U.S. Department
of Transportation
Federal Highway
Administration

PRECAST CONCRETE SEGMENTAL LINERS FOR LARGE DIAMETER ROAD TUNNELS

Literature Survey and Synthesis

FHWA-HIF-20-035
September 2020

Technical Report Documentation Page

1. Report No. FHWA-HIF-20-035	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Precast Concrete Segmental Liners for Large Diameter Road Tunnels – Literature Survey and Synthesis		5. Report Date September, 2020 6. Performing Organization Code
7. Principal Investigator(s): William Bergeson, PE (FHWA), John Wisniewski, PE (WSP), Sotirios Vardakos, PhD, (WSP), Michael Mooney, PhD, PE (Colorado School of Mines), Axel Nitschke, PhD, PE (WSP)		8. Performing Organization Report
9. Performing Organization Name and Address WSP USA, Inc. One East Pratt Street Suite 300 Baltimore, MD 21202		10. Work Unit No. (TRAIS) 11. Contract or Grant No. DTFH6114D00048
12. Sponsoring Agency Name and Address Federal Highway Administration U.S. Department of Transportation 1200 New Jersey Avenue, SE Washington, DC 20590		13. Type of Report and Period Covered Final Report 09, 2018 – 09, 2020 14. Sponsoring Agency Code DOT/FHWA/HIF
15. Supplementary Notes Cover photo: SR 99 Tunnel, Seattle, Washington. Photo: John Wisniewski.		

16. Abstract

Increased roadway traffic demands have led to a notable increase of large diameter tunnel boring machine-driven tunnels across the world. The technological advancements of tunnel boring machines have made them a viable technical option for tunneling in difficult conditions in urban environments at ever increasing diameters. Such tunnels utilize precast concrete segmental linings. Although precast concrete segments have been widely used and designed in the US since the mid-1970s, the significant increase in diameter demands brings about new challenges in design and construction. Various international publications and practice manuals have been authored about segmental lining design. The present document is the first phase of an FHWA research initiative focused on the design of large diameter precast concrete segmental linings. This document provides an overview of the literature survey and synthesizes the current state of the practice along with raising potential knowledge gaps for future research.

17. Key Words Tunnel, lining, precast concrete, segmental lining, fiber reinforced concrete, literature, research, knowledge gap, design, codes, standards.	18. Distribution Statement No restrictions.

19. Security Classif. (of this report) UNCLASSIFIED	20. Security Classif. (of this page) UNCLASSIFIED	21. No. of Pages 154	22. Price

Form DOT F 1700.7(8-72) Reproduction of completed page authorized

FOREWORD

The Federal Highway Administration (FHWA) sponsors research about the use of large diameter precast concrete segmental tunnel linings in highway tunnels. The basic technology of conventionally reinforced precast concrete segments is relatively matured for smaller diameter tunnels. It has been in use in the United States for nearly 40 years (and even longer in other parts of the world). However, recent advances in the use of steel fibers for concrete reinforcement, joint hardware and details, gasket technology, high-strength concrete mixes, and material durability warrant study to provide uniformity of application, identification of practices and details for use in large-diameter tunnels. The work of this research includes a literature survey to identify gaps in the current body of knowledge (“knowledge gaps”), computer modeling and laboratory research, and engagement of industry stakeholders through a workshop that will be used to solicit input on the research plans. The work herein will also build on prior research work performed on design approaches for Tunnel Boring Machine (TBM) excavated tunnels.

The objective of this research is to provide technical expertise to advance the current state of practice for the analysis, design, detailing, fabrication, installation, inspection, and maintenance of precast concrete segmental tunnel linings for large diameter highway TBM-tunnels in the US. The research includes several elements:

- conducting a literature survey and development of a literature synthesis of the current state of the practice;
- development of computer modeling and laboratory testing workplans;
- hosting an industry workshop to solicit input from technical organizations, designers, contractors and researchers regarding the workplans;
- executing research workplans and presenting research results in reports that summarize the finding of the research;
- development of document presenting suggested practices for design of large diameter precast concrete segmental tunnel linings.

This document provides an overview of the conducted literature survey and synthesizes the current state of the practice. Thus, it may potentially lead to additional research.

Notice

This document is disseminated under the sponsorship of the U.S. Department of Transportation (USDOT) in the interest of information exchange. The U.S. Government assumes no liability for the use of the information contained in this document.

The U.S. Government does not endorse products or manufacturers. Trademarks or manufacturers' names appear in this report only because they are considered essential to the objective of the document. They are included for informational purposes only and are not intended to reflect a preference, approval, or endorsement of any one product or entity.

Non-Binding Contents

The contents of this document do not have the force and effect of law and are not meant to bind the public in any way. This document is intended only to provide clarity to the public regarding existing requirements under the law or agency policies. However, compliance with applicable statutes or regulations cited in this document is required.

Quality Assurance Statement

The Federal Highway Administration (FHWA) provides high-quality information to serve Government, industry, and the public in a manner that promotes public understanding. Standards and policies are used to ensure and maximize the quality, objectivity, utility, and integrity of its information. FHWA periodically reviews quality issues and adjusts its programs and processes to ensure continuous quality improvement.

ACRONYMS

2D	Two Dimensional
3D	Three Dimensional
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AFTES	French Tunneling and Underground Space Association
ASCE	American Society of Civil Engineers
ASTM	ASTM International
BAST	German Federal Highway Research Institute
BSI	British Standards Institute
BTS	British Tunneling Society
Caltrans	California Department of Transportation
CBBT	Chesapeake Bay Bridge Tunnel
CEB	Comité Euro-International du Béton
CFD	Computational Fluid Dynamics
CMOD	Crack Mouth Opening Displacement
CNR	Italian Research Council
DAfStb	German Committee for Reinforced Concrete
DAUB	German Tunneling Committee of the International Tunneling Association
DB	Federal German Railway Authority
DBV	German Society for Concrete and Construction Technology
DOT	Department of Transportation
ECIS	East-Central Interceptor Sewer
EN	European Commission Joint Research Center
EPB	Earth Pressure Balanced
FDOT	Florida Department of Transportation
FE	Finite Element
FEA	Finite Element Analysis
FEE	Functionality Evaluation Earthquake
FHWA	Federal Highway Administration
fib	International Federation for Structural Concrete
FIP	Fédération Internationale de la Précontrainte
FRC	Fiber Reinforced Concrete
FRP	Fiberglass Reinforced Plastic
GFRP	Glass Fiber Reinforced Polymer
HFRC	Hybrid Fiber Reinforced Concrete
HPFRCC	High-Performance Fiber Reinforced Cementitious Composite
ICE	Institute of Civil Engineers
ITA-AITES	International Tunneling Association
JSCE	Japan Society of Civil Engineers
kN	Kilo Newton
LRFD	Load and Resistance Factor Design
MDE	Maximum Design Earthquake
Mpa	Mega Pascal
NAT	North American Tunneling Conference
NCHRP	National Cooperative Highway Research Program
NEIS	Northeast Interceptor Sewer
NFPA	National Fire Protection Association

NTI	National Tunnel Inventory
NTIS	National Tunnel Inspection Standards
NYCDOT	New York City Department of Transportation
NYSDOT	New York State Department of Transportation
ÖVBB	Austrian Society for Concrete and Construction Technology
PCTL	Precast Concrete Tunnel Lining
PFDHA	Probabilistic Fault Displacement Hazard Analysis
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PIRAC	World Road Association
RC	Reinforced Concrete
RETC	Rapid Excavation and Tunneling Conference
RIFD	Radio Frequency Identification
RILEM	International Union of Laboratories and Experts in Construction Materials, Systems and Structures
SEE	Safety Evaluation Earthquake
SFRC	Steel Fiber Reinforced Concrete
SIA	Swiss Society of Engineers and Architects
SSR	Stress Strain Relationship
STUVA	Research Association for Tunnel and Transportation Facilities
Svensk	Swedish Standards Institute
TAC	Tunneling Association of Canada
TBM	Tunnel Boring Machine
TOMIE	Tunnel Operations, Maintenance, Inspection, and Evaluation
UK	United Kingdom
ULS	Ultimate Limit State
US	United States
WTC	World Tunnel Congress

UNIT CONVERSIONS

SI	US
1 m	3.28 ft (‘)
1 mm	0.039 in (“)
1 m^3	35.32 ft^3
1 N	0.2248 lb
1 kN-m	737.56 lb-ft
1 W	0.00134 hp
1 tonne	2,204.62 lb
1 Pa	0.000145 psi
°C	9/5(°C)+ 32 °F

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION

Tunnels are major capital investments for owners and they involve large capital expenditures for construction, operation and maintenance. Because of the investment involved, tunnels are typically limited to important transportation routes, used to reduce the impact on important existing development or infrastructure or to provide passage through natural obstacles such as mountains or under bodies of water. Once in place, it is important that the tunnel is sufficiently robust and resilient to serve its purpose for the envisioned service life and beyond. Some highway tunnels in the United States (US) are approaching 100 years of service life. The voluntary and non-binding 2017 American Association of State Highway and Transportation Officials (AASHTO) LRFD Road Tunnel Design and Construction Guide Specifications recommends a design life of 150 years. This guide describes design life as the "period of time on which the statistical derivation of transient loads is based", while the guide describes service life as "the period of time that the tunnel is expected to be in operation". Although the guide does not specify a service life, because the durability of tunnels is not well quantified, a service life for the structure itself of 100 to 125 years (the voluntary and non-binding Federal Highway Administration (FHWA Technical Manual for Design and Construction of Road Tunnels – Civil Elements) or more is typical for major infrastructure elements, like tunnels. However, it should also be noted that not all elements of the tunnel (tunnel systems such as lighting, signage, communications, signals and ventilation) have the same service life as the tunnel structure.

During its service life, a tunnel can be expected to experience permanent loads (i.e. dead loads, earth pressure, surcharge loads, etc.), live loads (i.e. vehicular loads, live load surcharges, etc.), or transient loads (i.e. water loads, earthquake, superimposed deformations, blast, fire, construction loads, etc.). Growing the body of knowledge of how loads are supported by segmental tunnel linings during the design life will provide improved design methodologies for traditional materials such as conventionally reinforced concrete (RC) while deepening the understanding of the behavior of newer materials such as fiber reinforced concrete (FRC), glass fiber reinforced polymer bars (GFRP) and combinations of reinforcing materials. To increase knowledge in a systematic manner, research should be performed to analyze current state of design practices versus the actual performance and identify existing knowledge gaps. Conventionally reinforced segmental concrete tunnel linings have been in use in Europe prior to their introduction into the US. About the same time that the first conventionally reinforced tunnel linings were built in the US, fiber-reinforced concrete (FRC) segments were introduced in Europe. Extensive research and over 30 years of experience in Europe with FRC segmental linings is available. The U.S. market has utilized FRC segments in some sectors, such as water and waste water tunnels. However, owners of transportation systems, such as road or rail, have not yet followed this trend.

A literature search about segmental tunnel linings of primarily large diameter road tunnels and segmental lining design approaches was performed to create an understanding of the state of the practice. This includes commonly used or proposed design practices, but also understanding of material behavior, practices associated with fabrication, handling, installation and maintenance, concerns of designers, constructors and owners, the economic impact of the various materials and the risks associated with various applications and practices. In addition, the search revealed existing knowledge gaps that offer potential topics for additional research based on computer modeling and laboratory testing.

2 RESOURCES AND METHODS

2.1 General Approach and Resources

The literature survey involved an extensive and collaborative process performed by a multi-discipline team of designers and researchers. The literature investigated included a wide range of publicly available and published sources, along with project specific information available from the research team. A database was developed to catalogue and organize the material that was discovered. The information was categorized by content, applicability to the topic and usefulness to future research work, specifically the development of computer modeling and laboratory testing research plans. The national and international sources of information include (alphabetically):

- Academic research reports, theses and dissertations
- Conference proceedings including various research and case history papers
- Guidelines and manuals
- Material manufacturer catalogs and data sheets
- Prints of presentations
- Project specific data (reports, plans, specification, design criteria)
- Published books
- Scientific journal published articles
- Transportation project studies
- Various web resources including tunneling trade magazines and articles

More specifically, relevant reports, guidelines and material were researched from the following national and international authorities, agencies, associations, and organizations:

United States:

- American Association of State Highway and Transportation Officials (AASHTO)
- American Concrete Institute (ACI)
- American Society of Civil Engineers (ASCE)
- ASTM International (ASTM)
- Federal Highway Administration (FHWA)
- National Cooperative Highway Research Project (NCHRP)
- National Fire Protection Association (NFPA)

International:

- Austrian Society for Concrete and Construction Technology (ÖVBB)
- British Standards Institute (BSI)
- British Tunneling Society / Institution of Civil Engineers (BTS, ICE)
- European Commission Joint Research Center: Eurocode (EN)
- Federal German Railway Authority (DB)
- French Tunneling and Underground Space Association (AFTES)
- German Committee for Reinforced Concrete (DAfStb)
- German Federal Highway Research Institute (BAST)
- German Society for Concrete and Construction Technology (DBV)
- German Tunneling Committee of the International Tunneling Association (DAUB)
- International Federation for Structural Concrete (*fib*)
- International Tunneling Association (ITA-AITES)
- International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM)
- Italian National Research Council (CNR)
- Japan Society of Civil Engineers – Standards (JSCE)
- Swiss Society of Engineers and Architects (SIA)
- World Road Association (PIARC)
- Research Association for Tunnel and Transportation Facilities (STUVA)

Published research articles from the following scientific journals were obtained and included in the research database:

- *ASCE Journal of Materials in Civil Engineering* (American Society of Civil Engineers)
- *ASCE Journal of Structural Engineering* (American Society of Civil Engineers)
- *Composite Structures Journal* (Elsevier)
- *Engineering Structures Journal* (Elsevier)
- *Geomechanics and Tunneling Journal* (Austrian Society of Geomechanics)
- *Geotechnical and Geological Journal* (Springer)
- *International Journal of Numerical and Analytical Methods in Geomechanics* (Wiley)
- *Journal of Tunneling and Underground Space Technology* (Elsevier)
- *JSCE Journal of Tunnel Engineering* (Japan Society of Civil Engineers)
- *Materials and Structures* (Springer / RILEM)
- *Structural Concrete – The Journal of the fib* (Wiley / *fib*)
- *Structural Concrete Journal* (Ernst & Sohn)

Additionally, relevant papers from international conferences were reviewed from the following sources:

- Rapid Excavation and Tunneling Conference (RETC)
- North American Tunneling Conference (NAT)
- ITA – World Tunnel Congress (WTC)
- Tunneling Association of Canada Conference (TAC)

This diversity of sources provided information while also identifying knowledge gaps and independent research in similar topics discussed later in this report.

The literature survey was conducted using the following stepped approach:

1. Assembling technical publications including code standards, scholarly papers, project specific design criteria, manufacturers' data, national and international experience,
2. Synthesizing collected information to summarize historical development and highlight knowledge gaps, inconsistencies and discrepancies of:
    - Historical development of large diameter segmental lined road tunnels;
    - Current segment design approaches, including determination of load effects, load combinations, and load factors;
    - Current ring detailing approaches, including segmentation, hardware, and geometry;
    - Materials, material testing, theoretical and actual material behavior, durability, corrosion and deterioration factors, fire resistance/damage and concrete mix designs;
    - Fabrication and construction practices, including handling, installation practices, quality processes (including inspection during construction, commissioning and service), worker safety, cost and schedule,
3. Summarizing key findings and identifying major knowledge gaps and suggestions for future research.

2.2 Literature Survey Database

Based on the literature survey, a database of some of the largest diameter shield-driven tunnels was created. Based on the review of projects, a synopsis of the current state-of-the-art in the construction of large diameter tunnel projects was developed. Information from this study also assisted in assessing latest trends in large diameter bored tunneling and segmental lining across the world. This element of the research work is described in detail in Chapter 3. The project database is included in Appendix A.

More than 630 references were identified and reviewed to produce this synthesis report. To assist with the review process, the sourced material was organized based on the following database fields:

1. Document serial number.
   (This number is also embedded in the filename for later ease of retrieval)
2. Title of document
3. Document type
   - Book chapter/section
   - Brochure
   - Case history paper
   - Conference paper
   - Construction safety document
   - Guideline - Manual
   - Other web resource
   - Presentation
   - Project drawing/photos
   - Project report
   - Project specification/criteria
   - Research article/paper
   - Trade Magazine/ Newsletter
   - Thesis
4. Review priority index.
   A simple index 1 to 3 to assigned initially based on a brief cursory review, to assist prioritizing later review work
5. Directly applicable to large OD tunnels? (Y/N)
6. US or international related work? (Y/N)
7. Design Code (ACI, ASTM, EN-Eurocode, FHWA, *fib*, LRFD, other ULS/SLS, AASHTO)
8. Liner design related? (Y/N)
9. Construction practice related? (Y/N)
10. Material type / Strength related? (Y/N)
11. Material type (Combination, RC, FRC, Hybrid)
12. Ring beam analysis (Y/N)
13. Finite element analysis (n/a, Structural analysis FEA, Geotechnical/tunneling analysis FEA)
14. Fire performance related? (Y/N)
15. Seismic performance related? (Y/N)
16. Large scale testing related? (n/a, Ring, Segment, Joint)
17. Cost related? (Y/N)
18. Diameter (min – max)
19. Filename
20. Publication date
21. Publication entity
22. Location of publication
23. Summary

3 LARGE DIAMETER BORED TUNNELS

3.1 Geometric Aspects for Highway Tunnels

Diameters of tunnels constructed utilizing tunnel boring machines (TBM's) have been increasing during the past two decades. Refer to Appendix A for information gathered as part of this literature survey that illustrates a trend of increasing tunnel diameters over the past 26 years. Establishing what is generally considered "large" for tunnel diameters was the first topic addressed in this literature survey. Various authors, including Grübl, 2012; Herrenknecht, 2012; Bäppler, 2014 and others, cite tunnels with diameters greater than 12 m (40') as fitting into this category. Highway or multi-modal purpose tunnels with two- or three-lane single deck or twin-deck configurations are often constructed with diameters larger than 14 m (46'). A synopsis on the history of large diameter tunneling can be found in Herrenknecht, 2012. One of the earliest examples of large diameter tunnel projects discussed in Herrenknecht (2012) is the mass-scale Trans Tokyo Bay Highway Tunnel Project. Work on the project was performed between 1994 and 1997 and illustrates what was technically feasible at the time.

Figure 3-1: Artistic exhibit of a 14.1m TBM cutterhead used at the Trans Tokyo Bay Highway Tunnel, at the Kawasaki artificial island in Japan. Photo: FHWA.

The minimum tunnel cross section is largely a function of roadway geometrics, internal service space needs, fire safety, emergency egress, ventilation, constructability and cost. Roadway geometrics are directed by national and state highway requirements. The voluntary and non-binding AASHTO DCRT-1 Technical Manual for Design of Road Tunnels (2010) which draws on the policies set forth in AASHTO's Policy on Geometric Design of Highways and Streets (currently in its 2018 version which is voluntary and non-binding) contains information for determining space needs. The latter, also referred to as the Green Book, presents the general design considerations used for road tunnels from the standpoint of service level and suggests dimensions for road tunnels that do not differ materially from those used for grade separation structures. Per AASHTO 2018, the same design criteria for alignment and profile and for vertical and horizontal clearances generally apply to tunnels except that minimum values are likely used because of high cost and restricted right-of-way. Users of these documents should determine the applicability of the information they contain and ascertain which documents are incorporated by reference into FHWA regulations.

The AASHTO Green Book contains basic geometric information about cross-section elements and other items specifically for road tunnels. AASHTO (2010) suggests that in addition to the information contained in the Green Book the geometric configurations should consider the following:

- A Policy on Design Standards—Interstate System (AASHTO, 2005)
- Standards issued by the state or states in which the tunnel is situated
- Local authority standards, where these are applicable
- National and local standards of the country where an international crossing tunnel is located

Tunnels utilize fire life safety elements, including ventilation, lighting, traffic control, fire detection and protection, communication, and others. Therefore, planning and design of the alignment and cross section of a road tunnel should also consider the National Fire Protection Association 502–Standard for Road Tunnels, Bridges, and Other Limited Access Highways (NFPA, 2008).The following geometric information is suggested in the 2018 Green Book:

- The minimum roadway width between curbs, should be at least 0.6 m (2 ft) greater than the approach traveled way, but not less than 7.2 m (24').
- Where sidewalks are provided for emergency egress by pedestrians, they should be designed to be accessible to and usable by pedestrians with disabilities
- In long tunnels, 60 m (200') or more in length, the sidewalk width should be at least 1.2 m (4') with passing sections at least 1.5 m (5') wide every 60 m (200'). Since varying the tunnel cross section to provide passing sections may be impractical, sidewalks with a continuous width of 1.5 m (5') should be provided.
- The total clearance between walls of a two-lane tunnel should be a minimum of 10.5 m (34.5') for long tunnels.
- The roadway width and the curb or sidewalk width can be varied as needed within the total tunnel width; however, each width should not be less than the minimum value stated above.
- The minimum vertical clearance is 4.9 m (16') for freeways. However, the minimum clear height for all tunnels should not be less than that on the road leading to the tunnel, and it is desirable to provide an allowance for future repaving of the roadways.

In both the AASHTO Green Book (2018) and the DCRT-1 Technical Manual (2010), it is noted that costs associated with tunneling, especially in the case of long tunnels may be prohibitive. As result many factors are evaluated by designers, including but not limited to design speed, lane and shoulder width, tunnel width, horizontal and vertical alignments, grade, stopping sight distance, cross slope, super-elevation, and horizontal and vertical clearances, on a case-by-case basis to establish a safe roadway structure.

Considering these aspects, AASHTO's (2018) minimum recommended space for a two-lane vehicular tunnel, is shown in Figure 3-2 and suggests an interior width in the range of 15 m (49'). Several international guidelines, such as from BTS (2000), DAUB and TBM experts (Maidl et al.,

1996) note that due to the TBM drive and lining ring construction, tolerances in the order of ±50 to ±100 mm (2” to 4”) on radius can be expected. As a result, some international guidelines, which are not binding, suggest that the interior diameter of the tunnel be increased by 100 to 200 mm (4” to 8”). As will be described later, for tunnels in the diameter range that will be considered for this study, a precast segment thickness in the range of .6 m (24”) can be assumed for preliminary planning. The total TBM overcut (tail clearance, shield plate thickness) should also be added to the above for calculating the total bored diameter. Using the AASHTO-recommended geometry for a two-lane tunnel and the assumptions above results in an external (bored) diameter of more than16 m (52’).

As described in the following section, several large diameter tunnels with a variety of configurations were reviewed in this study. These projects had diameters in the range of 11.2 m (36’-9”) to 17.6 m (57’-9”), with a typical diameter for most being more than 15 m (49’-2”). As such, and in the context of technical developments in the TBM and tunneling industry, this research work will focus on the performance of precast concrete segmental lining of diameters greater than 12 m (40’).

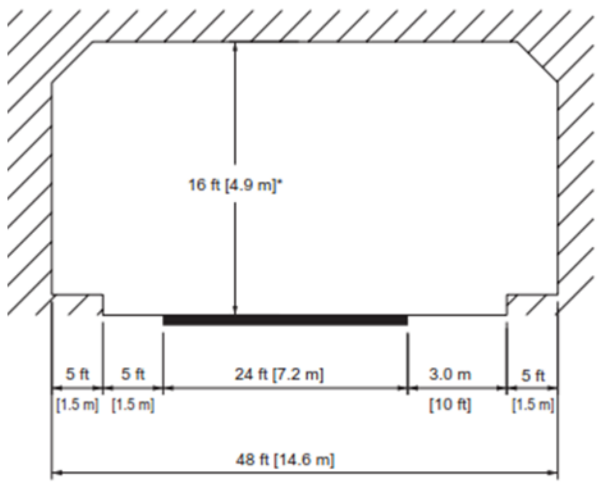


Figure 3-2: Desirable clearances for long tunnels greater than 200 ft (AASHTO Green Book, 2018). Figure courtesy of AASHTO.

3.2 State of the Art – Large Diameter Tunnel Projects

Several large diameter tunnel projects have been completed or are currently in-progress in the United States, while feasibility studies have also been performed for various sites, exploring a large diameter TBM option. Below is a brief synopsis of these projects:

3.2.1 Completed and In - Progress Tunnel Projects

Alaskan Way Viaduct Replacement Program – State Route 99 Tunnel

The Alaskan Way Viaduct Replacement program was a joint effort of the Washington State Department of Transportation, the Federal Highway Administration, King County and the City of Seattle. The project entailed the replacement of an aging, reinforced concrete, double-level viaduct in downtown Seattle – the Alaskan Way Viaduct. The replacement involved cut and cover approach tunnel structures and a massive single bore main tunnel in the center. The bored tunnel was 1.8 miles long, with an inside diameter of 16 m (52’)., and excavation diameter of 17 m (57’-

3''). Segmental lining was composed of universal tapered rings with .6 m (24") thick, steel rebar reinforced concrete segments. Each 2 m (6.5') wide ring employed ten segments: seven rectangular, two trapezoidal and one key segment. The large diameter "Bertha" earth pressure balance (EPB) TBM broke through in April 2017. The SR-99 tunnel is currently one of the two largest diameter TBM tunnels in the world. The tunnel has a two-deck, two-lane per deck configuration, with side and invert level spaces designated for egress, utilities and ventilation.

Figure 3-3: Alaskan Way Viaduct replacement project alignment in downtown Seattle. Figure courtesy of WSDOT.

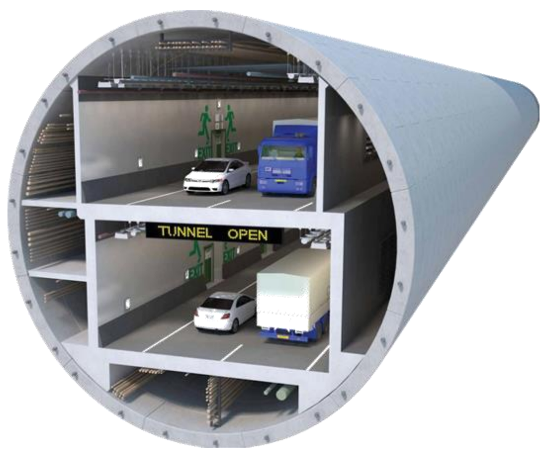


Figure 3-4: Cross section rendering of the Alaskan Way Viaduct. Figure courtesy of WSDOT.

The Port of Miami Tunnel

The Port of Miami tunnel is a Public-Private Partnership program that included the construction of a new twin ¾-mile long vehicular tunnel between Watson Island and Dodge Island and provides a link to the local interstate highway system and the Port of Miami. The bored tunnels with an inside diameter of 1 m (') and excavation diameter of 13 m (42.3') Segmental lining was composed of universal tapered (single side tapered) rings with .6 m (24") thick, steel rebar reinforced concrete segments. Each 2 m (5.6') wide ring employed eight segments: five rectangular segments, two trapezoidal segments and one key segment. The two tunnels were completed in May 2013. The tunnel has a single deck, two-lane configuration, with sidewalks.

The Chesapeake Bay Bridge Tunnel (Parallel Thimble Shoal Tunnel)

The Parallel Thimble Shoal Tunnel is an on-going Design-Build Program by the Chesapeake Bay Bridge Tunnel (CBBT) district, located in Virginia's Hampton Roads area. The project entails the construction of new approach structures and a new immersed roadway tunnel under the Thimble Shoal Channel, parallel to existing, immersed tube Thimble Shoals tunnel. The new tunnel will serve the southbound traffic and the existing tunnel will continue carrying only the northbound traffic. The new tunnel will be approximately 1.1 miles long, with an internal diameter of 12 m (39') and external diameter of 13 m (42') Segmental lining is to be composed of universal tapered rings with .5 m (18") thick trapezoidal segments (based on published preliminary information). The precast segments will be composed of fiber-only reinforced concrete. Each 2 m (6.5') wide ring is configured with nine segments: five rectangular segments, two trapezoidal segments and one key segment. The tunnel will have a single deck, two-lane configuration with separate egress corridor space. Construction is expected to be complete by 2023.

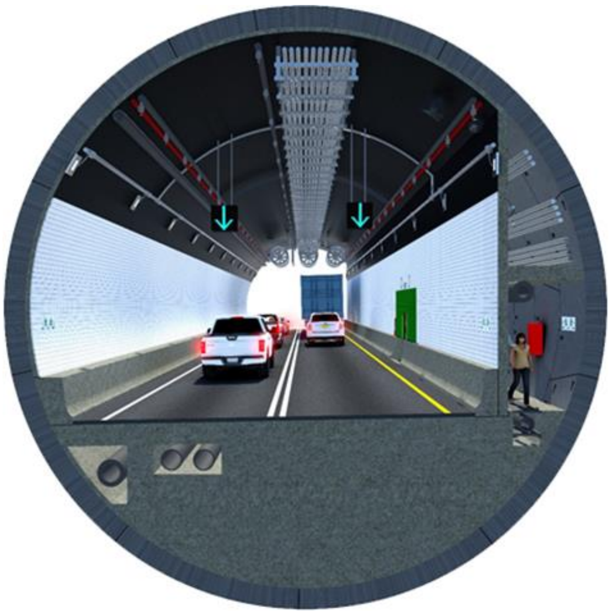


Figure 3-5: Cross section rendering of the Parallel Thimble Shoal Tunnel (CBBT, 2018). Figure courtesy of CBBT.

Hampton Roads Bridge Tunnel Expansion

In 2018 the Virginia Department of Transportation announced that this new Design-Build project will proceed with a bored tunnel option for the next connection across the Hampton Roads Harbor. A new tunnel connection with a four-lane capability has been prescribed in design-build tender documents. The internal diameter of the bored tunnel is not less than 13 m (41.5'). The project has been awarded and is currently undergoing final design.

3.2.2 Feasibility Studies in the United States

Several feasibility studies have been completed or are underway across the country, exploring options involving large diameter tunnels. The following is a summary of such known cases.

California

- State Route 710 North Project: The California Department of Transportation (Caltrans) in collaboration with the Los Angeles County Metropolitan Transportation Authority, is leading the environmental review process for multi-modal options for the State Route 710. Single and dual bore large diameter tunnel alternatives have been presented as options for an unfinished segment of State Route 710 in Los Angeles (Caltrans, 2018). In this concept the tunnel has a length of 6.3 miles and it would have a double-deck, two-lane per deck configuration with an outside diameter of approximately 18 m (60’).
- BART Silicon Valley Phase II: The Valley Transportation Authority’s Silicon Valley Extension program includes an extension of the BART subway system through downtown San Jose. A single bore large TBM option with an estimated external diameter approximately 14 m (45’) (TunnelTalk, 2019) was recently selected as the preferred option (VTA, 2018). The underground structure would be configured as a twin-deck, twin-track per deck tunnel.

New York

- Brooklyn-Queens Expressway: In 2016, a geometric feasibility study was released by New York City DOT, exploring different tunnel size and alignment options as possible replacement options for the ailing bridges of the Brooklyn-Queens Expressway Corridor. A single-bore, double-deck, two-lane per deck tunnel geometry with a 16 m (54’) diameter was presented as a viable but technically challenging option (NYCDOT, 2016).
- Long Island Sound Crossing: A feasibility study by the New York State DOT was released in 2017, exploring the potential of a Long Island Sound Crossing from the northern shore of Long Island to the northern shore of the Sound. An option with a single-bore double-deck, two-lane per deck tunnel with 18 m (58’). diameter was presented in this study (NYSDOT, 2017)
- I-81 Viaduct Replacement – City of Syracuse: In 2017 a feasibility study was released by the New York State DOT, for tunnel and depressed highway options for the Viaduct carrying the I-81 Interstate, which currently passes through Syracuse. Among the feasible options the study presented alternatives with a single-bore 17 m (57’) diameter tunnel or two 13 m (44’) diameter bores.

3.2.3 Literature Survey of Large Diameter Tunnel Projects

A list of 28 international projects involving large diameter tunnels was developed based on published information and WSP project data. This list contains projects that are complete or still in progress. Where available, additional data including roadway and tunnel layout, segment type and segmentation, reinforcement type, liner dimensions, water pressure, TBM torque and thrust, were included in the database, based on various online sources, site visit documents and other published literature. Due to the relevance or availability of data, select cases of lesser diameter projects were also included (i.e. the Port of Miami Tunnel and the A86 East Tunnel Paris). Detailed information about specific design aspects of some of these tunnels was not available as many of these projects were completed within recent years and information is not yet published. The intent of this section is to provide a current-state-of-practice in the construction of large diameter tunnels, highlighting the range of diameters achieved in modern projects, some design features and trends observed between the increasing size and geometry of the lining. The TBM-specific technology are not discussed here.

The advancement of TBM technology in the past twenty plus years has led to a continuous increase in tunnel diameters. The prospect of a single underground structure that accommodates multi-lane roadway traffic or multi-modal service, eliminating the need for multiple tunnels with cross connections between them, has led to international adoption of large diameter tunnels. A summary of TBM diameters during the period 1985-2015 is presented in Herrenknecht and Bäppler, 2012 showing a clear trend for roadway and multimodal tunnels. A similar trend is shown in Berger et al. (2018) which describes the increasing demand of tunnel boring machines with diameters greater than 14 m (46') in the period 1994 to 2017 (Burger et al., 2018). Within the last decade, there is a notable increase in the use of large diameter boring machines. New large diameter tunnel projects are currently in progress in many parts of the world (TunnelTalk, 2019), especially in Asia and Australia. Two of the largest diameter tunnels constructed to date, are the Tuen Mun – Chek Lap Kok subsea tunnel in Hong Kong with a diameter of 17.6m (58.1') and the Alaskan Way Viaduct (SR-99) Tunnel in Seattle at 17.48m (57.7') diameter. In Japan, Tokyo's new Ring Road project also involves multiple large diameter tunnels with excavation diameters just over 16 m (52.5'). The Orlovsky Tunnel in St. Petersburg in Russia, which has been revised to use two smaller TBM's, (TunnelTalk, 2019) would have been the largest on the list at a 19.25m (63.2') diameter.

3.2.4 Typical Precast Lining in Large Diameter Tunnels

This section presents a discussion on observed trends in the segmental lining of international tunnel projects. Typical design elements and practical sizing rules often used in preliminary engineering of segmental lining, are presented in Section 4.2 for the general case of segmental lining.

The excavation diameter of the tunnel projects reviewed varied from 11.6 to 19.25 m (38' to 63.2') (the latter corresponding to a recently revised concept for Orlovsky tunnel in St. Petersburg, Russia) with the most frequent diameters in the range of 13 to 15 m (42.7' to 49.2'). Internal diameters varied from 10.4 to 17.3 m (34.1' to 56.8'). As shown in Figure 3-6, the increase in diameter shows a corresponding increase in lining thickness which varied in the range of 0.4 to 0.76 m (16" to 30"). Of the tunnels reviewed, all but the Trans Tokyo Bay Tunnels are "single pass lining" types of tunnel structures. The Tokyo Bay tunnels also included a second pass cast-in-place lining. Interestingly, projects with FRC or hybrid reinforced segmental linings, such as the Brisbane Airport Link and West Gate in Australia, and the Waterview Tunnel in New Zealand make use of Steel Fiber Reinforced Concrete (SFRC) technology resulting in a thinner lining. On the opposite side, large diameter projects, including the Alaskan Way Viaduct in Seattle, the Tuen-Muen Check Lap Lok tunnel in Hong Kong, the Galleria Sparvo Tunnel in Italy, the 4th Elbe Tunnel in Germany, the M-30 "Calle" Tunnel in Spain and the Bund Tunnel in Shanghai, most of which use conventional rebar reinforcement, have a higher thickness to diameter ratio. As will be discussed in Chapter 6, a common metric used for preliminary dimensioning of segmental lining is the lining's "slenderness ratio" (referred to as the tunnel lining's "aspect ratio"), which is the ratio of the lining system thickness over the internal diameter and is plotted in Figure 3-7. This ratio varied in the range of 1/29.1 to 1/17.6, with most values observed in the range of 1/24 to 1/20 which closely follows the industry practice.

The number of segments increases as the tunnel diameter increases as shown in Figure 3-8. The large diameter tunnels examined used 8 to 13 segments (the latter for the recently revised Orlovsky Tunnel), with most projects using 9 or 10 segments per ring. Ring segmentation is a design process involving not just structural design performance, but also other factors such as segment delivery logistics and weight limitations, as well as compatibility with a specific TBM thrust cylinder array. Based on these records it can be projected that larger tunnels in the future may exceed the apparent twelve-segment ring threshold today in order to utilize manageable sized segments. Although the exact ring type was not available for all projects, both universal tapered ring and left/right tapered ring types are used successfully. The universal tapered ring appears to be a popular choice by designers and contractors.

Although exact segment weight data were only available for a few projects, certain trends can be established in Figure 3-9 which shows the reported maximum segment weight based on the reviewed sources for nine projects. The segment weight clearly shows an increasing trend with diameter. To compensate for the lack of segment weight data, the back-calculated total ring volume (excluding ring taper effects) is also plotted against the number of segments per ring in Figure 3-10. Despite the clustering around projects with 9 to 10 segments a mild trend of increasing volume (thus weight) vs number of segments can be observed.

As the diameter increases, the TBM performance needs increase as well, as demonstrated in Figure 3-11 to Figure 3-13. This in turn affects the design of the segmental lining and possibly in new ways not previously considered. Figure 3-11 shows the rapid increase in thrust capability large diameter shields have vs. tunnel diameter. Modern large diameter shield machines are outfitted with increasingly more powerful (greater torque capacity) cutterheads as observed in Figure 3-12 which also shows an expected difference between Earth Pressure Balanced (EPB) shields and Slurry or Mixshield TBM units. Typically, the EPB TBMs utilize greater torque than similarly sized Slurry TBMs. The use of more powerful TBMs, able to cope with mixed ground profiles and large face areas, translates to significant torque increases as shown in Figure 3-13, which also demonstrates the higher torque performance trend observed in EPB vs Slurry machines. From these plots, it can be inferred that in large diameter tunneling, the interaction level between these large TBMs and the segmental lining is much more intense. As pointed out by Bambridge, (2013) with the advent of large diameter machines, greater torque transfer may be experienced by the segmental lining, creating design issues for the tunnel lining.

Finally based on the data collected, both the glued-on and cast-in (anchored) types of segment gaskets have been successfully used in various projects. More information regarding gaskets and their design is presented in Chapters 6 and 7. Glued-on gaskets were more frequent amongst the projects reviewed given that cast-in gaskets are a newer technological development. In most cases a single gasket is the typical standard in waterproofing. In three cases, the Eurasia Tunnel in Turkey, the 4th Elbe Tunnel in Germany and the Trans Tokyo Bay Tunnels, twin gaskets were used in the segments.

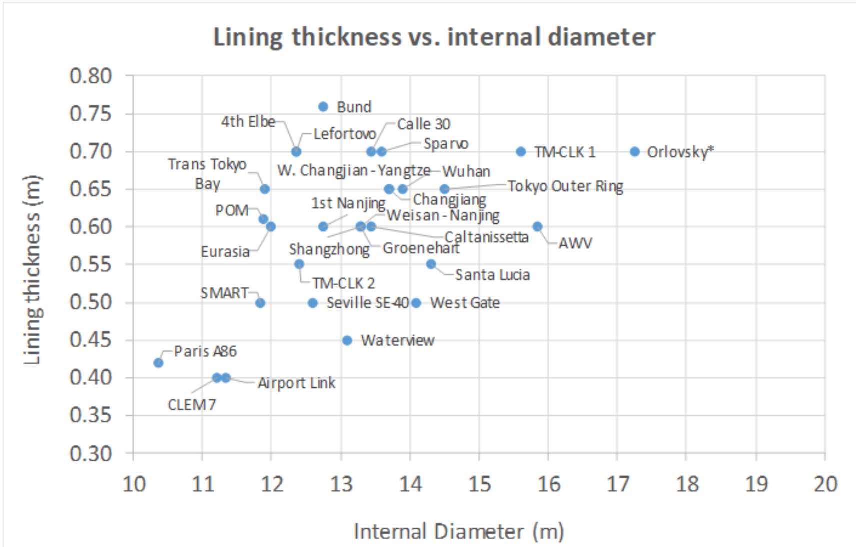


Figure 3-6: Lining thickness vs internal diameter in large diameter tunnel projects. Figure: FHWA.

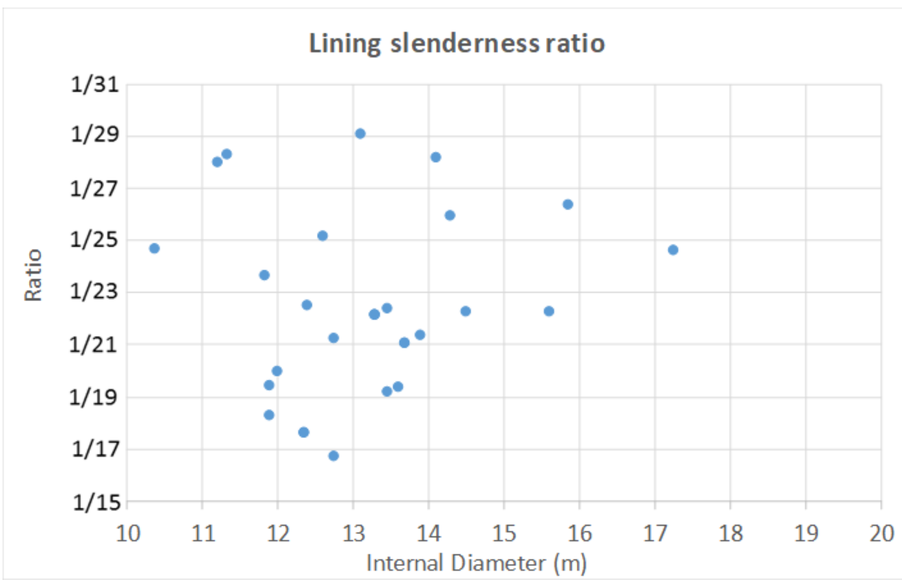


Figure 3-7: Distribution of lining slenderness ratio vs internal diameter. Figure: FHWA.

