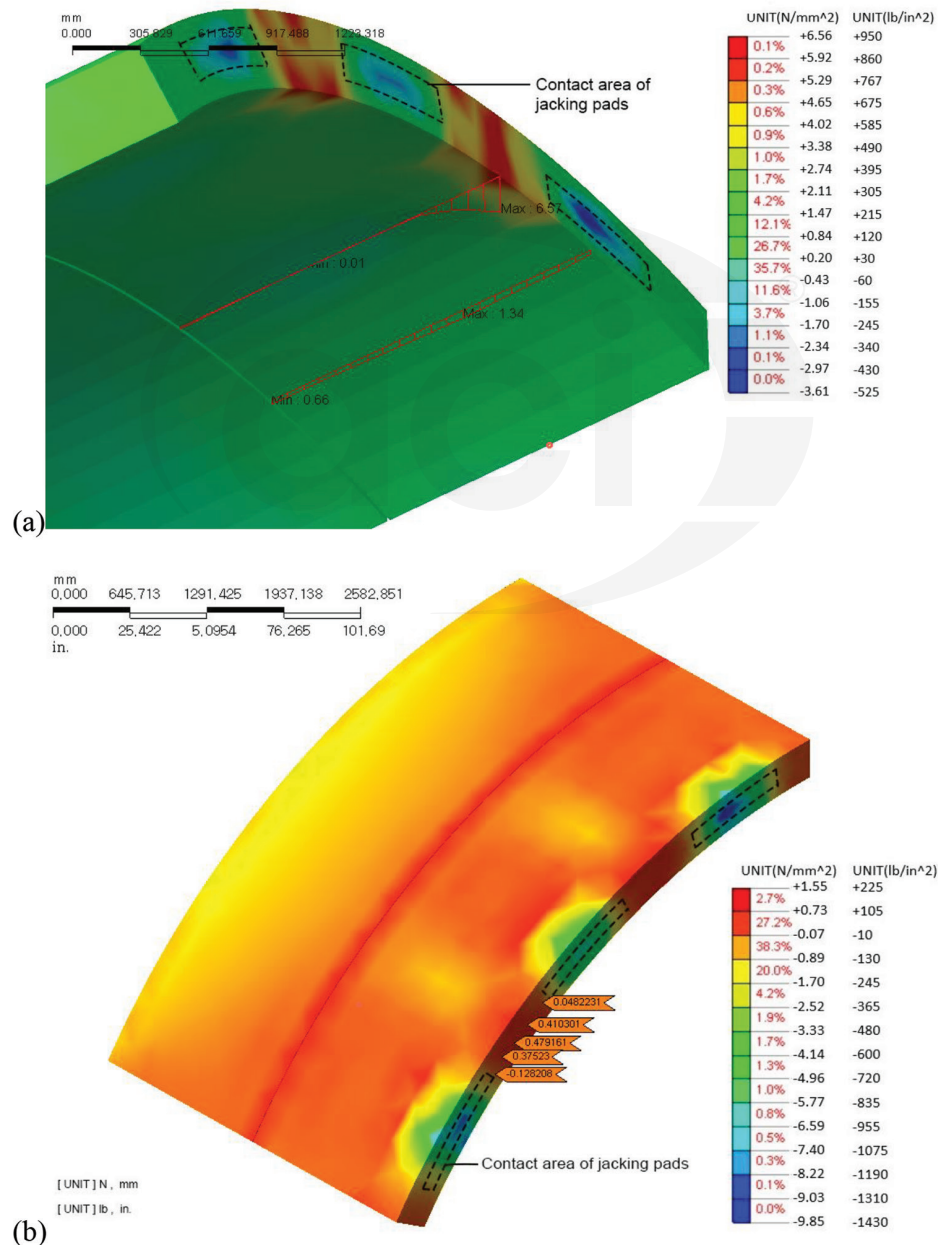


Fig. 5.1.3c—Example of bursting and spalling tensile stresses developed in segments due to TBM thrust jack forces (Bakhshi and Nasri 2013b, 2014d): (a) transverse stresses; and (b) radial stresses. (Note: Size of contact area of jacking pad is 7.8 x 30 in. [0.2 x 0.8 m]; segment thickness is 15.7 in. [0.4 m]).

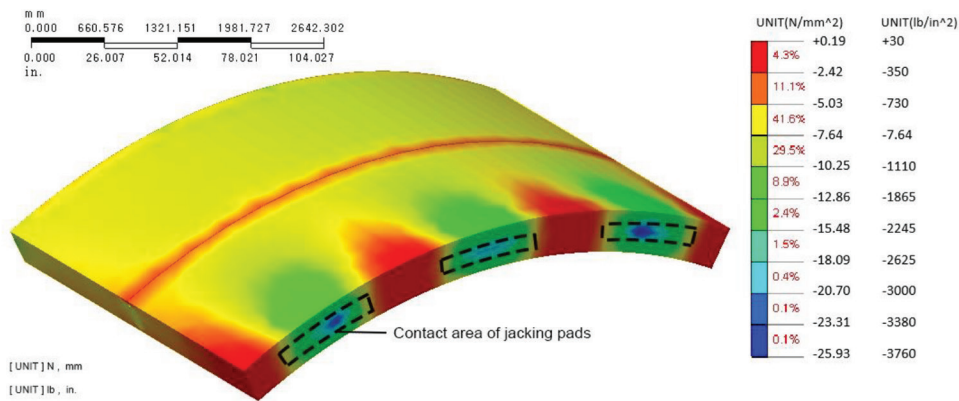


Fig. 5.1.3d—Typical compressive stresses developed in tunnel segments due to TBM thrust jack forces (Bakhshi and Nasri 2013b, 2014d). (Note: Size of contact area of jacking pad is 7.8 x 30 in. [0.2 x 0.8 m]; segment thickness is 15.7 in. [0.4 m]).

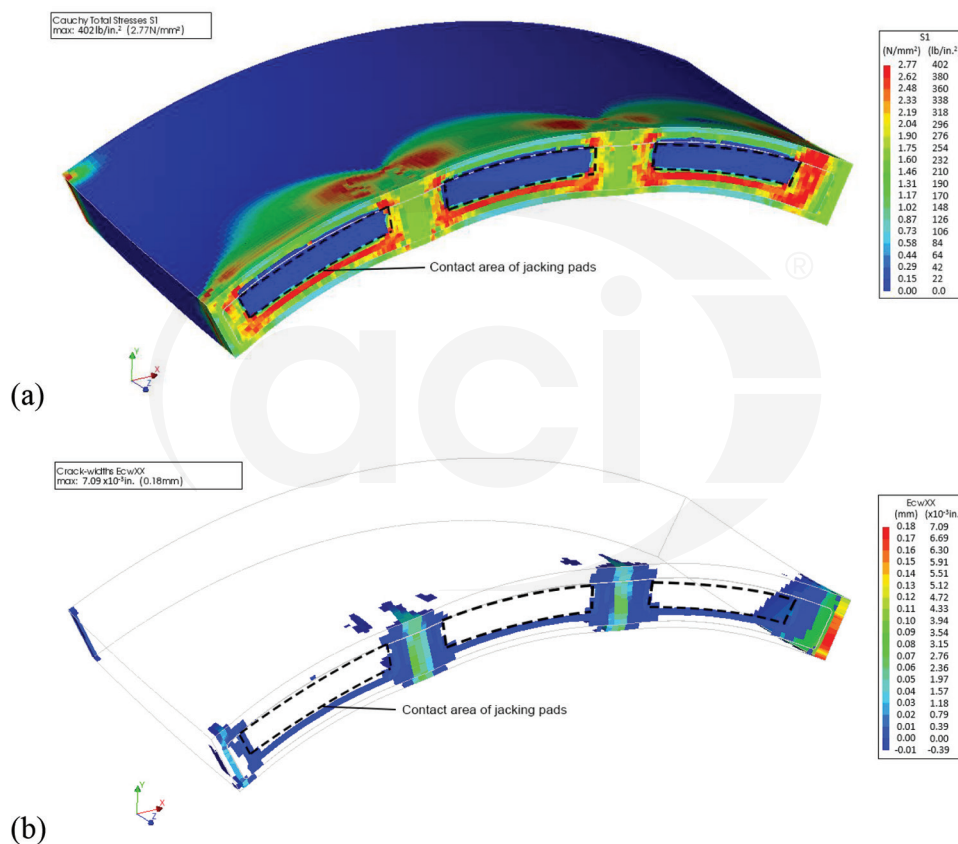


Fig. 5.1.3e—Example of a three-dimensional nonlinear FEM simulation for the load case of TBM thrust jack forces applied on fiber-reinforced concrete segment: (a) transverse tensile stresses; and (b) crack width dimension. Size of contact area of jacking pad is 6.8 x 24 in. (0.17 x 0.6 m) and segment thickness is 12 in. (0.3 m).

this approach, the translational degrees of freedom are fixed in all directions behind the previously installed segment. Figures 5.1.3c(a) and 5.1.3c(b) show typical results consist of the transverse and radial bursting stresses under the jack pad and the spalling stresses in the areas between the jacking pads as a result of an elastic FE analysis. Typical compressive stresses due to this load case are shown in Fig. 5.1.3d (Bakhshi and Nasri 2013b, 2014d).

Results from the three-dimensional FEM simulation indicate that spalling tensile stresses between the jack pads, and the jack pads and end faces, can be more significant than

the transverse bursting tensile stresses under the jacking pads (Bakhshi and Nasri 2013b). Precast tunnel segments should be designed to withstand these high tensile stresses. Reinforcement bars are designed for the tensile forces determined by the integration of stresses through the tensile zone, similar to **Iyengar (1962)** diagram method. When segments are only reinforced with fibers, the spalling stresses are likely to exceed the tensile strength. Therefore, a three-dimensional nonlinear FEM simulation, such as the one shown in Fig. 5.1.3e, should be performed to validate results of the crack width dimensions to the allowable values set

forth by standards, guidelines, and project specifications or design criteria. In the example shown in Fig. 5.1.3e, tensile cracking strength of fiber-reinforced segments is 403 psi (2.77 MPa) and its post-cracking tensile strength is 233 psi (1.6 MPa). Figure 5.1.3e(a) shows the transverse tensile stresses, and Fig. 5.1.3e(b) shows the crack width dimension due to jacking force. As expected, cracks are observed at the regions with highest spalling stresses in the linear analysis—that is, between jacking pads, and jacking pad and edge of segment (Fig. 5.1.3e(a)). In this example, the size of contact area of jacking pad is 6.8 x 24 in. (0.17 x 0.6 m).