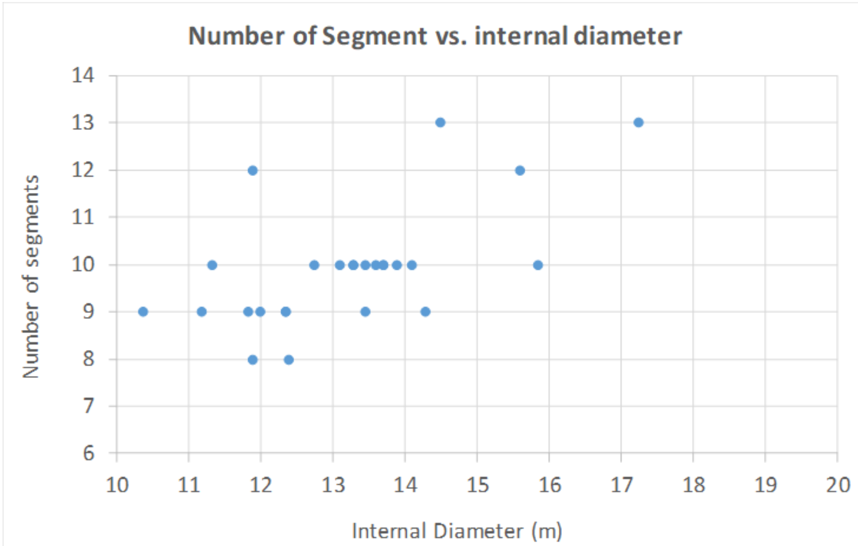


Figure 3-8: Number of segments vs internal diameter. Figure: FHWA.

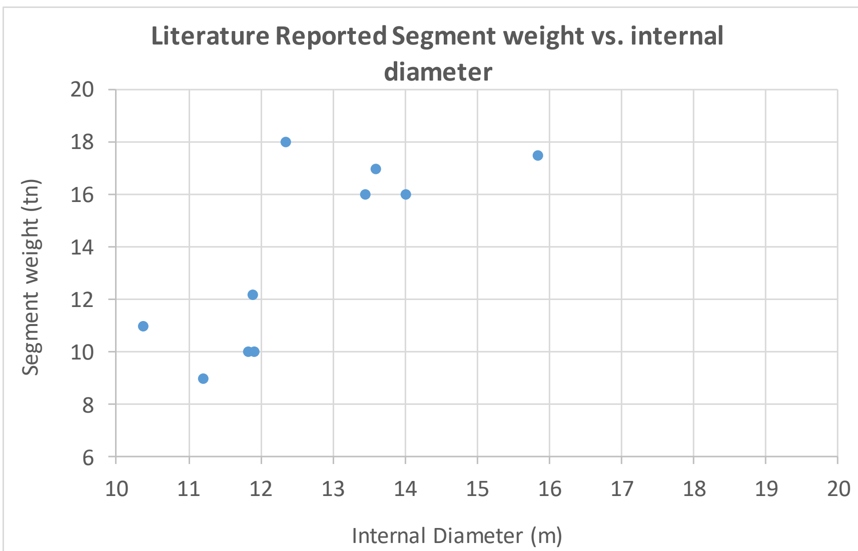


Figure 3-9: Reported segment weight vs internal diameter. Figure: FHWA.

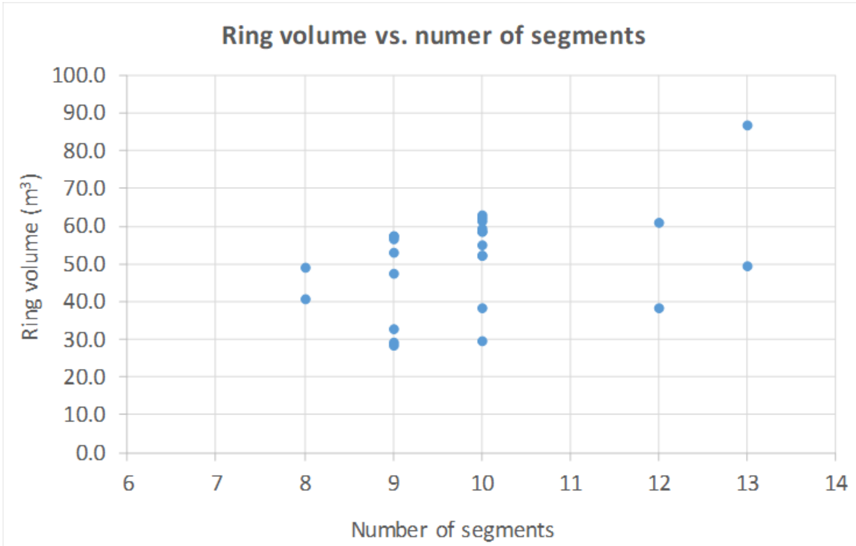


Figure 3-10: Average ring volume vs number of segments. Figure: FHWA.

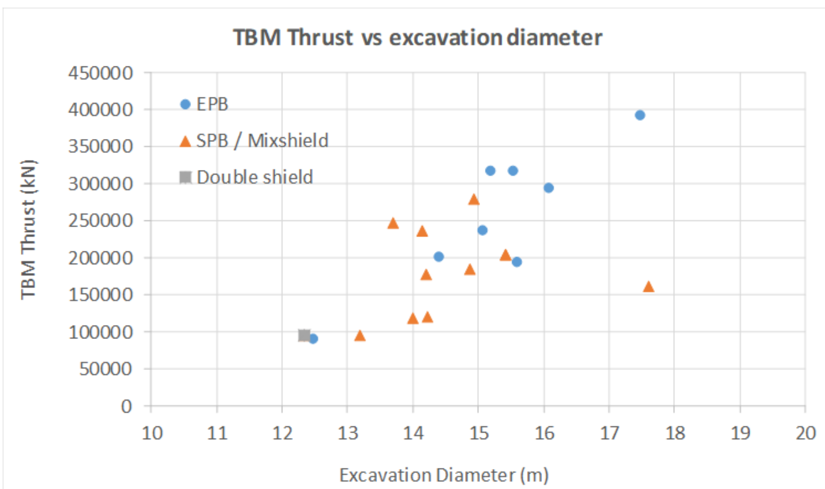


Figure 3-11: TBM thrust capacity vs excavation diameter. Figure: FHWA.

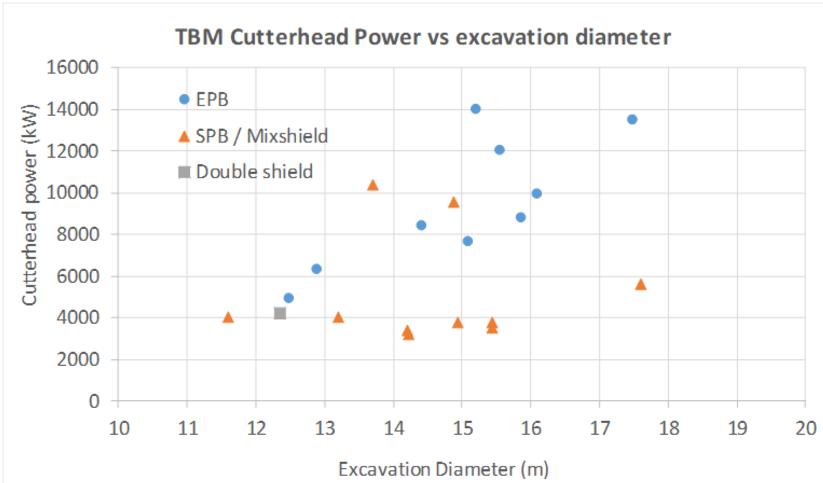


Figure 3-12: TBM cutterhead rotational power vs excavation diameter. Figure: FHWA.

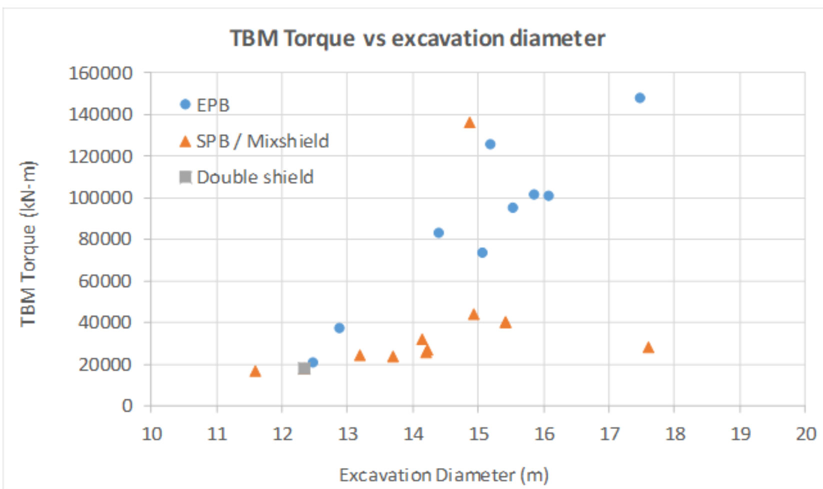


Figure 3-13: TBM torque vs excavation diameter. Figure FHWA.

4 DESIGN OF SEGMENTAL LININGS

4.1 General Approach of Structural Design

4.1.1 Introduction

There are no generally accepted standards or guidelines for the detail design of precast segment lining geometry and reinforcement in use in the US. Design of segmental linings has developed independently of codification. This chapter discusses both binding and non-binding, voluntary standards or documents issued by private organizations, FHWA, or DOT. This chapter also discusses non-binding, voluntary international documents. Analysis techniques used are widely accepted in the industry for determining load effects, but the design of the segments is left to the Engineer of Record or owner to dictate how the segments are sized. The American Concrete Institute and AASHTO have published standards that are adopted by tunnel owners for use in the design of segmental linings. These non-binding standards provide strength reduction factors to be applied to materials and load factors and load combinations to be applied to loads. Although experience has proven these standards produce safe designs, there is no specific standard of practice for the design of segmental liners. In addition, there are differences between the codes and designers are cautious not to mix codes due to the calibration of the load and resistance factors used within a specific code. Furthermore, the non-binding standards were not developed for the design of underground structures, so the load combinations are often not directly applicable to underground design.

4.1.2 Available References

4.1.2.1 References Issued by U.S. Private Organizations, FHWA, or DOT

American Association of State Highway and Transportation Officials (AASHTO)

The AASHTO-DCRT-1 “AASHTO Technical Manual for Design and Construction of Road Tunnels – Civil Elements” (2010) is based on the FHWA publication FHWA-NHI-10-034, incorporating editorial improvements and minor technical revisions. Both documents are of these documents voluntary and non-binding. The scope of the manual is limited to civil elements of design and construction not including fire life safety, ventilation, lighting, drainage, etc.

The binding requirements in AASHTO-LRFD-8 “AASHTO LRFD Bridge Design Specifications,” 2017 (23 CFR 625.4(d)(1)(v)) are intended for the design, evaluation, and rehabilitation of both fixed and movable highway bridges. Mechanical, electrical, and special vehicular and pedestrian safety aspects of movable bridges are not covered. Provisions are not included for bridges used solely for railway, rail-transit, or public utilities. The design provisions of this document employ the Load and Resistance Factor Design (LRFD) methodology. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance. Methods of analysis and modeling techniques are included.

The voluntary and non-binding AASHTO-LRFDTUN-1 “LRFD Road Tunnel Design and Construction Guide Specifications” (2017) is intended for the design, evaluation, and rehabilitation of highway tunnels constructed using the cut-and-cover, bored, mined, and immersed tunnel construction methodologies. It includes loads and load combinations, materials, geotechnical considerations, ground support and improvement, seismic considerations as well as planning considerations. The design provisions of these Specifications employ the Load and Resistance

Factor Design (LRFD) methodology. The load factors have been calibrated using structural analysis modeling for a limited number of loading conditions that take ground/structure interaction into account. This document refers to AASHTO-LRFD-8 for the design of structures internal to tunnels that support roadways over ventilation plenums, roadways, or other openings in the tunnel. The load effects of these internal structures should be applied to the tunnel lining, walls, or other supporting members in accordance with these Specifications. These Specifications are an initial attempt standardize highway tunnel design. As such, as future data produced in a systematic fashion in accordance with these Specifications become available, recalibration may be implemented based on statistical evaluation of these data.

The AASHTO-GDHS-7 "A Policy on Geometric Design of Highways and Streets" (2018 – voluntary and non-binding) provides geometric design information based on established practices supplemented by recent research. The document is intended as a comprehensive reference manual to assist in administrative, planning, and educational efforts pertaining to design of new construction projects on new location or designing reconstruction projects on an existing location. This policy is not intended as a prescriptive design manual or a policy for resurfacing, restoration, or rehabilitation (3R); traffic engineering; safety; and preventive maintenance-type projects that include very minor or no roadway work.

The voluntary and non-binding AASHTO RSDG-4 "Roadside Design Guide" (2011) focuses on roadside safety. This document discusses costs, geometry, topography, drainage, roadside features and barriers, railings and transitions, end treatments, safety features for work zones, and other safety related topics.

American Concrete Institute (ACI)

The ACI 318-14 and ACI 318R-14 "Building Code Requirements for Structural Concrete and Commentary" (2014 – voluntary and non-binding) provides information for the materials, design, and detailing of structural concrete buildings and, where applicable, nonbuilding structures. This Code addresses structural systems, members, and connections, including cast-in-place, precast, plain, non-prestressed, pre-stressed, and composite construction. Among the subjects covered are: design and construction for strength, serviceability, and durability; load combinations, load factors, and strength reduction factors; structural analysis methods; deflection limits; mechanical and adhesive anchoring to concrete; development and splicing of reinforcement; construction document information; field inspection and testing; and methods to evaluate the strength of existing structures.

The ACI 350-06 "Code Requirements for Environmental Engineering Concrete Structures and Commentary" (2006 – voluntary and non-binding) provides information for the materials, design, and detailing of structural concrete exposed to environmental conditions, materials or chemicals. Like ACI 318-14, this code covers the design and construction for strength, serviceability, and durability; load combinations, load factors, and strength reduction factors; structural analysis methods; deflection limits; mechanical and adhesive anchoring to concrete; development and splicing of reinforcement; construction document information; field inspection and testing; and methods to evaluate the strength of existing structures. Examples of when this code should be used in lieu of ACI 318-14 are in regions where above average snow fall/ice conditions occur, near or exposed to ocean salt water, where de-icing salts are used or where ground water has a high-level concentration of saline or other contaminates.

The ACI-544.4R-18 "Guide to Design with Fiber-Reinforced Concrete" (2018 – voluntary and non-binding) aims at providing practicing engineers with design guidelines and recommendations for fiber reinforcement used in structural and nonstructural applications and made of steel fibers and polyolefin synthetic macro-fibers that comply with ASTM C1116/C1116M. Typical dosages for fibers, general material properties, and available test methods for characterization of FRC are discussed. This voluntary and non-binding document focuses on design concepts and existing guidelines for fiber reinforcement, design for specific applications, and construction practices for specifying and building with FRC.

The ACI 544.5R-10 "Report on the Physical Properties and Durability of Fiber-Reinforced Concrete", (2010 – voluntary and non-binding) addresses the physical properties and durability of FRC. The effects of fiber reinforcement are evaluated for various physical, short-term, and long-term benefits they impart to the concrete mixture based on various testing methods. The various properties listed, in addition to the wide variety of the choices available in formulating matrix systems, allow performance-based specification of concrete materials using fibers to become a viable option. This document provides a historical basis and an overview of the current knowledge of FRC materials for tailoring new, sustainable, and durable concrete mixtures.

The ACI 544 7R-16 "Report on Design and Construction of Fiber-Reinforced Precast Concrete Tunnel Segments" (2016 – voluntary and non-binding) offers general information on the history of FRC precast segments from tunneling projects throughout the world and proposes a procedure for designing FRC tunnel segments to withstand all the appropriate temporary and permanent load cases occurring during the construction and design life of tunnels. A procedure for structural analysis and design based on governing load cases is provided along with a description of the material parameters, tests, and analyses to be performed to complete the design. The document is based on the knowledge gained from experimental research, analytical work, and the experience gained on numerous FRC precast tunnel projects.

The ACI 544.8R-16 "Report on Indirect Method to Obtain Stress-Strain Response of Fiber-Reinforced Concrete (FRC – voluntary and non-binding)" presents existing methods for estimating characteristic tensile stress-strain or tensile stress crack width response of strain-softening FRC using flexural beam test data. Methods are proposed for strain-softening FRCs that do not exhibit distributed or parallel micro-cracking when tested in flexural loading conditions, and strain-softening FRCs that do exhibit distributed or parallel micro-cracking when tested in flexural conditions. The report concludes with the relationship between the parameters for the stress-strain diagram and the experimental flexural residual strength.

Federal Highway Administration (FHWA)

The non-binding FHWA-NHI-10-034 "Technical Manual for Design and Construction of Road Tunnels – Civil Elements" (2009) is intended to be a single-source technical manual providing information for planning, design, construction, structural rehabilitation, and repair of road tunnels incorporating the LRFD methodology when applicable. The technical manual presents recommendations for mined, bored, cut-and-cover, immersed, and jacked box tunnels. It covers tunnel geometrics; investigative techniques and reports for planning, design and construction of road tunnels; seismic design; construction issues; instrumentation and monitoring; identification, characterization, and rehabilitation of structural defects.

The binding FHWA-HIF-15-005 "Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual," 2015 (23 CFR 650.517(c)(1)) was developed in accordance with the mandatory

National Tunnel Inspection Standards (23 CFR Part 650) to serve as a resource in the areas of tunnel operations, maintenance, inspection, and evaluation. The manual discusses the operational aspects of highway tunnels and common maintenance and repair issues for highway tunnels. It provides information for developing a comprehensive tunnel inspection program and load rating of highway tunnels.

The binding requirements in FHWA-HIF-15-006 "Specifications for the National Tunnel Inventory," 2015 (23 CFR 650.517(c)(2)) were developed in coordination with the FHWA NTIS regulation 23 CFR 650 Subpart E and the TOMIE Manual. It is intended to supplement the NTIS and provide the specifications for coding data to be submitted to the National Tunnel Inventory (NTI). Data in the NTI will be used to Federal reporting requirements set forth in the National Bridge and Tunnel Inventory and Inspection Standards (Section 144 of Title 23, United States Code) and provide tunnel owners, the FHWA and the public with information on the number and condition of the Nation's tunnels.

National Fire Protection Association (NFPA)

The NFPA-502 "Standard for Road Tunnels, Bridges and Other Limited Access Highways" (2017 – voluntary and non-binding) was prepared by the Technical Committee on Road Tunnel and Highway Fire Protection. This standard establishes minimums for fire protection and life safety for limited access highways, bridges and elevated highways, road tunnels, and roadways beneath air-right structures.

United States Department of Transportation (USDOT)

The DOT-TSC-UMTA-83-16 "Design Recommendations for Concrete Tunnel Linings Volume II: Summary of Research and Proposed Recommendations" (1983 – voluntary and non-binding) was developed from analysis and model testing of concrete tunnel linings in rock and soil at the University of Illinois at Urbana-Champaign. Aspects addressed include methods of analysis, design practices, loads, and load factors.

4.1.2.2 International References

All international references are voluntary and non-binding in the United States.

British Tunneling Society (BTS)

The PAS f:2016 "Tunnel Design - Concrete Segmental Tunnel Linings - Code of Practice" (2016) was developed specifically to cover the design of segmental tunnel linings and aims to bring together existing standards and industry documents into a single, usable standardization document while simultaneously reducing unnecessary administration and delay by streamlining, clarifying and standardizing the design process for segmental lining design. It covers design considerations from project inception through to the end of the service life of the tunnel. Clauses 4 to 8 cover the more general aspects of tunnel design and Clauses 9 to 12 provide specific, technical information on precast concrete lining elements for segmental tunnel linings.

The BTS "Tunnel Lining Design Guide" (2004) was created to cover the design of structural linings for driven tunnels and shafts. Consideration is given to fire resistance, failure mechanisms, tolerances, durability, water tightness, design and construction of junctions, portals and shafts, methods of analysis, and more. Some case histories are presented (limit state design).

The BTS "Specification for Tunneling Third Edition" (2010) is a model document intended to serve as a basis for materials and workmanship quality for tunnel projects including bored tunnels and shafts. In conjunction with the Specification component of the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls, the scope can be extended to include cut-and-cover tunnels and similar underground structures. The document covers materials and methods including but not limited to design and construction of segmental lining and gaskets, ground stabilization, and working environment conditions.

International Federation for Structural Concrete (*fib*)

The Model Code for Concrete Structures is an initiative taken by *fib*'s predecessors CEB (Comité Euro-International du Béton) and FIP (Fédération Internationale de la Précontrainte) at a time when there were hardly any international codes. Both organizations were aiming to synthesize international research and experience and to convert it into practical documents for design, so that national code commissions could utilize it. The first code-like recommendations in 1964 and 1970 were used in this way. The Model Code 1978 also contributed to international harmonization. The Model Code 1990 served as an important basis for the most recent version of Eurocode 2-Design of Concrete Structures. Model Code 2010 is the latest model code published by *fib*. The Model Code 2010 and other important *fib* documents relevant to segmental liner design are introduced below in more detail.

The *fib* "Structural Concrete, the Textbook on Behaviour, Design and Performance – Bulletin 51" (1999) includes three volumes and is written in modular form. Volume 1 considers the design process and material behavior. Volume 2 covers the basis of design, including limit states and detailing. Volume 3 includes chapters on durability, fire resistance, concrete member design, assessment, maintenance, repair, and practical aspects including tolerances. It is intended that the whole document be incorporated into contract documents by reference.

In 2002 *fib* established Task Group 5.6 "Model code for service life design of concrete structures" with the objective to develop a model code document on probabilistic service life design, which published the *fib* "Model Code for Service Life Design - Bulletin 34" (2006) in 2006. The approach developed in Bulletin 34 is also the basis for the service life design approach of the subsequent *fib* Model Code 2010.

The Model Code for Service Life Design identifies durability related models, prepares the framework for standardization of performance-based design approaches, and covers design for environmental actions leading to degradation of concrete and embedded steel. The basic idea of service life design is to establish a design approach that minimizes deterioration caused by environmental action comparable to designing for strength. That means quantifiable models on the load side (these are the environmental actions) and on the resistance side (this is the resistance of the concrete against the considered environmental actions). The first step in the design approach is to quantify the deterioration mechanism with realistic models describing the process physically and/or chemically with sufficient accuracy (e.g. ingress of carbonation into the concrete depending on the environment and the relevant concrete quality parameters). In the same way, models for the environmental actions with statistically quantified environmental parameters (e.g. temperature, relative humidity, splash rain events etc.) should be developed.

The second step is determining the limit states to be used for the design of the structure. Appropriate limit states would be de-passivation of reinforcement caused by carbonation, cracking due to reinforcement corrosion, spalling of concrete cover due to reinforcement corrosion, and collapse due to loss of cross section of the reinforcement. The objective of the *fib* "Model Code for Service Life Design" is to identify agreed durability related models and to prepare the framework for standardization of performance based design approaches. It treats design for environmental actions leading to degradation of concrete and embedded steel, the calculation of the probability that the limit states described above occur (determination of the probability of occurrence). The last step is the description of the type of limit state (SLS, ULS) of the limit states described above.

The *fib* "Practitioner's Guide to Finite Element Modeling of Reinforced Concrete Structures - Bulletin 45" (2008) provides an overview of concepts and techniques relating to computer-based modelling of structural concrete. The report is written primarily for the benefit of the practicing engineer, rather than as a state-of-the-art for researchers, concentrating more on practical application and less on subtleties in constitutive modelling.

The *fib* "Structural Concrete: Textbook on Behavior, Design and Performance - Bulletin 54" (2010) was prepared in the intermediate period from the "CEP-*fib* Model Code 1990" to *fib* "Model Code 2010" (see below). The updated Textbook provides the basics of material and structural behavior and the fundamental knowledge needed for the design, assessment, or retrofitting of concrete structures. It includes the following three sections: "Design of concrete buildings for fire resistance", "Design of members", and "Practical aspects" covering the phases of design process from conceptual design to structural analysis and design.

In 20l2, fib published "Model Code 2010 - Bulletin 65" (2012). The main intention of the *fib* Model Code 2010 is to contribute to the development of improved design methods, the application of improved structural materials, and to serve as a basis for future codes for concrete structures. Therefore, adequate attention is given to new innovative materials like high-strength concrete, steel fiber reinforced concrete and non-metallic reinforcement. Constitutive relations are given for concrete up to strength classes of C120 [120 MPa = 17,400 psi] for normal density concrete and LC80 [80 MPa = 11,600 psi] for lightweight concrete. In addition, design rules for fiber reinforced concrete are provided, which apply to the higher strength classes, too. Another important new aspect is the life cycle concept (see model code for service life design of concrete structures above), which serves as a basis for a holistic design approach. The *fib* Model Code 2010 covers the whole life cycle of a concrete structure, from design and construction to conservation (assessment, maintenance, strengthening) and dismantlement, and is applicable for buildings, bridges and other civil engineering structures. Design is largely based on performance of the feature. The chapter on materials is particularly extended with new types of concrete and reinforcement (such as fibers and non-metallic reinforcements). Most relevant and used in FRC segmental liner design is chapter 5.6, which covers fibers and fiber reinforced concrete and introduces the concept of evaluation of the notched 3-point bending test per EN 14651 using Crack Mouth Opening Displacement (CMOD) values and a concept to transfer these test results into constitutive material laws, including a stress-strain relationship. The importance of the *fib* Model Code 2010 lies in the fact that it provides a closed approach from tests to material laws that can be used for the structural design of fiber reinforced concrete for the service limit state (SLS) as well as the ultimate limit state (ULS).

Due to its closed design concept for FRC, the *fib* Model Code 2010 has sparked great interest in the tunneling community and several documents consider Model Code 2010 as a reference and tunneling projects use it as a design basis. For this reason, *fib* Task Group 1.4 “Tunnels" decided to create a working group on “Tunnels in Fibre-Reinforced Concrete”, which developed the state-of-the art report “Precast tunnel segments in fibre-reinforced concrete – Bulletin 83” (2017). The document aims to support designers, contractors and clients with information for the use of steel fiber-reinforced concrete in precast segmental lining tunnels constructed by TBMs. The document is intended to complement the *fib* Model Code 2010 considering it as the basis for the design of FRC segmental lining. The bulletin covers a wide range of design related topics and provides information for FRC segmental lining design in chapters about materials, design for transient load situations during production, TBM thrust phase, final state loading, fire exposure, connectors, durability, quality control, and sustainability. The bulletin also provides two case studies and appendices providing additional information to FRC segmental liner designers.

International Tunneling and Underground Space Association (ITA)

The ITA has produced several documents related to segmental tunnel linings.

The ITA “Guidelines for the Design of Shield Tunnel Lining” (2000) is a general report consisting of three parts that cover the design procedure, detailed design methods including allowable stress design and limit state design, and design examples, respectively, for reinforced concrete initial lining and cast-in-place secondary lining. In April 2019, the ITA released a revision of this report (ITA Report No. 22) titled “Guidelines for the Design of Segmental Tunnel Linings”. The report builds up on the elements of the 2000 report and further focuses on design methods, segment and ring geometry, loading conditions, reinforcement, testing, gasket materials and design, connection hardware and tolerance.

The ITA “Recommendations and Guidelines for TBMs” (2001) consists of four individual reports contributed by representatives from Japan, Norway, Germany, Switzerland, Austria, Italy, and France that provide comprehensive guidelines and recommendations for evaluating and selecting Tunnel Boring Machines (TBMs) for both soft ground and hard rock.

The ITA has also published guidelines and practice reports specifically on the design and use of fiber reinforced concrete in segmental lining. The ITAtech Report No 7 “ITAtech Guidance for Precast Fibre Reinforced Concrete Segments - Vol. 1: Design Aspects” (2016) is a report on the benefits and limitations of FRC and the combination of fiber reinforcement with steel reinforcement for precast concrete segmental linings. This document includes but is not limited to material properties, design, and testing.

The ITA Report No 16 “Twenty Years of FRC Tunnel Segments Practice: Lessons Learnt and Proposed Design Principles” (2016) discusses the advances and benefits of the use of FRC for precast segmental linings in terms of structural behavior and industrialized production. This document aimed to enhance existing standards and guidelines with lessons from real cases.

Bundesanstalt für Straßenwesen - Federal Highway Research Institute (BAST)

The “Additional Technical Contract Conditions and Guidelines for Civil Engineering Works - ZTV-ING - Part 5 Tunnel Construction - Section 3 Mechanical Shield-Driving Processes” (2010) applies to the design of road tunnels created by means of shielded TBMs. The document covers geotechnical considerations, permanent and temporary loads, construction monitoring, mechanical considerations of the TBM, waterproofing and drainage, structural fire protection, and segment design with respect to reinforcement, joints, and tolerances.

The "Technical Delivery Conditions and Technical Test Specifications for Sealing Profiles" (2010) contains provisions for the delivery of suitable elastomeric sealing profiles and describes the nature and scope of the tests to be performed for their use in precast concrete segments pursuant to ZTV-ING Part 5 Tunnel construction, Section 3. The document provides a list of standards and technical provisions, quality control and detailed description of tests to be performed with respect to gasket suitability.

Deutscher Ausschuss für Underirdisches Bauen – German Committee for Underground Construction (DAUB)

The "Recommendations for the Design, Production and Installation of Segmental Rings" (2013) by DAUB, gives an overview of the state of the art for design (limit states), production and assembly of precast segmental lining, including construction and dimensioning fundamentals, cross passages and portals, fire protection, and use of steel fibers.

The "Recommendations for Selecting and Evaluating Tunnel Boring Machines" (1997) examines the key factors that should be considered when selecting tunneling machines, such as route, gradient, geotechnical conditions, and environmental compatibility.

The "Recommendations for Design and Operation of Shield Machines" (2000) is related to the reports above and provides information on the application of systems and components as well as optimized possibilities of adjustment for operation and changing geotechnical conditions, including some project examples.

The "Recommendations for the Selection of Tunneling Machines" (2010) is a newer related document which includes types of TBM and criteria for selection.

Deutscher Beton- und Bautechnik-Verein E.V. - German Society for Concrete and Construction Technology (DBV)

DBV published the GSCCT "Guide to Good Practice-Steel Fibre Concrete" (2001), which covers materials, manufacturing, dimensioning and structural design (limit states with partial safety factors) including fire resistance, execution and monitoring of steel fiber concrete or combination with steel reinforcement as well as the tests to be performed and a design example. The document also classifies the steel fiber concrete in classes by means of equivalent tensile strengths.

Österreichischen Vereinigung für Beton- und Bautechnik - The Austrian Society for Concrete and Construction Technology (ÖVBB)

ÖVBB issued the "Concrete Segmental Lining Systems" (2011), a document on the design and implementation of precast segmental lining projects. This document incorporates findings from successful Austrian projects and addresses lining systems and geometry, waterproofing, loads, analysis and design, materials, testing, joints, production, installation, grouting, and tolerances.

Association Française des Tunnels et de l'Espace Souterrain - French Tunnelling and Underground Space Association (AFTES)

AFTES has published several relevant recommendations. The GT9R4A1 "AFTES Recommendations: Segmental Gaskets" (1993) provides general information for water tightness using compressible or water swelling gaskets, including groove tolerances and other key parameters as well as tests.

The GT9R6A1 "AFTES Recommendations: Watertightness of Precast Concrete Lining Segments" (1998) deals with potential causes affecting water tightness related to segment material, segment transport and storage, grouting, and others, and proposes preventative methods.

The GT9R9A1 "AFTES Recommendations: Hydrophilic Swelling Gaskets for Tunnel Lining Segments - Gasket Assessment and Quality Control Procedures" (1998) covers the use of hydrophilic swelling gaskets alone or in conjunction with conventional compression gaskets, including testing procedures.

The GT18R1A1 "AFTES Recommendations: The Design, Sizing and Construction of Precast Concrete Segments Installed at the Rear of TBM" (1997, 2005) is based on lessons learnt from the design, sizing and construction of precast concrete segmental linings. More specifically this document covers the main parameters influencing sizing design assumptions, limit state design, available methods, interaction of different elements of the lining, transitional sections to other structures, monitoring and instrumentation.

The GT38R1A1 "AFTES Recommendations: Design, Dimensioning and Execution of Precast Steel FRC Arch Segments" (2013) follows the previous recommendation published on reinforced concrete segments and draws from the Model Code 2010 (*fib*) on design, assembly, durability, fire performance, dimensioning, testing, production, installation and other key aspects of the use of FRC segments.

Chongqing Communications Research & Design Institute

The JTGD70 "Specifications for Design of Highway Tunnel" (2004) is formulated to provide technical criteria for the design of highway tunnels in China. It is applicable to two-lane highway tunnels, ranging from short length less than 500 m to extra-long over 3 km, with major excavation means of drilling and blasting method, and can be taken as reference for other forms of highway tunnels.

Japan Society of Civil Engineers (JSCE)

The "Japanese Standard for Shield Tunneling" was enacted in 1977, developed from the "Japanese Guideline for Shield Tunneling" (1969). It was revised in 1986, 1996, and 2000 and covers loads, materials, dimensioning, analysis, detailing, production, storage, transport, handling, tolerances, and other practical considerations. The allowable stress method is generally used in this manual. Limit state design was added in the latest version to complement the allowable stress method. The document suggests that limit state design should be used for design under extreme level earthquake. However, it does not suggest combining both methods for one structure. The 2007 English version was reviewed for the purposes of this synthesis.

Eurocode 2 – Design of Concrete Structures (BS EN 1992-1-1:2004)

The Eurocode 2 (2004) applies to the design of buildings and civil engineering works in plain, reinforced and pre-stressed concrete and is only concerned with the resistance, serviceability, durability and fire resistance of concrete structures, not considering thermal or sound insulation. The "Eurocode 2: Design of concrete structures Part 1-1: General Rules for Buildings" (EN 1992-1-1:2004) gives a general basis for the design of structures in plain, reinforced and pre-stressed concrete made with normal and light weight aggregates together with specific rules for buildings. The "Eurocode 2: Design of concrete structures Part 1-2: General Rules -Structural Fire Design" (EN 1992-1-2:2004) deals with the design of concrete structures for the accidental situation of fire exposure and only deals with passive methods of fire protection.

DB Netz AG

The "Richtlinie 853 Eisenbahntunnel planen, bauen, und instand halten, Deutsche Bahn DB (Guideline 853 Railroad tunnel design, construction, and maintenance, German Railroad)" provides mandatory technical provisions for planning and design, construction, and maintenance of railroad tunnels in Germany under consideration of the demands of the authority having jurisdiction for railroad in Germany, the Eisenbahn Bundesamt (EBA) (Federal Railway Authority). It is applicable for mined (independently from the length) and cut-and-cover tunnels (longer than 250 m (820')) on railroads with an operating speed up to 300 kph (186 mph). Chapter 12 covers segmental linings. The document provides general provisions regarding geometrical layout of joint, concrete cover, material properties, design for eccentric loads in joints, checks for tolerances, labelling, construction tolerances, acceptance, bolting, annulus grouting, waterproofing and gaskets. Reinforcement with FRC is not excluded, but is treated as construction with a non-regulated product.

Deutscher Ausschuss für Stahlbeton eV. (DAfStb) - German Committee for Reinforced Concrete

The "Richtlinie Stahlfaserbeton" – "Steel Fiber Reinforced Concrete Guideline" provides information for the properties and applications of steel fiber reinforced concrete as an amendment to DIN 1045-1 (German concrete design standard, comparable to ACI 318 in the US)), DIN EN 206-1, as well as two other DAfStb documents that also consider the utilization of steel fiber reinforced concrete. The document provides a classification in two performance classes based on the post-cracking behavior for either small deformations or large deformations respectively the combination with conventional reinforcement. Amendments and changes regarding DIN 1045-1 and EN 206-1 cover, amongst others, safety factor for an LRFD design concept, durability, a concept to evaluate section forces, material properties, ULS and SLS design, fresh concrete, classification, and quality control.

Italian National Research Council (CNR)

The CNR-DT204/2006 "Guide for the Design and Construction of Fiber-Reinforced Concrete Structures" focuses on the material behavior, composition and design of fiber reinforced concrete. Materials including steel, polymer and carbon fibers are described in the document, along with a detailed analysis on behavioral aspects of FRC, strength and elastic properties, physical properties, design approach and limit states. The document also provides direction for hybrid reinforcement designs with fiber and conventional reinforcement.

Studiengesellschaft für Underirdische Verkehrsanlagen - Research Association for Underground Transportation Facilities (STUVA)

The "STUVA Recommendations for Testing and Application of Sealing Gaskets in Segmental Linings" (2005) provides recommendations for testing the suitability of gaskets used in single-layer segmental linings. The document covers the necessary demands and related tests for selecting a suitable sealing gasket, provides pointers on the practical applications and recommendations for production control, quality monitoring, and delivery conditions. The recommendations are intended to ensure that the optimal gasket is chosen in accordance with comparable results.

The "STUVA Recommendations for the Use of Gaskets for Sealing Segmental Linings" (2006) provides recommendations for sealing single shell segmental linings. The document covers the course of joints, the demand on segment grooves, gasket geometrical recommendations, gasket assembly, gasket fire protection, application of anchored gaskets, and remarks on keystone gaskets.

STUVA released the 2005 and 2006 specifications for gaskets, as draft working specifications and as means to receive feedback from the industry. In 2019 TUVA released the final document.

Swedish Standards Institute (Svensk Standard)

The Swedish Standards Institute publication "Fibre Concrete – Design of Fibre Concrete Structures" SS 812310:2014, applies to the design of buildings and other civil engineering works in concrete with steel fibers and or polymer fibers per SS-EN 14889-1 and SS-EN 14889-2. In addition to fiber reinforcement, this manual provides information on design of hybrid fiber and steel rebar reinforced concrete.

Swiss Society of Engineers and Architects (SIA – Schweizer Norm)

In Switzerland, the 2004 version of the SIA codes 197, 197/1,197/2 and 198 govern the design principles for underground structures with SIA 197/1 focusing on rail and SIA 197/2 focusing on road tunnels. The basics of underground project execution and materials such as concrete segments, aspects of installation, monitoring and production tolerances are covered in SIA 198. SIA code 260 deals with general structural design (including limit states) that is also adopted for underground structures. Per Anagnostou and Ehrbar (2013) the structure of the SIA 197 presents important basic principles of design but is not highly descriptive, allowing for flexibility in the methods selected for structural design.

The International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM)

The "RILEM TC 162-TDF: Test and design methods for steel fibre reinforced concrete" (2000) is a RILEM recommendation for the design of steel fiber reinforced concrete per the (sigma)-(epsilon)-method. The European pre-standard ENV 1992-1-1 (Eurocode 2: Design of Concrete Structures - Part 1: General Rules and Rules for Buildings) has been used as a general framework for this design method. The method is valid for steel fiber concrete with compressive strengths of up to C50/60. This publication is intended for cases in which the steel fibers are used for structural purposes and does not apply for other uses of steel fiber such as increased resistance to plastic shrinkage or increased resistance to abrasion or impact.

4.2 Typical Design Aspects of Segmental Linings

4.2.1 Lining Dimensioning

Beside the internal diameter, a major design aspect of the lining design is the geometry of the segments and rings, including the lining thickness, the taper, segmentation, and ring length.

Thickness: The thickness of segments is generally determined per structural and constructional criteria. Initially and if not mandated by the specification, a reasonable thickness is assumed, which is then optimized through an iterative design process. The minimum thickness is mostly governed by the need to transfer the thrust cylinder forces and the load-bearing area of the thrust pads. AASHTO-DCRT-1 (2010) gives indicative segment thickness of 20 to 30 cm (8" to 12"), which is known to be low for large diameter tunnels. Typical thickness of 20 to 50 cm (8" to 20") is given by Maidl et al. (2012). DAUB (2013) states that the segment thickness is based on static and structural factors (e.g. sealing details, durability) and is generally between 15 cm (6") and approximately 75 cm (30"), while suggesting that it should not be under 30 cm (12") for one-pass lining. ZTV-ING (2010) also recommends a minimum of 30 cm (12") for one-pass lining. A range for FRC precast segment thickness between 15 and 40 cm (6" and 16") is given in ACI 544-7R-16 for existing tunnels with internal diameters ranging between 2.2 and 11.4m (7.2' ft and 37.4') based on the literature.

Lining slenderness ratio: There is a correlation between lining thickness and tunnel diameter typically referred to as lining slenderness or lining aspect ratio. The dimensionless slenderness ratio of the lining thickness to the internal tunnel diameter (ID) typically falls into a range of 1/25 to 1/18 for tunnels with ID of more than 5.5 m (Bakhshi and Nasri, 2018). JSCE (2006) suggests a slenderness ratio, (the thickness divided by the diameter) of about 4% (1/25) or more. For FRC or hybrid segments ACI 544-7R-16 has recorded values between 1/31 to 1/12 for diameters between 2.2 and 11.4m (7.2' and 37.4'). Four cases with diameters between 10.9 and 11.4 m (35.8' and 37.4') involved ratios in the upper end of the range, between 1/31 to 1/28. Only one of them is FRC alone and three are hybrid. Figure 4-1: shows the slenderness ratio over the external diameter by JSCE displaying results from actual projects.