5.2—Tail skin back grouting pressure

Tunnel rings are assembled within the shield of the TBM. At this location, the excavated diameter of the tunnel is larger than the external diameter of the tunnel ring. As shown in Fig. 5.2a, a tail void is created between the ground and the tunnel lining. This load case is generated by back grouting or filling of the annular space using semi-liquid grouts under high pressure to control and restrict settlement at the ground surface as well as to ensure complete contact between the ring and the ground. Grouting material generally consists of sand, water, cement, and several additives such as bentonite or plasticizers. This material is not actually a liquid per se, but it does have very low shear yield strength of 0.003 to 0.015 psi (20 to 100 Pa), prior to its hardening. When grout flows, it can be continuously injected into the annular space behind the TBM by means of grout pipes routed through the tail skin.

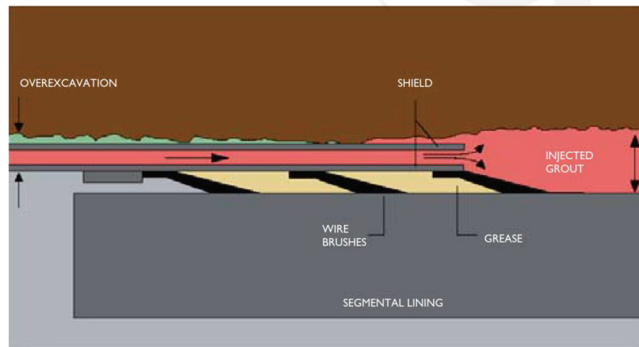


Fig. 5.2a—Backfilling of tail skin void (Guglielmetti et al. 2007).

To enter the annulus space created around the tunnel lining, the grout pressure needs to be greater than the water pressure surrounding the liner. The grout pressure should also typically be less than the overburden pressure to prevent pushoff of the soil, heave, hydrojacking, or all of these. Grouting models (Zhong et al. 2011) can be developed that predict the optimal grout pressure by considering the combined effects of groundwater level, plasticity of grout, rate of advancement of the TBM, and the filling rate of the tail void. Using Eq. (5.2a), the equivalent specific weight of the grout is determined by taking the equilibrium condition between the upward component of the total grout pressure, tunnel self-weight, and tangential component of the grout shear stresses (Groeneweg 2007). These forces are schematically shown in Fig. 5.2b.

$$\frac{\pi}{4} D_e^2 b \rho_{eq} = \pi D_e h b \rho_{concrete} + 2 D_e b \tau_{yield} \quad (5.2a)$$

The vertical gradient of the radial grout pressure between the crown and the invert of the tunnel is determined by Eq. (5.2b).

$$\Delta P_{g, invert} = \rho_{eq} \cdot D_e \quad (5.2b)$$

Note that AASHTO DCRT-1 specifies that 10 psi (69 kPa) above the groundwater pressure is the maximum permissible grouting pressure applied to the segmental ring. However, a maximum permissible grouting pressure of up to 22 psi (150 kPa) above the maximum groundwater pressure has been also considered (Ninić and Meschke 2017). For this load case, the lining is evaluated in a cross-sectional plane perpendicular to the longitudinal direction of the tunnel and modeled as a solid ring with a reduced flexural rigidity to account for the segment joints. Because the lining is surrounded completely by semi-liquid and fresh grout materials at this point, no interaction is taken into account between the ring and ground. As shown in Fig. 5.2b, the back-grouting load condition is modeled by applying radial pressure varying linearly from the minimum grout pressure at the crown to the maximum grout pressure located at the invert of the tunnel. The self-weight of the lining and the grouting pressure are the only loads applied to the tunnel lining at this stage of the analysis. For a load combination

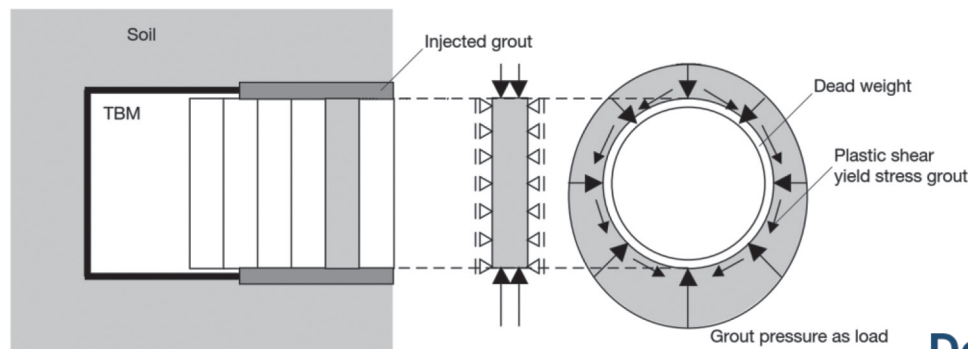


Fig. 5.2b—Forces and definitions for load case of tail void back-grouting (Groeneweg 2007).

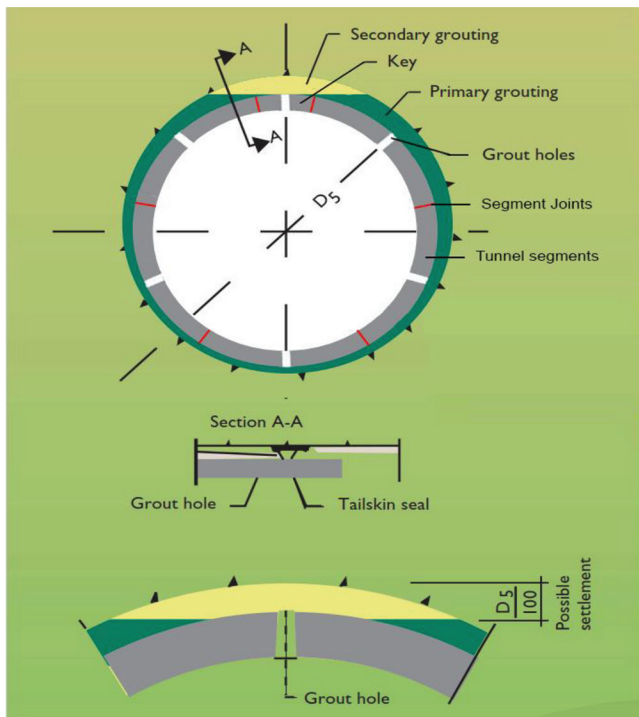


Fig. 5.3a—Secondary grouting through segment grout hole (Guglielmetti et al. 2007).