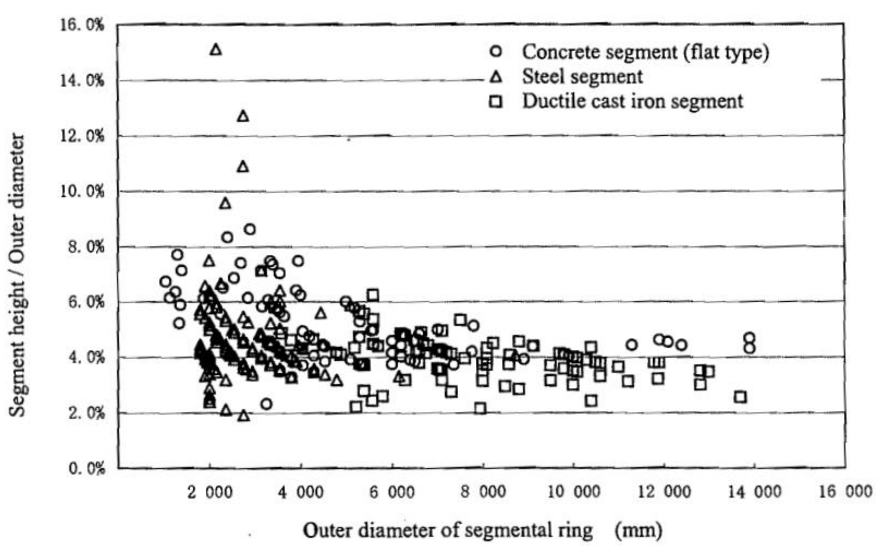


Figure 4-1: Relationship between outer diameter and segment height (thickness) (JSCE, 2007). Figure courtesy JSCE.

Segment Slenderness Ratio: When looking at segmentation, an important parameter is the segment slenderness or segment aspect ratio of the tunnel segment (λ), (the arc length of the segment divided by its thickness.) This empirical parameter can be used for the initial design of the segments but should be verified in final design. Bending can be limited by controlling the aspect ratio. The ITA tech Report No 7 (2016) reports that a segment aspect ratio of less than 10 is usually suitable for segments made of FRC alone based on experience from numerous projects, but higher segment aspect ratios may be possible depending on methods used and applied loads. Bakhshi and Nasri (2018) report that a typical aspect ratio is 8-13, with Fiber-Reinforce Concrete (FRC) segments around the lower boundary of this range.

Ring Length / Segment Width: Segment width or ring length is measured longitudinally, in the direction of the tunnel axis. AASHTO-DCRT-1 (2010) mentions typical ring lengths of 1 to 1.5 m (40" to 60") while an average length of 0.6 to 2 m (24" to 79") (AFTES GT18R1A1, 2005; Guglielmeti et al., 2008) or 1 to 2 m (39" to 79") (Maidl et al., 2012) is mentioned by others. DAUB (2013) indicates a range of 0.75 to 2.50 m (30" to 98") depending on the diameter. For large diameters segmental linings a width of 2 m (79" or 6.6') is commonly used. Recent projects have used larger ring lengths up to 2.2 m (86" or 7.2') by optimizing the segmental lining thickness (Bakhshi and Nasri, 2018). Historically, ring length appears to have been increasing over time (JSCE 2007), resulting in concentration of more secondary loading in the joints due to production and installation tolerances and thus leading to higher risk of cracking and spalling. The use of longer rings can also increase the risk of damage and cracking during handling and transportation. It tends to make installation more difficult when the segment needs to be turned, reduces the "margin" for driving curves and increases the necessary stroke of the thrust cylinders (PAS 8810:2016; Maidl et al., 2012). It is in fact directly related to the TBM shield body and backup design in terms of layout, length, cost and other aspects (Comis et al., 2016). Since the key needs to be pulled back and slid in position, the TBM shield length is affected. At the same time, longer rings reduce production cost, increase construction speed, and improve water tightness as the total length and number of radial joints, as well as the number of bolt holes, is reduced. Data from more than 60 projects presented in JSCE (2007) demonstrates that there is no direct relationship between the ring length and outer diameter of the segmental lining, largely because for smaller diameters, the available space for segment supply and handling limits the ring length, whereas for larger diameters, segment weight and production are the limiting factors (Bakhshi and Nasri, 2018) as shown in Figure 4-2: and Figure 4-3:).

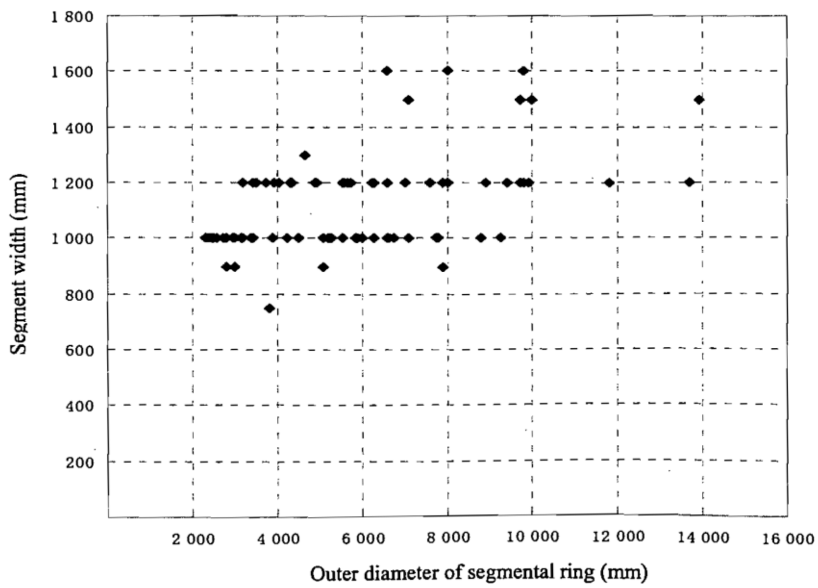


Figure 4-2: Segment width vs external diameter (JSCE, 2007). Figure courtesy JSCE.

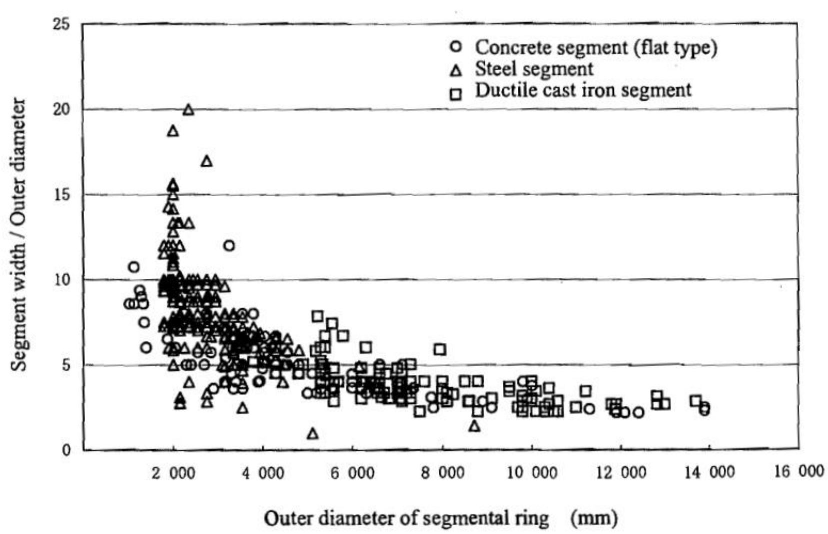


Figure 4-3: Normalized segment width vs diameter (JSCE, 2007). Figure courtesy JSCE.

4.2.2 Segmentation and Ring Design

Segmentation: Per AASHTO-DCRT-1 (2010), determining the number of segments used to form the ring aims to balance minimum number of pieces in a ring, short length of each segment for storage, transport and handling, and low enough segment weight to be handled by the type and size of machinery available inside the tunnel. The number of segments is a function of the ring diameter and to a certain extent, the contractor's preferences. It directly affects the design of major components such as the segment hoist (used to transfer the segments from the train cars to the ring building area) and the segment erector, and the number and arrangement of the thrust cylinders/rams should be considered (Comis et al., 2016). For large diameters, the maximum segment weight that can be transported is often the deciding factor. Maximizing segment weight minimizes the number of pieces and reduces ring build time.

A typical ring with rectangular segments consists of several A (“regular”) segments, two B (counter-key) segments and a K (key) segment. The number of segments in a ring varies widely from project to project (AFTES GT18R1A1, 2005). JSCE (2007) mentions a typical range of 6-8 segments for railway and road tunnels, but as described in Chapter 3, large diameter tunnels have been using higher numbers of segments per ring, i.e. 8 - 11. Special solutions may be needed for large diameters, such as dividing the ring into 8 segments and one of the ordinary segments into key and counter-key segments, thus remaining compatible with a TBM thrust jacking pattern of an 8-segment ring. From the list of international projects compiled, a 9+1 configuration is a very common configuration for large diameter. The SR-99 (Alaskan Way Viaduct) Tunnel rings are made of ten segments: seven rectangular segments (A), two right-trapezoid counter segments (B), and one isosceles-trapezoid shaped key segment (K), (7+2+1) (Pilotto and Jiang, 2012). However higher numbers have also been used.

The segmentation influences the structural design of the lining for temporary and permanent loads. Ductility in the lining allows for the creation of “hinges” at points of high moment that relieve the moments so that the primary load action is axial force (AASHTO-DCRT-1, 2010). Fewer and longer segments and fewer joints result in a stiffer segmental ring, reduced production cost, increased construction speed, less hardware for segment connection, decreased gasket length and fewer bolt pockets that are at high risk of leakage (Bakhshi and Nasri, 2018). More segments per ring means a more flexible support system, lower moments, lower transverse forces in the joints, higher deformations and rotations, and lower ground loads (Comis et al., 2016; Grübl, 2012). Comis et al. (2016) refer to a design example, where the influence of numbers of segments per ring was optimized by calculating the deformations and bending moments for several ring geometries.

Segments: Hexagonal, rectangular, trapezoidal, and rhomboidal segments have mostly been used to form segmental tunnel linings.

Rectangular systems are assembled into rings of rectangular or slightly tapered segments (unilateral or bilateral conicity) with a wedge-shaped (trapezoidal) key segment or a rectangular invert key segment (Swiss stacking system). For example, the SR-99 (Alaskan Way Viaduct) Tunnel is made of rectangular segments, two right-trapezoidal counter segments, and one isosceles-trapezoid shaped key segment. In general, segments are assembled from bottom to top, alternating the key stone location between left and right (ÖVBB, 2011). The simple radial joint geometry is one of their advantages. As described later, to avoid cross joints between rectangular segment rings, there should be a rotation for each subsequent ring installed. The main disadvantage is that time-consuming bolt assemblies are necessary and commonly used. Based on this literature survey, the rectangular segment ring is the most widely used form of segmental lining in large diameter projects including projects such as Alaskan Way Viaduct, Santa Lucia, Tuen Mun Chek Lap, West Gate, Sparvo, Port of Miami, SMART, and Yangtze River. As described below, with rectangular segment systems abutting rings should be installed in a way that avoids cross joints (a condition where four segment corners meet).

Trapezoidal segments are assembled with the first set of segments as an open-tooth row and the second set inserted in the gaps to form a complete ring. In general, an even number of trapezoidal segments is used, half of which are key type and half are counter type. The non-continuous, staggered radial joints are advantageous, along with every other segment acting as a key segment. The two "sharp" exposed edges on each segment may be a disadvantage and the segment alignment can be challenging (ÖVBB, 2011; Bakhshi and Nasri, 2018; AFTES GT18R1A1, 2005). Since tunnel excavation and lining erection can be simultaneous with use of a trapezoidal system (stroke of thrust cylinders adapted to two ring lengths), the counter segments support the thrust cylinders. If the total thrust force cannot be mobilized using only some of the thrust cylinders, simultaneous advancement of the TBM is limited to conditions using reduced thrust (AFTES GT18R1A1, 2005). Additionally, trapezoidal systems under pressure from gaskets between the segments might rotate which may lead to misalignment between rings and the position of TBM thrust cylinder ram pads (AFTES GT18R1A1, 2005).

Hexagonal systems are assembled continuously from hexagonal elements, alternating bottom/top and left/right, forming a ring. Each element serves as a key. The invert segment is often constructed differently for operational reasons (Maidl et al., 2012). These systems are mainly applicable as the outer layer of a two-pass system or as unsealed single-shell segmental linings, while the previous three systems can be sealed. Only one type of segment, each element acting as a keystone, no sharp exposed edges, coupled stability of the tube through staggered circumferential joints are some of the advantages of this system. However, the increased need for space within the TBM tail-skin and the transport and assembly problems with increasing diameter are some of its negative attributes (ÖVBB, 2011; Maidl et al., 2012).

Rhomboidal or parallelogram systems commonly use plugs between rings and consist of standard rhomboidal segments, one trapezoidal counter key, and one trapezoidal key, while erection occurs in the same order from one ring to another (AFTES GT18R1A1, 2005). AFTES recommends alternate segment erection with respect to first segment placed. A typical assembly procedure starts with the counter key trapezoidal element followed by placing parallelogram segments next to the counter key, alternating left and right. Dowels are often used for rhomboidal segments to avoid early crawling of the gaskets during the segments-approach phase of the ring assembly. Again, the non-continuous, staggered radial joints are advantageous. The continuous ring build from bottom to top is another advantage compared to trapezoidal systems, but the exposure of the two sharp edges of each segment is a disadvantage that these two systems share (ÖVBB, 2011). Both rhomboidal and trapezoidal systems have angled joints that lead to kinematic lack of fit and stresses when a ring is deformed in its plane by a curve (Maidl et al., 2012).

The key segment is generally inserted longitudinally. Although radially installed key segments are presented in the literature (JSCE, 2007) they are less common in construction of large tunnels. The longitudinal taper angle is in a range of 7-22º based on JSCE (2007). More narrow ranges have also been reported (8-11º in ÖVBB, 2011, 8-12º in Bakhshi and Nasri, 2018). Bakhshi and Nasri (2018) suggest that the same taper angle selected for the key segment be used for determining the geometry of other segments in the ring for rhomboidal or parallelogram-trapezoidal systems. There is normally no radial joint angle but when the arc and thickness increase an angle is sometimes set because the insertion angle becomes too large or the length of the shield machine becomes too long (maximum of 21.5º from construction experience in JSCE (2007)). If this angle increases excessively, the axial force acting on the segment may work as a sliding force on the joint (ITA, 2000).

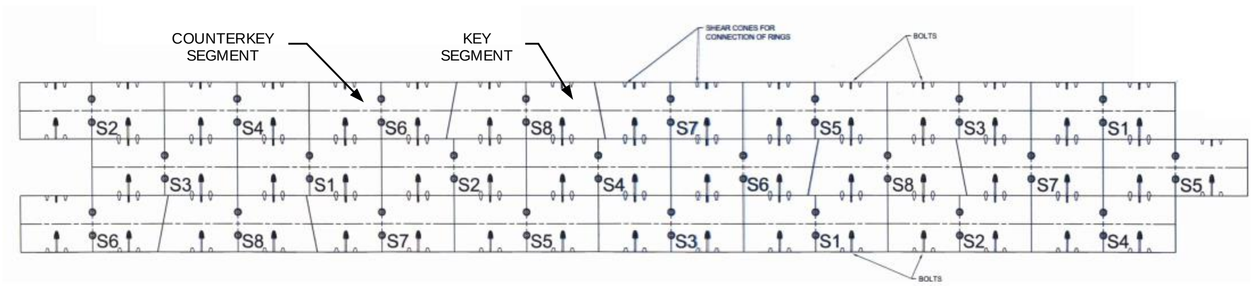


Figure 4-4: Typical ring configuration with rectangular segments. Figure: FHWA.

Rings: In parallel ring systems, the tunnel lining is assembled from parallel rings, meaning that the radial (or circumferential) joints are parallel to each other in longitudinal direction. For directional corrections and curves, packers can be placed in the circumferential joints (shifting). However, that generally prevents waterproofing of the tunnel except under certain conditions. Some of the advantages are the simple construction and that a special invert segment can be incorporated if needed mostly for railroad tunnels. Different sets of formwork are used for this system (ÖVBB, 2011). DAUB (2013) recommends to only use straight rings (parallel rings) as special rings, for example when steel segments are used in the region of later crosscuts.

When curves are present, which is the typical condition in nearly all tunnels today, tapered rings are used. Right/left ring systems are assembled from rings with one circumferential joint orthogonal to the tunnel axis and the other one inclined to the tunnel axis. The maximum/minimum ring length is arranged 90° from the key segment axis (DAUB, 2013). Simple construction is a key advantage but two different sets of formwork are used (ÖVBB, 2011). The key position was historically limited to the upper semi-circumference (AFTES GT18R1A1, 2005) to eliminate perceived difficulties of inserting a key segment at the invert, or to avoid high concentrated load on a key, for example from floating track slab pads. However, a modern TBM system is considered capable of placing the key segment at the invert with little difficulty (PAS 8810:2016). A combination of straight and tapered rings is sometimes used. Grübl (2012) recommends only tapered rings be used for large diameter tunnels, even for straight alignments.

Universal ring systems are assembled from rings with circumferential joints inclined to the tunnel axis on one or both sides. Any alignment angle can be achieved through appropriate rotation of the ring. Orientation of each ring is calculated in advance by a ring-building software in combination with the TBM steering system. Especially when using rectangular segment rings, to avoid the opportunity of cross joints (a condition where four joint corners meet), not all ring rotation combinations should be allowed, and typically a matrix of all admissible ring positions is provided in design drawings. This positioning restriction is followed by the segment delivery logistics and the TBM erector system and control crew, so that at all times the proper rotation of one ring relative to the previously installed one, is followed.

Universal ring systems can negotiate curves and can be made watertight but cannot incorporate a special invert segment (ÖVBB, 2011). The key segment can be erected in any angular position (AFTES GT18R1A1, 2005), but if desirable can always be placed above springline along straight drives by adjusting the drive error of less than a few millimeters in two or three rings (Bakhshi and Nasri, 2018). The position of the key segment may be at the invert, in which case erection begins at the crown and additional measures are needed to protect the crew in that area (Maidl et al., 2012). When offset longitudinal joints are specified, a compromise is often necessary regarding the rotation of the ring for the optimal steering of the shield. Universal rings need mold fabrication for each segment but only one mold set. Universal rings are most frequently used. Parallel and left/right rings involve higher logistical complexity and segment production costs (Maidl et al., 2012). Development (control of supply to the workface) tends to favor the use of the universal tapered ring (AFTES GT18R1A1, 2005). Per JSCE 2007 the minimum width of concrete tapered rings is 750 mm (30") due to the physical precast concrete production limitations.

Taper angle, curve radius: The ring taper and taper angle depend on segment type, width, outer diameter, curve radius, rate of tapered ring usage, manufacturing considerations, tail clearance, etc. (PAS 8810:2016; JSCE 2007). A tighter tunnel curve involves more tapering, which results in larger clearance between the segment and the tunnel shield (Comis et al., 2016). Bakhshi and Nasri (2018) observed that the minimum curve radius can be limited to 80 m, 160 m, and 300 m when shield outer diameter is less than 6 m, between 6 - 10 m, and more than 12 m, respectively, based on a review of more than 100 tunnel projects with different sizes in JSCE (2007). A plot of taper angle vs diameter is shown in Figure 4-5:. JSCE, 2007 provides suggested and past record taper values by diameter. An equation is provided for correction of the actual tunnel alignment that has deviated from the planned alignment. The general ring taper equation assumes that the key is always installed at the optimum location for maximum taper (i.e. tunnel crown). To avoid cross joints this condition is not possible. Thus, a correction factor is generally applied to the equation, depending on the connection dowels and dowel spacing.

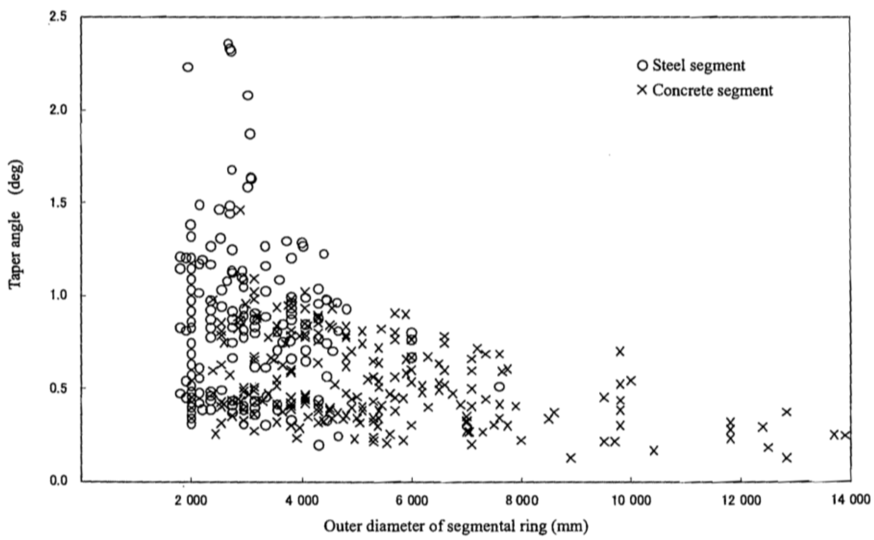


Figure 4-5: Taper angle vs external lining diameter (JSCE, 2007). Figure courtesy JSCE.

4.2.3 Reinforcement Type

Selecting the most suitable type of reinforcement appropriate for each specific project is critical for the structural stability and durability of the tunnel. During the design, reinforcement selection is determined from a wide variety of considerations that include the concrete analyses and designs, construction means and methods, manufacturing methods and costs, and durability.

The most common types of reinforcement for precast concrete segmental lining are Steel Reinforcement Cages and Steel Fiber Reinforcement (SFR). Another emerging option in the segmental lining design is the use of hybrid reinforcement - a combination of both fibers and steel reinforcement presents the ideal hybrid solution per ACI 544 7R-16, ACI 4R-18 and AFTES GT38R1A1.

Steel Reinforced cages can utilize various types of steel reinforcement, such as epoxy coated or stainless steel, which are commonly governed based on the need for durability, and are bent and shaped to exact geometric specifications for each segment. Cage assembly is performed by means of tied or welded bar reinforcement, using skilled labor or automated equipment. Despite the higher construction costs associated with it, the overall advantage of segments with reinforcing steel cages is that it is a proven technology with over 100 years of successful project applications when compared to segments with FRC.

Fiber reinforced concrete (FRC) refers to a type of concrete that uses discontinuous discrete fibers that have various shapes and can come from different types of materials such as plastic, glass, steel, or other natural or synthetic materials.

Less common is the use of glass fiber reinforced polymer (GFRP) bar reinforcement, unreinforced concrete or segments that use fiber-reinforced polymers (FRP) only, which have little to no durability issues. However, under the current practice for FRC these options are not widely used and are not adequate for large diameter tunnels due to the limitation arising out of low tensile strength.

FRP consists of composite materials that are typically of strong fibers of various sizes embedded in a resin matrix. The fibers provide strength and stiffness to the composite and generally carry most of the applied loads. The matrix acts to bond and protect the fibers and to provide for transfer of stress from fiber to fiber through shear stresses. The most common types of FRP materials are made of glass, carbon, and or synthetic fibers. The most common use of FRP, micro-polypropylene, in precast segmental tunnel linings is typically to address spalling as a mitigation for fire protection. Fibers are added at the appropriate dosage to the concrete mix.

The use of larger sized macro-fibers in concrete provides considerable improvements in the reduction of crack widths, but does not significantly increase the peak compressive and tensile strength. Despite the advantages, the use of segments with FRP should consider the multitude of design, manufacturing and construction limitations that make this not commonly used for tunnels. The aim of complete replacement of conventional reinforcement, including cases with tunnels of internal diameter greater than 7 m (24') and longer segment length with high slenderness ratios (up to 12 to 13) can be achieved using high performance materials. The current practice, if FRC is not adequate by itself, is a hybrid solution of fibers and reinforcing bars that can be used to achieve the design residual flexural strength of the lining at ultimate limit state and to improve the controlling the crack width at serviceability limit states (ACI 544 7R-16). In addition to published materials as mentioned above, research on the behavior of hybrid reinforced concrete segmental lining has been performed by various researchers including the work by Plizzari and Tiberti (2007) and Tiberti et al. (2008). The latter research demonstrates the effectiveness of combined reinforcement in resisting higher moments at the same crack width, compared to sole rebar reinforcement as the service limit state. The authors conclude that at ultimate Limit State, fiber contribution to bending resistance is negligible since the localized stresses (due to bending moment) are better contrasted by conventional rebar reinforcement.

Steel fiber-reinforced concrete (SFRC) is a durable and cost-effective alternative to concrete with conventional reinforcing bars. The fibers are short lengths of wire, about the size of an unwound paper clip, that are combined with plain concrete materials during the mixing process. The fiber length, or "nominal length," of a deformed fiber measured directly out-to-out is typically around two to three times the maximum aggregate size. When SFRC containing smaller length fibers is poured, the relatively large pieces of aggregate can force the fibers to orient around them. Cracks typically propagate between pieces of aggregate, and the capacity of SFRC is reduced if a disproportionately large number of fibers are aligned parallel with the crack. Longer fibers are preferable as they are not as influenced by the action of aggregate and keep a more random orientation. Fiber lengths for precast segments are typically between 50 mm and 65 mm (2" and 2.5"). If fibers are much longer than this, difficulties can arise with mixing and with standard SFRC testing. Extensive testing has shown SFRC to be highly resistant to corrosion and to have excellent impact resistance during handling. When SFRC is subjected to tension, fibers transfer tensile stress across any cracks that develop, which enables the composite material to resist bending moments and tension. Galvanized steel and stainless steel fibers are available on the market for very high corrosion protection.

Figure 4-6: Dry concrete mix with steel fiber reinforcement. Photo: FHWA.

SFRC tunnel liners are becoming increasingly common due to construction cost savings, however the capacity under bending of SFRC is significantly lower compared to segments with conventional steel bar reinforcement. The strength of SFRC segments limits their use to conditions where only moderate bending stresses resulting from the combination of moment and thrust exist (see Chapter 6). The number of segments per ring increases if SFRC segments are used in comparison to cage reinforced segments; this increases the number of joints, which increases the potential for leakage especially if ring-build quality is poor. ASTM A 820 - Standard Specification for Steel Fibers for Fiber Reinforced Concrete covers the 5 classification types of steel fibers; Type 1 is more commonly used in SFRC segments. The ends of each fiber should be properly restrained to resist pull out if cracks occur; different shaped anchors include hooks, twists, deformations and crimpled.

Figure 4-7: Types of steel fibers with anchors. Photo: FHWA.

Per AFTES GT38R1A1 in service cracks are much narrower and less straight in hybrid segments than steel reinforced concrete, thus making it more difficult for aggressive agents to reach the rebar. Mechanisms by which cracks are sealed and heal occur more easily in hybrid segments.

Compared to steel, GFRP presents higher tensile capacity, lower elastic modulus and lower weight. However, it is weak in compression and bars can tend to buckle. In addition, fiberglass is non-conductive for electricity and non-magnetic. In recent years, a research project supported by the EU's Horizon 2000 research and innovation program called, COMPOSKE (COMPOsite SKEleton), has seen the joint collaborative efforts between ATP and the University of Rome focus on a strong case to advance the use of GFRP reinforcement technology as an innovative solution that can be proposed as an alternative to traditional steel reinforcement, in segmental lining, mainly when resistance to the environmental attack is needed. Full laboratory testing (see Chapter 6) and model simulations have been performed as noted in the below case study.

GFRP reinforcement may suffer static fatigue when subjected to high-level long-term tensile stresses and the structural performance can be affected by the low value of the Young's modulus and by the poor bond behavior. The material durability can be improved and controlled through a suitable choice of the composite constituents. The application of GFRP reinforcement as a substitute of the traditional steel reinforcing bars in the precast segments of tunnel lining, could represent a suitable solution to the challenges of underground construction in terms of maintenance cost and durability (Caratelli, 2017). Despite the advantages and disadvantages compared to traditional steel reinforcement, GFRP segmental linings cannot be implemented in large diameter tunnels at this time, due to lack of research and testing.

A design procedure for tunnel concrete segments reinforced with GFRP bars was proposed and demonstrated in Spagnuolo et al. (2017). Based on the results and the comparison of two experimental full-scale tests featuring GFRP reinforced concrete and steel reinforced concrete construction, the following observations were made:

- For the flexural structural behavior, there were no significant differences when the steel reinforcement was appropriately substituted with GFRP reinforcement. In fact, although steel reinforced concrete sections are commonly under-reinforced to ensure yielding of the steel before failure, providing ductility and a warning of failure, the lack of ductility of the GFRP reinforced concrete segment was compensated by increasing the strength reserve. Additionally, the warning of failure was guaranteed by extensive cracking due to the significant elongation that GFRP bars experienced before failure;

- The simplified check procedure for the tunnel segment based on interaction diagrams built on the constitutive relationship of the materials, appeared suitable for predicting the capacity of a tunnel segment;
- The use of GFRP reinforcement in precast concrete tunnel segments appears to be a promising solution that can be proposed in situations often present in tunnel construction. To obtain successful results in this type of application, an adequate conceptual design should be performed with the aim of highlighting the advantages of GFRP use and, at the same time, maintaining a sustainable cost level.

4.2.4 Joint Geometry

In general, joints can be categorized as per ITA (2000) under flat (with or without connecting elements or with guiding bars), tongue and groove joints, pin joints, and hinge joints (convex-concave, convex-convex, with or without centering elements). Flat circumferential/longitudinal joints are considered state of the art (DAUB, 2013; Grübl, 2012).

Based on AASHTO-DCRT-1 (2010), radial joints between segments are mainly flat or concave/convex. Flat joints are more efficient at transferring axial load between segments and may result in less end reinforcement. They can also transfer bending moments, which reduces the moments in the segments. The load transferring width is normally 1/3 to 1/2 of the total segment thickness; the smaller the reduced section the higher the rotation capability which can negatively affect waterproofing (Maidl et al., 2012). ZTV-ING (2010) recommends flat longitudinal joints for single lining systems and AFTES GT18R1A1 (2005) states that flat longitudinal joints with a mechanical assembly are the most common. Cylindrical joints (concave-convex or convex-convex) facilitate rotation at the joint, allowing the segment to deform and dissipate moments. These are sometimes used when compressive forces and angle of rotation are high (Maidl et al., 2012) but may be deficient in terms of waterproofing. Concave-convex contact surfaces are mainly used for two-layer systems. The curved contact surface of these types of joints widens gradually via plastic deformation of the concrete and thus centralizes the load. ÖVBB (2011) provides some rules of thumb for the radius of the joint with respect to segment thickness. With convex–convex joints ring building tends to be more difficult and bolting may be necessary during construction. Other longitudinal joint types include convex-flat, tongue and groove or use of a guide rod. Tongue and groove elements are almost impossible to reinforce traditionally because of the necessary concrete cover (Maidl et al., 2012; DAUB, 2013). The tongue edges are at risk of tearing off in case of inaccurate assembly. When plastic guiding rods are used they can shear off due to low strength. Per PAS 8810:2016, the profile of the radial joint face is predominately governed by the structural behavior of the tunnel ring.

Circumferential joints are often flat contact joints as these allow the most efficient transfer of thrust loading from the TBM (PAS 8810:2016). The corners are sometimes chamfered to prevent damage by the TBM thrust jacks (BTS, 2004; ÖVBB, 2011). Flat joints combined with centering or coupling elements such as dowels or centering cones are sometimes selected (DAUB, 2013; Maidl et al., 2012). The use of mechanical systems like permanent bolting for shear transfer can help limit radial slippage (AFTES GT18R1A1, 2005; Maidl et al., 2012). Combined geometry contact joints (tongue and groove or cam and pocket, also referred to as pin and socket) are less common, because they make it difficult to reconcile erection tolerances and they pose a risk of high localized stresses. It is also difficult to reinforce the load transfer zones and the loads should be distributed over a reduced area (AFTES GT18R1A1, 2005). When a cam and pocket connection is used, if the load bearing capacity of the coupling is exceeded, the cam should shear off before the pocket edge is damaged in order avoid compromising the water tightness of the structure. The same is true for the tongue and groove type joint (ÖVBB, 2011). Tongue and groove systems in the ring joint need more accurate ring building, because in the case of slightly offset rings spalling often occurs during TBM advancement. This system is generally only used in very poor subsurface conditions (DAUB, 2013).

In some cases, stress distribution plates or packing is used in longitudinal and/or circumferential joints to avoid direct concrete to concrete contact, reduce high contact stresses and tensile cracking, and to minimize the effects of uneven loading or for slight plane correction (BTS, 2004; PAS 8810:2016; ÖVBB, 2011). Packing is described in more detail in Section 7.6. These interlayers can be made of fiberboard, plywood, or bituminous felt-fiber based. DAUB (2013) suggests that highly plastically deformable intermediate layers such as bituminous material should not be used for this purpose as they cannot maintain the intended spacing. BTS (2010) recommends that packing to be at least 2 mm (.1”) thick but not exceed 6 mm (.2”), or half the sealing capacity of the gasket, whichever is less, and that it should cover at least 80% of the longitudinal joint surface area. It is more commonly used in the circumferential joints, on the side of the segment facing away from the working face (ÖVBB, 2011). DAUB (2013) and Grübl (2012) state that based on experience, the amount of damage did not increase in cases where no packing was used in the circumferential joints, if production accuracy and exact ring build was achieved.

4.2.5 Coupling Hardware

Mechanical assembly systems are often used between segments and/or rings. The segments are temporarily or permanently connected in the radial and circumferential joints by means of connecting devices to ensure a certain degree of stability during installation, to prevent displacement of the segments, and to keep the gaskets compressed during construction and, if necessary, in the finished structure (ÖVBB, 2011). The *fib* Bulletin 83 (2017), notes the following with respect to the use and purpose of connecting elements as different conditions may prevail depending on project:

- Use of connectors only for alignment of the rings
- Support of segments during ring erection
- Resistance against TBM generated torque
- Retain gasket compression
- Provide redundancy and stability (ie. seismic or blast event)

Segment stability in the event of a temporary power loss in the TBM, is also another function offered by bolted joints.

In the same publication, connectors are not expected to serve the following functions, based on King, (2015) and (Maidl), 2014:

- Significant shear load transfer across joints
- Stabilize openings
- Pull joints and rings closed by post-tensioning
- Eliminate joint rotations and slipping

The durability and service life of coupling hardware should be the same as that of structure if they are considered as permanent (AFTES GT18R1A1, 2005).

Bolt and bolt insert pullout capacity is generally determined by calculation to maintain gasket closure, ignoring confining pressure around lining. Friction dowels are designed to resist gasket expansion forces in a completed segmental ring, initially with full joint closure assuming all TBM rams have been retracted. Shear capacity is determined based on segment erection forces and loads from TBM operation.

Guide rods and dowels can be used as centering aids for precise segment installation, but can also serve as connecting devices as guide rods can resist shear force in the radial joint and dowels can absorb both shear and tensile forces in the circumferential joint (ÖVBB, 2011). Dowels are only used in circumferential joints and are not limited to specific segment geometries. They aid in keeping segment installation tolerances low, but do not allow for subsequent ring offset to compensate for imperfections. This type of connector can offer advantages compared to bolts, such as no pockets in the intrados, easier reinforcement of the ring, simplified ring erection, good centering of rings, high shear strength, and improved safety of personnel because no human intervention is needed inside the ring (AFTES GT18R1A1, 2005). Most dowels cannot be removed and may inhibit some degrees of freedom while increasing concentrated stresses.

The use of bolts is most popular for both radial and circumferential joints (JSCE, 2007). Straight or curved bolts are used in the industry. However, straight bolts are more common in segmentally lined large diameter tunnels, because straight, inclined socket bolts lead to fewer pockets in the tunnel intrados and are safer for personnel as they are inserted under a fully erected ring (AFTES GT18R1A1, 2005). Bolts resist tensile and shear forces during installation and possibly during service. They generally have higher capacity than guide rods and dowels, which may be of higher importance for larger diameters where TBMs can transfer higher amounts of torque to the installed rings. New dowel types exist with dual purpose of alignment and shear resistance, which can be used in conjunction with bolts along the joint for large diameter tunnels. Bolts are placed immediately upon installation of the individual segment after application of the thrust jacks so that the necessary pressure can be applied and/or maintained. AFTES GT38R1A1 (2013) states that screw assemblies are more difficult for FRC segments and tests should be carried out to verify the resistance of the assembly areas. In waterproof systems with double gasket frames, the holes from temporary bolt connections and/or the entire bolt system in case of permanent bolt connections should be sealed (ÖVBB, 2011).

Coupling systems are usually only essential during construction (AFTES GT18R1A1, 2005; ZTV-ING, 2010; PAS 8810:2016). In general, bolts can be removed (in both longitudinal and circumferential directions) when the TBM is more than 200 m away per AFTES GT18R1A1 (2005) or approximately 2 tunnel diameters away per ÖVBB (2011) and grouting has been completed. In fact, DAUB (2013) recommends removal in the areas over the roadway to prevent bolts from loosening and falling out during operation. Exceptions include proximity to connections (2-3 tunnel diameters), where longitudinal assembly systems are usually necessary during operation to keep gaskets compressed. Based on PAS 8810:2016, the designer should assess the risk of segment damage against the need to remove the bolts once the ring is complete and grouted into place and advise the client of any identified risks. In practice, bolts are seldom removed even though they are typically no longer needed for structural stability or waterproofing. In special cases, such as for a load combination with "rock load on the tunnel roof area with partially bedded segment ring" and certain exceptional actions (e.g. flooding), the need for structurally effective coupling should be verified (ÖVBB, 2011). The ring joint bolt in most cases has an equal capacity to the segment joint bolt, but sometimes ring joint bolts with relatively small diameter and strength are used (JSCE, 2007). Per AASHTO-DCRT-1 (2010), recent lining designs have eliminated the need for longitudinal bolting and often use bolting in the circumferential direction, only. The thrust force of the TBM is usually sufficient to compress the gaskets in the longitudinal direction and to couple the rings by friction (Grübl, 2012) and the ground and water pressures are pressing the segments together in the circumferential direction.

The number and size of connectors depend on the construction method, the segment size and the recovery force of the gasket (ÖVBB, 2011). JSCE (2007) suggests that joint bolts be used in a single row for thin segments and double row for thicker segments. It also recommends that bolts for ring joints be placed in a single layer at 1/4 to 1/2 of the segment height regardless of ring type and thickness. AFTES GT18R1A1 (2005) states that assembly systems between segments comprise between one and three units. Per PAS 8810:2016 each segment (including the key segment) should have at least two bolt/dowel positions on the circumferential joint so that each segment can always be supported by at least one TBM thrust ram during assembly of the tunnel lining.

4.2.6 Gaskets

For a single-pass lining system of precast concrete segments, the highest risk of tunnel leakage lies in the lining joints. Waterproofing of segmental tunnel lining is primarily achieved through use of properly designed and installed gaskets arranged circumferentially along the four joint faces of the segments. The gasket can be either glued to the lining after manufacture of the segment or cast in the lining during production of the segment. The purpose of cast-in or anchored gaskets is to elongate the seepage path and to hold the gasket safely in place during installation, especially of the key segment (ÖVBB, 2011; STUVA, 2006). Sealing profiles that are inserted into the formwork and concreted into the segment with foot anchors have already been used but have yet to prove themselves in construction practice (STUVA, 2006; DAUB, 2013).

Gaskets for TBM segmental linings are nowadays mostly made of the elastomeric compound EPDM (Ethylene Polythene Diene Monomer), while hydrophilic material strips are also used in conjunction with the main gaskets for some projects. Some modern EPDM gasket types integrate a hydrophilic element within the contact face of the gasket. Gaskets are compressed when installed and during service life. Water-expansive gaskets are a compound of polymer that reacts with water and natural rubber or urethane (ITA, 2000). Per AFTES GT9R9A1 (1998) the material can either be a single component or compounded formulation (made of two hydrophilic swelling components or a hydrophilic swelling component combined with a conventional elastomer). This type of hydrophilic gasket expands in the presence of water and retracts in the absence of water during the service life. Initial watertightness is ensured by compression, if necessary. Hydrophilic gaskets should be protected from moisture during gasket and segment installation. Per BTS (2010) hydrophilic gaskets should be treated with a coating to delay the onset of swelling during erection of segments while elastomeric gaskets should be lubricated prior to erection. STUVA (2006) recommends gaskets with a low-friction special coating as an alternative to lubricant. Hydrophilic seals may deteriorate after repeated cycles of wetting and drying and should not completely dry out. Performance can also be affected by the salinity or chemical content of the groundwater, which is why different hydrophilic seals may be needed for saline and fresh water (BTS, 2004). Different tests and controls or variations are applicable to each of the main types of gaskets (AFTES GT9R4A1, 1993; ÖVBB, 2011).

The waterproofing system typically comprises a conventional or a combined hydrophilic EPDM gasket on the extrados side or less commonly a double system on both sides of the joint, if high water pressure is present or the project has a stringent waterproofing provision. For a double gasket frame ÖVBB (2011) recommends webs between the two gaskets arranged in such a way as to prevent water from flowing along the joint between them and states that all provisions should still be met by a single frame. If a second gasket is installed near the intrados fire protection should be considered for this gasket (STUVA, 2006). One sealing gasket near the extrados is enough in most cases (Grübl, 2012). When using combined compression-hydrophilic gaskets, typically at higher water pressures, the compressible gasket is the main component and the hydrophilic material is fitted into a groove formed in the former (AFTES GT18R1A1, 2005) or placed outside of the compression gasket, either as double or single gasket (AFTES GT9R9A1, 1998). When the usual practice of employing a single gasket is followed it can be difficult to locate the weakness if the sealing system is breached. Instead of a second gasket, a second preformed sealing groove with injection points can be provided as a means of remedial sealing (BTS, 2004).