of self-weight and grout pressure, a load factor of 1.25 is suggested to both loads (ACI 544.7R). The analysis is performed using general structural analysis packages. As a result of this load case, it can be shown that significant axial forces and small bending moments can be developed in the tunnel segmental lining. Precast segments are designed for combined maximum bending moments and axial forces.

5.3—Localized back grouting (secondary grouting) pressure

Localized back grouting, also known as secondary or check grouting, is performed through holes that are manufactured into the segments. As shown in Fig. 5.3a, segments can be fitted with grout sockets that are screwed into position and remain closed with nonreturn valves and plastic covers during the ring installation process. Prior to the introduction of modern pressurized face machines, this grouting method was used to fill the annulus; however, the collapse of unstable ground into the annular space can generate significant settlements. As such, this method is now primarily used for secondary grouting in closed-face tunneling. To model the effects of secondary grouting, this load case consists of the forces applied to segments to verify whether the annular gap has been closed, which is similar to the tail skin back grouting method when only one of the grouting pipes is used. Following the ITA WG2 guidelines, this load case can be simulated using the force distribution shown in Fig. 5.3b. In this figure, secondary grouting pressure is applied on one-tenth of lining perimeter on the crown.

The lining for this load case is modeled in the cross-sectional plane perpendicular to the longitudinal direction of the tunnel using a solid ring with a reduced flexural rigidity

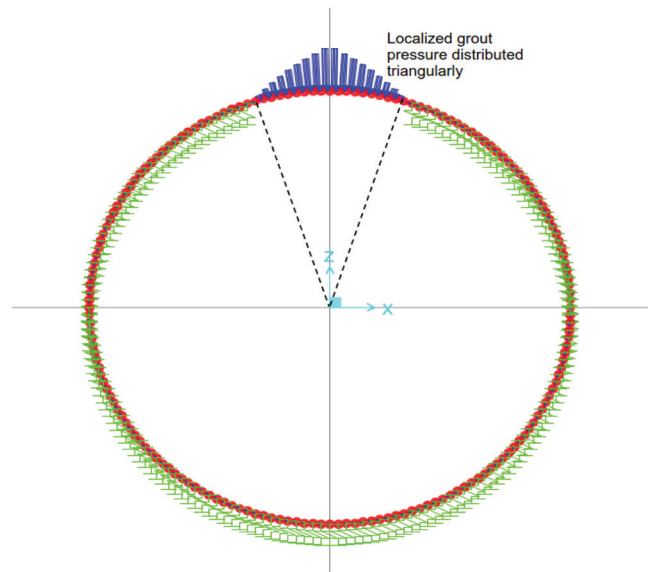


Fig. 5.3b—Modeling localized grouting pressure applied over one-tenth of the lining.

to represent the segment joints. Because secondary grouting occurs long after the primary grouting materials have cured, it can be assumed that tunnel lining is in full contact with the surrounding ground except in the local area where the secondary grouting is to be performed. To simulate the boundary condition for this case, the interaction between lining and surrounding ground or primary hardened grout can be modeled using radial springs with the segments supported radially. Linear translational springs have been used to represent this type of interaction. The method described by USACE EM 1110-2-2901 can be used, as an example, to determine the spring stiffness per unit of exterior tunnel surface. Using the same grout pressure on the crown as for the previous load case and with the radial spring stiffness, the bending moments and axial forces developed within the lining can be determined from the localized grouting operation. This loading case often results in small incremental axial forces with large bending moments. Precast segments are designed for this load case using axial force-bending moment interaction diagrams.

5.4—TBM backup load

TBM backup load is the self-weight of the backup equipment train behind the TBM shield, shown by schematics in Fig. 5.4a, applied as a concentrated variable load onto the segmental precast concrete lining. In general, TBM backup load may not be a critical or governing load case. However, in subsea and river crossing projects, to control buoyancy, an additional weight should be provided by the backup system inside the tunnel after ring installation and before installation of precast buoyancy unit. TBM drawings, such as Fig. 5.4b, show that this backup load is applied on specific locations in cross section on contact areas with tunnel lining intrados. In addition, longitudinal TBM drawings (Fig. 5.4b(b)) reveal that the backup load is applied longitudinally (with respect to the tunnel direction) on two wheels on each ring. For this load case, two-dimensional FEM analysis is often sufficient.