When installing the segments, the sealing profiles of adjacent segments are pressed against each other and their contact surfaces with the concrete. Because the sealing performance of the typical EPDM profiles, which are not bonded to the concrete and not welded to each other, is based solely on contact pressure, it is important that the contact pressure exceeds the expected water pressure along the entire segment circumference (DAUB, 2013; Maidl et al., 2012). The gasket design pressure is usually double the working pressure. The designer should verify the gasket design for all possible combinations of pressure, offset induced by the construction tolerance (lips and steps), and maximum gap due to birds-mouthing at the joint associated with ring diametrical deformations induced by construction tolerances and loading conditions (PAS 8810:2016). The performance of these seals with respect to water pressure, gasket compression characteristics, and joint gap tolerances is an important part of the lining design. Manufacturers generally specify conditions of installation and provide load-deflection curves and watertightness-gap diagrams. Positioning and size of gaskets can significantly reduce the joint cross-sectional areas available for load transfer and thus further influence lining design (BTS, 2004). In addition, the gasket service life should match that of the tunnel. Gasket aging tests can be performed to confirm the service life of the gasket.

A number of publications give specific recommendations regarding gasket and groove positioning, sizing, tolerances, and gasket initial compression (AFTES GT9R4A1, 1993; JSCE, 2006; STUVA, 2006; BTS, 2010). Grueber and Dienner (2012) present a rule of thumb for gasket width in relation to tunnel diameter. Specifically, for tunnel diameters over 10 to 12 m (33’ to 39’) a 44 mm (1-3/4”) gasket width is recommended, keeping in mind that gasket size is related to erection tolerances, gap and offset.

Per AFTES GT9R6A1 (1998) watertightness, however, does not depend solely on the gaskets; different factors may affect it at each stage of the lining life. The AFTES publication classifies the types of distress that can affect watertightness under mass porosity, cracking due to temporary (demolding, transport, storage, installation, thrust, grouting) or permanent loads, and flaws or distress affecting the gasket groove or contact surface. Some examples include poor segment geometry if mold manufacturing tolerances are not maintained, inadequate protection of hydrophilic gaskets resulting in expansion before fitting, impact damage of segments, poor bond between gasket and segment, upward movement of lining due to buoyancy, ageing, and displacement between segments or rings. Insufficient vibration of molds is also a potential factor so the publication recommends resilient mold mounting that does not dampen vibration for large segments. ÖVBB (2011) recommends installation of an all-around pre-seal to protect the gasket from grout, soil and grease. Avoiding cross joints is also generally recommended (STUVA, 2006).

4.2.7 Lining Tolerances

There are two types of tolerances when it comes to segmental lining: production or manufacturing tolerances and installation tolerances. Manufacturing tolerances are often subdivided into formwork and segment tolerances (ÖVBB, 2011). Bakhshi and Nasri (2018) report that segment production tolerances are 1.1 to 5.5 times greater than formwork tolerances, depending on the nature of measured reference dimension. They also suggest that specifying formwork tolerances may be especially helpful for dimensional control in the pre-production phase but, despite this advantage, current practice is to specify segment dimension tolerances in contract drawings and specifications, and control them directly during segment production. The range of tolerances can be obtained from discussions with manufacturers, but will vary from project to project, depending on the function, serviceability needs, size and depth of the tunnel (Goodfellow, 2011). Less information is generally available on installation tolerances than manufacturing tolerances. Per AASHTO-DCRT-1 (2010) gasketed segments should be designed to high tolerances to assure that gaskets perform as designed.

The importance of manufacturing tolerances lies mainly in the assumptions made in the design regarding ring stiffness, joints and load transference, and in damage prevention (Handke, 2012). Segment tolerances are not explicitly considered in the structural design. Installation tolerances are considered in the design and dimensioning of the segments as well as in the analyses and selection of the sealing gaskets. Vigl et al. (2016) suggest that segment tolerances are generally not responsible for damages, whereas ring installation tolerances may cause load concentrations and consequently segment damage. Handke (2012) on the other hand suggests that spalling and leakage are typical consequences of inaccurate segment production. ÖVBB (2011) suggests that both radial and circumferential joints are affected by production inaccuracies of the segments and ring installation inaccuracies.

Per AASHTO-DCRT-1 (2010), better tolerances can be attained for precast segments than in cast-in-place concrete construction, since they are cast and cured in a much more controlled environment. Some key manufacturing tolerances are with respect to inner and outer diameter, segment width, longitudinal joint taper/conicity, segment circumferential (arch) length, evenness of the contact surfaces and gasket groove dimensions and position. Other parameters that may be measured include perimeter, diagonal chord, ring assembly misalignment, and bolt or other connecting device positioning. Kolic and Mayerhofer (2009) recommend segment dimension tolerances that tend to increase with increasing tunnel diameter. Forms, segments and test rings are the three main objects that can be measured. Standard practice is to measure tolerances of every segment made between the 1st and 10th casting, and then measuring every 50th segment after that. For both segment and formwork dimension control, testing is usually resumed at the initial frequency soon after detection of any inadmissible deviation. For large or segments with are more complex geometry a three-dimensional laser scan survey can be utilized.

The current permissible segment tolerances for Germany are presented in Guideline 853 (DB Netz AG) for rail tunnels and in ZTV-ING (BAST) for road tunnels. DAUB (2013) also specifies a comprehensive set of tolerances, including access tolerance by means of a tolerance radius (R) (R = ±10 cm), i. e. the diameter of the tunnel is made 20 cm (8”) larger than the minimum theoretical internal diameter. It also differentiates manufacturing tolerances between inner diameters ≤ 8 m (26') and ≥ 11 m (36'). Some project specifications do not allow any use of segments that have been produced outside the tolerance limits, while DAUB (2013) allows these segments to be used in areas with low projected tunneling forces, as long as gasket performance is not compromised, re-design demonstrates sufficient reinforcement in the joints, and a deep-beam analysis is conducted, if joint distortion and segment width specified tolerances are exceeded (Bakhshi and Nasri, 2018).

Installation tolerances can be described as deviations from the nominal position of the segments. The installation tolerances depend on the future use of the tunnel and the segment design, and are specified by the designer in consultation with the client (DAUB, 2013). Joint misalignment/offset, ovalization, and joint opening are the main installation tolerances, but different types of eccentricities during storage or in supporting the TBM thrust are also mentioned in the literature. Eccentric placing of segments on top of each other may cause additional bending moments to develop and lead to cracks or deformations, especially for long segments in big tunnel diameters (Kolic and Mayerhofer, 2009). Ovalization tolerance depends on the system parameters such as diameter and number of segment joints in a ring, allowable tolerance for joint design, connecting devices, and gaskets. A common provision in different publications and recommendations is 0.25-0.5% of the internal diameter (Bakhshi and Nasri, 2018). Among the reviewed publications BTS (2010) and DAUB (2013) specify ovalization tolerance as 1% and 0.5%, respectively. Joint opening can be considered a function of ovalization. BTS (2010) specifies that a 1 mm (.04") feeler gauge should not pass through the gap between longitudinal joints. Joint misalignment can be adjusted based on sealing performance needs and maximum allowable gasket offset for each project. DAUB (2013) specifies 10 mm (.4") as the maximum allowable offset, while ÖVBB (2011) specifies 5 mm (.2") for tunnel diameters ranging between 3 to 8 m (10' to 26'). BTS (2010) specifies 5 mm (.2") between adjacent segments and 10 mm (.4") between the plane of each ring and the plane normal to the tunnel axis. BTS (2010) also recommends overall tolerance of ±35 mm (1.4") for tunnel diameters under 5 m (16.4') and ±50 mm (2") for diameters over 5 m (16.4') for any point on the internal structure from its planned center line after construction and grouting. However, per Kolic and Mayerhofer (2009), installation tolerances tend to become tighter for single-pass, watertight tunnels with connectors even if the diameter is significantly higher.

Cavalaro et al. (2012) performed a study showing the relation between structural damage and contact deficiencies via FEM explaining how the contact deficiencies found in practice are generated by the tolerances. A general design method is proposed for the estimation of production tolerances and general recommendations are derived for the maximum admissible production tolerance used to limit the incidence of structural damage. The authors state that the cracks in the axial direction are caused by contact deficiencies at circumferential joints, whereas the chipping at a segment corner results from contact deficiencies at longitudinal joints. At least part of the aforementioned contact deficiencies may be related to the tolerances introduced during the production or the installation process. Considering segment thickness, they suggest that thicker segments are less deformable, allowing lower angular imperfection at the longitudinal joints, while thinner segments can tolerate higher angular imperfections. Along circumferential joints, wider segments have a higher capability of deforming and adapting to the contact imperfection, which is aided by a less stiff packing material. The mechanism of accumulation of tolerances and imperfections was studied using a Monte Carlo analysis. Fewer segments per ring cause less propagation of imperfections between rings. Curves, where a point of abrupt reduction of the resistant capacity for a certain imperfection in the circumferential and longitudinal joint is identified, can be obtained through FEM, from which limit values can be determined. It is then possible to determine the segment tolerances from the maximum contact imperfections already assessed. The values recommended using this methodology are slightly above the ones found in the literature (from 0.03˚ to 0.05˚ for the angular deviation of the longitudinal joint of the molds and from 0.30 to 1.00 mm (.01" to .04") for the width tolerance of the molds).

Vigl et al. (2016) discuss a new approach that has been established as part of a research initiative of the ÖBB-Infrastruktur AG with respect to the risks and consequences arising from geometrical tolerances, to provide a reasonable basis for allowable segment and installation tolerances. The paper explains the concept of "tolerance classes" and the approach for their derivation based on a risk assessment. Risk of negative effects (damage) is classified by the so-called tolerance classes, which can be translated into smaller allowable tolerances at higher risk of damage and higher allowable tolerances at lower risk of damage. Exceedance of different production or installation tolerances causes different types of damage in serviceability and ultimate limit states. Depending on the system (single shell/double shell, sealed/unsealed) certain types of damage have low or high effect (consequences of damage). In collaboration with the German committees for underground structures, ZTV-ING and DAUB, a harmonization procedure for the terms and definitions of tolerances was carried out. Determination of allowable tolerances is discussed. The main criteria mentioned are concept conformity to the safety philosophy of the Eurocodes, consideration of the specific project needs, identification of possible impacts regarding tolerances (damage- and risk assessment), and adaptability of the tolerances if the design parameters are changed during the development of the project.

4.2.8 Durability – Service Life

Design life is determined by the relevant limit state, number of years, and the period for which the level of reliability for not passing the limit state is not exceeded as described in *fib* Bulletin 34. Durability/serviceability of the precast concrete segmental lining is the ability to maintain structural integrity when exposed to conditions, such as environmental corrosive chlorides that could result in concrete section loss or corrosion deterioration of the reinforcement, during the specified design service life. There are different types of exposure classes that determine the severity of the anticipated exposure of chlorides to structural concrete; these classes are specified differently in concrete codes and standards of various countries.

The mix design of a high-performance concrete can be developed, to specifically address durability per the conditions of the tunnel environment, by lowering the concrete porosity resulting in a lower chloride diffusivity by using a higher cement to water ratio or using cementitious substitutes such as fly ash, silica fume or slag. The concrete strength, section properties, the designed lining thickness, crack analysis, additional clear cover for rebar cages and anticipated section loss are additional methods of concrete durability mitigations.

According to published documents (ACI 544 7R-16, 544 4R-18, ITA No.16), the durability of SFRC segments is better than steel reinforcement cages. However, with moderate or severe exposure conditions, the ability of steel fibers to resist long-term tensile stresses should be carefully evaluated. The durability of segments with steel reinforcement cages is generally good but, in aggressive environments (tunnels under salt water, sewer tunnels, brine outfalls, etc.), measures should be taken to prevent corrosion.

Reinforcement steel in concrete is passivated (made un-reactive) because of the alkaline environment at the steel surface. The steel cannot corrode as long as this passivation is prevailing. However, the passivation can be interrupted in two ways:

- carbonation causing a drop in pH in the carbonated part of the concrete;
- chloride ingress causing a chloride content at the steel surface above a certain critical chloride content, the "threshold level".

After de-passivation, and corrosion initiation, the rate of corrosion depends on the concrete properties, the thickness of the cover and the temperature and humidity conditions at the steel surfaces and in the cover (RILEM TC 230-PSC). The steel corrosion can lead to catastrophic structural failure if the proper pre-emptive mitigations are not considered. These could include epoxy-coated rebar, coated or lined segments, corrosion inhibitors, specialty cement or other measures. Increasing clear cover for the placement of steel reinforcement and accounting for anticipated section loss should be considered during the analysis and design stages. In comparison with steel, GFRP does not suffer corrosion problems and its durability is a function of the concrete mix design (Spagnuolo, 2017).

Performance-based design for durability of concrete structures includes the measurement of relevant concrete properties in the design stage to assess the resistance of the material against deterioration. Various performance-based service life design models have been developed in different parts of the world. For example, the European performance-based design approach "DuraCrete" was developed to model both chloride ingress into concrete and carbonation. The models were slightly revised in the research project DARTS and are described in the *fib* Model Code for Service Life Design. Other models dealing with chloride ingress include the South African chloride prediction model and the Scandinavian model "Clinconc". Using these models for the prediction of chloride ingress or carbonation, the onset of the corrosion propagation period is predicted. Durability indicators of concretes in relation to constituent materials and mix proportions, which are needed as input parameters in the service life models, are determined through experiments, usually using laboratory-cured concrete at an age of 28 days. Longer curing periods may be needed if pozzolanic or latent hydraulic materials (such as fly ash, natural pozzolans, slag) are used. However, it should be checked to which extent such longer curing periods replicate site exposure conditions (RILEM TC 230-PSC).

Predicting damage progression with time (the "damage function") in reinforced concrete structures subject to steel corrosion damage is important to design and maintenance. Reliable methods for corrosion forecasting are especially desirable for service applications that involve exposure to chloride ions, which affects a large fraction of worldwide transportation infrastructure. Like Performance-Based design, the Sagües model is a method for durability in that it states that structures that are exposed to corrosion risk can be divided into a large number of individual elements of equal size, traced on the concrete surface, such that the corrosion initiation and propagation processes within each element are independent of those in any other element. The element size is assumed to be small enough that the concrete and reinforcement properties, as well as the concrete cover and surface exposure conditions, may be considered to be uniform. On the other hand, the element size is assumed to be large enough that when corrosion propagates and damage is eventually made visible in the form of concrete cracking or delamination, the damage does not extend into neighboring elements (Sagüés, 2003).

For the Port of Miami tunnel project, the design service life was predicted using three different models: Life-365, DuraCrete/DARTS and the Sagües prediction model. Analyses identified steel corrosion induced by chloride ingress at the extrados as the most severe deterioration mechanism. The specified service life was achieved by using a low-permeability concrete made with locally available materials complying with Florida Department of Transportation (FDOT) specifications. The accepted mixture design comprises Type II portland cement, slag cement, and Class F fly ash (318, 397, and 79 lb/ft^3 [188, 236, and 47 kg/m^3], respectively) and has a water-cementitious material ratio of 0.32. Analyses of chloride ingress through the 76 mm (3") extrados cover indicated, at a confidence level of 90%, a 140-year period before corrosion initiation. Taking the tunnel tail void grout into account, the confidence level was raised to 93%. Corrosion propagation would occur over an additional 10 years. Analyses conducted using the DuraCrete/DARTS model predicted that the carbonation front in the intrados cover would reach just one third of the cover depth at the same age. The results were accepted by FDOT as demonstrating that the segment lining design fulfilled the 150-year service life provision (Torrent et al., 2014).

The following are excerpts from various international publications and standards that discuss durability and how it has been applied to the project service life:

AASHTO-DCRT-1 (2010): Chemical attack in certain soils can reduce lining life. Surface cracking in the concrete lining can result in water infiltration that can reduce the life of the lining.

AASHTO LRFDTUN-1 (2017): Tunnel structural elements are designed for a service life based on consideration of the potential effects of material deterioration, leakage, stray currents, scour, natural and manmade extreme events, and other potentially deleterious environmental factors on each of the material components comprising the structure, as well as for load effects experienced as part of the construction process.

BTS "Tunnel Lining Design Guide" (2004): The design life is typically in the range of 60–150 years. It can be argued that linings that receive annular grouting between the excavated bore and the extrados of the lining, or are protected by primary linings, for example sprayed concrete, may have increased resistance to any external aggressive agents. Typically, these elements of a lining system are considered to be redundant in terms of design life. This is because reliably assessing whether annulus grouting is complete or assessing the properties or the quality of fast set sprayed concrete with time is generally difficult. Other issues to be considered in relation to design life include the watertightness of a structure and fire-life safety. Both will influence the design of any permanent lining. In situ concrete was first used in the United Kingdom (UK) at the turn of the century. Precast concrete was introduced at a similar time, but it was not used extensively until the 1930s. There is therefore only 70 to 100 years of knowledge of concrete behavior on which to base the durability design of a concrete lining. The detailing of the ring plays an important role in the success of the design and performance of the lining throughout its design life. The ring details should be designed with consideration given to casting methods and behavior in place. Some of the more important considerations are as follows:

- Eliminate all embedded metallic fittings and fixings, bolt sockets and grout sockets
- Gasket grooves: Too small a distance to the edge may result in the enclosing nib breaking under load or when transporting the segment.
- Joints: Detailed to achieve the specified watertightness considering the type of waterproofing material used.
- Joint bearings: Detail joints to achieve adequate bearing area but with reliefs or chamfers to minimize spalling and stripping damage.

- Overall detail: Consideration should be given to all tolerances of manufacture and construction.
- Positioning of fixings: Embedded fixings/holes should be positioned to allow continuity of reinforcement (where needed) while maintaining cover.

fib Model Code 2010 (Bulletin 65) – Final Draft 2012 Section 3.3.2 Service Life suggests that the period in which the structure continuously satisfies the performance criteria is considered the service life for new structures. The performance verification is to be conducted with proper consideration of the change of performance over time, for instance due to degradation or time-dependent effects and effects of creep and shrinkage of concrete on the structural performance over time. Currently, this proper consideration of the chronological change of performance is not fully possible, at least for the effects of material degradation. Therefore, a staggered approach is used with regard to the verification of performance needs for safety and serviceability. Verification of limit states associated with safety and serviceability is performed without considering a change of performance over time due to degradation. In parallel, verification of limit states associated with the time-dependent material degradation is performed by means of service life verification. Accordingly, the service life verification is performed as a justification of the assumption of time-independence of the structural performance, which is made when verifying safety and serviceability according to the procedures for serviceability of various types of concrete structures. Service life verification demonstrates that during the specified (design) service life (new structures) or the residual service life (existing structures) degradation does not result in violation of the performance criteria.

4.3 Structural Analysis

4.3.1 General

This section presents a synopsis of the current state of practice in structural analysis methods for segmental tunnel lining design as presented by various authorities and associations including the FHWA, AASHTO, ACI, JSCE, BTS, DAUB, AFTES, ÖVBB, and ITA-AITES. Relevant state-of-practice and research findings as presented in published research articles is also discussed. The analysis methods used in other large diameter tunnel projects across the world have also been considered in this section.

4.3.2 Simplified Analysis

Closed Form Solutions

Many of the closed form analytical solutions available for estimating tunnel lining member forces assume plane stress and/or plane strain conditions, and a homogeneous, and an isotropic and elastic ground medium. Certain models have been extended to exhibit a viscoelastic ground behavior as well. These solutions typically assume that the circular tunnel lining behaves in an elastic manner. The ground pressures that act on the lining are often assumed to be equal to the primary stresses in the undisturbed ground. The underlying assumptions behind internationally recognized analytical solutions have been discussed in detail by Duddeck and Erdmann (1982, 1985), Zhao et. al. (2017), Iftimie (1996), Çimentepe (2010), among others.

In 1926, Schmidt proposed one of the earliest available analytical solutions for the development of tunnel lining member forces assuming a homogenous elastic continuum. Morgan (1961) proposed a solution using continuum models with the assumption that the tunnel lining deforms into an elliptical shape. This method was further improved by Muir Wood (1975) by including the tangential stresses on the model, but ignoring radial deformations due to these stresses. The anomaly was later corrected by Curtis (1976). Around the same time, Hartmann (1970) independently came up with a similar solution including the effects of stress variation between the top and bottom of the tunnel due to gravitational field. Schulze and Duddeck (1964) provided closed form solutions for shallow tunnels, assuming a circular tunnel model with bedding except at the crown. This was extended by Windels (1966) to account for second-order effects (geometrical non-linearity). Windels (1967) also published solutions for a circular tunnel in an elastic continuum, which accounts for geometrical nonlinearity. Separate analytical solutions based on elastic continuum were also presented by Einstein and Schwartz (1979), among others.

The member forces and deflections of tunnel linings are often dictated by ground movements ahead of the tunnel face, stress relief prior to lining installation and soil-structure interaction. While some analytical solutions can implement these factors to a certain degree, there is some consensus that these closed form methods fall short of capturing the full complexity of the tunnel construction process and cannot be considered as a substitute to more complex numerical analysis solutions.

Among the various analytical methods presented internationally for the case of a circular tunnel in homogeneous ground, the Muir Wood/Curtis approach is recognized as one of the most widely used approaches for estimating tunnel lining member forces.

Wood Curtis

In 1975, Sir Alan Muir Wood presented a simple approach to the problem of estimating axial forces, bending moments and deflections in tunnel linings. Muir Wood assumed that a circular lining deforms into an elliptical shape in homogenous elastic ground and that interaction between the lining and surrounding ground exists. Using the condition of plane strain and the assumption that no shear stress exists between the lining and the surrounding soil, the bending moment, axial forces and deflections in the lining can be estimated. However, the correction for the shear interaction between the ground and the lining, i.e. the inclusion of radial deformations due to tangential stresses, was incorporated in the solution proposed by Curtis (1976), to obtain more realistic estimates of the bending moments, axial forces and deformations in the tunnel lining. This approach has been mentioned in publications by authorities and associations such as the PAS 8810:2016, ITA (2000), ACI (544.7R-16), AASHTO (2010) and FHWA. As such the Muir Wood-Curtis approach is frequently used as a first approach in the design of segmental lining. Based on this literature survey, the Muir Wood-Curtis approach has been used in the design of large diameter tunnel projects such as the Shanghai-Yangtze Tunnel (Frew et al., 2008), Waterway Connection (Auckland), CLEM7 (Brisbane), Airport Link (Brisbane) and Cross Island Line (Singapore).

The analysis is typically carried out under drained conditions, with superimposed horizontal and vertical earth pressures, hydrostatic pressures, and vertical surcharge pressures. Design groundwater tables as mandated by the design codes and specific project design criteria are used as well. Separate cases are typically analyzed for forces imposed on the bored tunnel at all critical sections. The material properties of the ground and the lining are assumed to be elastic and the effects of joints are incorporated by utilizing an effective moment of inertia proposed by Muir Wood (1975). For preliminary design considerations, a stiffness reduction to account for joints is oftentimes not considered for a conservative estimate of member forces. It is also a design practice to neglect the stiffness reduction in cases of very weak soft ground, as in these cases it is important to minimize deflections considering the limited ground reaction.

Elastic Equations Method

JSCE (2006) provides multiple methods for evaluating the lining member forces through simplified equations. This approach is also endorsed by the ITA (2000) and ACI (2016) and has been used in the design of large diameter tunnels such as the Shanghai-Yangtze Tunnel in China (Frew et al., 2008). The two methods of implementing this analysis approach are described below:

1. Typical Calculation Method: This method provides a simplified analytical approach using elastic equations for calculating member forces and deflections of circular tunnels, applicable only for cases with symmetrical loading. This method assumes that the tunnel squats and hence the horizontal bulging of the tunnel induces a reaction from the soil. Equations are provided for estimating the member forces along with deformations from vertical loads, horizontal loads, self-weight and horizontal soil reaction, based on the assumption that the ring is modelled as having uniform rigidity without accounting for any reduction for the segmented joints. This approach tends to underestimate the member forces in the main section while overestimating those at the joints.
2. Modified Calculation Method: This method is like the Typical Calculation Method discussed above, except that the equivalent bending rigidity of the entire ring reflects the reduction of rigidity due to segmented joints. In addition, a redistribution of the bending moments is expressed by introducing a transfer ratio of bending moment (to transfer bending moment to the adjacent ring), which is determined based on the staggered arrangement of the rings. In comparison with the Typical Calculation Method, the member forces in the main section become larger, while those in the segmented joints become lower.

Empirical Method for Ring Deformation in Soft Ground

An empirical approach to determine the residual moments that arise because of the deformation of the ring, based on Morgan (1961) has been mentioned in publications by FHWA (2009), AASHTO (2010), ACI (544.7R-16) and the PAS 8810:2016. In the event of plastic deformations of the ground, the lining, which is typically more flexible than the surrounding ground, distorts as the ground displaces. This distortion of the ring results in residual moments building up in the lining, which is accounted for by assigning an arbitrary change in radius and calculating the theoretical moment resulting from this change in radius. The effective moment of inertia of the ring provided by Muir Wood (1975) is often used in the analysis.

Moments by Joint Rotation

The relation between bending moments and joint rotation has been the subject of many research studies (see Chapter 6) including those by Leonhardt and Reimann (1965, 1966), Janssen (1983), Blom (2002) and Jensen (2017). Ireland and Asche (2011) discuss a method to determine the moments due to ovalization of the ring, which could be caused by in-situ stresses or build inaccuracies. As the ring ovalizes, bird’s mouthing occurs at the radial joint. The angle of opening of the joint is a function of the percentage of ovalization to be designed for. This information is used in combination with the expected axial (hoop) force to determine the design bending moments.

The Ireland and Asche method is capable of accounting for the stiffening effect of the ring due to variations in the axial load. The bending moment calculated in the segments is a function of the assumed ovalization and does not depend on an assumed ring stiffness. This method, as discussed by Ireland and Asche (2011), is applicable only for flat radial joints and rectangular segments.

4.3.3 Structural Modeling

This section presents the various structural computational methods and related concepts that are frequently used to determine the member force effects for segmental lining design. The analysis for both permanent load conditions as well as construction related loads are discussed here.

Bedded Spring/Beam Spring Analysis

The concept of a bedded beam-spring analysis for tunnel lining analysis was first introduced by Bull (1944) and expanded by Schulze and Duddeck (1964). In the conventional use of the beam-spring analysis method, the structural lining is represented as beam elements, which are connected to radial ground springs to simulate the soil-structure interaction. It is assumed that the ground reaction is generated from the displacement of the lining, which is proportional to the deformation of ground. The tensile behavior of the ground springs is often neglected. Tangential ground springs may also be used to bring stability to the model, especially in the case of asymmetric loading, as described in publications such as ÖVBB (2011), DAUB (2013). The determination of the radial ground spring design values is typically done using elastic properties of the ground and the tunnel radius or using numerical analysis methods.

There are two different approaches to perform the beam-spring method of analysis to determine the lining forces.

1. **Single Ring Analysis**

   This approach involves modelling the lining as a single ring to determine the design member forces. Several published documents, as presented by authorities and associations such as ITA (2000), PAS 8810:2016, JSCE (2007), ÖVBB (2011), FHWA (2009), AASHTO (2010), AFTES (2005), DAUB (2013) mention performing the analysis using a single ring. This approach has also been used in the design of large diameter tunnel projects such as the Alaskan Way Viaduct tunnel (Jiang et al., 2018, 2012), Shanghai-Yangtze tunnel (Frew et al., 2008), Cross Island Line (Singapore) tunnel, Waterview Connection tunnel (Auckland), CLEM7 (Brisbane) tunnel and Airport Link (Brisbane) tunnel. The use of this method has also been discussed by Grübl (2011, 2012), Behnen et al. (2015), Bakhshi and Nasri (2014), among others.

   There are multiple methods available to implement this approach. The lining could be modelled as a single ring without any reduction in bending stiffness to account for the behavior of the joints. This method tends to attract the highest bending moments and is often considered appropriate for preliminary analysis. Alternatively, a reduction in ring stiffness as recommended by Muir Wood (1975) may be used in the analysis to account for the behavior of the segmental joints. However, some publications by the JSCE (2007), ITA (2000) recommend modelling the connection between segments in the lining as hinges to simulate joint behavior, although this approach is often restricted to grounds with high stiffness. The final method involves modelling the longitudinal joints in the ring using rotational springs, to achieve a more realistic behavior at the segmental joints.

2. Double Ring Analysis

The approach using a single ring cannot represent circumferential joints and the staggered arrangement of segments in adjoining rings, which can result in a coupling behavior. Arnau and Molins (2012) show that this coupling effect of adjacent rings become more significant in grounds with low stiffness. The behavior is simulated by modelling two adjacent rings. Such an approach has been mentioned in published documents presented by the JSCE (2007), DAUB (2013), and used in the lining analysis of large diameter tunnel projects like the Shanghai Yangtze Tunnel (Frew et al., 2008) and the Waterview Tunnel in Auckland, NZ. Discussions on the use of this method are also presented by Blom (2002), Bakhshi and Nasri (2014), Grübl (2006), Arnau and Molins (2012), Poel et al. (2006), Smarslik et al. (2018), Gall et al. (2018), Horichi et al. (1989), Koyama and Nishimura (1997).

This approach involves modelling two adjacent rings using beam elements, connected to the ground through radial, and if necessary, tangential ground springs. The longitudinal joints are modelled using rotational springs while the circumferential joints are modelled through shear springs, thus capturing the "coupling" action between adjacent rings. The design moments predicted though this method could be higher than those obtained through the modelling of a single ring using full rigidity, as shown by Klappers (2006).

Further discussions on the implementation of longitudinal joint behavior and the coupling effects between the rings in numerical modelling are presented in the following section.

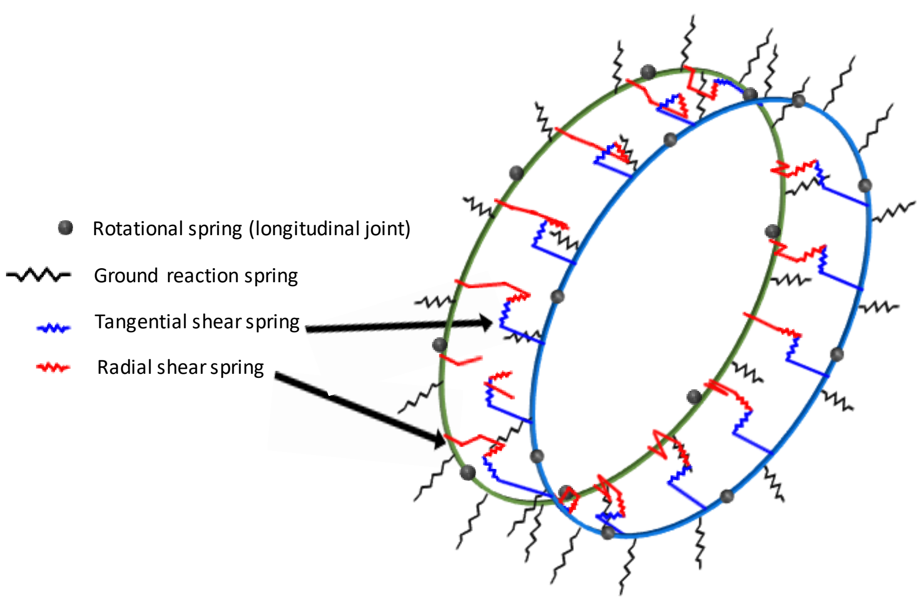


Figure 4-8: Double ring bedded beam model Figure: FHWA.

Three-dimensional modelling

The use of three-dimensional modelling for estimating lining member forces has been mentioned in various publications by various authorities and associations such as the AASHTO, FHWA and BTS. Detailed discussions on this method can also be seen in the work of Blom et al. (1999), Arnau and Molins (2012), Chen and Mo (2008), among others.

The use of a three-dimensional model enables a more realistic implementation of behavior at radial joints, as well as the coupling effect of adjacent rings due to the staggered arrangement of radial joints in the longitudinal direction. The modelling of joint behavior can be performed using the same recommendations as that for a two-dimensional beam-spring analysis (FHWA, 2009), as described briefly below.

Methods proposed by Leonhardt and Reimann (1965), Janssen (1983), Blom (2002), Jensen (2017), Iftimie (1996), Arnau and Molins (2011) and DAUB (2013) are available for estimating the radial joint properties to be used in the modelling. In comparison, limited literature exists for the implementation of the coupling behavior of the rings in the model. Grübl (2006) suggests representing the coupling of the rings by using non-linear lateral springs, which represent the shear stiffness and the maximum bearing capacity of the coupling. While using a plane joint with plywood hardboards, the spring stiffness is given by the shear stiffness of the plywood. This approach was also used by Cimentepe (2010) in his research. Even in the absence of mechanical coupling, the rings are coupled through the friction between the joint surfaces and the normal forces from the TBM thrust. The value of the frictional coefficient can be determined through tests and has been the focus of many studies, such as those performed by STUVA (1996) and Gijsbers and Hordijk (1997).

The modelling of the lining elements can be performed using plate or shell elements. Many commercial structural FEM packages enable the implementation of both thick and thin shell formulation. The PAS 8810:2016notes that 3D solid elements can be used instead of shell elements, especially when a more refined analysis of the joint behavior is needed. The use of 3D solid elements permits the modelling of the full behavior of the longitudinal and circumferential joints with the use of contact (interface) elements. Blom et al. (1999) presented the three-dimensional analysis of the Green Heart Tunnel using solid elements and the implementation of contact elements to simulate the interaction between segments in both directions. Plizzari and Tiberti (2007) discuss the use of 3D finite element analysis using the Non-Linear Fracture Mechanics method to model steel fiber reinforced and hybrid reinforced segmental lining for a subway tunnel lining application in Italy.

The number of rings used in the finite element analysis model is left to the designer’s judgement. Typically, an odd number of rings is selected for the analysis, with the design moments reported for the middle ring (Arnau and Molins, 2012). Three-dimensional analyses are also performed by practicing engineers for evaluating the impacts of thermal and seismic loading (Jiang et al., 2018; Kim et al., 2016), in addition to the evaluation of bursting and spalling stresses, as discussed later.

The use of three-dimensional bedded spring shell models is also permitted for modelling openings in segmental linings, such as for cross passages, and for intersecting tunnels, as presented in AASHTO (2010), FHWA (2009). Concerns about the use of this method include the inability to capture the non-linear behavior of the surrounding ground and ground movements with time.

Compression and equivalent ring bending stiffness

The compressive forces acting on a circular tunnel lining are generally large enough to have compression over its entire thickness. Therefore, the area and moment of inertia are calculated using the gross, uncracked dimensions of the lining.

Muir Wood (1975) proposed an empirical equation which accounts for a reduction in the bending stiffness of the ring based on the joint segments, when the number of segments in the ring is greater than four. This method has been recognized in design publications by the PAS 8810:2016, AFTES (2005), AASHTO (2010), FHWA (2009) and ÖVBB (2011). However, PAS 8810:2016 also permits the use of an alternative concept proposed by Muir Wood (2000), which considers the rotation for a unit circumferential length when a unit bending moment is applied. It is also common design practice especially in case of very weak ground and soft soils to entirely neglect the effects of the joints in the moment of inertia and perform the analysis assuming the full ring, as the intent is to minimize lining deformation.

The following topics in this Section deal with the member forces that develop within the segments during the construction and ring erection process. The ensuing discussions about the various load cases to be analyzed for the production and transient stages as part of the design of segments are based on discussions in ACI (2016), AASHTO (2010), FHWA (2009), BTS (2016), ITA (2000), Nasri (2016), among others.

Segment stripping/demolding

Segment stripping refers to the effects of lifting systems on stripping precast concrete segments from the forms in the segment manufacturing plant. Design is performed using specified early concrete strength. The analysis considers the cantilevering of the unsupported end sections under self-weight on either side of the lifting equipment contact edges. A high level of quality control of the machines and equipment is assumed by the ACI (2016), and hence a dynamic factor is not used for the design loads. If high quality procedure is not assured, dynamic load factors can be used, per Nasri (2016).

Segment storage

Following segment stripping, the segments are stored in the stack yard for attaining necessary strength prior to transportation to the construction site. Typically, all segments comprising a full ring are piled up within one stack. An eccentricity is typically assumed to account for possible imperfect alignment between the location of the stack supports for the bottom segments and the segments stacked above. The analysis is performed under the assumption that a simply supported beam is loaded by the weight of the segments above.

Segment transportation

During the transportation process of segments from the stack yard to the TBM trailing gear, the segments may experience dynamic forces. In large diameter tunnels two to three segments are transported stacked together. The loads are analyzed like the segment storage phase. The design, however involves the use of a dynamic shock factor per ACI (2016). Segment transport is discussed more in Chapter 9.

Segment handling

Segment transportation from the stack yard to the trucks or rail cars, and inside the TBM are performed using vacuum lifters, specially designed lifting devices or using two-point handling devices such as forklifts, cradles and slings. The load analysis is like the segment stripping/demolding case, except that critical moment could develop at the center of the segment also, depending on the lifting mechanism. A dynamic shock factor is recommended for design by ACI (2016).

Tail void grouting and secondary grouting

The diameter of the excavated bored tunnel is slightly larger than the extrados diameter of the segmental lining due to the thickness of the shield, over-excavation, and gap between the intrados of the shield and the extrados of the segmental ring, resulting in a grout filling the gap between the two surfaces. Annular grout is injected to fill this void, but it has the potential to cause high pressure to be applied to the ring segments.

Tail void grouting is typically performed by injecting grout through the tail of the TBM shield and it exerts relatively uniform pressure on the lining. The loads experienced by the lining at this stage are purely grout pressure and self-weight. ACI (2016) reports that the tail void grout pressure is generally greater than the hydrostatic pressure around the lining and at the same time, less than the overburden pressure to prevent soil expansion, heave or hydrojacking. BTS (2010) recommends that the primary grout pressure should not be greater than the prevailing hydrostatic pressure by more than 14.5 psi. AASHTO (2010) and FHWA (2009) suggest that the maximum permissible grouting pressure applied be limited to 10 psi to reduce the possibility of damage to the ring. BTS (2016) notes that this scenario is not expected to govern, unless the hydrostatic pressures are very high.

Secondary grouting refers to subsequent re-grouting of rings (after tail void grouting). It is performed for specific rings or segments when the primary grouting proves insufficient and there are concerns of remaining voids behind the ring. For secondary grouting, grouting ports formed in concrete segments are utilized to perform this work. The remaining voids or loosened zones between the segmental lining and excavated area are filled through this process. Secondary grouting produces localized bending moments as the circumferential grout pressure is not uniform. ACI (2016) endorses the ITA WG2 (2000) recommendation of a triangular pressure distribution over the segment. BTS (2016), however, recommends checking for possible punching shear due to secondary grouting along a 1m x 1m section.

Bearing and bursting stresses

Segment bearing and bursting stresses develop at areas in proximity to the circumferential joints (due to the TBM thrust ram loads) and at radial joints (due to permanent hoop forces between segments caused by external loading). Immediately below the loaded area, the concrete is in compression. As the compression stress is distributed (spread) over a large area in the segment, and away from the bearing surface, tensile splitting forces develop perpendicular to the compression forces, causing significant tensile stresses deep within the section. In addition, spalling tensile forces act between adjacent jack pads along the circumferential joint. The available bearing area for the thrust ram (circumferential joint) and segmental contact area (radial joint) are restricted by the gasket grooves and recesses.

Bursting forces acting on the segment due to the TBM thrust ram loads can be determined through multiple approaches. The ACI 318-14 provides equations for determining the bursting force and it's centroidal distance from the major bearing surface. Similar equations are also available in publications by the DAUB (2013) and the British Standards implementation of Eurocode 2 (1992-1-1:2004).

Nasri (2014) proposes that tensile (bursting) stresses from TBM loads can also be estimated using analytical methods such as Iyengar Diagram (Iyengar 1962), which has been used in design of tunnels in the Netherlands (Groeneweg, 2007).

The bursting forces that act on radial joints due to hoop loads are estimated similar to that for circumferential joints due to the TBM ram loads, except that consideration is given to the non-uniform load transfer across the joints. Publications such as PAS 8810:2016 discuss accounting for the opening of the radial joint as the tunnel ovalizes, which results in non-uniform stress distribution across the joint.

Alternatively, three-dimensional structural finite element analysis can be used to determine bursting and spalling tensile stresses, as discussed by Grübl (2006, 2012), Nasri (2014, 2016). Bursting stresses at the vicinity of the longitudinal joints can also be analyzed separately for the case of maximum normal force and gasket pressure, using two-dimensional FEM as discussed by Nasri (2014). The tensile stresses developed are compared against the tensile strength of concrete to determine if bursting reinforcement is necessary.

In addition to using three-dimensional modelling, the spalling stresses that develop between the adjacent TBM jack pads can also be estimated using a strut-and-tie model or using an approximation of the post-tensioned anchorage zone design per AASHTO (2017).

Moments due to build inaccuracies

Inaccuracies in the ring installation, such as an offset between the radial joints due to misalignment, create an eccentricity of the thrust force resulting in an additional moment that should be considered in the analysis. BTS (2010) and ÖVBB (2011) recommend a design eccentricity of 5 mm (.2") between joints during installation. Higher design eccentricity values may be established on a project specific basis.

4.3.4 Geotechnical Numerical Modeling

Geotechnical-based ground-structure-interaction numerical modeling is being used extensively for the analysis and design of underground structures, as it provides the capability of simulating complex geometry, loading, construction sequence, ground-structure interaction, and sophisticated constitutive behavior of soil and structure materials. Use of geotechnical numerical analyses is becoming increasingly popular among project owners as a viable option of performing or verifying the design of the segmental tunnel lining, especially in the case of large diameter tunnels. Large diameter tunnels have a higher probability of encountering mixed ground conditions or ground of mixed stiffness compared to smaller diameter tunnels. Therefore, soil structure interaction analysis with geotechnical-based computer programs has become more relevant than before for large diameter tunnel designs and it is typically included in most project specifications as a valid design methodology.