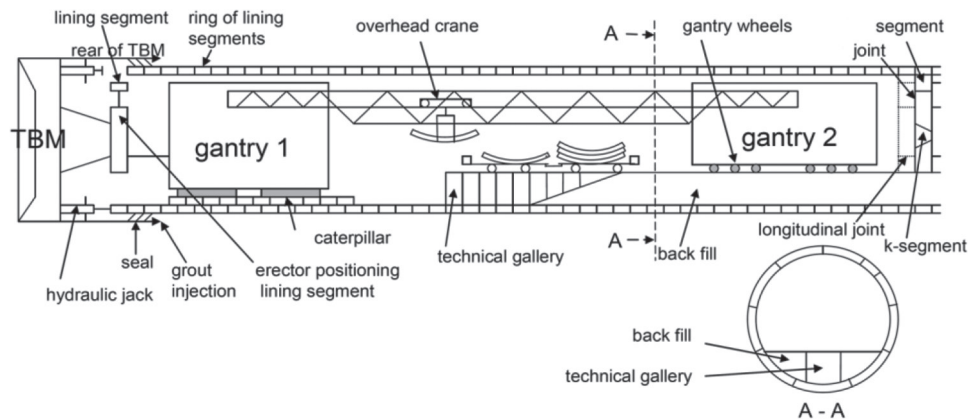


Fig. 5.4a—Schematics of TBM, backup train (gantries), and tunnel lining at the Groene Hart Tunnel as segmental lining is being erected inside the TBM (Talmon and Bezuijen 2011).

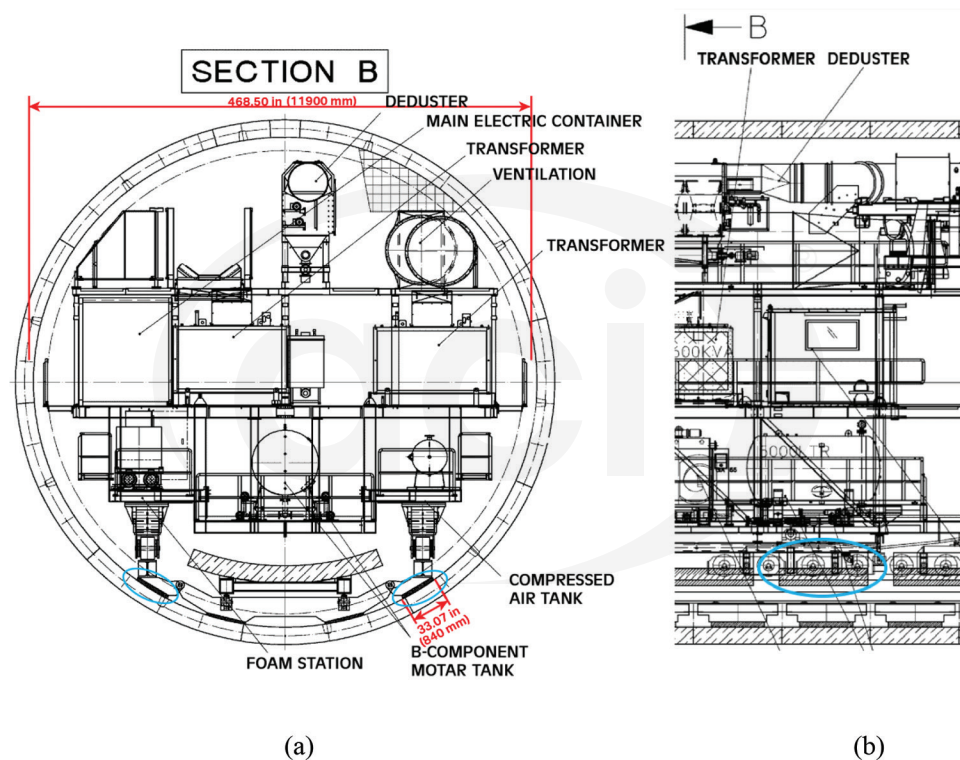


Fig. 5.4b—Sections behind the shield through the heaviest part of backup train (gantries) where heaviest backup loads are applied on segmental ring before backfilling: (a) cross section; and (b) longitudinal section.

with the assumption that backup load is applied uniformly on this area. Typical analysis results including deformations, axial forces, bending moments, and shear forces are shown in Fig. 5.4c. Resulting factored bending moments and axial forces for all critical cases (for example, shallow cover and deep tunnel) are compared with axial force-bending moment interaction diagram of segmental lining. Results should confirm that precast concrete tunnel segments can withstand axial forces and bending moments induced by TBM backup load or otherwise design should be modified. Punching shear strength of the segmental lining should be also assessed following concrete design codes such as ACI 318 and BS EN 1992-1-1:2004.

For the TBM backup load consideration, the early stage strength (setting time) of the tail void grout and the TBM

advance rate are also critical. The advance rate can be limited to the tail void grout strength needed for support of the first and second gantries.

CHAPTER 6—DESIGN FOR FINAL SERVICE STAGES

The final service stages are represented by the long-term loads imposed on the lining from the ground; groundwater; surcharges; and other factors, which generally are specific to the particular tunnel. Hoop force transfer along the longitudinal joints between segments, additional distortion, and other load cases specific to projects are also considered during this stage. Some common load cases specific to projects include earthquake, fire, explosion adjacent tunnels, and longitudinal bending moments. For the most part, service loads

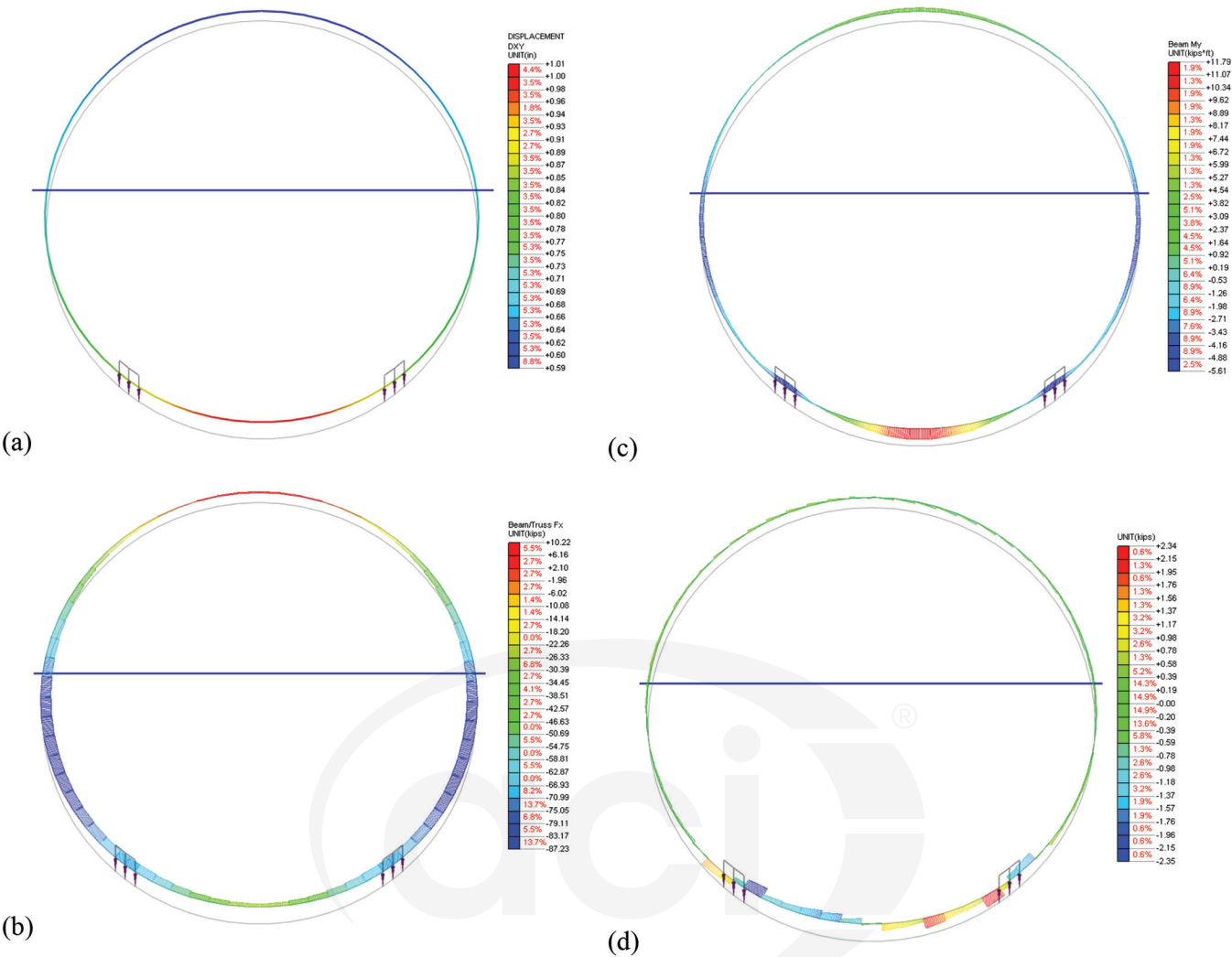


Fig. 5.4c—Typical results of FEM analysis for backup load case: (a) deformations; (b) axial forces; (c) bending moment; and (d) shear forces. (Note: Units are inches, kip, and ft-kip; multiply by 25.4, 4.45, and 1.36, respectively, for converting to mm, kN, and kN.m.)

Table 6.1—Load factor and load combination table for final service stages (AASHTO DCRT-1)

Load combination limit states	w, WAp		EH, EV		ES	
	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum
ULS	1.25	0.90	1.35	0.90	1.50	0.75
SLS	1.0		1.0		1.0	

generally result in axial forces and bending in the segmental lining. With the hoop force transfer in the longitudinal joints, significant bursting tensile stresses can be developed.

6.1—Earth pressure, groundwater, and surcharge loads

Precast concrete segments are used to withstand various loads from vertical and horizontal ground pressure, groundwater, self-weight, surcharge, and ground reaction loads. In accordance with load and resistance factor design (LRFD) principles and as explained in 3.1, 3.2, and 3.3, load factors and load combinations shown in Table 6.1 should be used to compute the ultimate limit state (ULS) and serviceability limit state (SLS).

Methods for analysis of segmental tunnel linings are presented in accordance with standards and guidelines from Europe, Asia, and America (Bakhshi and Nasri 2014a). The effect of ground, groundwater, and surcharge loads on segments is analyzed using elastic equations, beam-spring models, finite element methods (FEM), and discrete element methods (DEM). Other acceptable methods of analysis include Muir Wood’s (1975) continuum model with discussion from Curtis et al. (1976), Duddeck and Erdmann’s (1982) model, and an empirical method based on tunnel distortion ratios (Sinha 1989; Deere et al. 1969) that was originally developed by Peck (1969). The results of these analyses are used to specify the concrete strength and reinforcement. Reduction in bored tunnel segmental lining

moment of inertia also known as reduced flexural rigidity, where relevant, will be in accordance with Muir Wood's (1975) proposed method as presented in following equation.

$$I_r = I_j + (4/n)^2 \times I \quad (6.1)$$

where I_j is moment of inertia at joint (taken as zero in the design); I is the moment of inertia for nominal lining thickness; and n is the number of segments per ring excluding key segment ($n \geq 4$).