Geotechnical numerical programs commercially available and typically used by practitioners for the design of tunnels in soft ground include the Finite Element (FE) and Finite Difference (FD) methods. Among the reviewed documents, the following publications refer to the use of geotechnical numerical modeling for tunnel design and analysis: FHWA-NHI-10-034 (2009),

FHWA-HIF-15-005 (2015), AASHTO-DCRT-1 (2010), AASHTO-LRFDTUN-1 (2017), ACI 544-7R-16 (2016), DOT-TSC-UMTA-83-16 (1983), PAS 8810:2016, BTS Tunnel Lining Design Guide (2004), ITA Guidelines for the Design of Shield Tunnel Lining (2000), AFTES GT18R1A1 (2005), ÖVBB Concrete Segmental Lining Systems (2011), Japanese Standard for Shield Tunneling (1996).

AASHTO-DCRT-1 (2010) briefly introduces numerical modeling programs available for the design and analysis of tunnels in homogeneous ground and media with discontinuities. AASHTO-LRFDTUN-1 (2017) makes note that numerical modeling, including finite element or finite difference analyses, should be used to account for ground-structure interaction and the final design of tunnel linings. Due to the complexity of numerical modeling, appropriate modeling techniques are based on advanced modeling capabilities and experience, time and computational effort along with reliable interpretations of geotechnical data by experienced designers to produce a robust and reliable model.. As with every modeling approach, geotechnical numerical modeling for design of segmental lining is subject to in-accuracies as simplifications and assumptions are necessary, starting with, but not limited to, the complex problem geometry, quality of input data, material constitutive modeling, and interpretation of the results. PAS 8810:2016 highlights that numerical models are often considered more useful as tools to investigate mechanism rather than obtaining precise predictions about the tunnel performance.

Geotechnical numerical models are often developed to estimate ground movements during tunneling and construction as well as to examine the effects to existing structures and facilities. Numerical modeling for estimation of ground movements with respect to building protection, however, is beyond the scope of this synthesis report. Complex numerical methods are sometimes performed for seismic soil-structure interaction cases and are discussed in the Seismic Performance Analysis Section 4.3.5. The value of numerical modeling with respect to segmental lining design lies in the fact that the liner response is directly connected with the response of the surrounding ground. Regardless of design methodology (SLS/ULS or LRFD), the objective of geotechnical-based numerical analyses is to obtain input for structural analyses or obtain resultant loads in the structural members for dimensioning and designing of the lining. Despite the acceptance and recommendation of geotechnical numerical analysis by many international publications, there seems to be no uniform approach to the proper implementation of the numerical analysis results and findings in tunnel lining design, especially so in precast concrete segmental lining design. As discussed later, the fundamental difference between an un-factored input geotechnical numerical analyses performed for the "Service" case and an LRFD structural design is another issue facing designers. As highlighted in Asche and Ireland (2013), a typical design practice involves analysis of the ring using FE/FD analyses to determine the ground loading applied to the ring and followed by beam spring models to analyze various load cases.

Geotechnical numerical models provide the computational environment to study bored tunnels by means of two-dimensional (2D) or three-dimensional (3D) analysis. AASHTO-DCRT-1 (2010), PAS 8810:2016, BTS Tunnel Lining Design Guide (2004) recommend the use of 3D models when 3D stress regimes are expected such as at junctions, connections and cross passages. Plane strain (2D) models are applicable as long as the assumption of a relatively long tunnel is valid. This is justified on the grounds, that the transverse stress and strain distributions for a tunnel of reasonable length in homogeneous ground corresponds to a two-dimensional plane strain condition at distances greater than two or three diameters from the point at which the tunnel ring is closed (BTS Tunnel Lining Design Guide, 2004).

A state-of-the-art in the techniques that can be used in conjunction with a 2D geotechnical numerical code for tunneling analysis, can be found in Möller (2006), and Möller and Vermeer (2008). One of the more popular methods - The Convergence-Confinement Method (Panet, 1995) is frequently applied in analysis of bored tunnels, to provide some equivalent stress relaxation to the tunnel excavation prior to installation of the segmental tunnel liner. The Convergence-Confinement method can be generally applied as a direct stress-controlled method in FEA/FD 2D analysis, as a means of providing some level of stress relaxation prior to the mobilization of lining. Publications such as AFTES GT18R1A1 (2005) and PAS 8810:2016 note that standard analyses are usually limited to 2D modeling in which the influence of the advancing face is considered by applying the “convergence-confinement” method. Other methods including the “Contraction” method or the method recommended by Addenbrook et al. (1997) can also be used as their primary metric is the volume loss at the tunnel level, which is a key performance indicator in all soft ground tunneling with TBMs.

Kunst (2017) explored the possibilities of modelling the construction phases of bored tunnels with respect to lining forces through different finite element programs. Two commonly used FE programs were selected for detailed evaluation and the limitations of 2D and 3D models associated with capturing the soil behavior during construction or modeling the segment joints were discussed in detail.

Regardless of the excavation and lining installation modeling approach and based on the literature survey, the current practice for modeling segmental lining in 2D or 3D geotechnical-based computer programs involves implementation of similar methodologies as in conventional bedded beam structural analysis. As the liner is not monolithic, the effect of segmentation may be reflected by a reduction of the flexural stiffness either by modeling an equivalent ring or by explicitly modeling the segments. For an equivalent ring calculation, AASHTO-LRFDTUN-1 (2017) and ÖVBB Concrete Segmental Lining Systems (2011) recommend that the effective moment of inertia be calculated as per Muir Wood (1975) for use in numerical analysis. However, the Muir Wood (1975) empirical ring stiffness formula has been demonstrated to be not applicable to many design cases because it usually underestimates the moments induced in the lining (Asche and Ireland, 2013). AFTES GT18R1A1 (2005) states that this behavioral assumption is not valid in the case of adjacent rings incorporating combined radial contact joints in association with rigid assembly systems between rings, nor in the presence of very soft ground.

Alternatively, the joints may be modeled explicitly in a numerical model by introducing local discontinuities in the lining body by way of simple rotational hinges or rotational springs. While this methodology is more widely established in 2D and 3D structural numerical programs, explicit segment modeling in geotechnical FE or FD models is more complex. Modeling of segmental linings even in modern 3D geotechnical software becomes particularly challenging. Such 3D models are complex to develop, time-consuming to analyze, and oftentimes challenging to interpret. It is project dependent whether explicit 3D modeling of segmental lining is necessary; further research is necessary to identify the trade-off in terms of computational effort and accuracy and the limitations of simplified methodologies.

As part of this research several journal and conference papers, and research theses, demonstrating the use of two dimensional (2D) or three dimensional (3D) geotechnical numerical analysis for lining design, were reviewed. The most relevant findings are summarized below:

Kramer et al. (2003) discussed aspects of the design of the Los Angeles East-Central Interceptor Sewer (ECIS) and the Northeast Interceptor Sewer (NEIS) sewer. FD models were used to provide a design envelope of the anticipated conditions. Results obtained from the FD analyses were treated as dead loads in accordance with the City of Los Angeles Bureau of Engineering Structural Design Manual and a load factor of 1.4 was applied for structural design along with ACI provisions considering the lining to be a "wall" or shell" for calculation of capacity and reinforcement. Separate FD analyses of radial joints were performed by modeling two short portions of segments meeting at the radial joint to verify that the joints are capable to withstand rotations calculated from the previous FD models. Strut-and-tie models were then used to size reinforcement along the radial joints.

Ding et al. (2004) proposed a 2D finite element (FE) model for the analysis of shield tunnels by considering the construction process in four stages. The soil was assumed to behave as an elasto-plastic medium whereas the shield was simulated by a beam–joint discontinuous model in which curved beam elements and joint elements were used to model the segments and joints, respectively. To assess the accuracy of the method, the shield tunneling in the No. 7 Subway Line Project in Osaka, Japan, was used as a case study. The numerical results were compared to field measurements and showed that the proposed numerical procedure can be used to effectively estimate the deformation, stresses, and moments experienced by the surrounding soils and the concrete lining segments. The authors suggested that although the predicted lining bending moment values were generally greater than those measured on site, the method can be used for a safe design.

Chiaia et al. (2009) discussed the application of 2D FE for the design of the Faver – S.S. 612 tunnel lining in Italy. The design of both primary support and final lining was based on multi-stage 2D FE models. For each loading condition that was analyzed, the FE analysis provided the principal stresses in all the cross-sections of the final lining. After computing the stress components normally oriented with respect to the cross-section, two different mechanical models were used to compute the minimum reinforcement area and the crack width of Reinforced/SFRC members.

Grübl (2012) discussed the older design of the Orlovsky-Tunnel, which has been but would have been the world's largest bored tunnel in St. Petersburg, with an outer diameter of 18.65 m when constructed. FE modeling was used to support the vertical earth pressure calculations for the design of the tunnel.

Pilotto and Jiang (2012), Jiang et al. (2013), Jiang at al. (2018) discussed the design of SR-99 Tunnel in Seattle using FE modeling. The design followed a two-step approach for static and seismic conditions that separated the structural from the geotechnical modeling. For the static design, the first step involved a FD model to quantify the soil loads and the hydrostatic loads by simulating the tunnel construction sequence. The ring was modeled with beam elements with reduced stiffness after Muir Wood to consider the effects of the joints. The ring elements of the tunnel were connected to the surrounding soil through non-linear springs in both the radial and tangential directions at each of the nodes. During the second step, structural 2D and 3D beam spring models were created by using springs with stiffness obtained during the first step and different load factors and various combinations were used in the structural model. For the seismic design, deformations of the soil surrounding the liner due to the seismic waves propagating from bedrock through soil media and in absence of the liner were computed with a continuum FD model. The resulting ground deformations were imposed on the liner through supporting elements (non-linear springs) with a 2D beam-spring model to analyze the soil-structure interaction. Jiang at al. (2018) provided a comparison of the forces and deformations on the liner from the geotechnical and structural models on an unfactored basis. Good agreement was achieved in terms of axial, shear forces, bending moments and deformations, between the structural models producing more conservative results. After this verification process, the results from the structural model were used to design the liner, with and without interior structures, using the LRFD method.

Asadollahi and Kaneshiro (2014) discussed the design of the Anacostia River Tunnel of internal diameter of 7 m (23') using closed-form solutions, as well as 2D and 3D FE analyses. In 2D FE analyses, the segmental liner was simulated using structural monolithic elastic plate elements and the Muir Wood stiffness reduction. The axial, shear forces, and bending moments were determined from undrained and drained FE analyses performed at all critical sections and after applying appropriate load factors, all axial forces and bending moments were compared to the capacity of the segmental liner. 3D FE analyses were performed for the critical sections and the results were used to demonstrate the adequacy of the designed steel fiber reinforced segmental liner.

Do (2014) investigated the behavior of segmental tunnel lining by developing 2D and 3D numerical models using the finite difference method (FDM) under static and dynamic loads. The influence of the segmental joints, in terms of joint distribution and stiffness characteristics was studied through 2D models pointing out the importance of taking into consideration the effect of the joints during segmental tunnel lining design. 3D models focused on the interaction between twin tunnels and the ground displacements around the tunnels.

Susetyo et al. (2014) performed numerical simulations to evaluate the precast concrete tunnel lining (PCTL) structural capacity of the Eglinton-Scarborough Crosstown twin tunnels (internal diameter 6 m (19') in Toronto, Canada, during a fire event. Solid elements were used to model the ground and concrete and linear-elastic cable elements simulated the reinforcing steel. Each segment was explicitly modeled with the PCTL rings connected one to another through sixteen dowels uniformly spaced along each circumferential joint. Compression-only frictional interface elements were provided at the extrados, the circumferential joints, and the radial joints of the PCTL to model the stress transfer between ground and the PCTL as well as between PCTL segments. The methodology used in the analyses was considered to yield slightly conservative results, based on the results of the verification study done by Phan (2008).

Almong et al. (2015) discussed the application of the "convergence-confinement method" in 2D FE simulations for TBM segmental lining design purposes. It was recommended that for lining structural design, both lower and upper bound convergence should be checked to ensure a robust design. In that context, it is vital to assume a relaxation value for lining design that is not a function of the contractual volume loss but of the range of TBM confining pressures which can be directly monitored during construction. The methodology was investigated through a case study for a typical tunnel in Hong Kong.

Vazaios and Vlachopoulos (2015) presented a 2D FE to simulate segmental liners in shield-driven tunnels at high overburdens within rock masses. For the purposes of this paper, two different types of concrete liners, (i) monolithic and (ii) segmental liners, were adopted in order to investigate the influence of the in-situ conditions on the structural forces developing in the liner under different ground-tunnel interface conditions. It was generally observed that the presence of the joints reduced the magnitude of the generated moments and altered the distribution. The authors highlighted that 2D numerical analyses cannot capture effects such as the presence of the TBM in the excavation or the joints connecting the rings along the tunnel axis in the out of plane direction and how it affects the rigidity of the overall tunnel structure. Therefore, 3D numerical modelling is crucial if a shield-driven tunneling process is to be simulated as realistically as possible.

Gall et al. (2018) presented a holistic modeling procedure that may be used to incorporate not only structural factors, but also serviceability factors, such as crack development in linings. To showcase this approach, a 3D finite element model was used to calculate the predicted forces on the tunnel lining of a tunnel with diameter of 10.6 m. The tunnel lining was represented by volume elements that formed a continuous ring and were activated in a step-wise manner during the process simulation. The loadings obtained from the 3D tunnel model were, in the form of displacements, transferred to a high resolution non-linear composite FE model of a single lining segment to investigate the nonlinear structural response of the segments, particularly with respect to the load-transfer mechanism between segments at the longitudinal joints. The results were also compared to traditional methods.

In his doctoral thesis, Blom (2002) presented a new approach to implement explicitly the rotational stiffness of the longitudinal joints and the lateral interaction between the rings for a lining system in an elastic soil continuum The new analytical solution for the segmented linings of shield driven tunnels, with explicitly integrated longitudinal joints, lateral ring joint interaction and elastic soil continuum offers a very powerful tool to calculate the lining behavior in the serviceability state.

Afshani et al. (2014) performed a study on the effects of earth pressure balanced tunneling on the stress path and drainage condition of the soil during tunnel advancement through 3D finite element modeling. Through this study, the soil around the tunnel face was found to be in the elastic domain and a numerical experimental equation was proposed to determine the drainage conditions during tunneling that was verified with field data from a case study.

Aggarwal (2017) presented the design process of an irrigation tunnel using a TBM and consisting of segmental concrete tunnel lining 3D finite element models to evaluate the construction stage stresses induced in the segmental lining. A dynamic analysis was also carried out considering wave motions generated in ground by high-speed train passages.

In summary based on this literature survey, the following general modeling approaches have been used for modeling segmental lining of bored tunnels.

Table 4-1: Segmental tunnel lining options in geotechnical numerical computer programs

Liner modeled with beam or plate element	Liner modeled as solid volume element
Solid ring with equivalent flexural stiffness (Muir Wood)	Solid ring with reduced stiffness properties
Segmented ring with hinges	Ring with segment to segment and ring to ring interfaces
Segmented ring with rotational springs	

Earth Pressures from Numerical Modeling

The ground loads imposed on the tunnel lining are a function of the stress variation with depth, tunnel depth, ground-lining relative stiffness, tunnel geometry, and construction sequence. Per the voluntary and non-binding AASHTO-LRFDTUN-1 (2017), the vertical earth pressure for mined tunnels in soft ground may be taken as the pressure resulting from the total height of ground directly over the tunnel crown, if the height of ground over the tunnel crown is two times or less the excavated width of the tunnel. If the height of ground directly over the tunnel crown is greater than two times the excavated width of the tunnel, the arching action of the soil should be evaluated to determine if an increase in pressure is necessary.

Practice and literature show that moderate changes in earth loads may be responsible for large changes in predicted lining loads. This becomes especially critical for large diameter tunnels. Grübl (2012) highlights that for large diameter tunnels with segmental lining, a marginal change of lateral pressure leads to notable changes of ring bending moments. As soil-structure interaction models can capture soil arching and redistribution of ground stresses, they can be a practical method in estimating ground loads acting on lining. The voluntary and non-binding AASHTO-LRFDTUN-1 (2017) permits the use numerical analysis computer software for the determination of earth pressures and resulting load effects for the design of mined soft ground tunnels. For instance, geotechnical numerical modeling was used to derive the earth loads in the two-step design approach followed for the design of the SR 99 Tunnel in Seattle (Pilotto and Jiang, 2012; Jiang at al., 2018). The Orlovsky-Tunnel study showed through FE calculations that a 25% reduction of earth loads should be possible versus conventional methods (Grübl, 2012).

When numerical analyses are performed for the design of a tunnel lining the designer should consider the different stress states that the tunnel lining needs to accommodate starting from the initial ground stress regime, the change in stress during excavation, and the change in groundwater pressure. The estimation of ground loads becomes more complicated as the tunnel diameter increases and the probability of encountering ground of mixed stiffness becomes higher compared to smaller diameter tunnels. In that context, the use of geotechnical numerical models versus the conventional practice of empirically derived stress reduction due to arching, becomes more relevant than before. Further research should be performed to identify the instances when the use of numerical modeling for calculation of earth loads is necessary and provide the designers practical information for the application of this methodology.

Lining Loads from Numerical Modeling

In recent design-build projects, design criteria often recommend numerical analyses as a valid method for prediction of lining member loads. ACI 544-7R-16 (2016) discusses the use of FE and FD methods to calculate forces in the tunnel lining in soft ground, loose rock, and partially homogeneous solid rock based on recommendations from the PAS 8810:2016, the ÖVBB Concrete Segmental Lining Systems (2011), and AFTES WG7 (1993).

However, as there is no well-established methodology to treat the geotechnical numerical analysis output, designers tend to treat the results in different ways. An often-adopted approach is to use an overall factor of safety on the FE/FD model output. However, roadway tunnel design in the United States is headed toward Load and Resistance Factor Design. In general, this is a well-established procedure when employing conventional design calculations. However, it becomes an issue in tunnel design since soil acts as both load and resistance and factoring output is not straight forward.

Currently there is no clear approach in the United States on how LRFD can be applied when numerical methods are employed. The voluntary and non-binding AASHTO-LRFDTUN-1 (2017) states that when commercial numerical modeling is used, a rational method for incorporating the load factors into the analysis should be developed by the Engineer. PAS 8810:2016 suggests that load factors be applied to force effects (resultant structural forces and bending moments) when numerical analyses are performed. The Eurocode 7 presents the semi probabilistic concept of partial factors, referred to as "Design Approaches" that is formulated in a way that combinations of partial factors for Actions (Loads), Materials, and Resistances may be applied depending on local design rules.

Several references focus on the concept of applying partial factors in tunnel design; specifically, in connection to numerical modeling calculations, and compared methodologies and highlighted matters for further investigation (Cheung et al., 2010; Städing and Krocker, 2010; Schweiger et al., 2010; Walter, 2010; Schweiger, 2014; Brinkgreve and Post, 2015). Schweiger et al. (2010) notes that the application of partial factors on structural material should be examined regarding the mechanical behavior of the material and depending on the problem different design approaches may lead to significant differences. In the same context, Schweiger (2014) discussed the influence of the EC7 design approaches on the design with FEM and highlighted that the choice of the design approach seems to be less significant when employing advanced elasto-plastic constitutive soil models incorporating strain hardening behavior as compared to ideal-elasto-plastic failure criteria.

In summary, the connection between geotechnical numerical modeling and Load and Resistance Factor Design warrants further investigation, to provide designers with comprehensive information for designing segmental tunnel linings, particularly for large diameter tunnels.

4.3.5 Seismic Performance Analysis

In general, ground embedded tunnels are known to behave better during earthquakes than exposed, above ground structures. Inertial effects generally do not govern underground structure behavior, as these structures are lighter than the ground they replace, are enclosed in the surrounding medium, and are forced to follow the ground movement to some degree. The significant length of tunnels also differentiates them from surface structures. Bored tunnels are often preferred when the excavation depth is significant, which usually means that soil conditions improve and ground motion amplitude decreases. Another distinct feature of bored tunnels is that they are constructed without significantly affecting the soil or rock above the excavation (Hashash et al., 2001). The circular shape and the joints of segmental lining also contribute to the low observed levels of damage. Earthquake-induced damage has been recorded nonetheless and seismic tunnel design is vital to ensure satisfactory performance (AASHTO-DCRT-1, 2010).

Ground shaking alone has produced damage in tunnels, but ground failure such as fault rupture, land-sliding, and liquefaction cause the most severe damage. Ground shaking is transient ground deformation induced by seismic wave passage, whereas ground failure involves large permanent displacements and is more common at tunnel portals and shallow tunnels. Design against ground failure is more complex. Potential effects include localized displacements, tectonic uplift and subsidence, increased pressures, loss of passive resistance, floating or sinking, lateral displacements, permanent consolidation settlement, and compression/tension failure. AASHTO-DCRT-1 (2010) and other publications discuss seismic evaluation procedures for ground shaking and ground failure effects separately. Sharp curves, junctions and other connections or transitions may also need special attention. The most common solution to these interface problems involves the use of flexible joints.

Seismic design and analysis is generally based on the ground deformation approach. A key factor is the relative stiffness of the tunnel – ground system. An underground structure in stiff soil or rock may follow the movement imposed by the ground for low levels of shaking but soil structure interaction should be taken into account for a structure in relatively soft soil, especially for strong shaking.

The first step of any seismic analysis is the determination of the seismic hazard or design earthquake(s), which goes hand in hand with the desired performance level (seismic design criteria). A deterministic or probabilistic seismic hazard analysis may be performed for a site. Alternatively, existing results may be used if up to date and available for the specified location and hazard levels. Site-specific hazard analysis is more typically performed if near-field effects need to be incorporated for a structure near an active fault. Typical performance levels are life safety and functionality, sometimes referred to as maximum design earthquake (MDE) and operating design earthquake (ODE) respectively, or Safety Evaluation Earthquake (SEE) and Functionality Evaluation Earthquake (FEE) in the voluntary and non-binding AASHTO LRFDTUN-1 (2017). Indicatively, Wang (1993) developed criteria for MDE and ODE with respect to both strength and ductility, including seismic load combinations, and similarly the voluntary and non-binding AASHTO LRFDTUN-1 (2017) established criteria for the SEE and FEE. Besides the seismic hazard level, geologic conditions and tunnel design, construction, condition and importance influence the seismic performance of tunnels and the approach to their seismic evaluation (AASHTO-DCRT-1, 2010).

Any type of analysis includes the evaluation of the ground response. Depending on the type of analysis to be conducted, design ground motion parameters such as peak ground acceleration (PGA), peak ground velocity (PGV), design response spectra or time histories at the depth of the tunnel should be determined corresponding to the design earthquake events. The design and analysis of underground structures is based on ground deformations/strains rather than ground acceleration values, but these are typically produced using the aforementioned ground motion parameters. Free field deformation can be calculated in a multitude of ways, most commonly using linear, equivalent linear, or non-linear one dimensional site response analysis, which only considers vertically propagating body waves. Simplified calculation methods can be used for homogeneous ground conditions (Newmark, 1968; Kuesel, 1969; Hendron, 1985) and numerical analyses can also be performed to estimate free field deformation.

With respect to ground shaking, the main modes of underground structure seismic deformation are ovaling/racking, axial, and curvature deformation, depending on the seismic wave component and its direction of propagation. The general types of structural analysis are not that different from how other loading conditions are approached. For instance, the relevant seismic section of JSCE (2007) references the general structural calculation sections for analyzing the effects of earthquakes on the lining in both the transverse and longitudinal directions.

The most critical seismic deformation is generally caused by vertically propagating shear waves that cause a circular tunnel to oval. The vertical component of ground motion has become an important issue in seismic design as well (Hashash et al., 2001). The procedures used to calculate the lining response can be grouped under simplified analytical or numerical modeling (finite element or finite difference analyses). Simplified analysis may or may not take soil structure interaction into account.

The free-field deformation method simply assumes that the tunnel follows the ground deformation. The lining is designed for the maximum diameter change corresponding to the free field deformation (with or without a cavity present in the ground). This is considered a reasonable methodology for low shaking intensity and lining that is less or nearly as stiff as the surrounding medium, but it may overestimate the deformations if the rigidity of the structure is high relative to the ground.

Soil structure interaction modifies the free field deformations and is often considered by utilizing the compressibility ratio, C, and the flexibility ratio, F, of the lining, which quantify the stiffness of a tunnel relative to the surrounding ground (Hoeg, 1968, and Peck et al., 1972). The first is mostly related to thrust response and the second to resistance to distortion. Typically, F is large enough so that the tunnel-ground interaction can be ignored and the tunnel behaves almost like a cavity. If F is low, the tunnel lining will deform less than the free field and tunnel-ground interaction should be considered (AASHTO-DCRT-1, 2010). Wang (1993) proposed equivalent static closed form analytical solutions based on the assumption that the ground is infinite, elastic, homogeneous and isotropic, the lining is an elastic thin tube under plane strain conditions, and either full-slip or no-slip conditions exist between ground and lining. For most tunnels, the interface condition is between full-slip (maximum bending moment) and no-slip (maximum thrust). Other researchers have developed similar closed-form elastic solutions. Maximum thrust, bending moment, corresponding strains, as well as diametric strain can be calculated from the maximum free field shear strain using these expressions. Methods that simplistically account for the effect of joints on the lining stiffness have also been developed (e.g. Muir Wood, 1975). The free-field deformations may alternatively be used in a beam-spring model to take soil-structure interaction into account via appropriate springs.

Numerical modeling may be used if the complexity of the problem cannot be adequately captured using the simplified procedures. Such complexities may arise due to the tunnel shape, variable soil conditions, seismic hazard, interaction with other structures, etc. For transverse ovaling/racking analysis, two-dimensional finite element or finite difference continuum method of analysis is generally considered adequate (AASHTO-DCRT-1, 2010). The beam-spring model is sometimes incorporated into a finite element model. The three main types of numerical analyses are the pseudo static seismic coefficient deformation method, pseudo dynamic time history analysis, and dynamic time history analysis. In the first method, the maximum ground acceleration profile with depth is applied to the soil-tunnel system in a pseudo static manner. In the second method, the system is statically subjected to displacement time history steps. The model may include both, the soil medium and the tunnel, or be performed in two separate steps, during which the ground displacements are first computed around the cavity and then applied to the tunnel through springs. The third method is mostly used when inertial effects are deemed significant, in which case ground motion time histories are dynamically applied at the base of the soil-tunnel model.

Similarly, longitudinal tunnel response can be estimated using simplified analytical methods or numerical modeling. The free-field deformation method assumes that the lining follows the free field ground deformation. That is a reasonable assumption as long as the tunnel lining stiffness is considered low compared to the stiffness of the ground (AASHTO-DCRT-1, 2010). In some cases, the structure may be sufficiently long that the motion could vary significantly in amplitude and phase along its length (Hashash et al., 2001). Axial and bending strains can be estimated from free field deformations using closed-form solutions (St. John and Zahran, 1987). The bending component of strain is, in general, relatively small compared to the axial strain, but its contribution to axial strain increases as the radius of the tunnel increases (Hashash et al., 2001).

A simplified beam-on-elastic-foundation procedure used in conjunction with a ground displacement spectrum can be used to account for soil structure interaction in the longitudinal dimension. St. John and Zahran (1987) developed reduction factors that take wavelength into account for the free field axial and curvature strains. These equations are applicable to structures built in soft ground.

Numerical modeling is mostly used in the longitudinal direction when the tunnel or ground stiffness changes abruptly (e.g. connection to a station, soil/rock interface). Numerical analysis methods include lumped mass/stiffness methods and finite element/difference methods. A three-dimensional pseudo-dynamic time history analysis is typically performed to capture axial compression/extension and curvature deformations. The surrounding soil/ground is represented by linear or nonlinear springs, where the free field displacement time histories are applied in static steps.

When it comes to ground failure effects, design measures may consist of flexible joints, ground stabilization, soil replacement, drainage, or bypassing the problematic zone. Analytical procedures, some of which were originally developed for buried pipelines (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984), are generally used for evaluating the effects of fault displacement on lining response. First, the free field displacement of the fault should be estimated. Empirical relationships have been developed to estimate the fault displacements (Wells and Coppersmith, 1994). Probabilistic fault displacement hazard analysis (PFDHA) and the displacement approach are newer methods for evaluating fault displacement (Coppersmith and Youngs, 2000). For detailed evaluation of transportation tunnels crossing faults, it is generally believed that the finite element method is more appropriate than other methods. Transverse and axial springs connected to the tunnel are generally used. Analyses using discrete element models may be considered for discontinuities in the soil/rock medium. If a tunnel is at risk from slope movements due to landsliding or lateral spreading movements due to liquefaction, the potential effects are like those of fault displacement. Liquefaction may also increase the lateral earth pressure and cause uplift or settlement of a tunnel. Simplified procedures or numerical modeling can be utilized to determine the liquefaction potential.

The segmental lining joints should accommodate the anticipated ground deformations. In cases where high levels of precision are desirable, three-dimensional numerical analyses can be performed to evaluate the performance of the joints in the transverse and longitudinal directions, potentially incorporating connections and gaskets in the model. The joint behavior can be designed to remain within the elastic range or, if inelastic response is anticipated, by a more detailed model of the joint, considering lining ground interaction. Takada and Abdel-Aziz (1997) present such analysis, showing that plastic joint deformation can cause water leakage after a seismic event (Hashash et al., 2001).

The discussed methods and variations or combinations of these methods provide the forces and deformations that develop in the lining due to seismic excitation. Some considerations for the structural design include providing sufficient ductility to absorb seismic deformations, structural detailing of internal structural members or connections, and prevention of water leakage.

4.3.6 Extreme Event Analysis

In certain types of transportation tunnels, the project's Client or Owner may mandate specific investigative analyses that consider low occurrence, extreme level conditions, or exceptional conditions, that deal with high risk impacts to the structural stability of the tunnel lining to avoid a potential collapse. Typical events are fire and blast impact.

Fire

Concrete behavior during a fire, when heated to a high enough temperature, will spall explosively. This produces a hazardous condition for motorists attempting to exit the tunnel and to emergency response personnel responding to the incident. In addition, the concrete cover shielding the steel reinforcement is lost and the reinforcement is exposed to the heat of the fire which can alter the mechanical properties of the reinforcement.

Spalling is caused by the vaporization of water trapped in the concrete pores being unable to escape and can also be caused by fracture of aggregate and loss of strength of the concrete matrix at the surface of the concrete after prolonged exposure to high temperatures. The use of polypropylene fibers in the concrete mix can reduce vaporization of entrapped water, as the fibers melt during a fire and provide a pathway for water to escape and is discussed further below.

Reinforcing steel that is heated above certain temperatures will lose strength. Spalling and loss of reinforcing strength can cause changes in the shape of the lining, redistribution of stresses in the lining and possibly structural failure. A sacrificial layer can be applied to the intrados to aid and protect the reinforcing steel.

Fire Design Analysis can be typically divided into three main components:

1. Heat transfer to Tunnel Lining – This consists of an independent analysis using Computational Fluid Dynamic (CFD) modeling to the tunnel internal geometry.
2. Distribution of Temperature – This considers the how the increased temperature over time distributes through the tunnel lining using finite difference.
3. Response of Tunnel Lining to Temperature Distribution – This applies the finite difference coupled with the CFD analysis to the beam spring model to determine how the temperature induced thrust effects the deteriorated capacity of the tunnel lining.

Fire protection mitigations can be applied to the tunnel lining on the intrados/internal surfaces. in the form of coatings or post installed protection board and can provide a measure of thermal protection against relatively low temperature fires. These are specialty products and manufacturers should be consulted to ascertain the exact level of protection that they can provide. These products also hide the structure and inhibit inspection and maintenance. Fixed Fire Fighting and deluge systems can be used as alternative and should be coordinated with the fire and life safety, ventilation and tunnel drainage.

Blasts and Explosions

Post 9/11 considerations of possible terrorist attacks to transportation infrastructures, specifically tunnels, has become an increasing concern for Federal, State and local agencies and or Owners, often resulting in additional blast analysis being performed. This analysis considers the high-risk scenario that a localized explosive device is detonated inside the tunnel or near the outside of the tunnel such as in a shallow marine or harbor environment. Explosions from inside the tunnel are often considered more sensitive than those from outside of the tunnel due to the differences between a confined energy blast vs. a non-confined energy blast.

The Client or Owner, specifies the various parameters (strength, size, location) to be analyzed for the tunnel lining.

Due to the sensitivity of the topic, there is often a high level of security clearance surrounding the performance of the blast analysis for the tunnel during design and post design phases.

4.3.7 Loads and Load Combinations

A TBM segmental lining is subject to a variety of loads, largely driven by the multicity of stages between segment production and tunnel operation. Even though the primary load carried by the precast segments is axial load induced by ground forces acting on the circumference of the ring, loads imposed during construction should also be accounted for in the design. In keeping with the AASHTO DCRT-1 terminology, the main loads are grouped into transient and permanent. Load factors are also discussed later in this section.

In the US, loads and load factors for design for roadway bored tunnel lining design are recommended in AASHTO LRFDTUN-1 (2017) and AASHTO-DCRT-1 (2010). However, AASHTO notes that the load and the resistance factors specified in LRFDTUN-1 (2017) were calibrated to provide designs with member proportions consistent with the current practice in tunnel design. Load factors that differ from the ordinary LRFD Bridge Design Specifications do so due to this calibration. Load factors that were not calibrated were carried over from the ordinary LRFD Bridge Design Specifications. This is an initial step in the calibration process and further work is warranted to obtain a comprehensive calibration. Information regarding the process can be found in NCHRP Report 12-89. Per AASHTO the Engineer may use pre-LRFD practice to verify/validate the design.

The ACI 544.7R -16 report, which is focused on design of fiber reinforced precast concrete segmental lining, proposes an approach using both LRFD-based load factors and inelastic flexural resistance at the ultimate limit state (ULS) for fiber reinforced concrete. It is also understood that there is work in progress for a possible future publication by ACI working group 533, specifically on precast tunnel segments (presumably tailored to conventionally reinforced lining). A draft report of this upcoming recommendation was not available for review during preparation of this synthesis report and literature survey. LRFD load factors for design of segmental lining can be found in AASHTO LRFDTUN-1 (2017) Chapters 3 and 7 and AASHTO-DCRT-1 (2010) Chapter 10 and are reproduced in Tables 3 to 5.

For reference, a brief discussion of how load factor design is treated in other national publications is provided in the following paragraphs. The below summary is based on publications including ACI 544.7R-16, ACI 318-14, DOT-TSC-UMTA-83-16, PAS 8810:2016, BTS Tunnel lining design guide (2004), JSCE Standard Specifications for Tunneling (2007), DAUB (2013), DBV GSCCT "Guide to Good Practice-Steel Fibre Concrete" (2007), ZTV-ING (Additional technical conditions of contract and guidelines for engineering structures, Part 5 Tunnel construction, since 2008), Ril 853 (Design, construction and maintenance of rail tunnels, since 2007), ÖVBB (2011), ITA "Guidelines for the Design of Shield Tunnel Lining" (2000), AFTES GT18R1A1 (2005), as well as Städing and Krocker (2010) and Goodfellow (2011).

Transient Loads

Demolding, Transport, Storage, Installation

Tunnel segments are designed to resist loads imposed by handling, storage, lifting and erecting. PAS 8810:2016 summarizes these stages as follows: a) segment lifting and turning during curing and mold stripping; b) handling stages from precast plant to storage areas; c) segment stacking and insertion of timber spacer (dunnage) between units; d) removal from storage and unloading on site; e) transportation along the tunnel; f) segment erection in the TBM. These operations are critical and can often be the determining factors of the design.

Typical lifting methods of a segment include vacuum lifting or mechanical lifting (e.g. single-point lifting, clamping or the use of a forklift). Lifting points are identified to minimize flexural loads on the segments (typically at quarter points) (Goodfellow, 2011). Particularly for segments at an early age, made with SFRC, or with a high slenderness ratio, vacuum lifting is often preferred, which treats the segment uniformly and minimizes bending moments. It is suggested that the use of chains or lifting devices that will damage the concrete be avoided.

Segments are normally delivered to the TBM on rail-mounted cars, although in larger diameter tunnels rubber-tired vehicles can be used. A key consideration is the strength at which segments are demolded and their capability of being safely handled and stacked without stress. Some specifications recommend a minimum strength, but the exact specification depends on loads imparted by the type of lifting and handling equipment, the method of stacking segments in storage, and the location on the segment where these loads will be applied (Goodfellow, 2011). Segments are conventionally lifted from the mold and rotated to be stacked with the intrados face up. A full ring of segments is typically stacked together as a single stack. Supports (dunnage) are often selected to minimize bending moments. Some eccentricity occurs and should be considered (ACI 544.7R-16).

For the case of fiber reinforced concrete segmental lining ACI 544.7R-15 provides a detailed list of factored loads and load cases including transient (construction induced) loading. In general, ACI 544.7R-15 incorporates provisions per the AASHTO-DCRT-1 and other ACI publications.

AASHTO-DCRT-1 (2010) recommends a dynamic factor of 2.0 to be applied to the dead weight of the segment to account for dynamic shock loading during stages that it may occur. The BTS Tunnel lining design guide (2004) states that a factor of safety of at least 3.0 on the dead loads is usually used.

TBM Thrust Loads

Ram loads are applied to the precast concrete segmental lining to propel the TBM forward against friction caused by the dead load of the machine and the ground and water pressures. The force imparted by the hydraulic ram provides a concentrated variable load onto the circumferential joint face of the lining (PAS 8810:2016). Loads from the jacking forces of the TBM are significant and can cause segments to be damaged resulting in the need for replacement or rehabilitation. These forces are unique to each tunnel and are a function of the ground type and the operational characteristics of the TBM.

The TBM jacks bear against the jacking pads typically placed along the exposed circumferential joint. High compression stresses develop under the jacking pads and result in the formation of significant bursting tensile stresses deep within the segment. Additionally, spalling tensile forces develop between pads. Different methods can be used to estimate these forces due to the various geologic materials encountered in TBM tunneling.

A consideration mentioned in the BTS Tunnel lining design guide (2004) is the eccentricity between the segment and the thrust rams as the segmental rings constructed within the TBM tend to sit in the invert of the tail-skin. This eccentricity varies around the diameter. Excessive eccentricity (e.g. at sharp curves) may be uneconomical. Tail-skin seals tend to reduce this eccentricity. In tunnels below the water-table in stable strata the tunnel lining may float to the crown of the excavation resulting in an inverse eccentricity.

Since concrete segments have large rigidity and resistance to compression, buckling failure due to thrust force of shield jacks seldom occurs (JSCE Standard Specifications for Tunneling, 2007). For large tunnel sections and deep tunnels, the thrust force can become especially large.

Grouting Pressure

Grouting pressure load is generated by back-grouting or filling of the annular space using semi-liquid grouts under high pressure, primarily to control settlement at the ground surface and to ensure complete contact between the lining and the ground (ACI 544.7R-16). The anticipated grouting pressure is added to the ground loads applied to the lining. Primary grouting is commonly carried out before the ground load is fully transferred to the lining, unless the ground is very soft. The primary grout load is therefore considered to be hydrostatically applied to the lining.

Grouting is treated as a construction condition related loading in AASHTO - LRFDTUN-1 (2017). Maximum pressure values are recommended in some publications, including AASHTO DCRT-1.

It is often suggested in the literature that the grouting pressure shouldThe tendency of the ground to move into the excavated opening affects the loads on the lining. The size of the opening and the type of ground influence the tendency for this movement. The subgrade reaction is sometimes subdivided into the reaction independent of ground displacement and the reaction dependent on ground displacement (ITA, 2000; JSCE, 2007). Under rock conditions, unstable blocks and discontinuities above the tunnel crown may need to be considered.

Concrete segments possess both high rigidity and the ability to resist compression and buckling failure due to earth pressure (JSCE, 2007). The JSCE Standard (2007) also states that for large diameter tunnels, section forces caused by dead weight tend to be bigger than those caused by earth pressure and water pressure. Therefore, deformations caused by dead weight can be considered for soil reaction calculation.

Water Pressure

This load represents the hydrostatic pressure expected outside the tunnel structure. Mined tunnels are usually detailed to be watertight without provisions for relieving the hydrostatic pressure. As such, the tunnel lining is subject to full hydrostatic pressure normal to its surface. When a relief system is included, it is evaluated to determine the hydrostatic pressure to be applied to the tunnel.

Some publications recommend that both maximum and minimum hydrostatic loads be used for structural calculations. For circular tunnels setting the groundwater level higher is not always conservative (JSCE, 2007). The Austrian standard (ÖVBB, 2013) refers to ultimate limit state and serviceability limit state design water levels.

Buoyancy

The resultant water pressure acting on the lining is the buoyancy. The buoyancy force is evaluated to ensure that the dead load, primarily by the weight of the structure, is larger to avoid that the tunnel becomes buoyant. The weight of soil and water over the tunnel is considered in the resisting forces. The DAUB "Recommendations for the Design, Production and Installation of Segmental Rings" (2013) consider the verification of sufficient buoyancy safety of primary interest for tunnels.

Other Soil Effects

Consolidation, creep, swelling and squeezing may add to the lining pressure in the long term. These ground conditions are typically related to shales, marls, anhydrite, basalts and clay minerals such as corrensite and montmorillonite. AASHTO DCRT-1 mentions squeezing ground in relation to weak ground or faults. Time-dependent loads are a function of the overburden pressure, the restraint provided by the lining, and the creep characteristics of the material. The ring thrust may approach that for full overburden in soft plastic clays and may increase very little with time in a sandy soil (DOT-TSC-UMTA-83-16). For tunnels in soft ground the effects of settlement can be studied in the transverse direction in the case of consolidation settlement, or in the longitudinal direction in the case of differential settlement (JSCE, 2007). Planes of weakness are an additional soil effect that may need to be considered (AASHTO DCRT-1).

Surcharge

A variety of surface loads may be considered as surcharge, such as embankments, earth removal and weight of existing or future buildings. The Austrian standards (ÖVBB, 2013) recommend that future buildings be at a distance of at least one tunnel diameter from the outer side of the segment.

Proximity Loading

It is becoming increasingly common for shield tunnels to be constructed concurrently with or after the construction of a nearby tunnel. In this case, the soil conditions, relative position, outer diameter, construction method and timing of the tunnels is typically considered, as additional pressure may be applied to the lining due to changes in ground stresses. The BTS Tunnel lining design guide (2004) suggests that two tunnels constructed at a distance of two diameters or less between centerlines are expected to be affected by each other. The ACI 544. 7R-16 Report suggests that segmental tunnel linings are designed to take an additional diametrical distortion that can result from various mechanisms including ground movement caused by the construction of an adjacent tunnel. The Singapore Land Transport Authority (2010) specifies an additional distortion on the diameter to allow for future development near the tunnel. Large diameter tunnels have a greater effect on an existing tunnel if constructed near one each other (JSCE, 2007).

Nonuniform Loads and Distortions Due to Misalignment of Segments

Segmental tunnel lining distortion may occur during segment assembly under the self-weight of the segments due to construction-related events such as joint misalignment or yielding of joint connectors (ACI 544 7R-16). Los Angeles County Metropolitan Transportation Authority (2012) specifies a minimum additional diametrical distortion that must be considered due to imperfect lining erection.

Openings

The construction of a cross passage between one tunnel and another, a shaft, or other underground structure intersecting the tunnel involves complex construction procedures. Each stage of the construction should consider stability of the ground and the structure. Segmentally lined tunnels that have permanent openings, should have detailed analyses for the areas around openings, along with special construction methods, such as segmental lining propping, special steel segments or increased capacity alignment bicones.

Grouting Pressure

The AFTES GT18R1A1 (2005) suggests that if long-term grouting pressures are higher than the ground pressure, they should be considered in the lining design.

Recovery forces from gaskets, bolt forces

Time dependent recovery forces of the gaskets and the bolt forces may have to be taken into consideration when calculating longitudinal segment strain (ÖVBB, 2013).

Some other permanent or long-term loads that may need to be considered are:

- Lining prestressing measures
- Shrinkage
- Internal or external environment causing tunnel lining deterioration

Load factors

Three main methods have been used in the design of structures; namely service load or allowable stress design, load factor design, and load and resistance factor design. The latter takes the statistical variation of both the strength of the structural member and of the magnitude of the applied loads into account. The AASHTO LRFD Specification describes four limit states; service, fatigue and fracture, strength, and extreme event. The non-binding and voluntary AASHTO LRFDTUN-1 (2017) describes two strength, three extreme, three service and one fatigue limit state. Each of these limits states contain several load combinations. The recommended load cases for the design of linings for bored highway tunnels are given in AASHTO DCRT-1 Table 10-3.2-1 and Tables 3.4-1 and 3.4-2 of AASHTO LRFDTUN-1 (2017). Load factors are predominantly equal to 1 for the serviceability limit states (with the exception for some load cases like creep, shrinkage, uniform temperature, and temperature gradient) and vary between 0.5 and 1.75 for the ultimate limit states.

Each load factor depends on the degree of accuracy to which the load effect can be calculated and the load variation that might be expected during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are for example assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to calculate moments and shear forces (ACI 318-14).

As described by Hurt and Hart (2011), the ACI 357 “Guide for the Design and Construction of Fixed Offshore Concrete Structures” states that dead loads include “external hydrostatic pressure” and provides a load factor of 1.2. This is in line with the approach taken by international standards. Because neither ACI 318 nor AASHTO DCRT-1 has developed a load factor to be applied for grouting pressures, the load factor for water pressure has been incorporated in ACI 544.7R-16. For a load combination of self-weight and grout pressure, a load factor of 1.25 is applied to both loads. ACI 544.7R-16 refers to AASHTO DCRT-1 for load cases not covered by ACI 318. ACI ULS load factors vary between 0.75 and 1.4.

Besides the US standards, many international publications use comparable design methods and factors. The Japanese standards generally apply allowable stress design complemented by limit state design (for Level 2 seismic design). The Japanese standards state that combining both methods for one structure is not allowable and that use of the limit state design method has been increasing. In the limit state design chapter of this document, material factors, member factors, load factors, structural analysis factors, and structure factors are provided. A load factor of 1 is commonly used in the serviceability limit state, similar to US publications. Load factors for the ultimate limit state range from 0.8 to 1.3.

The PAS 8810:2016 similarly gives load factors of 1 for SLS and 0.9 – 1.5 for ULS (NA to BS EN 1990:2002+A1:2005, Annex A and NA+A1:2014 to BS EN 1997-1:2004+A1:2013).

For road tunnels in Germany, design and construction is regulated by the ZTV-ING document on tunnel works since 2008. For rail tunnels, the main document has been Ril 853 (Design, construction and maintenance of rail tunnels) since 2007. For structural design, other documents that apply are DIN 1054 (Ground – Verification of the safety of earth-works and foundations, issue 01/2005 with corrections to 10/2008), DIN-Fachbericht 100: Beton (DIN report 100: Concrete), issue 2005, DIN-Fachbericht 101: Einwirkungen auf Brücken (DIN report 100: Actions on bridges), issue 03/2009, DIN-Fachbericht 102: Betonbrücken (DIN report 100: Concrete bridges), issue 03/2009, DIN 1055-1: Action on structures – Part 1, issue 06/2002, and Ril 836: Erdbauwerke und sonstige geotechnische Bauwerke planen, bauen und instand halten (Design, construction and maintenance of earthworks and geotechnical structures), issue 10/2008 (Städing and Krocker, 2010).

In summary, a factor of 1 is used for permanent actions in the SLS and factors in the range of 0.2 – 1.5 are used for ULS. The DAUB "Recommendations for the Design, Production and Installation of Segmental Rings" (2013) refer to DIN 1054 regarding limit states. In accordance with DIN 1054, verifications in SLS generally refer to deformations or displacements to be complied with. It is pointed out in the standard that in individual cases other criteria may be relevant. The crack width verification is of importance when it comes to the design of a reinforced concrete segmental lining (DIN EN 1990 and DIN EN 1992-2). Again, factors for SLS are 1 and factors for ULS are 0.9 – 1.8.

The ÖVBB "Concrete Segmental Lining Systems" (2011) references the Austrian Standard ÖNORM EN 1992-1-1 with respect to partial safety factors and offers some modifications under special cases.

AFTES GT18R1A1 references BAEL 91 ("Règles techniques de conception et de calcul des ouvrages et constructions en béton armé suivant la méthode des états limites", Technical rules for the design and calculation of reinforced concrete structures with limit state method), Eurocode 2 (BS EN 1992), and the 1979 Common Directives ("Instruction technique sur les Directives Communes relatives au calcul des constructions", Engineering Guide to Common Directives covering structural design, French Govt. Circular No 79-25) and adopts a factor of 1 for SLS and 0.6 – 1.5 for ULS design. This document has a separate loading section for assembly systems, including gaskets.

4.4 Knowledge Gaps and Research Needs

Based on the evaluated literature the following knowledge gaps were identified and are suggested for further study:

4.4.1 General Design Related

1. Load and resistance factors are applied in the design of tunnel lining segments. The development of these factors was partially addressed in the NCHRP 12-89 research. The conclusion at that time was that there was not sufficient data available to recommend values outside of the existing AASHTO Bridge Specifications. For example, NCHRP 12-89 researched LRFD design practices for road tunnels (including fiber reinforced concrete segmental linings), while at the same time ACI sponsored research into the design of fiber reinforced concrete segments. The research projects were run un-coordinated and independently, and used many of the same technical papers to reach similar conclusions. However, the NCHRP research was looking at tunnels globally, not specifically limited to FRC segments and developed load factors and load combinations that were advanced from previous research, especially FHWA's Technical Manual for the Design of Road Tunnels – Civil Elements. Since the ACI research was not coordinated with the NCHRP research, load combinations, load and resistance factors published in the ACI report are not coordinated with the FHWA and NCHRP research. Identifying similar on-going research as part of the literature survey could potentially provide the benefit of a coordinated approach to the topic of this task. The non-binding/voluntary AASHTO LRFDTUN-1 (2017) was a move in the direction of producing calibrated load factors for design. However, ACI 544.7R-16 is another recent research product, that presents a load and resistance factor proposal for fiber reinforced segmental lining, generally based on AASHTO LRFD principles, and goes into detail in terms of the specific load factors used especially for transient loading. Coordination between these two documents could be performed to harmonize load factor design and provide clear suggestions for practitioners. By way of example, the JSCE Standard (2007) also states that for large diameter tunnels, section forces caused by dead weight tend to be higher than those caused by earth pressure and water pressure. Therefore, deformations caused by dead weight can be considered for soil reaction calculation. The load factor for dead weight (in this example) is a candidate parameter for calibration.

2. Large diameter tunneling presents new challenges due to the weight of the segments and the scale of the lining. Given the tremendous difference between TBM-imposed transient loading on the segments compared to smaller sized tunnels (thrust, torque), load factors for design should be studied in detail for either conventionally reinforced, steel fiber reinforced, and hybrid reinforced designs. The computer modeling task, in conjunction with the laboratory testing should result in data that can be used to evaluate and develop load factors.

4.4.2 Structural Design Analysis

1. While the Muir Wood/Curtis analytical method is effective and widely used in tunnel projects internationally, it is limited to uniform ground conditions over the full face of the structure. This limitation becomes particularly pronounced in the case of large diameter bored tunnels. At sections where the tunnel experiences mixed-face conditions or ground strata with significant inhomogeneity in stiffness, closed-form analysis solutions do not

provide satisfying results, because the method can only reflect one, homogeneous and isotropic stratum in the analysis. The use of ground stiffness corresponding to the stronger strata produce lower-bound results, while that of the weaker strata provide upper-bound results. Parametric studies are often performed to determine the sensitivity of the results to varying ground stiffness values.

2. In addition, the approach by Muir Wood/Curtis ignores the gradient of a gravitational stress field between the crown and the invert. In other words, the bending moments, axial forces, stresses, and displacements at the crown are identical to those at the invert. This issue can be particularly magnified in the case of large diameter tunnels. The solutions provided by Hartmann account for changes in the gravitational stress field. Studies performed by Asche and Ireland (2013) reveal that significant differences in the design thrust forces are observed in large diameter tunnels at low cover between results obtained through Curtis method and a corrected version of Hartmann's solution.
3. The empirical ring stiffness equation provided by Muir Wood (1975) ignores the stiffening effect of axial load which results in an overestimation of the design moments using the Muir Wood/Curtis equations. The use of the equivalent ring stiffness often encourages designers to opt for additional segments to reduce the design bending moments. This problem can be avoided by explicitly calculating the moments resulting from joint rotation, a methodology which has been outlined in Ireland and Asche (2011) and could be further examined and verified for large diameter tunnels.
4. Also, as discussed by Bambridge et. al. (2013), the extent and magnitude of nonlinear ground response increases with tunnel diameter. The applicability of analytical solutions such as Muir Wood/Curtis, which is based on elastic ground response, could be verified for large diameter tunnels.
5. Often, a limit of 1% (BTS, BTS Specification for Tunneling, 2010, Singapore LTA, or Table 10-2 in the FHWA TM-DCRT) of the change in radius in soft ground (and 0.5% in rock) has been set as the design standard. The applicability of this standard for large diameter tunnels could be investigated further.
6. The use of Morgan's (1961) equations for ring ovalization with the empirical ring stiffness equation provided by Muir Wood (1975) does not account for the stiffening effect of axial load and can result in large moments and areas where axial forces are low. This problem can also be avoided by explicitly calculating the moments resulting from joint rotation, a methodology which has been outlined in Ireland and Asche (2011), and could be the subject for further research in the context of large diameter tunnels.
7. Damage in segmental lining is observed typically during the construction and ring installation process, and not during the permanent condition. Hence, it could be argued that priority should be shifted to a design of the ring providing sufficient tolerance to accommodate ring installation deficiencies. One option is the Method of Joint Rotation. However, the applicability of this method for non-flat joints and trapezoidal segments could be studied and evaluated to be able to assess this approach.
8. The beam-spring analysis method was introduced by Duddeck and Schulze (1964) as a means of estimating the lining member forces meant for shallow tunnels. Zhao et al. (2017) point out that while bedding models are typically preferred in Germany for shallow tunnels and continuum models for deep tunnels, their study shows that bedding models are appropriate for both shallow and deep tunnels. The applicability of this method for deep, large diameter tunnels could be a topic for further research.

9. Additionally, Duddeck and Schulze (1964) recommend ignoring the subgrade reaction at the top 90 degrees of the tunnel. Poel et al. (2006) discusses that this assumption is not necessarily always conservative. This assumption should be evaluated and assessed for large diameter tunnels.
10. FHWA (2009) raises concerns about the validity of the beam-spring method in soft ground for cases, where the soil is assumed to be elastic and homogeneous, instead of plastic and non-homogeneous. The empirical method suggested in FHWA (2009) is intended to account for the residual moments that develop in the lining due to plastic deformations in the soil. Yet, the method is still widely used in the design of large diameter tunnels. Further study of the topic and limitations of either option could be performed.
11. The conventional approach of determining radial ground spring stiffness values based on the elastic properties of the ground and opening radius (eg. ÖVBB 2011, DAUB (2013)) may not be ideal in soft ground. FEM programs, which can model the non-linear behavior of the ground, can be used to provide acceptable ground spring stiffness values, as shown by Jiang et al. (2012), Poel et al. (2006). Further studies should be conducted to develop and a suggested procedure for producing FEA-based spring constants for design.
12. In addition, as discussed by Bambridge et al. (2013), the probability of encountering multiple geologic units within the tunnel cross-section are higher for a large diameter tunnel. Localized high bending moments may be observed in the lining, while transitioning between softer and stiffer grounds. Hence, varying ground loads and stiffnesses corresponding to the actual geologic units should be used in the beam spring analysis. It is suggested, to investigate this approach versus conventional practice of using upper and lower bound spring constants in bedded beam analysis.
13. The impacts of simulating segmental joint behavior in a single ring beam-spring analysis using hinges or rotational springs, as opposed to using a uniform reduced ring bending stiffness per Muir Wood (1975) are not well studied and understood. The applicability of the equivalent ring bending stiffness per Muir Wood (1975) and Muir Wood (2000) to large diameter tunnel linings should be studied in further detail.
14. Also, the considerations between opting for a single ring beam-spring analysis as opposed to a double ring beam-spring analysis, especially for large diameter tunnels, need to be understood in more detail. Certain authors including Blom (2002), Klappers and Grübl (2006), Arnau and Molins (2012) present studies indicating that the coupling effects between rings, achieved by staggered joints, cause an increase of the lining stiffness, resulting in increased bending moments and reduced deformations. In other words, a three-dimensional, coupled double ring beam-spring analysis can yield higher design moments, than those predicted by single ring beam-spring analysis using full ring rigidity. The two models could be studied in parallel for large diameter tunnels.
15. Deciding to perform a single ring analysis in conjunction with a 3D analysis, or rather a coupled double ring beam-spring analysis, replacing the need for a 3D analysis should be studied. The work presented by Grübl (2006), as pointed out by Arnau and Molins (2012) , would suggest that a double ring beam-spring analysis could replace a coupled three-dimensional analysis. 3D FE analysis is time consuming and sometimes computationally expensive. Given the popularity of the twin-ring bedded beam approach, it should be investigated, if this method can replace a complex 3D numerical analysis of segmental lining and provide sufficient results.
16. One of the challenges of performing structural numerical modelling with the simulation of realistic joint behavior is the difficulty in estimating the mechanical properties of the

longitudinal and circumferential joints. Methods proposed by Leonhardt and Reimann (1965), Janssen (1983), Blom (2002) , Arnau and Molins (2011) , Jensen (2017) and DAUB (2013) are available for estimating the longitudinal joint properties. However, limited literature exists on the implementation of the coupling behavior between segments in the model. Grübl (2006) (cited by Climentepe (2010)) suggests representing the coupling of the rings by using non-linear lateral springs which represent the shear stiffness and the maximum bearing capacity of the coupling. Investigation of large scale testing to produce realistic input parameters for rotational or longitudinal coupling spring constants is suggested.

17. The use of large diameter TBMs implies that greater pressures are to be absorbed by the precast segments in a smaller circumferential joint area compared to regular sized TBM tunnels (Bambridge et al., 2013). In other words, higher tensile and splitting stresses can be anticipated for large diameter tunnel segmental linings. By means of parametric studies and FEA structural modeling different possible layouts of thrust pads, thrust forces for one or two typically large diameter tunnels should be investigated. A production of tabulated results that can be used as a rough preliminary aid for designers, is considered very useful to the industry.
18. Nasri (2014) suggests the use of FEM methods or analytical solutions such as Iyengar Diagram for determination of tensile bursting stresses that result in more cost-effective reinforcement distribution compared to the simplified equations presented in ACI/DAUB/Eurocode/British Standards. This maybe particularly important for economic designs in large diameter tunnels and should therefore be further studied.
19. As observed by Bambridge et al. (2013), TBM torque increases cubically with the tunnel diameter in order to excavate the ground and overcome the friction between the rotating cutter head and the ground. For smaller tunnels, the TBM torque has typically not been a governing factor in the segmental tunnel lining design as the torque could generally be counteracted by the friction between the shield and the ground. However, for large diameter bored tunnels, especially with shield gap injection for reducing the ground loss and surface settlement, there is a potential need to react a portion of the total TBM torque against the segmental lining. This creates a new loading scenario that has not traditionally been considered in the tunnel lining design process. Additionally, this consideration could impact the detailing of the circumferential joints and the joint connectors. More research is warranted in this area to better understand the mechanism of torque transfer and if and how this should be implemented in the design.
20. The buoyancy of large diameter tunnels should be further explored. The difference between the buoyant force and the lining self-weight increases with the tunnel diameter. Sufficient ground cover should be provided to resist flotation. Current literature suggests for the preliminary design and planning purposes, that a minimum cover of 0.4 to 0.5 times the tunnel diameter should be provided. Work should be performed to explore typical tunnel portal conditions and determine if specific safety factors against floatation, exclusive to large diameter tunnels should be developed.
21. The weight restriction for handling and transporting the segments from the fabrication facility to the TBM portal often dictates the number of segments comprising a full ring. Consequently, the number of segments per ring grows with the tunnel diameter as evident from the presented data. Per Grübl (2012) fewer segments per ring results in higher bending moments and lower deformations. According to the same, a 9 + 1 ring (9 normal segments, one key) has an about 20% higher bending moments and about 10% smaller

deformations than a 12+1 ring. As the rotation in the longitudinal joints for a ring with 13 segments is about 20% higher than in the 9+1 ring, the allowed transverse forces in the joints are lowered by about 40%. Large diameter tunnels equal or larger than 18.3 m (60') appear likely in the future. Therefore, investigation of segmentation under an assumption of standard overburdened condition should be considered.

4.4.3 Geotechnical Numerical Analysis

1. Geotechnical numerical analysis is necessary to meet design criteria of projects (for example SR-99) for predicting tunnel lining forces and loads. Given that numerical analysis is becoming increasingly important for large diameter tunnels, and given that the roadway tunnel design in the US is anticipated to follow the voluntary and non-binding LRFD method, the question arises, how should the loading output on the lining, resulting from the numerical modeling be treated? Since the implementation of a true load factor design in an FEA model is challenging, research could be performed on how to implement geotechnical numerical modeling results in the design. In Europe, Eurocode 2 offers provisions for designers to implement alternative ULS principles with FEA modeling. Another approach is to use a "weighted average" load factor based on SLS and ULS- derived pressures, on the lining loading results of the geotechnical numerical analysis, regardless of load type (surcharge, water or effective earth load). The optimal process to develop reliable load factors is currently lacking.
2. The main question pertaining to earth loads is whether modern design criteria for large diameter tunnels should rely on conventional practice (i.e. full overburden or reduction based on arching) for calculation of earth loads or rather use geotechnical numerical modeling for the determination of these loads? The ways geotechnical numerical modeling may be used as a tool to predict earth loads for design of large diameter tunnels should be investigated further.
3. In the case of three dimensional geotechnical numerical programs, there is often a limit as to the possible level of segmental lining modeling detail that can be achieved. For example, the most refined model would include all segments with their proper geometry however at the expense of preparation time, modeling execution time and post-processing time. It is not uncommon that continuous non-segmented rings are used with three-dimensional geotechnical modeling which points back to the question of the proper ring stiffness value to use. If three-dimensional soil structure interaction is to be used for lining load calculations incorporation of the LRFD approach into geotechnical numerical modeling, should be researched.

4.4.4 Effect of Tolerances in Design

A literature survey was performed on the two main types of segmental lining tolerance – production and installation. Tolerance is a crucial part of segmental lining behavior and as the diameters increase, the TBM transient loads increase, too, and the tolerance plays an even bigger role in performance. Work by Kolic and Mayerhofer (2009) suggest that segment dimension tolerances tend to increase with increasing tunnel diameter. The same research also notes that installation tolerances tend to become tighter for single-pass, watertight tunnels with connectors, even if the diameter is significantly larger. It has been noted in the literature that specially in the case of SFRC segments, tight tolerances can potentially be more critical compared to conventionally reinforced segments. The development of a multitude of new coupling hardware (i.e. alignment dowels and bicones) finds an increasing use in controlling installation tolerances in large diameter tunnels; an effect that ordinary bolt connections do not offer. This seems to be an indication, that with increasing diameters and in addition to a specification of tolerances, strict tolerance control should be embedded in US practice. In addition to a formal review of typically cited tolerance limits (production and installation), the opinion of TBM manufacturers, precast segment mold and connecting hardware manufacturers should be compiled and analyzed to gain a comprehensive view on the subject. Work could be performed to assess if specific production and installation tolerance levels should be developed for large diameter tunnel designs.

5 MATERIALS

This Chapter discusses materials, material testing, theoretical and actual material behavior, durability, corrosion and deterioration factors, fire resistance/damage and concrete mix design as they appear in the literature surveyed.

5.1 Reinforced Concrete

5.1.1 General Description and Typical Concrete Mixes

Per AASHTO DCRT-1, concrete mixes for precast segments for initial linings do not need special designs and can generally conform to the structural concrete mixes provided in most state standard construction specifications. Compressive strengths are typically specified for various stages. 28-day strength values in the range of 5,000 to 7,000 psi (34 to 48 MPa) are generally used in the US. Higher strengths can be obtained on a project specific basis. In typical mixes for conventionally reinforced precast concrete lining, designs specify:

- Type of cement (typ. portland cement, of type depending on project)
- Concrete compressive strength
- Minimum water to-cement (incl. ash) ratio
- Silica fume content
- Fly ash content
- Total cementitious content
- Chloride ion penetrability
- Air entrainment (if allowed)
- Alkali and Chlorides content
- Aggregates (distribution, type, chemistry, water absorption)
- Polypropylene fiber content (type, geometry, and dosage)
- Other admixtures

Segment formwork consist of steel molds constructed for each project. The reinforcement is placed into the form and the concrete is poured in the pre-cast concrete facility. In the case of tapered ring segments, each mold is specifically made for a specific segment. Each mold should meet or exceed the finished segment tolerance specifications as specified by the project specifications and plans.

Curing specifications may differ depending on projects, and adequate curing is essential for the concrete segment to gain early strength to resist demolding and early handling stresses. Curing is a well-controlled process and may involve moist curing, curing compounds, or a steam curing process. Per AASHTO air entrainment in precast segmental lining is desirable since segments may be stored outdoors for extended periods of time, and final lining segments may be exposed to freeze/thaw cycles during service.

Per AASHTO, reinforcing steel bars should conform to ASTM A 615 grade 60 and if used, welded wire fabric should conform to ASTM A 185. ASTM A706, Grade 60, low-alloy deformed bars is a potential option in the US for precast segmental lining. However, some corrosive environments may call for the use of galvanized, epoxy-coated, or stainless steel reinforcement. If used, typical segmental lining designs call for steel welded wire fabric per ASTM 496 and ASTM 497 with minimum tensile strength 90 ksi and minimum yield strength 80 ksi.

Reinforcement cage welding or tying is used in projects, with welding being often the preferred option due to the rigidity it imparts to the reinforcement cage. Precast facilities can efficiently weld reinforcing steel cages.

Figure 5-1: Steel rebar reinforcement for the Galleria Sparvo tunnel in Italy. Photo: FHWA.

Figure 5-2: Segment mold carousel at the concrete plant of the Galleria Sparvo. Photo: FHWA.

Figure 5-3: Segment mold at the Galleria Sparvo. Photo: FHWA.

5.1.2 Testing

Pre-production concrete mix testing and production testing is the current typical practice in the US with respect to Quality Control of precast concrete segments. Pre-production concrete testing is performed to establish a concrete mix that will provide the project specified strength levels and meet other specified items such as air entrainment, slump and unit weight. The test mix should include any polypropylene fibers at the proper dosage if specified in the design. A certain number of test cylinder specimens is cast and cured in the same way as the complete segment. The samples are demolded at select time intervals in pairs and subject to compression tests.

Production testing involves a frequent testing program that ensures that the specified concrete strength is maintained throughout the production of precast concrete segments. Typically, several specimens are prepared at a predetermined time interval or concrete volume used, cured and tested in compression. Additional weekly testing is often specified to ensure the specified early strength for demolding is achieved.

5.1.3 Theoretical and Actual Material Behavior

In general, conventionally reinforced concrete for precast segmental lining is a material well studied for its behavior and it follows the well-established principles and proven mechanical models of typical reinforced concrete.

5.1.4 Durability

Durability of precast segmental lining is discussed in detail under section 4.2.8. Various US and international tunnel lining publications discuss design life and durability of rebar reinforced segmental tunnel lining (AASHTO, AFTES, BTS, BSI, DAUB, JSCE, ÖVBB). A brief synopsis of these discussions is provided here.

Generally, many of the recommendations to improve durability as indicated in the various publications apply to both conventionally and fiber reinforced segmental lining. Based on this review, there is no direct correlation between specifics of durability design and large diameter tunnels, other than the service life itself. Depending on the project, these projects due to complexity and large capital investment, can have a specified service life of more than 125 years.

In the US, it is common practice in design-build projects for design builders to prove through durability reports that their selection of materials, lining components and/or Design Life prediction methods, results in the final lining product for the specified service life as specified by the owner. As explained in 4.2.8, durability of precast segmental lining with conventional reinforcement is generally good, but as steel is a material highly susceptible to corrosion, corrosion control measures are necessary.

Typically, the following segmental lining aspects are analyzed to evaluate the overall durability of precast segmental lining

- Concrete mix
- Additives including corrosion inhibitors
- Specified strength
- Fabrication/ curing protocol or methods
- Reinforcement cover
- Cage - reinforcement material
- Permeability of cast concrete
- Waterproofing
- Corrosion control (considering the chloride content, alkalinity of surrounding soil and groundwater, presence of sulphates or other project specific aspects of the chemistry of the soil and groundwater).

- Other environmental factors or chemicals including temperature, hydrocarbons, micro-organisms, condensation, frost, salt and de-icing agents, fire.
- Serviceability
- Long term maintenance plan of tunnel structure

AASHTO DCRT-1 discusses the use of High Density Concrete and Corrosion protection methods as means of reducing the risk of deterioration. The AFTES GT18R1A1, BTS Tunnel Lining Design Guide (2004), note a list of factors contributing to the durability of segments and discuss durability of steel bar reinforcing.

Per AFTES, the durability of the steel bars is dependent on the permeability of the concrete, the depth of concrete cover, type of cement, and ultimately the combination of the factors above. AFTES notes that the composition and surface condition of the steel bars should ensure good weldability for fabricating the reinforcement cage (a challenge when epoxy treated bars are specified). The depth of the concrete cover specified varies from project to project and is typically in the range of 3 – 8 cm (1" – 3") in examples of large highway tunnels in the US. Although increased concrete cover of conventionally reinforced segments is a desirable feature, its selection should be weighed against the total increase in excavation area that results, its effect in the structural performance of the segment and its potential for damage during transportation and installation.

Steel corrosion protection measures including paint coatings, metallization (i.e. galvanizing), epoxy-coating, or cathodic protection systems, may be needed in certain applications.

Per BTS (2004) and BSI (2016) three types of attack affect the durability of metal constituents of concrete tunnel lining:

- Corrosion of exposed metals
- Chloride-induced corrosion
- Carbonation-induced corrosion

Other mechanisms have the potential to attack the concrete material itself such as:

- Sulphate
- Acid attacks
- Alkali-silica reactions.
- Freeze-thaw cycle
- Wearing stresses of inner surface of lining
- Impact
- Cracking

Specifically, for precast concrete lining, the BTS Tunnel Lining Design Guide (2004) discusses that the detailing of the ring plays an important role in the success of the design and performance of the lining throughout its design life. The ring details should be designed with consideration given to casting methods and behavior in place. Some of the more important considerations are as follows.

- Eliminate all embedded metallic fittings and fixings, bolt sockets and grout sockets
- Thickness and segments size particularly related to handling and transportation.

- Gasket grooves: Too small a distance to the edge may result in the enclosing nib breaking under load or when transporting the segment.
- Joints: Detail joints to achieve the specified water-tightness considering the type of waterproofing material used.
- Joint bearings: Detail bearings to achieve adequate bearing area but with reliefs or chamfers to minimize spalling and stripping damage.
- Consideration in the overall detailing should be given to all fabrication and installation tolerances.
- Embedded fixings/holes should be positioned to allow continuity of reinforcement (when needed) while maintaining cover.

DAUB (2013) discusses that although dense concrete mixes are generally advantageous for durability, high strength mix designs can produce concrete segments with more pronounced brittle behavior, thereby increasing the actual risk of corner or edge spalling which even if repaired become themselves weak points thus compromising durability. AASHTO LRFDTUN-1 also notes that dense concrete mixes can be subject to shrinkage cracking. When designing dense concrete mixes, the effects of heat of hydration and the formation of shrinkage cracking should be considered, and the mix designed to minimize shrinkage cracks

Documents including the JSCE (2006), ÖVBB (2013) and BSI (2016) specifically callout attention to segment cracking as a path to durability reduction. Cracks increase permeability, allow water inflow and result in risk of rebar reinforcement corrosion. Crack width limits are recommended in these publications depending on environmental exposure classes.

5.1.5 Fire Resistance and Damage

When exposed to the high temperatures created by fires, concrete has a natural tendency to spall in an explosive manner. Spalling occurs due to the rapid formation of water vapor within the concrete matrix which is not able to escape. Increased heat levels may also cause aggregate breakage and steel reinforcement relaxation, further contributing to the lining distress and possible failure. The ITA Guidelines for Structural Fire Resistance for Road Tunnels (2014) provide a detailed discussion on lining material behavior under fire conditions. An excellent review of structural performance of segmentally lined tunnels under fire conditions, is given in the thesis work by Lottman (2007) as part of research performed at Delft University. This work included a case study of the Groene Hart large diameter TBM tunnel in the Netherlands. Segmental lining material behavior under fire is also discussed in Tarada and King (2009), Gipperich et al. (2010), Yan et al. (2013, 2015, 2016). An approach to practical design of precast segmental lining considering the effects of fire loading is given by Monckton (2018).

Per AASHTO DCRT-1 external and internal protection measures can be employed to provide structural fire resistance to precast segmental lining. External measures (fireproof cladding) are discussed more in Section 5.8. Polypropylene fibers are widely acknowledged in the industry and by most tunnel lining publications as effective means of minimizing the risk of explosive spalling providing integrated fire safety in segmental lining. Polypropylene fibers are discussed in Section 5.2.5.

5.2 Fiber Reinforced Concrete

5.2.1 General Description and Typical Concrete Mix Designs

Fiber Reinforced Concrete (FRC), especially Steel Fiber Reinforced Concrete (SFRC), has been used for a couple of decades for precast segmental linings all over the world, i.e. in Europe, the US, and Asia. FRC allows either to reduce the traditional reinforcement in combined or hybrid solutions or completely replace it. Generally, FRC is a composite material consisting of discrete (discontinuous) fibers made of steel, polymer, carbon, glass or natural materials and concrete. However, currently most commonly used in precast segmental liners are steel and polymer (synthetic) fibers. Steel and so-called macro-synthetic fibers are improving the structural properties of the hardened FRC, while micro-synthetic fibers improve shrinkage cracking of the fresh concrete and improve the fire resistance.

Steel or macro-synthetic fibers change the mechanical properties of concrete. However, for fiber dosages typically used for pre-cast segmental linings (less than 60 kg/m^3), the primary improvement of mechanical properties is gained in the post-cracking behavior under direct or flexural tension. The failure behavior is changed from brittle to elasto-plastic and adds “toughness”. At the above fiber dosages, the failure mode in a bending test shows strain-softening. Higher fiber dosages are necessary to achieve strain-hardening under pure bending. The improvement of mechanical properties of the FRC with typical fiber dosages in the elastic phase and under compression is negligible. Therefore, the mechanical properties in the elastic phase and under compression are typically tested, modelled, and designed like plain concrete. The structural advantages of FRC for precast segments are cracking control during handling, installation, and long term loading. The crack width control and reduced permeability leads improvement of the durability and improvement of the impact resistance i.e. during transport of the segments or long term. Other advantages are the reduction in material and labor costs, sustainability due to lower steel material usage.

Historically, FRC was specified primarily based on the fiber dosage and many smaller diameter tunnels solely reinforced with SFRC have a fiber dosage between 40 to 100 lb/yd^3 (25 to 60 kg/m^3). However, it should be noted that the fiber dosage is not an objective value for specification, because the fiber diameter, length, geometry, material properties and types differ. Therefore, recent projects move towards a performance based specification of FRC as proposed in i.e. Model Code 2010. Typical dosage for mono-filament or fibrillated macro-synthetic fibers ranges from 8 to 10 kg/m^3.

Micro-synthetic fibers are typically added to precast segmental liners to improve the performance under fire conditions as a means for passive fire protection, which is especially problematic in road tunnels. It should be noted, that micro-synthetic fibers do not serve a structural purpose like steel or macro-synthetic fibers. Micro-synthetic fibers are discussed in more detail further below.

Compared to unreinforced or rebar reinforced concrete macro- and micro fibers add significant surface area to the concrete mix that binds water in the fresh concrete and therefore changing the rheology. The concrete mix has therefore to be adapted if using fibers, primarily by increase of fines in the mix, but also adding admixtures to ensure proper workability of the mix.

Steel fibers should be Type I (cold drawn wire) per ASTM A820 Specification for Steel Fibers for Fiber-Reinforced Concrete.

Macro- and micro-synthetic fibers are specified per ASTM C1116, Type III.

The following sub-chapters are focused on steel and macro-synthetic fibers. Micro-synthetic fibers are not further discussed.

5.2.2 Theoretical and Actual Material Behavior

Fibers change the mechanical properties of concrete. However, the modification of the elastic properties and the compressive strength is for practical purposes negligible for fiber dosages typically used for FRC segmental linings (i.e. < 60 kg/m^3 for steel fibers). The fibers are influencing, however, crack development and post-cracking behavior under tension and bending. The improvement of the mechanical properties also positively influence the structural behavior of segments regarding impact loading (i.e. damages during handling and transport) and splitting loading (i.e. due to TBM jacks).

For the design of FRC typically well-established concepts from regular reinforced concrete structures are adapted. FRC on the compression side and in the elastic state is modelled like plain concrete and classical concepts and simplifications apply. The biggest change, however, is the expansion of the stress-strain-relationship (SSR) on the tension side, beyond the linear elastic flexural strength. The post-cracking behavior or toughness of the FRC is modeled based on an equivalent flexural strength derived from beam-tests. There is, however, a mechanical disconnect between the evaluation of beam-tests (and according assumptions as an ideal-elastic material) and the actual material behavior. The theoretical model assumes ideal-elastic behavior and linear stress distribution over the cross-section height of a homogeneous and isotropic material. However, the cracked beam is (around the crack) neither isotropic nor homogeneous. In a simplified assumption, the deformations around the crack are transferred in an “equivalent strain” by using a so-called integral approach. The actual deformations around the crack, which include primarily plastic but also elastic components, are distributed evenly over a “structural characteristic length”. This assumption of the characteristic length is important, because it will also be used to determine the crack width primarily in the SLS design, but also the USL design. The structural characteristic length based on Model Code 2010 is the minimum of the mean distance value between cracks or the distance between the neutral axis and the tensile side of the cross section (height of the cross section under tension). RILEM (2000) also uses the height of the cross-section under tension as the characteristic length. In addition, equivalent strains are derived under the assumption of a linear distribution of the of the stresses over the cross-section height, which is clearly not the case in the post-cracking phase. Therefore, the term “equivalent strength” is used. However, utilization of the equivalent strength in the SSR would lead to an overestimation of the bearing capacity. Therefore, the equivalent strain is transferred into a corresponding strength to be used in the SSR and is called the “residual tensile strength” in Model Code 2010 or “post-cracking stress” in RILEM 162-TDF. Typically, the equivalent flexural strength is multiplied by a factor to gain the residual tensile strength, which is derived from certain simplified assumption of the stress distribution over the cross section.

5.2.3 Durability

Durability and corrosion of fibers, especially steel fibers, has been researched widely. The corrosive effects and passive protection of steel in the concrete matrix known from rebar reinforcement are basically identical. As described in Section 4.2.8, contrary to steel rebar reinforcement, fibers are spread through the concrete matrix, disconnected from each other. This provides a distinct corrosion mitigation effect. However, there are two major differences between rebar reinforcement and fiber reinforcement. Like rebar, the corrosion of fibers will create rust, which comes with an increase of volume of the rust products. However, since the diameter of fibers is much smaller than the diameter of rebar the stress created by the increased volume is too small to create spalling of the concrete cover. Secondly, inevitably some fibers can be located at or close to the concrete surface with minimal or no concrete cover. This leads to a – primarily aesthetic -problem due to rust stains at the surface.

The depth where fibers corrode in un-cracked FRC extends from the surface to around 0.1 to 0.2 mm (.004” to .008”). More critical, however, is the corrosion of steel fibers crossing open cracks. It was found that steel fibers do not corrode in cracks up to 0.15 to 0.2 mm (.006” to .008”) in width. Maximum acceptable crack widths for the SLS design, chosen based upon environmental conditions, are typically at or below these crack widths.

Synthetic fibers are not subject to corrosion effects which are typically observed for steel fibers.

5.2.4 Fire Resistance and Damage

Macro-synthetic fibers quickly lose their mechanical properties when exposed to high-temperatures and are therefore generally not used as permanent reinforcement of road tunnels where they can be exposed to large tunnel fires.

Steel fibers and macro-synthetic fibers provide very little or no influence on the prevention of explosive spalling.

Fire resistance can be improved using micro-synthetic fibers (monofilament polypropylene). High temperatures and remaining moisture in the segments create a water vapor overpressure during fire events that leads to explosive spalling of the concrete at the surface. To improve performance, polypropylene micro-fibers are added into the concrete mix. During a fire event the micro-synthetic fibers melt and create microcapillaries that allow the developing vapor steam pressure to relief and thus avoiding explosive spalling.

The above theory is commonly used as an explanation to describe the positive effect of the micro-fibers. However, Naaman (2018) quotes conflicting research from Dehn and Koenig that leads to doubts about this explanation. The authors report that a large portion of the free and chemically bound water in the concrete would have already evaporated at temperatures between 100˚C to 200 ˚C before the beginning of the decomposition of the polypropylene fiber at 205 ˚C.

Typical fiber dosages for micro-synthetic fibers for fire protection range from 2.5 to 3.5 lb/yd^3 (1.5 to 2.0 kg/m^3) (ITA tech Report No.7).

5.3 Other Concrete Reinforcement Materials

5.3.1 General Description and Past Research

As noted in Section 4.2.3 the current practice, if FRC is not adequate by itself, is a hybrid solution of fibers and reinforcing bars that can be used to achieve the needed flexural strength of the lining at ultimate limit state and to improve the crack control at serviceability limit states. The use and design of hybrid reinforcement systems is described in various design publications including ACI 544 7R-16, ACI 4R-18 or the Swedish Standards 812310 (2014).

Typically, segments with steel fiber reinforcement only are used for rings with low bending moments and compression forces. However, high ram forces and partially loaded joints may include additional convention steel bar reinforcement, which add to the cost of fabrication. Many large diameter tunnels today including the Barcelona Line 9 subway and the Madrid Subway in Spain, the Waterview Connection, the CLEM7 Tunnel and Airport Link Tunnel in Brisbane, utilize some hybrid reinforcement. In large diameter projects, SFRC segments are utilized for most the alignment, and hybrid segments are used in areas of high moment and thrust such as areas of high overburden or in sections close to a connection with another tunnel, cross passages or station.

Typically, rebar reinforcement in hybrid designs takes the form of bent steel rebar ladders positioned along the long curved edges of the segment, or an overall circumferential rebar frame that includes all sides of the segment and structurally connects them. Hybrid designs with a full conventional steel rebar cage and steel fibers in the concrete mix, are also possible. The ITA Working Group 2, in producing the ITA Report No.16 (2016) on fiber reinforced segmental lining, performed an extensive research on projects where pure fiber and hybrid reinforcement solutions were used. The ITA Report No.16 contains a list of international projects examples where FRC and hybrid reinforcement was utilized. Similar lists can be found in ACI 544.7R-15 and in Gong et al. (2017). Figures in ITA's Report No. 16, demonstrate a clear increasing trend of larger diameter projects involving hybrid reinforcement.