6.1.1 Elastic equation method—The elastic equations method (ITA WG2; JSCE 2007) is a simple method for calculating member forces of circular tunnels. As shown in Fig. 6.1.1, the load distribution model consists of applying uniform vertical ground and groundwater pressures, a linearly varying lateral earth pressure, self-weight of the lining, and a triangularly distributed horizontal ground reaction between 45 and 135 degrees from the crown. Member forces are calculated using the elastic equations contained in Table 6.1.1 (JSCE 2007; ITA WG2). For this method, the segmental tunnel lining is modeled using a uniform reduced bending rigidity (Muir Wood 1975) that takes into account the effect of longitudinal joints between the segments. Subgrade reaction modulus (spring stiffness) formulations recommended by different guidelines are as follows

USACE EM 1110-2-2901:1997:

$$K_r = E_r / (R(1 + \nu)) \quad k_t = 0.5k_r / (1 + \nu) \quad (6.1.1a)$$

ÖVBB 2011 (Austrian):

$$K_r = E_s / R \quad k_t = 0$$

Groeneweg (2007):

$$K_r = E_s / R \quad k_t = 0$$

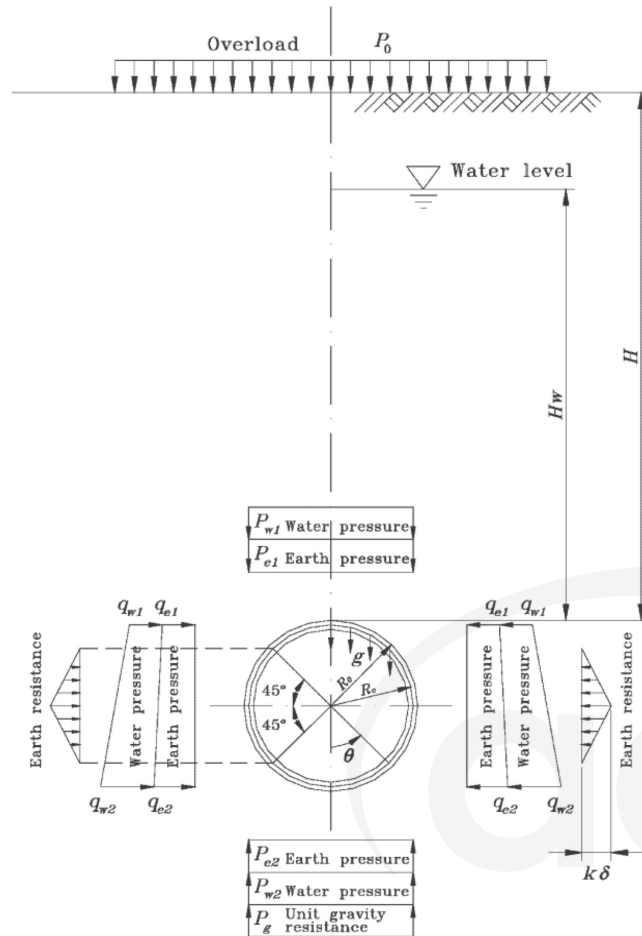


Fig. 6.1.1—Distribution of loads used in elastic equations method (JSCE 2007).

Table 6.1.1—Equations for calculation of member forces using elastic equation method (JSCE 2007; ITA WG2)

Load	Bending moment	Axial force	Shear force
Vertical load ($P = p_{e1} + p_{w1}$)	$(1 - 2S^2)PR_c^2/4$	S^2R_cP	$-SCR_cP$
Horizontal load ($Q = q_{e1} + q_{w1}$)	$(1 - 2C^2)QR_c^2/4$	C^2R_cQ	$-SCR_cQ$
Horizontal triangular load ($Q' = q_{e2} + q_{w2}$) ($Q - Q'$)	$(6 - 3C - 12C^2 + 4C^3)(Q - Q')R_c^2/48$	$(C + 8C^2 - 4C^3)(Q - Q')R_c/16s$	$(S + 8SC - 4SC^2)(Q - Q')R_c/16$
Soil reaction ($P_k = k\delta_h$)	$0 \leq \theta \leq \pi/4$ $(0.2346 - 0.3536C)R_c^2k\delta_h$ $\pi/4 \leq \theta \leq \pi/2$ $(-0.3487 + 0.5S^2 + 0.2357C^3)R_c^2k\delta_h$	$0 \leq \theta \leq \pi/4$ $0.3536CR_c k\delta_h$ $\pi/4 \leq \theta \leq \pi/2$ $(-0.7071C + C^2 + 0.7071S^2C)R_c k\delta_h$	$0 \leq \theta \leq \pi/4$ $0.3536CR_c k\delta_h$ $\pi/4 \leq \theta \leq \pi/2$ $(SC - 0.7071C^2S)R_c k\delta_h$
Dead load ($P_g = \pi \cdot g$)	for: $0 \leq \theta \leq \pi/2$ $(3/8\pi - \theta S - 5/6C)R_c^2g$ for: $\pi/2 \leq \theta \leq \pi$ $(-\pi/8 + (\pi - \theta)S - 5/6C - 1/2\pi S^2)R_c^2g$	for: $0 \leq \theta \leq \pi/2$ $(\theta S - 1/6C)R_{cg}$ for: $\pi/2 \leq \theta \leq \pi$ $(-\pi S + \theta S + \pi S^2 - 1/6C)R_c^2g$	for: $0 \leq \theta \leq \pi/2$ for: $\pi/2 \leq \theta \leq \pi$ $(-(\pi - \theta)C + \theta S + \pi SC - 1/6S)R_{cg}$
Horizontal deformation at springline (δ_h)	$\delta_h = \frac{(2P - Q - Q' + \pi g)R_c^4}{24(EI + 0.045kR_c^4)}$		

Note: θ is angle from crown; $S = \sin\theta$; $S^2 = \sin^2\theta$; $S^3 = \sin^3\theta$; $C = \cos\theta$; $C^2 = \cos^2\theta$; $C^3 = \cos^3\theta$; EI is flexural rigidity in unit width.