Hybrid reinforcement is an emerging trend in the tunneling industry and there has been continued research interest on the subject over the past few years. A series of research papers was collected and reviewed for this synthesis report and a brief synopsis of some of these is provided here. Gettu (2016) describe the hybrid reinforcement and fiber mix used at the Barcelona Line 9 metro. Chiaia et al. (2009) use two different mechanical models, introduced with the purpose of computing the minimum reinforcement area and the crack width of hybrid members. Both models have been successfully applied for designing some cast-in-situ hybrid tunnel linings in Italy. Mobasher et al. (2015) performed research on hybrid reinforcement and provide analytical expressions for load–deflection response explicitly derived based on simplified bilinear moment–curvature curves. In their work, parametric studies demonstrate that the use of discrete fibers to increase residual tensile strength is not as effective as continuous reinforcement in improving the moment capacity. However, the ability of fibers to distribute cracking leads to higher stiffness and strength compared to plain reinforced concrete. Liu et al. (2018) performed testing of hybrid reinforced segments and concluded that assuming a constant steel rebar reinforcement, any addition of fiber has little effect on the cracking moment of segments. However, there is obvious increase on the stiffness and bearing capacity of reinforced concrete segments after adding steel fiber or synthetic fiber, especially in the control of the crack width and the ultimate bearing capacity. They also report that the fiber reinforced concrete segments with reduced traditional reinforcement exhibit the same capacity as traditionally reinforced concrete segments without reducing the amount of reinforcement. This includes the bending moment when crack width reaches 0.20 mm (.008") and the ultimate bending moment, and the mixed reinforcement method is considered able to meet the design demands. The bearing capacity and stiffness of segments increases with the increase of dosage of fiber if traditional reinforcement remains unchanged.

Bernard (2016) performed testing to investigate the behavior of hybrid reinforced concrete beams using macro-synthetic fibers. Neu et al. (2018) performed numerical analysis testing of hybrid reinforced segments to investigated their structural performance. Both authors highlight the importance of installation tolerance with specific numerical predicted results for the performance of hybrid segment designs.

Yao et al. (2018) presents a novel approach with analytical closed-form solutions to construct full range thrust-moment interaction diagram for hybrid segments based on parametric multi-linear material models for tension and compression of FRC matrix and steel model for rebar.

Yan et al. (2015) investigated hybrid fiber reinforced performance under fire conditions. The authors conclude that further research is needed to derive quantitative recommendations regarding the optimum dosage and the mixing ratio of the hybrid fibers, as well as detailed effects of the combined steel-reinforced and hybrid designs to maximize the spalling resistance while maintaining a desirable structural behavior of the tunnel lining segments when exposed to fire.

Harding and Francis (2014) discuss fiber and hybrid reinforced segment design for the Brisbane Airport Link. De la Fuente et al. (2016) report that although the cost of the self-consolidating fiber reinforced concrete (SC-FRC) concrete mix is some 15% higher than that of FRC, the greater spatial efficiency of the fiber distribution in the case of SC-FRC reduces by 10% the quantity of fibers needed to achieve mechanical characteristics equivalent to those of FRC.

Bakhshi and Nasri (2016) report that if the aspect ratio of a segment (the developed segment length divided by its thickness), is higher than 10, it is generally necessary to adopt a hybrid reinforcement of fibers and conventional steel bars. However, some researchers have proposed to increase the slenderness limit up to 12 – 13. Full-scale tests are needed to validate the usage of fibers with such slenderness conditions.

5.4 Connection Hardware

The design and utilization of connection hardware including bolts and dowels is discussed extensively in section 6.2.5. Generally, in the US it is commonly specified that bolts should conform with ASTM A307 or ASTM A325 and hot-dip galvanized for corrosion protection. Bolt nuts are typically specified using ASTM A563 (hot-dip galvanized) and washers are specified to meet ASTM F436 (hot-dip galvanized).

Bolt inserts can be specified by either ASTM A36 steel, zinc coated (hot-dip galvanized) in accordance with ASTM A153, or non-ferrous.

A wide variety of dowel systems exist today that accommodate a range of pullout and shear forces. Besides the load transfer, the main purposes of dowel systems are the quick, easy installation in precast sockets, compared to bolts, and as an aid for alignment control. Depending on size and design, dowel systems can handle pullout forces in the range of 20 – 120 kN and shear forces up to 160 kN. Certain products today fall under the category of "bi-cones" developed as alignment control hardware with a high shear capacity in the range of 150 to 500 kN. Such elements can handle increased shear forces between segments due to high TBM torque reaction or at openings in the segmental lining , i.e., at cross passages and other connections.

Connection hardware is generally subject to the same design life as the overall tunnel structure. Although not very common, specialized performance testing of connection bolts under fire and high heat conditions may be specified.

5.5 Gaskets

5.5.1 Materials

Waterproofing gaskets are discussed in section 4.2.6. The gasket recommendations by STUVA and AFTES, are generally adopted in the gasket manufacturing industry. The elastomeric gasket material is typically EPDM (Ethylene Propylene Diene Monomers) formulated to provide high retention of elasticity and low stress relaxation properties. Hydrophilic gaskets are inexpensive and do not need a high compression force to be applied by the erector arm or TBM thrust jacks. They were widely used in some regions in the 1990s and 2000s. However, in recent years large diameter tunnels almost exclusively use EPDM gaskets. The long-term durability of hydrophilic gaskets under high hydrostatic pressures has not been demonstrated. They are glued onto the segments, are not under any tension, and only swell to fill the gasket groove when wet. As a result, hydrophilic gaskets can occasionally become loose and sag into the tunnel, becoming non-functional. Discussions regarding example uses of gaskets in large diameter tunnels can be found in Bomben and Bringioti (2013) and Datwyler (2012).

Manufacturers produce a variety of gasket profiles, in either glued-on or anchored types. Glued-on gaskets typically need a special gasket pressing frame that aids in the installation of the specific gasket onto each individual segment with precision. Specialized adhesive compound is applied on the segment gasket groove either by brush or by pneumatic gun. The gasket should be lightly tensioned to ensure that it grips and fits snugly into the groove.

Anchored gaskets have gradually emerged in the market providing advantages such as installation time savings (there is no need for a crew to spend time fitting the gasket into the groove or to repair the gasket grove if needed), cost savings as no adhesive is used, better waterproofing behavior as the potential seepage path is longer, and a more robust connection to the segment. Gaskets should perform per the specifications at all possible gap and offset conditions. Savings in materials can be realized if installation tolerances are reduced with appropriate means and connecting hardware (Datwyler, 2012).

Modern gaskets are fully vulcanized with tight geometric tolerance control. Significant progress has been made by manufacturers at the corner areas of the gaskets which in early products had a tendency to cause unneeded concentration of stresses in the segment extrados edges that often resulted in segment damage. Technique development and testing trials have resulted in corners that have an extruded profile form with typically lower stiffness than the main gasket section. That form allows them to stay in proper shape and be strain compatible with the abutting gaskets during installation without leading to stress concentrations in these areas.

5.5.2 Durability

Typical materials tests used to demonstrate the strength and durability of gaskets are listed are:

- Tensile Strength: ASTM D412
- Elongation: ASTM D412
- Hardness: ASTM D2240
- Compression Set: ASTM D395; Method B
- Aging: ASTM D573, 70 hours at 100-degrees Centigrade (212 degrees F). Limit
- Water Absorption: ASTM D471, 48 hours at 70-degrees Centigrade (158
- Oil Absorption: ASTM D471, 70 hours at 70-degrees Centigrade (158 degrees

- Ozone Resistance: ASTM D1149, by method described in ASTM D518

Gaskets should also be resistant to specific concentrations of compounds in the ground and groundwater chemistry of each project. The gasket should be resistant to alkalinity of the tunnel lining concrete and tunnel grout.

5.6 Load Distribution Plates and Packing

Various standards describe the use of what is commonly referred to as load distribution material, which may be used in the form of glued-on sheets of compressible material on the joint faces. Sometimes the terms packing or joint inserts is used interchangeably with the term load distribution plates, to describe the same product and functionality. Information explaining the difference between load distribution sheets and packing can be found in Babendererde and Hahn (2012).

Load distribution plates are applied at select positions usually on the circumferential face of segments facing away of the working face with two main purposes: First to engage more surface area of the segment during the application of the thrust cylinder action and avoid concentrating the force at the center of the segment. Secondly, these materials under compression prevent the direct contact of concrete faces between adjacent segment rings. These sheets are essentially meant to provide a load distribution effect during the application of thrust from the shield, but also contribute to shear load transfer from ring to ring (Maidl et al., 1996). Per DAUB (2013) and Maidl et al., 1996), when cam and pocket joints are used, plastic or bituminous sheets are used so that the rings can transfer shear loads from one to another. These load distribution sheets compress under applied loading to the specified joint closure limit per each design.

Packing is a more general term describing similar materials (i.e fiberboard, plywood) that can be used locally to compensate for small angles or gaps between segments. Per ÖVBB (2011) packing can be used as a mechanism of joint adjustment to accommodate tunnel alignment corrections. DAUB (2013) does not recommend the use of deformable bituminous materials as packing.

There is debate in the tunneling industry if the use of packing products brings true benefit to the quality of the lining. Based on this literature survey many large diameter projects have been successfully built with or without using either of these materials. Load distribution plates on circumferential joints are more commonplace today and per Babendererde and Hahn (2012), damaged segments can be greatly reduced if load plates are introduced. Per DAUB, if segment production tolerances are kept very low resulting in highly planar segment surfaces, such intermediate materials can be dispensed with. Typical products include bituminous fiber board (felt), polyethylene, hardboard and marine grade plywood.

Figure 5-4: Example of load distribution plates on the circumferential face of segments. Photo: FHWA.

5.7 Tail Void Grout

Backfill grouting is used in segmentally lined tunnels to fill the annular gap between the lining extrados and the excavated ground. Backfill grouting is described in detail in many segmental tunnel lining design publications. The primary functions of the grouting can be summarized as follows (AFTES GT18R1A1):

- Confinement of the segmental lining against the ground, eliminate ring displacement and distortion while TBM advancement
- Reduced risk of ground surface displacement
- Confinement pressure control in pressurized face TBMs
- Long term ground-lining bond, load distribution

Some items that should be considered when specifying backfill grout are as follows:

Rheology:

Grout should be sufficiently fluid to allow placement and injection under pressure into the annular gap. Also, it should be sufficiently firm so that it does not flow through the tail seals, the gaskets, or flow outside the shield towards the front of the TBM. Backfill grouts should be capable of resisting washout by groundwater.

Strength and composition:

Grout type and chemistry are selected based on ground conditions and anticipated construction rates. Stiffness, gel and set time and strength should be compatible with the advance rates of the TBM. The grout should have properties that prevent early on lining ovalization or other lining deflections as the TBM advances with its trailing gear. Generally, the grout should support all loads applied by the TBM and backup equipment, and to support the precast segmental liner within specified tolerances.

Typical 28-day grout strength used in large diameter tunnel projects (ie. SR-99 Alaskan Way Viaduct, Autostrade Sparvo Tunnel, Eurasia Tunnel) is in the range of 2.0 – 3.0 MPa (300 – 440 psi).

Backfill grout mixes can be broadly categorized as follows (AFTES GT18R1A1, Guglielmetti et al., 2007):

1. Inert grouts: Non-cementitious grouts contain bentonite, polymers, sand, filler and plasticizers.
2. Active grouts: Cement based grouts contain, in addition to cement, fly-ash, sand, fillers, polymer, bentonite, lime and plasticizers as well as retarding or accelerator agents
3. 2-component grouts

Grouting through the tail shield of the TBM is the typical method of grouting the annulus in large diameter tunnel projects as it increases the efficiency of the installation and the machines at these diameters can easily accommodate this functionality. In this case the tail shield has evenly spaced outlets around its perimeter, that allow continuous injection of grout at constant pressure during the advancement of the TBM (Figure 5-5). With two component grouts, dual outlets at each perimeter location, feed two components of grout which are mixed together real time at the point of injection. Component A can be a mixture of cement, bentonite, water, and polymer and component B an accelerator. Secondary grouting of the annulus, can take place by grouting through special grouting ports integrated in the segments.

Figure 5-5: Backfill grout dual port outlet embedded in the shield of a large diameter TBM. Photo: FHWA.

5.8 Fire Resistant Cladding

Per AASHTO DCRT-1, NFPA-502 Standard for Road Tunnels, Bridges and other Limited Access Highways is used for planning and design of highway tunnels in the US. NFPA 502 provides a series of fire protection and life-safety provisions that significantly reduce the risks associated with fires in tunnels. There has been extensive research on in-tunnel fire dynamics. Borghetti et al. (2017) present a research testing program performed at the Morgex North Tunnel in Italy and demonstrate analytical and semi-empirical models used in tunnel fire analysis. Extensive work on tunnel fire dynamics, testing, calculation methods and mitigation systems can be found in Ingason et al. (2015.

External measures of structural fire protection are today common and specified by owners and designers in many large diameter tunnels. These systems in the form of specialized fire resistant boarding, are attached to the intrados of the lining or other internal tunnel structure built inside the segmental lining, thus providing structural resistance to the lining in the event of a fire. Structural fire protection works in conjunction with all other fire safety, suppression and monitoring systems. Various international publication for tunnel lining design describe the use of structural fire protection (AASHTO, ITA, DAUB, BSI, BTS, ÖVBB, PIARC). Per AASHTO the ITA Guidelines for Structural Fire Resistance for Road Tunnels (2004) should be followed for designing the structural fire protection. According to DAUB (2013), fire resistance cladding (including fireproof plaster or fire protection paneling) helps in preventing the inner reinforcement layer from reaching temperatures over 300°C. Compliance with this criterion ensures the stability of the tunnel construction during the fire and that major permanent deformations after the fire are avoided. Structural fire protection for highway tunnels is outlined in AASHTO LRFDTUN-1 which lists a series of factors that should be considered when such products are evaluated for use in a tunnel, some of which are:

- Sacrificial layers and applied protection layers occupy space. Additional space comes with added cost to the project
- Applied protection layers usually are installed after the major construction work is finished. This secondary work element adds time to the construction schedule in addition to the cost of the materials and their installation.
- Protection layers can be integrated into a system of finished architectural panels.
- Specially designed materials typically are more expensive than conventional materials.
- Attachments for applied protection layers should be suitable for the service conditions as well as for the fire conditions. Attachments should be coordinated with the structural components and can contribute to tunnel leakage. Leakage behind the layers can add weight to the layer which will be transmitted to the supporting structure, as well as to the layer.
- Protection layers will obscure the structure being protected, making direct observation and inspection difficult, even if the layers are designed to be easily removed.
- Protection layers should be capable of surviving vehicle impacts and tunnel maintenance washing.
- The fire protection afforded by specially designed materials and sacrificial layers is immediate, whereas protection layers are not effective until installed.
- Protection layers function to reflect heat away from the structural elements and back into the tunnel environment. This heat reflection should be accounted for in the design of the tunnel ventilation system.

Per DAUB (2013) the life of panel coverings averages 25 to a maximum of 35 years. Therefore, the covering should be replaced multiple times within the lifetime of a tunnel. A discussion on the application of such materials in the case of a large diameter roadway tunnel in the US can be found in Promat (2016).

5.9 Knowledge Gaps and Research Needs

1. The incorporation of international codes for FRC design (like RILEM 162 TDF or *fib* Model Code 2010) in US codes and standard framework, like ACI, ASTM, and AASHTO has not occurred. Internationally the *fib* Model Code 2010 provides a closed concept for FRC segmental lining design. However, the design is based on a notched three-point beam test per EN 14651 and there is currently no applicable standard in the US for this test. Nonetheless, more and more projects with FRC segmental linings refer to and specify *fib*

Model Code 2010. An existing knowledge gap is the proper implementation of these foreign standards into the US based design publications.

2. Hybrid reinforced segmental lining is receiving more attention today due to its design efficiency. More research is warranted to investigate the behavior and establish an approach for design.
3. Further research should be performed to better understand the contribution and exact mechanisms of polypropylene fibers as integrated fire protection of precast segmental lining. Naaman (2018) quotes research from Dehn and Koenig that leads to doubts about the typical explanation that the fibers melt during high heat thereby providing capillary pathways for water to evaporate. Given the safety related nature of this technology and considering that polypropylene use in segmental lining is becoming a practice in the industry, a careful study of the mechanisms is warranted.
4. A great deal of international research work has been performed on the subject of service life analysis and durability (ie. *fib* Model Code 2010 Service Life, RILEM TC 230-PSC) and in many design-build projects, contractors employ a variety of methods to demonstrate service life compliance with the specs. These methods are adapted from international practice and subject to review by authorities and design review teams on a project by project basis. Additional research on service life analysis of segmental linings is suggested.

6 PERFORMANCE TESTING

More than 40 publications describing testing of large diameter and metro size tunnels (large diameter is considered any tunnel over 13m in diameter and metro size is around 6.5m), using conventional reinforced concrete (RC), steel fiber reinforced concrete (SFRC), and glass fiber reinforced polymer (GFRP) bar segments were identified and reviewed for this literature survey. The tests include single segments, two-segment joint configurations, and full ring assemblies. Only five of these 40 publications explicitly focused on large diameter; however, many of the topics addressed in the broader body of papers are assumed to be transferable to large diameter tunnels.

Testing has been undertaken on RC, SFRC and GFRP concrete segments to better understand flexural behavior including first crack and crack width characteristics, compressive loading by TBM jacks, response during and after exposure to fire-induced heat, blasting, and durability. Full ring structural testing has been performed at four locations worldwide including Tongji University, China, Delft University of Technology, Netherlands, STUVAtec, Germany, and Southwest Jiaotong University, China. Blast loading on full rings has been performed at the State Key Laboratory of Disaster Prevention and Mitigation of Explosion and Impact, and College of Defense Engineering, PLA University of Science and Technology in Nanjing, China.

This section summarizes the key findings from this literature survey and identifies knowledge gaps. It is organized into the following sections: concrete material testing, segment flexural and point load testing, segmental tunnel lining as a system, segmental lining under blast loads, connection hardware, and waterproof gaskets, seismic resistance, knowledge gaps and research needs.

6.1 Concrete Material Testing

6.1.1 Strength

Strength testing for concrete used in segmental lining is a typical process before and during-segment production as described in Sections 6.1.2 and 6.2.3.

6.1.2 Durability

Durability testing of reinforced concrete is described by the Portland Cement Association as the materials ability to resist chemical and weathering attacks while maintaining its engineering properties. There are two standardized tests to assess durability, namely the rapid chloride ion permeability test (ASTM C1202) and the salt ponding test (AASHTO T259). In addition, there are a number of additional tests that have been performed that are not standardized. Limited published research is available on durability of precast concrete tunnel segments, both for traditional rebar reinforcement and alternative reinforcing, such as steel fibers or glass fiber reinforced polymer bars.

Abbas and Nedhi (2016) performed salt ponding testing (AASHTO T259) on cored specimens from SFRC and RC tunnel segments. The SFRC was observed to have enhanced durability due to less chloride penetration compared to RC. The specimens showed no signs of penetration at a depth of 25 mm (1”) (for RC) and a depth of 5 mm (.2”) (for SFRC). Abbas and Nedhi (2016) also performed rapid chloride ion permeability testing (ASTM C1202), finding that cores from exposed surfaces showed lower permeability than the cores taken from the internal surfaces by 17% (RC) and 26% (SFRC). They found that exposed surfaces of SFRC specimens showed 23% lower electrical resistance than RC specimens. Abbas and Nedhi concluded that SFRC samples showed enhanced durability due to the addition of steel fibers. The steel fibers were determined to provide a barrier for the penetration of chlorides inside the concrete compared to rebar reinforced concrete segments.

6.2 Fiber Reinforced Concrete Testing

Four point or three point bending tests on beams are typically used to evaluate the post-cracking behavior or toughness of FRC for pre-cast segmental linings. If cracking occurs a lot of elastic energy is suddenly released and the beam deflects rapidly when going into the plastic phase. Since the post-cracking bearing capacity under pure bending is typically lower than the peak elastic bearing capacity, the tests cannot be run load-controlled, because the post-cracking behavior could not be properly tested. Therefore, FRC beam tests are generally conducted deformation controlled. For the deformation control either the deflection of the beam in the center is used or the Crack Mouth Opening Deflection (CMOD).

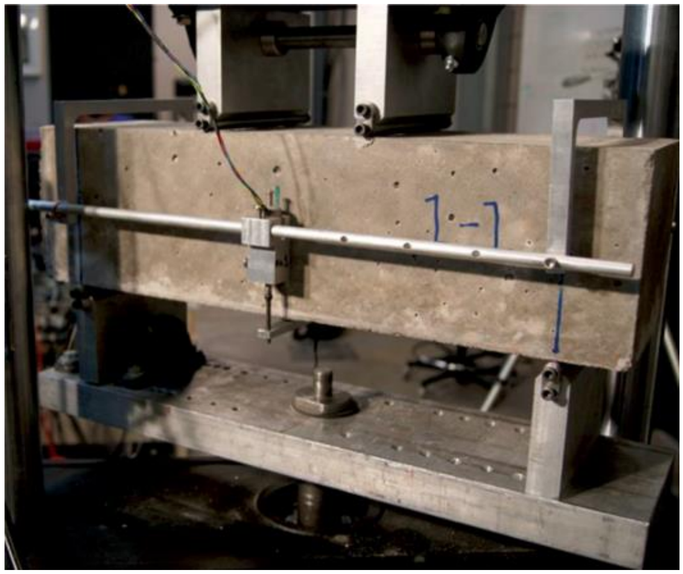


Figure 6-1: Deflection controlled four-point-beam test per ASTM C1399 (ACI 544.8R). Photo courtesy ACI.

The deflection setup is used for the four-point-beam test setup by per ASTM C1399 (see Figure 6-1). The test creates a constant moment distribution over the center area of the beam between the two load introduction points. The first crack and subsequent failure appears within the center area of the beam, where the moment is constant over the center third of the beam, and will appear at the weakest spot. The exact location of first crack cannot be predicted and is random.

The three-point-beam test in ASTM C1609 is also deflection controlled. Differently to the four-point-beam test the maximum moment is in the center of the span. However, the origin of the initial crack can still appear slightly off the center due to inhomogeneity of the material. If the CMOD should be used to control the test, the location of the crack should be known prior to the start of the test.

The CMOD is measured on a three-point bending test per EN 14651. To ensure the location of the first crack in the center of the beam, the 150 mm (6”) deep beam is notched by a 25 mm (1”) deep saw-cut at the bottom. The notch and the loading location of the three-point-beam test ensures that the crack appears in the center of the beam, where the CMOD monitoring gauge is located. The advantage of the test is that the recorded monitoring provides a directly measured load crack-width relationship, rather than a load-deflection curve. However, since the location is not random, the comparable peak-load is statistically to be expected higher compared to a non-notched four- or three-point beam-test.

There are also two major differences how to evaluate the results of a beam test and transfer them into an equivalent flexural strength.

RILEM 162-TDF bases the evaluation method on the energy absorption capacity, which is represented by the area underneath the load-deflection curve. By dividing the non-elastic part of the area at two typical deflections by the deflection itself, effectively and average “non-elastic” load is determined.

Model Code 2010 on the other hand develops characteristic loads at specific CMODs (0.5 mm (.02”); 1.5 mm (.06”); 2.5 mm (.1”); and 3.5 mm (.14”).

The following table provides an overview of currently available international standards for testing, FRC design, and FRC segmental lining design. Those standards and recommendations are neither federal regulations, government guidance statements, nor official policy statements. ACI 544.7 provides an overview of FRC segmental lining design. It does not provide or recommend any specific standard to be used and how it is embedded into existing codes and design publications in the US, like ACI 318 and the AASHTO LRFD design publication. Internationally, *fib* Model Code 2010 provides a closed concept for FRC segmental lining design. However, the design is based on a notched three-point beam test per EN 14651 and there is currently no applicable ASTM or other organization’s standard in the US for this test. Nonetheless, more and more projects with FRC segmental linings refer to and specify *fib* Model Code 2010. An existing knowledge gap is a process for using these documents for US designs.

Table 6-1: Overview of Current FRC Standards and Recommendations related to FRC Segmental Lining Design

Voluntary, Non-Binding Standards and Recommendations		
Evaluation of Post-Cracking FRC Residual Strengths	Design of FRC	Design of FRC for Tunnel Linings
EN-14651	*fib* Model Code 2010	AFTES recommendation*
ASTM C1609/C1609M	RILEM TC 162-TDF	DVB recommendations*
ASTM C1399/C1399M	CNR-DT-204	DAUB recommendations*
ASTM C1550/C1550M	DafStb Guideline	ACI Report 544.7R-16
JCI-SF4		*Refers only to the design of SFRC
DIN 1045-2		

6.3 Segments

Flexural and point load compression testing has been completed on tunnel segments in three locations worldwide, namely in Ontario, Canada (Abbas and Nedhi, 2016; Abbas et al., 2014; and Nehdi et al., 2015), in Rome, Italy (Caratelli et al., 2010; Caratelli et al., 2016; and Meda and Rinaldi, 2015) and in Tokyo, Japan (Dobashi et al., 2007). All testing was performed on segments sized for tunnels between 5.7 and 8.8m in diameter. The data and results are assumed to be applicable to large diameter tunnel segments. The main objective of this research was to compare behavior of SFRC segments or glass fiber reinforcement bar segments to traditional RC tunnel segments.

Overall, flexural testing results demonstrated that crack widths are generally smaller in SFRC than RC segments. SFRC segments exhibited lower ultimate flexural capacity than RC segments. Increased fiber content resulted in greater load levels at first cracking and higher peak load capacities. RC segments exhibited stiffer behavior than the GFRP segments due to the higher steel rebar bond. Flexural test results showed that GFRP bars were suitable as a reinforcing substitute to improve durability in an aggressive environment. Single point loading compression testing showed that both SFRC and GFRP bar segments exceeded the expected maximum jacking forces.

6.3.1 Comparison of Steel Fiber Reinforced and Reinforced Concrete Segments

SFRC and traditional RC tunnel segments tested by Caratelli et al. (2010), Abbas and Nedhi (2016), Abbas et al. (2014), Nedhi et al. (2015), and Dobashi et al. (2007) were subjected to three-point flexure and single point load longitudinal compression testing. Meda and Rinaldi (2015) tested only SFRC segments for three-point flexure and single point load compressive testing. Flexural testing was completed in a reaction frame where the tunnel segment was placed on hinge supports. Displacement transducers monitored the vertical displacement and crack widths. The primary goal of the testing programs discussed here was to investigate ultimate flexural capacity and crack width at max loading.

Caratelli et al. (2010) conducted flexural tests on RC and SFRC segments with a curved length of 3.64m, width of 1.5m and thickness of 0.2m. The RC segments had a compressive strength of 50 MPa and was reinforced with 8 mm (.3”) reinforcing bars at a 200 mm (8”) spacing. The SFRC segments had a fiber content of 40kg/m^3 and a compressive strength of 75MPa. The fibers were 0.35 mm (.01”) in diameter and 30 mm (1”) in length. The overall tunnel diameter was not reported. The RC segment observed a first crack at 70kN, yielded at 125kN and reached ultimate load at 175kN. The average crack width was 0.5 mm (.02”) at the time the segment yielded, which is greater than the maximum accepted crack width of 0.3 mm (.01”) per ACI 244. The SFRC segment observed a first crack at 95kN (35% higher than the RC), yield at 120kN (similar to RC) and reached maximum capacity at 140kN (29% lower than the RC). Observed crack widths were 0.2 mm (.01”) at yield and within the maximum accepted crack width per ACI 244.

Abbas and Nedhi (2016) and Abbas et al. (2014) also conducted flexural tests on both RC and SFRC segments. The skewed end segments had a curved length of 3.18m, width of 1.5m, a thickness of 0.235m, and an overall diameter of 5.7m. The fiber content was equal to 36 kg/m^3. Fibers were cold-drawn hooked-end steel fibers 60 mm (2”) in length, 0.75 mm (.03”) in diameter and with an ultimate tensile strength greater than 1050MPa. The compressive strength of the RC segment concrete was 60MPa and SFRC was 61MPa. The RC segment experienced initial cracking at 45kN, yielded at 210kN and reached ultimate load at 244kN. The crack width at ultimate capacity was equal to 8.2 mm (.3”). The SFRC segment experienced initial crack at 71kN (57% higher than the RC), yielded at 113kN and reached ultimate capacity at 119kN (52% lower than the RC). The SFRC segment crack width at ultimate capacity was equal to 0.25 mm (.01”), less than maximum accepted crack width per ACI 244.

Meda and Rinaldi (2015) tested SRFC segments with the length of 1.67m, width of 1.2m, and thickness of 0.25m. The SFRC segment had 40 kg/m^3 of steel fibers and had an average compressive strength of 61MPa. The fibers had a diameter of 0.75 mm (.03”) and a length equal to 60 mm (2”). The overall tunnel diameter was not reported. The SFRC segment observed a first crack at 170kN with an observed crack width of 0.05 mm (.002”). Yielding was observed at 225kN with a crack width of 0.4 mm (.02”). Maximum capacity was reached at 253kN with a maximum crack width of 0.9 mm (.04”).

Nedhi et al. (2015) conducted testing of SFRC segments with varying fiber contents. The segments were 1m in length, 0.5m in width and 0.1m in depth. Radius of curvature was not specified. Fiber content was varied between 24, 72 and 144 kg/m^3. The fibers had a consistent diameter equal to 0.2 mm (.01") and varying lengths of 8, 12 and 16 mm (.3", .5" and .6"). Fibers had a tensile strength of 2850MPa. The benchmark non-reinforced segment observed first crack at 17.0kN and peak load at 17.55kN. The following table summarizes the SFRC segment first crack and peak load results that indicate the linear load carrying capacity with increasing steel fiber content. Segments with shorter fibers exhibited higher first crack and peak loads. Yielding and crack width was not noted.

Dobashi et al. (2007) tested both RC and steel rebar/fiber hybrid segments. The segments were 3.7m in length, 1.5m in width and 0.4m in depth. Total internal diameter was equal to 8.8m. In the hybrid reinforced segment, the total weight of the conventional reinforcement in addition to the fiber content was 63kg/m^3 with a fiber diameter and length equal to 0.6 mm (.02") and 30 mm (1"), respectively. The RC segment observed first crack at 300kN while yielding occurred at a load of 1200kN and the ultimate loading was reached at 1334kN. Crack widths were not reported. The SFRC segment observed first crack at 450kN (150% higher than the RC), yielding occurred at a load of 1400kN (16% higher than the RC) and the ultimate loading was at 1495kN (12% higher than the RC). Crack widths were not reported.

Single point load compression testing has been performed to compare the tensile splitting forces that would be induced during TBM thrust jack loading. Caratelli et al. (2010) applied 4000kN single point loads to segments. The segments size was the same as in the flexural testing with width of 1.5m and thickness of 0.2m. To simulate the TBM loading a steel plate with the same geometry of the actual TBM shoe was placed into the testing frame and use to apply the single point load force at the middle of the segment. The actual dimensions of the steel plate were not discussed. No significant cracking was observed in RC or SFRC segments (actual crack width was not mentioned). The authors concluded that both RC and SFRC segments were able to tolerate loading greater than the maximum TBM jacking force expected (1130kN). Meda and Rinaldi (2015) simulated TBM jacking loading using single point compression testing. They applied 2500kN to a segment, exceeding the maximum expected TBM jacking force of 1130kN. The segment size was the same as in the flexural testing with width of 1.2m and thickness of 0.25m. To simulate the TBM loading the test adopted the same steel plates used by the TBM machine having dimensions of 0.504m in length and 0.25m in width. The loading was applied at 2 points along the segment. The steel plates were centered on the segment and placed so that the center of the plates were 0.903m apart. Maximum observed crack widths at load levels of 1000, 2000 and 2500kN, applied at each plate, were less than 0.05 mm (.002"), 0.10 mm (.004") and 0.35 mm (.01"), respectively. All cracks observed did not reach the cracking limit value of 0.5 mm (.02") per Model Code 2010.

6.3.2 Comparison of Glass Fiber Reinforced Polymer Bar Reinforced and Reinforced Concrete Segments

Glass Fiber Reinforced Polymer (GFRP) bars have been considered a possible alternative to traditional RC when durability concerns such as aggressive soils are present. Caratelli et al. (2016) subjected GFRP bar reinforced and traditional RC segments to three-point flexure and single point load longitudinal compression testing. The segment dimensions were 4.15m in length, 1.5m in width, and 0.4m in thickness. GFRP bar reinforcing was designed to provide the same ultimate bending resistance as the RC segment. The overall tunnel diameter was not reported. The RC segment first crack occurred at 175kN and the ultimate load was equal to 395kN. At an applied load of 300kN the maximum crack width was equal to 3 mm (.1"). GRFP segment first crack occurred at 130kN (25% lower than RC) and the ultimate load was 640kN (62% higher than the RC). At 195kN loading, the maximum crack width was equal to 1.3 mm (.05").

Point load thrust loading was conducted only on the GRFP bar reinforced segment to simulate installation via TBM. Segment dimensions were 4.15m in length, 1.5m in width and 0.25m in thickness. Loading was applied by twin steel pads and was increased to 1130kN on the first cycle and 2500kN on the second cycle. The maximum TBM jacking force (two jacks) was 1130kN. First crack was observed at 785kN and crack width opening was 0.05 mm (.002") at 1130kN. After the second loading cycle, the crack width was 0.35 mm (.01") at 2500kN. Upon load release, the residual crack opening was 0.05 mm (.002"). The authors concluded that the GFRP reinforced concrete showed significant strength and durability and did not reach the cracking limit value of 0.5 mm (.02") per GRFP codes (JSCE, 1997; CSA 2002; ACI 440R-06, 2006; CNR-DT203, 2007).

6.4 Segmental Tunnel Lining Systems

Various testing programs have been carried out to examine the behavior of full scale segmental tunnel lining systems. These tests can be divided into two main categories, namely two-segment testing to evaluate individual joint behavior and complete segmental ring testing (either single ring or multiple ring systems) to evaluate the overall ring system behavior. Only four full-scale, full ring testing facilities worldwide and one in-situ test section were found in this literature survey. There are about 20 available publications on full scale complete ring testing, five of which focused on large diameter tunnels. In most of the full segment ring testing programs, the objectives were to validate the load capacity of the ring system under the design load, and to investigate joint rotation and the influence of joints on ring deformation (Schreyer and Winselmann, 2000; Blom, 2002; and Lu et al., 2006). However, some key test results such as lining forces (e.g. axial forces and bending moments) are not reported. To study the behavior of individual joints in a more efficient manner, two-segment systems have also been studied. In most cases, more comprehensive results were published compared to the full ring tests. Based on these studies (both two segment joint systems and complete ring testing) existing behavioral models and new joint behavioral models have been developed and validated.

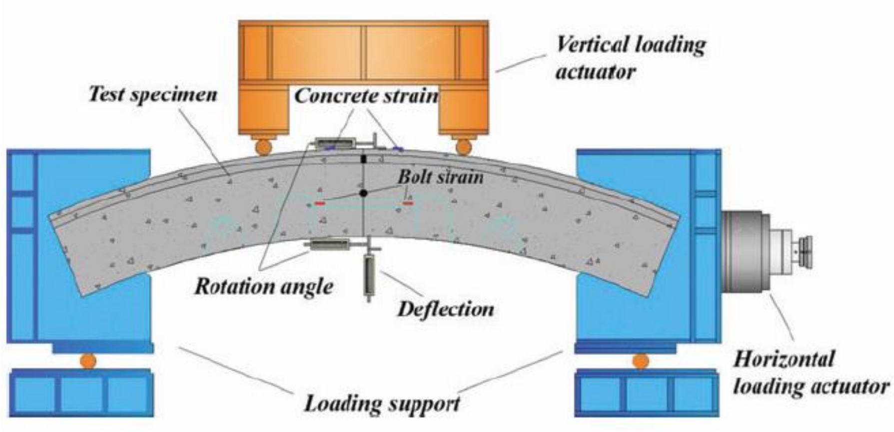
6.4.1 Individual Joint Behavior

The rotational stiffness of joints has been widely investigated experimentally. Several rotational stiffness models have been validated through large scale testing. However, the influence of connection bolts on the rotational stiffness is contradicted across several publications. The contradictory results is believed to be due to differences between the bolt strength, position, and joint asymmetrical geometry. The bearing capacity of RC and SFRC segmental joints has been studied experimentally. SFRC segment joints exhibited a higher ultimate load, while RC segment joints showed a more ductile failure.

Hordijk and Gijsbers (1996) published (in German) one of the first experimental research programs on segmental joint behavior, using two segment joint configurations. In this study, two segments were first subjected to a thrust force, and then loaded by an increased bending moment. The test results showed increasing moment capacity with thrust force, and were well matched by theoretical models. Observed moment-rotation response was linear until a certain rotation limit (thrust-dependent), which was followed by a non-linear response. Using these test results, Luttikholt (2007) found the Janssen (1983) model to represent the moment-rotation behavior of the joint most accurately. Hordijk and Gijsbers (1996) concluded that the initial rotational stiffness was only slightly affected by the presence of bolts (the M24 bolt was used in this test). However, the ultimate bending moment of joints with a bolt was higher than joints without a bolt. This increase in ultimate bending moment was in the order of 20kN-m/m for very low axial forces and was negligible in higher axial forces.

During the following decade, most joint rotation research studies concentrated on complete ring testing. Blom (2002) performed full-scale, complete ring tests on a 9m diameter tunnel to validate a newly developed analytical model to calculate thrust forces and bending moments. The complete ring test results showed good agreement with the analytical solution developed using non-linear joint rotational stiffness based on the theoretical model of Janssen (1983). With an overall good understanding of the non-linear rotational stiffness behavior of tunnel segmental lining under ideal conditions (as in Hordijk and Gijsbers, 1996; Blom, 2002; Lu et al., 2006), experimental programs have investigated joint behavior to complete failure of the joint, due to construction errors, with atypical bolt connections and under more varying loading conditions (Luttikholt, 2007; Li et al., 2015; Jin et al., 2017; Caratellia et al., 2017; Gong et al., 2017, and more).

Li et al. (2015) investigated the development of joint rotation up to complete failure of the joint, under both a positive and negative moment by full scale tests of a two-segment system (Figure 6-2). The two segments were connected by a straight bolt just below the neutral axis (towards intrados). Based on this test, a model for predicting joint rotation was developed, incorporating the influence of axial (thrust) stress, bolt positioning, and bolt pre-tension. While the authors do not discuss the difference in the experimental results between positive and negative joint rotational behavior, the results show positive rotational stiffness (squatting action; where the bolt is in tension) to be greater than negative rotational stiffness (egging; where bolt is in compression) by up to 70%. This is contrary to what Hordijk and Gijsbers (1996) found where the joint does not have significant influence. This is likely due to the configuration of the bolt in the tests performed by Hordijk and Gijsbers (1996) where the bolt runs through the neutral axis to the segment intrados.



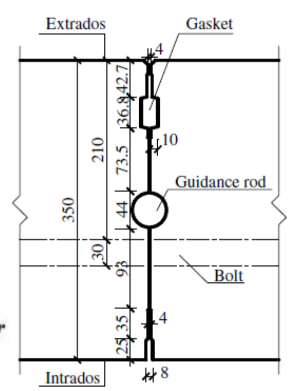


Figure 6-2: Left: the testing rig scheme used by Liu et. al. (2015), from (Liu et. al., 2018), Right: joint geometry with bolt position (Li et. al., 2015). Images courtesy Lui et. al. and Li et. al.

Jin et al. (2017) conducted full scale tests on a two-segment joints to study the effect of two bolt arrangements used in water conveyance tunnels on the joint rotational stiffness under varying internal loading conditions (see Figure 6-3). Under positive bending moment (opening of the edge connected by bolts), the specimen with more bolts exhibited greater joint stiffness than the specimen with less bolts. Under negative moment (compression at the bolted edge), both types of joints exhibited similar behavior.

Gong et al. (2017) investigated ultimate joint bearing capacity (compression) using a full-scale two-segment setup of both conventional RC segment joints and SFRC joints under different moment-thrust combinations. The ultimate bearing capacity was taken as the maximum bending moment achieved for a constant thrust force. Gong et al. (2017) found that the moment capacity of the SFRC joints at initial cracking and ultimate state were higher than that of the RC joints (Figure 6-4). They evaluated both the deflection ductility and rotational ductility for both the RC and the SFRC which is the ratio of the ultimate deflection or rotation over the cracking deflection or rotation. They found that the rotational ductility of RC and SFRC joints was almost identical, however, the RC joint deflection ductility was 12-40% greater than the SFRC joint deflection.

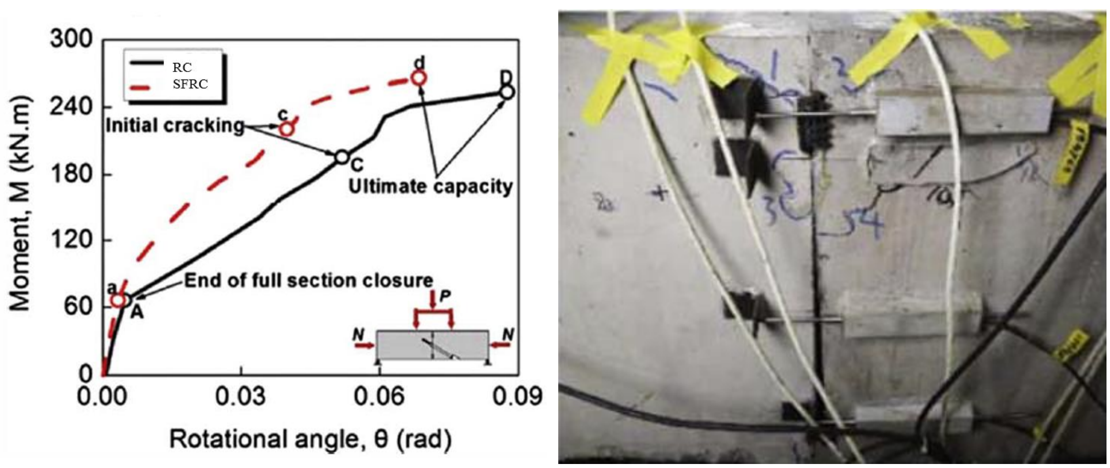


Figure 6-3: On the left a comparison of the moment-rotation of the joint between the RC specimen in black and the SFRC in red. On the right the RC segment at first crack (Gong et. al., 2017). Figure and photo courtesy of Gong et. al.