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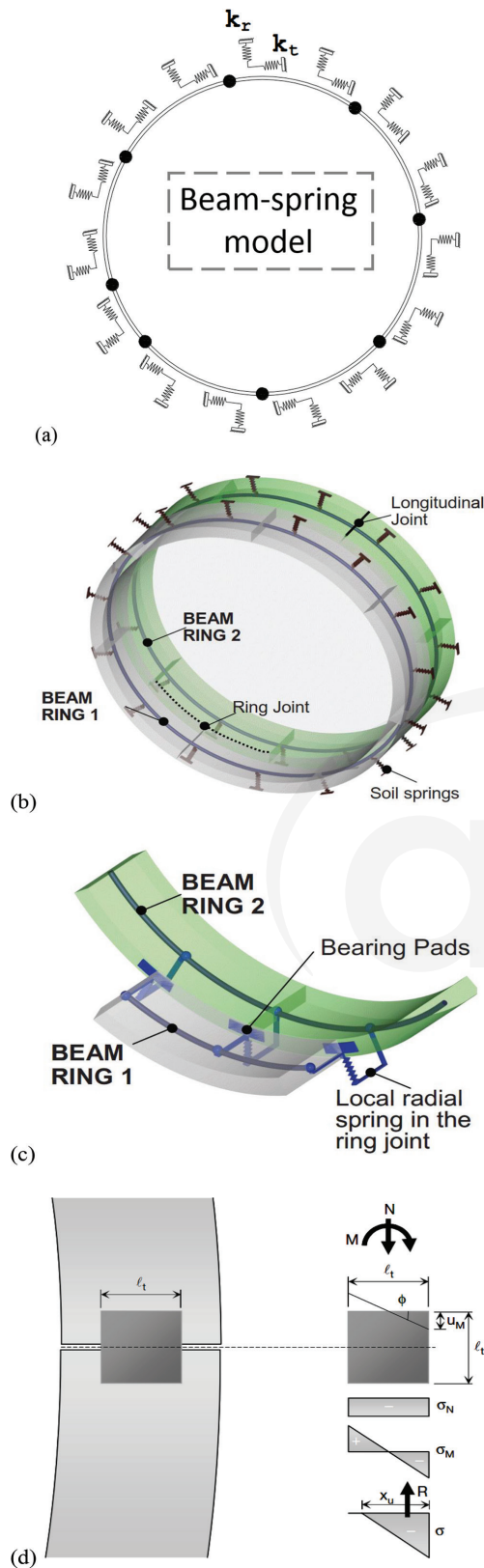


Fig. 6.1.2—(a) Demonstration of springs used for modeling of the interaction between ground and the lining; (b) double ring beam-spring model with radial springs simulating ground, and joint springs simulating longitudinal and circumferential joints; (c) scheme of ring joint (Plizzari and Tiberti 2009); and (d) presentation of parameters used in Janssen model for determining rotational spring stiffness.

JSCE 2007:

$$K_r = NA \quad k_t = 1/3K_r$$

RTRI 2008:

$$K_r = \text{varies} \quad k_t = NA$$

AFTES-WG7:

$$K_r = E_r / (R(1 + \nu)) \quad k_t = NA$$

DAUB:2005:

$$K_r = E_s / R \quad k_t = 0$$

where E_s or oedometer stiffness has the following relationship with surrounding ground Young's modulus (Duddeck and Erdmann 1982)

$$E_s = \frac{(1 - \nu) E_r}{(1 - 2\nu)(1 + \nu)} \quad (6.1.1b)$$

For example, assuming $\nu = 0.25$, $E_s = 1.2E_r$.

6.1.2 Beam-spring method—Using the beam-spring (also known as bedded beam) method endorsed by AASHTO DCRT-1, JSCE 2007, and ÖVBB 2011, the lining can be modeled in the cross-sectional plane as a series of beam elements that span between the longitudinal joints of the segments. As shown in Fig. 6.1.2(a), the interaction between the ground and the lining is modeled in two-dimensional domain using translational springs in the radial and tangential directions. When modeling in three-dimensional domain, interaction of lining with the ground in longitudinal direction of the tunnel is also modeled by longitudinal springs accordingly. Because the lining and ground are represented by a series of beams and springs, this method is referred to as the beam-spring or bedded-beam method. In the U.S. tunnel industry, the stiffness of the springs is generally calculated using formulas recommended by USACE EM 1110-2-2901 (Eq. (6.1.1a)). Furthermore, various two-dimensional approaches are used to evaluate effects of the segment joints, including solid ring models with fully bending rigidity, solid ring models with reduced bending rigidity (Muir Wood 1975), ring models with multiple hinged joints, and ring models with rotational springs. However, two-dimensional models cannot be used to represent circumferential joints or staggered arrangement of segments between rings.

As shown in Fig. 6.1.2(b) and 6.1.2(c), a two-and-a-half-dimensional multiple-hinged segmented double-ring beam spring has been used to model the reduction of bending rigidity and the effects from a staggered geometry. This manipulation is achieved by modeling the segments as curved beams, flat longitudinal joints as rotational springs