Caratelli et al. (2017) conducted full-scale tests of a SFRC two-segment system to study the bearing capacity of opened joints due to construction errors. In a test setup similar to Hordijk and Gijsbers (1996) as shown in Figure 6-4, an opening of one side of the joint was first induced by rotation, then axially loaded. An analytical interpretation of the joint behavior and bearing capacity was proposed, accounting for the height of the zone of compressed concrete. Salemi et al. (2014) performed a series of direct shear tests to study the mechanical behavior of longitudinal joints. They developed a relationship between the normal stress at the joint (a function of the thrust force at the joint and the joint cross sectional area) and the shear and normal stiffness of the joint, commonly used in numerical models to model the joint behavior. Tirpitz and Hestermann (1999) conducted shear tests on joints with tongue and groove configuration typically used in circumferential joints (without bolts or dowels) to optimize the concrete reinforcement layout near the segment joint. For the 0.7m thick, 1m wide segments and an average concrete compressive strength of 55MPa they found that by inserting six vertical 12 mm (.5”) diameter rods the ultimate shear load could be increased by 200%.

The contribution of full-scale testing of complete segmental rings to the overall understanding has included validation of joint rotational stiffness models in analytical solutions and numerical models. Varied findings have been published on redistribution of loads between adjacent rings, most likely due to differences in longitudinal forces between testing efforts. Full scale ring testing also revealed that segment placement offsets larger than 2 mm (.1”) at circumferential joints can result in excessive cracking.

One of the first full-scale tests of a complete segment ring was performed for the 13.75m diameter 4th Tube Elbe highway tunnel in Germany, at the time the world largest TBM-driven tunnel. Schreyer and Winselmann (2000) published the results of the five full-scale tests conducted at STUVAtec in Cologne, Germany. The test set up included a horizontally positioned full width ring at the center with one-half width rings on each side with a stagger joint configuration. The lining was conventional RC with a thickness of 0.7m, comprised of eight segments and one key segment, plane longitudinal joints without bolts, and tongue and groove circumferential joints. Two of the five tests simulated the anticipated loading conditions. In the three subsequent tests, the soil bedding stiffness was reduced by 80% (by force-adjustment of the jacks) and different loading or deformations were induced.

Schreyer and Winselmann (2000) observed in all five tests that ring deformation was uniform over the entire perimeter of the ring, despite large rotation angles measured at the joints (especially at the key segment joints that was 60% higher than the max rotation at the other joints). Despite not being discussed in the paper, the observation of uniform deformation of the jointed ring can give some support to the simplification of the jointed ring using a continuous lining ring as described by Muir Wood (1973). However, the magnitude of stiffness reduction should be studied.

Blom (2002) and Vervuurt et al. (2003) published results from full scale tests of 9 m diameter complete segment rings conducted at Delft University of Technology as part of the Botlek Railway Tunnel project. The assembly included three horizontally oriented adjacent rings. Each ring was composed of seven segments including one key segment. The conventional RC had a thickness of 0.4m. In every loading case the rings were loaded in the radial and axial (longitudinal) direction. Three loading cases were used, namely simultaneous ring loading, sequential loading and simulation of segment placing offsets. In the simultaneous ring loading test, all three rings were equally subjected to a uniform and ovalization load. In the sequential loading tests the bottom two rings were loaded first, followed by loading of the top ring to a greater loading level. From the sequential loading test, Blom (2002) found that after the additional loading of the top ring, 40% of the additional acting loads were taken by the top ring and about 60% were redistributed to adjoining rings. The directly adjoined ring dissipates 40% of the acting loading, while the next adjoining ring dissipates 20%. Vervuurt et al. (2003), published the results of the third load case where longitudinal gaps were introduced at six different locations between the top and the middle rings. Two tests were conducted, one with gaps of 2 mm (.1”), and the second test with gaps of 4 mm (.2”). These tests were compared with a benchmark test without segment placing offset. The results showed that cracking occurred from both the 2 mm (.1”) gaps and the 4 mm (.2”) gaps. The cracks observed in the 2 mm (.1”) gap test had a very small crack width and were not continuous (crack width not reported). In the test with 4 mm (.2”) gaps, the cracks were reported as wider and continuous (crack width not reported). The authors concluded that assembly placing offset tolerances should not allow for a gap larger then 2 mm (.1”)

Significant complete ring testing research has been conducted in China in the last decade, namely at Tongji University and Southwest Jiaotong University. Liu et al. (2015) conducted full-scale single ring testing of a standard metro size (6.2m diameter) at Tongji University to investigate ultimate capacity of jointed segmental tunnel linings. The RC ring had an average compressive strength of 55MPa, with an inner diameter of 5.5m, a thickness of 0.35m and segment width of 1.2m. The ring was composed of five segments and one key segment that were connected by two bolts at each longitudinal joint. Testing was performed using a horizontal configuration. The first of two load cases explored was a gradual increase in geostatic loading with lateral earth pressure ratio different than unity up to failure. The second was lateral earth pressure unloading to simulate a nearby excavation. The main conclusions the authors noted in this study was that the failure initiated at joints and the segmental ring structure ultimate capacity is dependent on the joint bearing capacity.

Liu et al. (2017) continued this study with a staggered joint ring system composed of one full ring and two half width rings. The circumferential joints were a tongue and groove, and included 16 bolts uniformly spaced around the circumferential joint. To simulate the longitudinal force, a vertical load equal to 15% of the maximum TBM jacking force was applied. All three rings were equally loaded with the same loading cases as in Liu et al. (2015). Comparing the results from this test to the single ring test of Liu et al. (2015), they found that the staggered assembly (the full ring and two half rings tested in Liu et al. (2017) resulted in a significantly higher ultimate capacity (84% higher than the design load) while the continuous joint assembly (the single ring testing from Liu et al. (2015), assumed to be representative of rings assembled with continuous joint) ultimate capacity was only 17% higher than the design load. The failure mechanism of the staggered ring system was observed to originate from the circumferential joints followed by failure at of longitudinal joints and the segment body at the same time. Under design load, the middle ring bending moments were consistent with that of a continuous ring (e.g. without joints) back-calculated to a certain stiffness reduction value. The magnitude of stiffness reduction was not discussed.

Chuan et al. (2011) reported the results of full scale complete ring testing for tunnels under high water loading carried out at Southwest Jiaotong University in China. A 14.5 m OD highway tunnel and 10.8 m OD railway tunnel was tested. Chuan et al. (2011) reported that the high-water pressure (resulting in pure hoop forces) played a significant role in reducing the rate of increase in deformation after the initiation of cracking.

Zhu et al. (2018), Huang et al. (2018) and Zhang et al. (2019) reported the results from a series of full scale tests on sub-rectangular segmental tunnel rings (flat crown and sidewall with a radius in the corners) to evaluate the design of a tunnel constructed in Ningbo, China. They report that the effective rigidity ratio (like what is calculated using Muir Wood equation in section 4.1) increases as the axial load at the joint is increased, and is not a constant value depending only on the geometry of the section. However, the Muir Wood equation is not discussed in this paper.

Molins and Arnau (2011a, b) conducted a unique in-situ test on (the only in-situ test conducted) a 15-ring test section constructed in the L9 metro project, Barcelona. The SFRC ring consisted of 7 segment and a key segment with an outer diameter of 11.6m, and a thickness of 0.35m. The SFRC average strength was 50MPa with steel fiber content of 60Kg/m^3. Three hydraulic flat jacks were placed between the extrados of the crown of one ring to load the tunnel lining (Figure 6-6). Molins and Arnau (2011) did not find any significant load redistribution between the loaded ring to the adjacent rings.

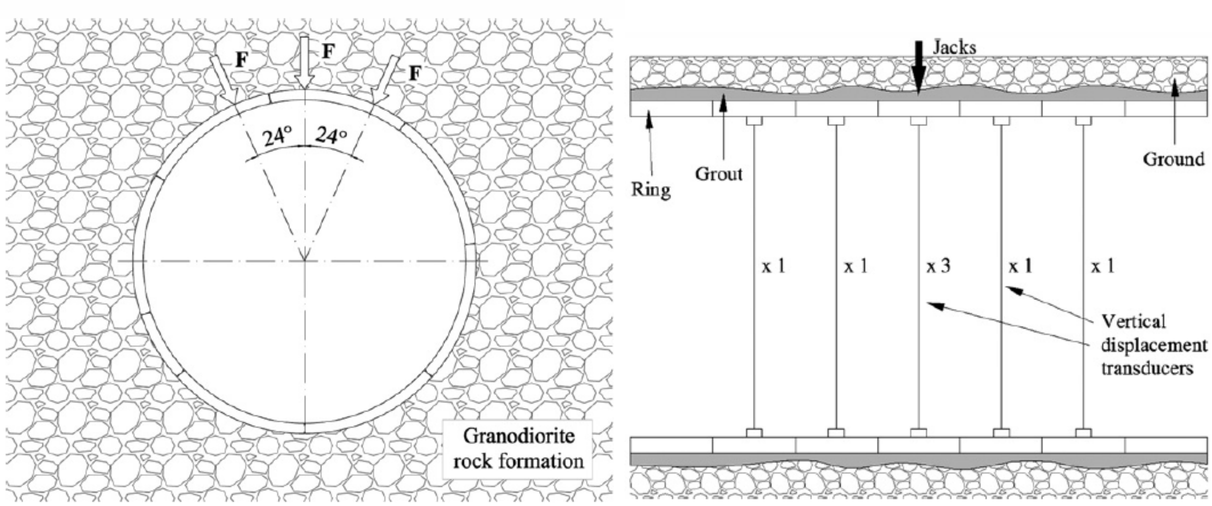


Figure 6-4: On the left is the conceptual test configuration cross section with hydraulic jack forces (F), and on the right a longitudinal section including the vertical displacement transducers (Arnau and Molins, 2011).
Figures courtesy of Arnau and Molins.

6.5 Segmental Lining Under Fire Exposure and Blast

The behavior of pre-cast segmental tunnel lining during and after fire exposure has great significance to tunnel safety. While fire damage to tunnel linings has been widely studied with various publications addressing this issue, only a few full-scale tests have been performed on the influence of fire exposure to precast segmental lining performance. Experiments show that the flexural (bending moment) capacity of SRFC is significantly reduced, much more so than conventional RC. Based on only one study, the bending capacity of RC is decreased by about 20% during fire-induced heating and is partially recovered after cool down. The bending capacity of SFRC segments, however, is reduced as much as 60% during heating, and also observed to have an additional reduction in bending capacity after cool down. The use of polypropylene (PP) fibers was found to almost eliminate the spalling phenomena.

Yan et al. (2012) conducted a full-scale test on RC tunnel segments subjected to a standard ISO834 heat curve with 45 min (max temperature 900˚C) and 90 min (max temperature 1,005˚C) heating durations to investigate fire damage to RC lining segments and rotational stiffness of joints (Figure 6-5). The tested specimens were loaded with a thrust-moment combination held constant throughout the heating and cooling process. After cooling, each specimen was then loaded to failure. The temperature of the intrados reinforcement (having a concrete cover of 60 mm (2")) exceeded the ISO specified failure temperature when exposed to the 90 min heat curve but did not exceed the failure temperature throughout the 45 min heat curve. The temperature of the waterproof gasket located at the extrados of the joint exceeded the product limit temperature in one out of the six tests yet remained intact. The measured maximum explosive spalling depth was between 26 to 51 mm (1" to 2") with an area within a range of 13.1–55.7%, of the segment surface area.

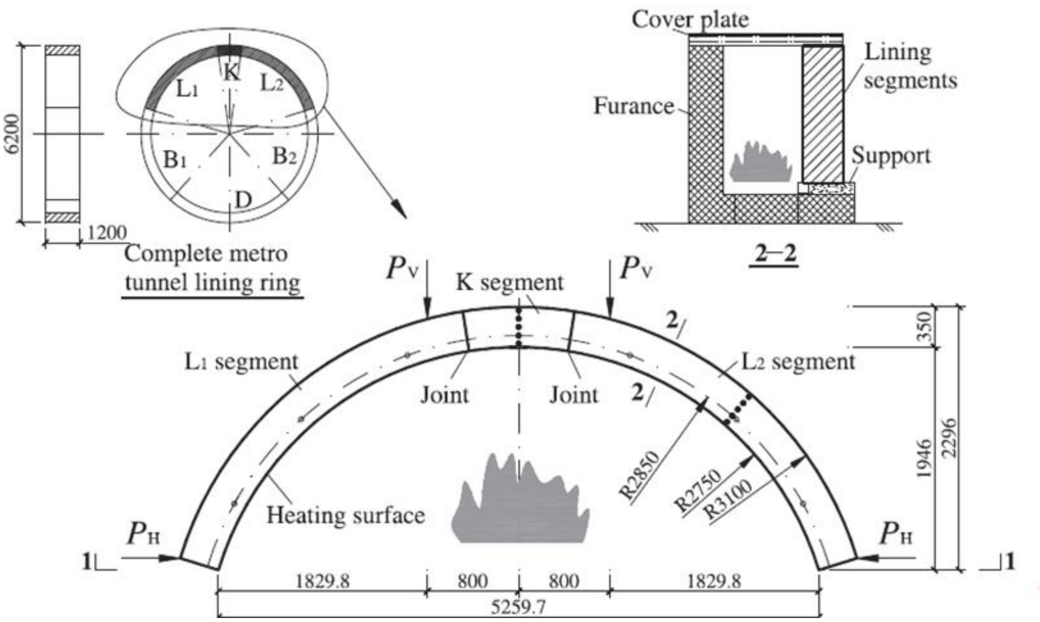


Figure 6-5: Fire exposure test of segmental liner. The thrust-moment loading was applied by a combination of horizontal load (PH) and a vertical load (Pv). Units in mm. (Yan et al., 2012). Figures courtesy of Yan et. al.

Yan et al. (2013) tested RC and SFRC lining segments (single segments and closed rings) subjected to a hydrocarbon heat curve with a maximum temperature of 1100°C. Two loading cases were investigated in this study. In the first case the specimen was heated without loading. After a heating duration of 60 min, the segments were loaded to failure under continued heating. In the second case, the segment was heated for 60 min under constant thrust-bending moment. After cooling, the specimen was loaded to failure. Yan et al.(2013) found that in the RC, the ultimate load was up to 30% greater in segments that were cooled before loading to failure, compared to segments loaded to failure under heating. SFRC segments experienced the opposite behavior. The ultimate load of SFRC segments was 33% lower after cool down than during heating. Unfortunately, no specimens were tested under ambient temperatures to quantify strength loss due to heating and heating-cooling.

Similar testing was performed by Yan et al. (2015) on two segment systems of RC and hybrid fiber reinforced concrete (HFRC) segments that included a mixture of steel and polypropylene (PP) fibers. The test program included six RC specimens and seven HFRC specimens, where one of each was tested in ambient temperatures to provide benchmark results. Of the remaining eleven specimens exposed to fire, six were loaded to failure under fire, and five were loaded post fire. The flexural capacity of HFRC segments during heating was 60% less than the benchmark HFRC results, while the flexural strength of RC segments during heating was 20% less than the benchmark RC results. This reduction in flexural capacity resulted in HFRC failure at the segment cross section by tension stresses at the inner fiber, while the RC specimen failed in compression at the joint. The significant reduction in bending moment capacity is attributed by the authors to degradation and destruction of the bond between the steel fibers and the concrete matrix when exposed to high temperature. Spalling of RC segments was observed while no visible spalling was observed in HFRC segments. This was attributed to the presence of the PP fibers. Tajima et al. (2006) tested RC segments with and without PP fibers exposed to fire and found that no spalling occurred in the segments with PP fibers while the segments without PP fiber experienced severe spalling.

Limited experimental research has been performed regarding the effects of blasting on concrete segmental tunnel linings, with only two studies completed: one in China (Zhao et al., 2016) and one in Italy (Colombo and Martinelli, 2014). The research completed was for metro size tunnel (±6.5m), however, the work described in this section is assumed to be applicable to large diameter applications also. Zhao et al. (2016) conducted a full-scale test on two parallel metro size tunnels vertically constructed. Four rings of the 6.3m diameter tunnel lining were assembled vertically. The longitudinal joints were bolted; however, no details about the bolt strength was reported. The lining thickness was 0.4m. Two charge location were investigated, namely at the center of the tunnel (centric) and eccentric (charge at the bottom corner of metro carriage) positions. Zhao et al. (2016) found that for both centric and eccentric charge positions, the damage occurred only at the joint areas; no deformation or damage was observed within the segments. The damage was observed to be due to stress concentrations around the bolts sockets. This was attributed to outward movement of the segments under internal explosive loading held together by the joint bolts.

Colombo and Martinelli (2016) tested the effectiveness of thin high-performance fiber reinforced cementitious composite (HPFRCC) plate in three applications (Figure 6-6). The first specimen consisted of a circular slab made entirely of HPFRCC. The second specimen included a layer of SFRC of 0.14m in thickness attached to the HPFRCC. The third specimen is the same as the second, with the addition of a second layer of HPFRCC. SFRC material properties consisted of compressive strength equal to 70MPa.

Steel fibers were low-carbon hooked-end fibers 30 mm (1”) in length with as aspect ratio equal to 45. The fiber content was equal to 50 kg/m^3 (0.64% by volume). Blast loading was applied by use of shock tubes. Low pressure blast load tests consisted of an average peak pressure of 0.36 MPa and an average specific impulse of 3.32 MPa ms. High pressure blast load tests consisted of an average peak pressure of 1.07 MPa and an average specific impulse of 6.09 MPa. Testing showed that HPFRCC thin panels (20 mm (1”)) shown in Figure 6-6 with an anchor span of 40 cm (16”) could break for blast pressures as low as 0.3 MPa. Testing on layered structures showed the reduction of the acceleration transmitted to the soil by 50% for the low pressure testing. High pressure testing of 1MPa showed a reduction of 60%.

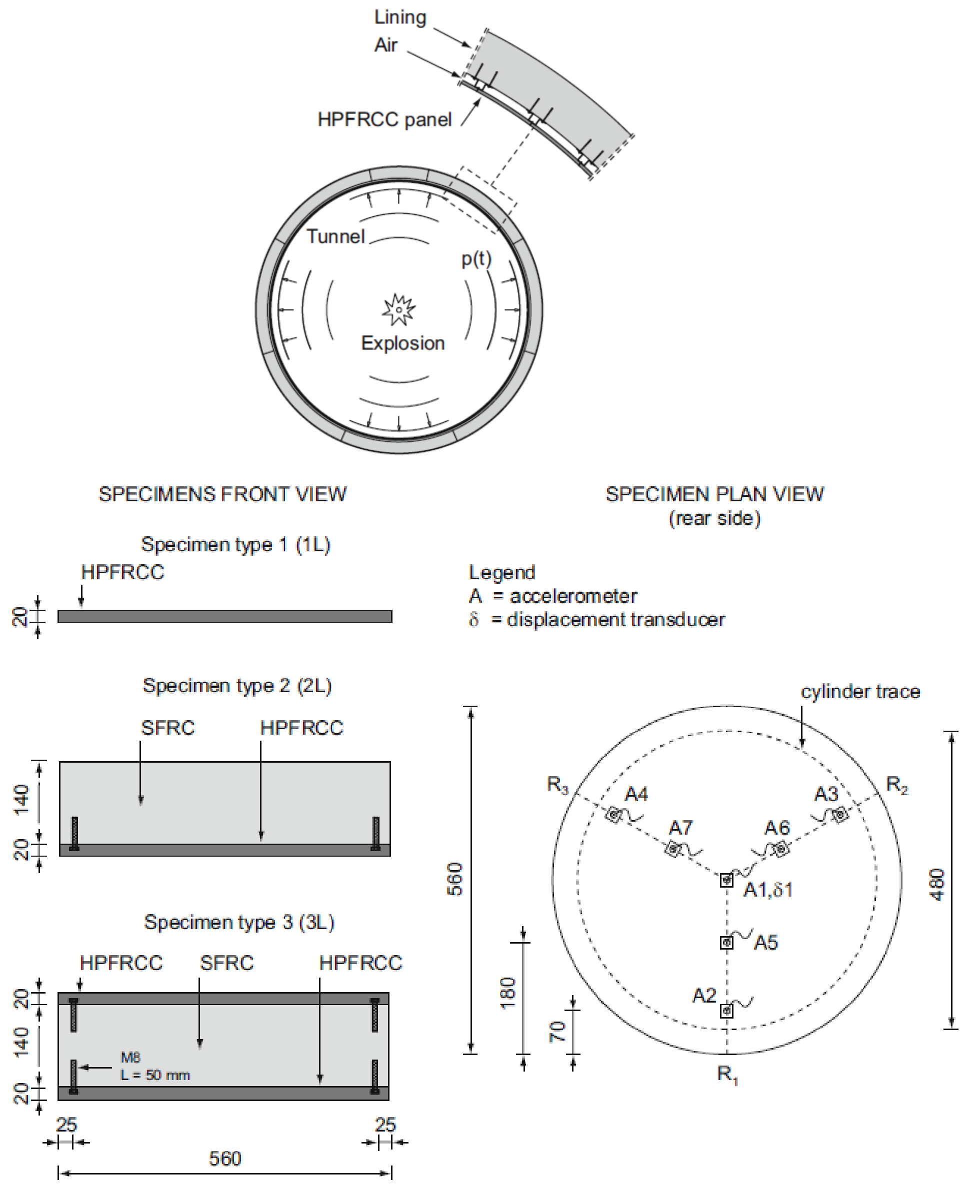


Figure 6-6:Top: Schematic layout for HPFRCC panel applied to the intrados of the tunnel. Bottom: Schematic layout for SFRC with HPFRCC panels applied to the intrados of the tunnel (Colombo and Martinelli, 2016). Figures courtesy of Colombo and Martinelli.

6.6 Connection Hardware

Connection hardware such as shear dowels (plastic or steel core), guiding rods, and joint connection bolts vary in strength and properties from manufacturer to manufacturer. Gehwolf et al. (2016) is one of the few published papers investigating the system behavior of segmental lining and shear dowels (Figure 6-7:. The capacity of the shear dowels themselves is sufficient in most cases, however, in combination with thin precast segments (commonly 20-40 cm (8" – 16") in thickness) the ultimate load capacity of the overall system (segment-shear dowel) poses a challenge due to failure of the concrete in shear. Four different reinforcement layouts were tested

(Figure 6-7). Gehwolf et al. (2016) found that the helical layout and longitudinal oriented stirrups and V-shaped stirrup layout displayed the highest capacity.

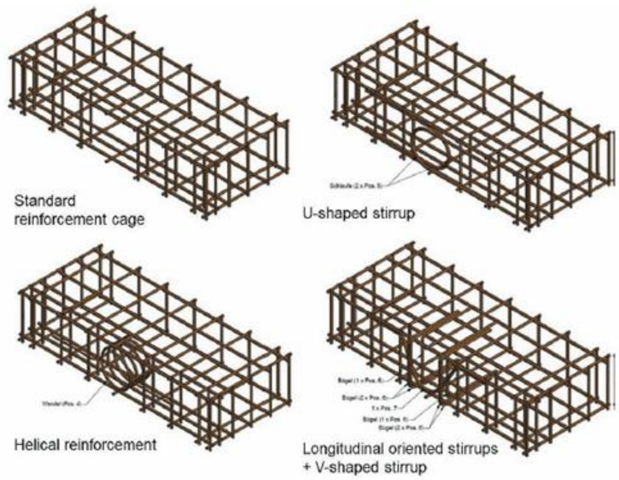


Figure 6-7: Four different reinforcement layout tested (Gehwolf et al., 2016). Figure courtesy of Gehwolf et. al.

6.7 Gaskets

Details with respect to watertightness testing of sealing gaskets can be found in the technical publications by STUVA, AFTES, and other segmental tunnel lining publications such as DAUB, Ril 853 and others. Testing equipment is configured typically to test gasket profiles on a straight joint or a "T" model joint.

Each gasket profile is generally characterized by a series of reference performance testing charts provided by the manufacturer, that can be used to aid in the proper selection. Charts, include the watertightness (pressure) versus gap length, the load-deflection diagram, short and long term relaxation diagrams.

Additionally, project specifications may call for analysis or experimental results demonstrating that the gasket will be watertight and will not exert a load that will damage the concrete gasket groove of concrete tunnel lining under any manufacturing and installation tolerances.

Experimental work performed on the sealant behavior of segmental lining gaskets was published by Shalabi (2001). Using these results, Shalabi et al. (2007) documented the behavior of gasket and groove mechanical and sealant behavior using a steel frame device. Shalabi et al. (2007) suggested a conceptual model for leakage behavior of a gasket in a groove as water pressure is applied. Shalabi et al. (2012) investigated the leakage behavior of a gasketed segmental tunnel lining subjected to static ground loads and seismic shaking using full scale testing. The testing program was developed to test the sealant behavior at the longitudinal joints, circumferential joints and the T-joint connection between longitudinal and circumferential joints. Shalabi et al. (2007) found that the gasket sealant capacity improved after a short cyclic test (12 cycles at ±0.05 in. amplitude), while after severe cycling (100 cycles and ±0.1 in. amplitude), gasket sealant capacity was reduced.

Ding et al. (2017) presented a newly developed testing apparatus to investigate the coupled leakage and mechanical behaviors (gasket load-deformation behavior) of gasketed segmental joints. Based on this testing apparatus, Gong et al. (2018) proposed a simplified design formula that quantifies the relationship between the contact stress of the gasket and the water pressure based on a series of tests on seven different gasket profiles, followed by computational modeling.

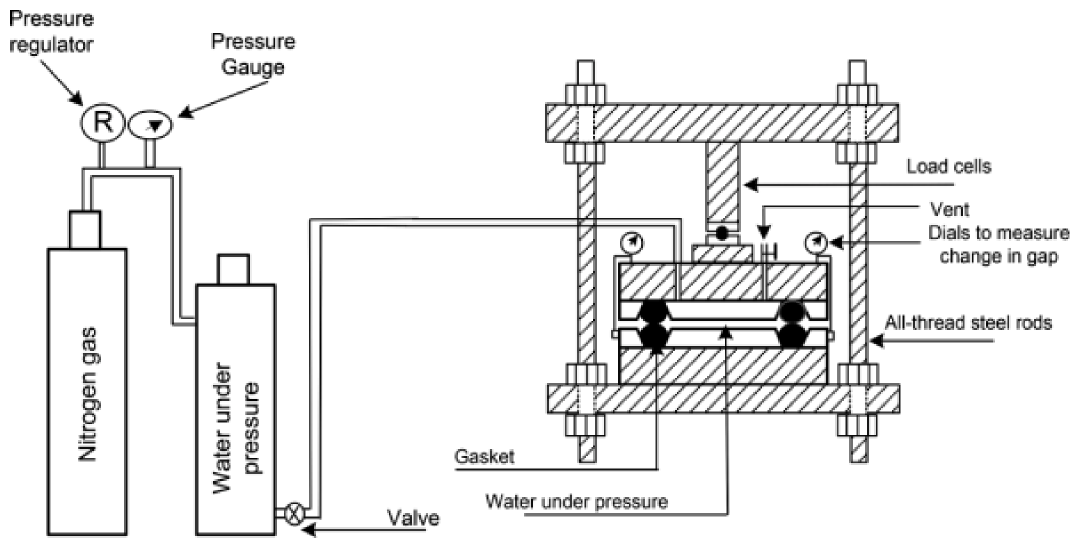


Figure 6-8: The steel picture frame device used for the water leakage tests (Shalabi et al., 2016). Figure courtesy of Shalabi et. al.

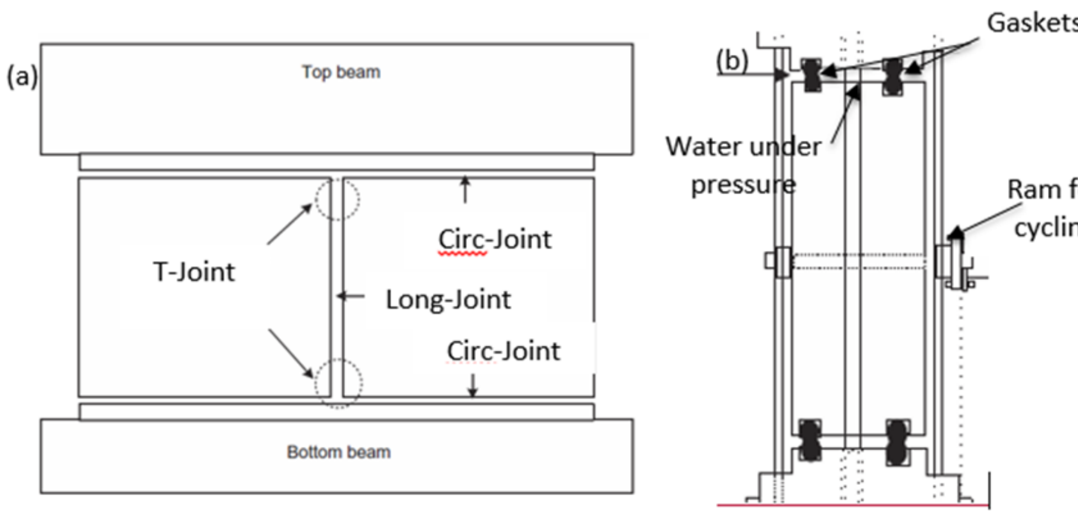


Figure 6-9: On the left the front view of the full-scale testing device, on the right the cross section of the testing device. (Shalabi et al., 2012). Figure Courtesy of Shalabi et. al.

6.8 Seismic Resistance

No experimental testing conducted on seismic resistance of tunnel segments and lining systems was discovered during this literature survey.

6.9 Knowledge Gaps and Research Needs

The main knowledge gaps identified for large diameter tunnels include:

- Influence of the following factors on joint stiffness and bearing capacity:
    - Joint geometry
    - Bolt configuration and bolt strength
    - Segment radius
- Influence of tunnel radius and joint geometry on joint shear capacity
- Influence of the number of joints on large diameter ring behavior, e.g.., flexural stiffness
- Investigation of simplification assumption on segmented tunnel lining flexural rigidity (continuous ring assumptions as Muir Wood Eq) depending on variable thrust forces, bending moments, number of joints, etc.
- Redistribution of loads between adjacent rings as a function of connection details (tongue and groove, bolts, etc.), longitudinal forces (full jacking forces or residual forces), packing material etc. and simplified calculation models.
- Behavior of segmented tunnel ring system without bolts
- Optimization of SFRC design. Experimental testing is commonly completed by testing only one fiber dosage.
- Durability/service life of tunnels- the influence of environmental aspects on life cycle based methodologies using safety factor adjustments and crack width (Spyridis, 2014)
- Seismic resistance
    - Segmental joint behavior under seismic loading
    - Influence of joint number under seismic loading
    - RC vs SFRC capacity under seismic loading
- SFRC segmental lining behavior under fire exposure using different steel fibers (length diameter, dosage, etc.) and additional syntactic fibers.
- Investigation of anchored gaskets leakage behavior
- Blast resistance of large diameter tunnels without connection bolts.
- Joint bearing capacity with different SFRC design and deferent RC design
- Repeatability. Typically, only 1 test was completed for each the traditional and alternative reinforcing methods. Multiple tests would ensure consistence of results.

7 CONTRACTOR FEEDBACK

As part of this literature survey , the research team collaborated with a major tunnel construction contractor in the US, discussed problems and concerns contractors have when dealing with large diameter tunneling and sought their feedback for the purposes of this synthesis report. The following section summarizes the points raised which have been categorized in two sections – Materials and Testing and Construction Aspects. The information presented in this chapter represents input from a single construction company and is not a synthesis of industry practice.

7.1 Comments

7.1.1 Materials and Testing

Full Scale Fiber Reinforced Concrete Testing

Some attention should be called to ACI 544R (or its current variant), which calls for full scale testing of any design using FRC prior to its use in permanent construction. Early Steel Fiber Reinforced Concrete (SFRC) projects implemented these full-scale tests. However, since these tests were done for projects with different designers and different contractors, the results were not published and/or publicly available for a systematic study. As diameters of tunnels get larger the lining thicknesses can get larger as well. In fact, some projects currently do not specify a full-scale testing program. This may be a result of comfort in the civil engineering community in the use of fibers, a change in ACI recommendations, or both.

As the technology stretches into larger diameters, the industry should consider a standardized set of full scale tests to 1) verify the behavior of steel fiber reinforcement in larger diameters and thicknesses, 2) to universally satisfy the full-scale testing provisions, and 3) performed in a manner that is open to industry review and reference.

A standardized set of near typical segment geometries and concrete strengths could be selected and tested to create a matrix of typical acceptable results. These standardized tests could be used to support designs that range outside the typical. Performing these tests as a group and publishing the results would reduce on-the-job testing budgets over many projects, as well as reduce schedule criticality of this testing in a project environment, and provide more confidence for assumptions during the bidding phase.

Connection Hardware Testing

Few projects perform independent Connection Hardware Testing, herein after referred to also as bolts and dowels. Bolts are typically utilized within a ring, from segment to segment. Dowels are typically used from ring to ring. Since there are several different manufacturers of bolts and dowels, testing equipment between manufactures can vary. The manufacturer typically performs these tests and are reported in project documentation. These reported values, however, can vary from the actual strength due to installation tolerances, jobsite contamination, variations in manufacturing, etc., all of which contribute to deviations. Dowels are typically sized (among other considerations) to maintain a compressed gasket until the annular grout can lock the rings into the final permanent position. The remaining margin of dowel compression strength can be questionable. Additional loads on dowels can also develop. For example, external hydrostatic pressure acting on the mating surface between the gasket and the extrados, effectively applying an additional line load compounding the loads from the compressed gasket. Although this load can be easily calculated and added to the expected gasket loads, the variability in dowel performance merits a provision for independent testing on each project for verification. Verification that arguably can only be done with the environmental, physical and chemical properties of the concrete to be used on any specific project.

Beam Testing of Fiber Reinforced Concrete

Past experience has shown that beams for testing flexural strength (both mix design purposes and production testing acceptance) can be representative of the actual cast concrete by the methods used to place the SFRC into the beam mold. Segments are typically cast inverted into forms with a narrow opening at the quadrant of the segment. Concrete is placed at this opening and the mix flows down towards the ends of the molds. As the wet concrete flows, aggregates pull and tease the fibers to generally align to this direction of travel. This anisotropy is a desirable effect as it generally orients the fibers to the principal direction of expected bending moments.

Tail Void Grout Properties

Large diameter TBMs can be constructed relatively short in length relative to the diameter. A TBM consists of two major components, the shield and the back-up gantries. As the tunnel gets "larger" in diameter, the space available on the gantry increases (through both area and number of levels to each deck), and consequently more equipment gets installed on a given gantry. The result is heavier wheel loading from the gantry to the segment. Since the segments have 28-day strength, this load gets transferred to the annular grout. Consideration should be given in grout and TBM design to allow the annular grout to cure long enough before these wheel loads are applied to fresh grout.

Regarding tail void grouting, some industry agreement is desired on the verification of annular grout quality. Some projects sample the grout at the batch plant on the surface and accelerate it (if needed) by hand. Some projects collect the same sample on the agitator tank on the TBM and accelerate it (if needed) by hand. Attempts have been made to drill a sample hole behind the TBM shield and collect the fresh grout through the sample hole. The mold rarely is filled properly. At worst groundwater or grout line flushing water dilutes what flows through the sample hole resulting in a poor sample with low strength. An even less desirable method is to drill a hole after the annular grout has cured. This results in both a hole in the segment and a hole through the grout, and introduces a permanent flow path of water into the finished tunnel. A desirable method of grout verification is an acceptable level of ring deformation.

Compression Packing Properties

Several recent projects have opted to eliminate compression packing altogether – both circumferential and longitudinal joints. These projects have, apparently, been successful. What criteria, fabrication tolerances or design elements made it successful? This may open some savings in the material's list for the following reasons:

- Bitumen packing simply doesn't work – it squeezes out of the joints
- Engineered materials are generally very expensive
- Marine grade plywood is frequently used, however:
    - High waste factor cutting circles from sheets of plywood
    - Marine grade plywood is made from sub-tropical woods, it's both imported and promotes rain forest deforestation
    - It should be affixed to the segment after transport (to prevent plywood flying off a loaded truck and possibly causing damage), which adds to the on-site labor costs

7.2 Construction Aspects

Transportation

Avoiding unnecessary transportation increases the durability of the segments. During transport, accidental loading should be prevented. In addition to the mechanical impacts during transportation, shock loads during transport and installation can also cause damage. Adding steel fiber reinforcement reduces the risk of damage caused by shock loads.

In the event of unplanned marginal pressures during installation or transport, high-strength concrete can be more brittle near the edges resulting in corner spalling. Subsequently, repaired damaged areas from spalling or wide cracks are often weak points in terms of durability. In summary, high strength concrete is more sensitive regarding transportation impacts. All transport damage should be documented.

Weight Limitations

Mega TBM drives tend to utilize an increase of segment width by minimizing the amount of segments per ring (segmentation) to allow for higher performances. Therefore, the weight of a single segment can approach an maximum limit. These limits are based on transportation restrictions such as weight limits on roadways and bridges.

Segment Transportation Logistics

The transportation cost can be a significant part of the segment price. Truck loads should be optimized. For contractors, it is usually beneficial to receive complete rings on one single truck/trailer. This does not always work for large diameter TBMs because of the weight of a single ring. An optimization of ring width selection and segmentation of the ring can minimize trucking costs if truck loads are close to the 100% efficiency rate of available trucks/trailers.

- Transportation from the plant to the site should be checked and depends on local road restrictions. To keep transportation costs economical, the availability of local trucks should be checked. Local road limitations (bridges, traffic load) can drive the decision where the segment plant will be established.
- Availability of local equipment and local road restrictions should be evaluated. For example: interference of available truck/trailer loads with allowable axle loads will drive additional load limits (weight of segments).

Segment Loading and Transport

- Equipment used to handle segments (forklift vs. crane and segment clamps) should be selected to avoid or minimize damage to the segemnts.
- Minimize additional handling by considering necessary storage area based on production schedule.
- Design checks of pads and cradles (segment clamp, on trucks, on site storage, on train or other equipment for tunnel transportation, quick unloading system).

Segment Handling Quality Assurance

Use of bar codes allows for seamless documentation of the segment life from the batching process, loading, deliveries to the site, segment storage and segment handling on site up to the installed product. Segment trackers will automatically store the time related to each stage of the process, as well as each damage occurred and repair procedures implemented. The initial cost of the segments might be increased by the implementation of such a system. The advantages for owners, as well as for contractors, can be significant. An example is the final tunnel acceptance e.g. proper tracking of each imperfection of a segment, focusing on areas with damaged segments, will make the process of final point and patch work the most efficient.

Segment cradles should have dunnage at 1/5th point of the segments for minimization of the bending moments in the segments.

Dunnage between segments should be lined up with the pads of the cradle to allow for a vertical load transfer from the top through the stack into the ground.

Safety Aspects

- Storage on site includes load checks of the existing ground surface. This should be performed for different seasons and rain events as they might decrease storage capacities. If there are any doubts, the ground surface should be improved and special attention should be paid to the design of the segment cradles.
- Height of a segment stack will lead to a high elevated center of gravity and will add risks to the transportation. Load pads should be designed to allow for proper loading of the segments.
- Stack heights on site should be checked based on the necessary access for:
    - Getting segment clamps or slings in place,
    - Installation of additional hardware (plywood packing, labels, etc.)
    - Removal of ice from pockets
    - Performance of repair procedures (replacement of gasket rings e.g.)
- In case of necessary re-stacking of segments during transport in the tunnel, the proper alignment of dunnage between single segments should be checked and corrected, if necessary. A proper location of the lowest segment on the cradle of the segment car is important. The center of gravity of the segments should be aligned to the center of the segment car. A clearance tolerance should be provided at each stage of transportation (train traffic and fixtures in the tunnel, as well as entering the gantries).
- Access to the segment stack inside of the TBM should to be checked. Segments should to be accessible to install dowels e.g., to clean vacuum cones or segment bolt pockets.

Tunnel contractors prefer repeatability in the supply chain logistics. But transportation is influenced by many factors, including local highway regulations (weight per axle). Having one truck-load deliver one ring may be efficient and cost effective. However, as segment sizes increase, it gets less and less possible to deliver one ring on one truck. In fact, it may take several truck-loads to deliver one ring, and on-site labor is needed to check and verify the ring has been reassembled (stacked) on site correctly for the ring build inside the tunnel. Using multiple trucks to deliver a single ring can result in a ring being erected in the wrong sequence, which may have a negative effect on ring quality and water-tightness. Management systems such as RFID tags embedded in the segments or bar code readers that are shared between contractor and pre-caster are potential solutions.

Ring Plane Alignment

One pass liners are generally held to a higher installation standard. As such, it is desirable to avoid cracked segments. As the TBM drive progresses, however, the leading face of the ring (that meets the TBM jack shoes) can get out of plane, promoting segment cracking. A series of tedious measurements at many locations around the ring can be done by hand and compression packing added to bring the next ring back into plane; but this is done after a segment or a few segments crack. Developing a process to avoid this situation is a potential topic for further research.

There are jack cylinders that have extensometers (instrumentation) for assisting with the ring orientation selection, but generally only 4 total have these extensometers (top, bottom, left and right) out of dozens of thrust cylinders. Devising a system that utilizes two or three times as many extensometers to give a proactive indication to correct the ring plane before it starts cracking segments could be beneficial.

The above condition is more pronounced with segments with a “higher degree of arc” and may be addressed in the segmentation section. Also, segments that have three dowels per segment, as opposed to two dowels per segment, tend to see more cracking. This also feeds into a liner "flexibility" discussion. In some situations, a flexible liner is desirable. Possibly in other situations, a more rigid style of liner is desirable.

The segment design drives the sensitivity regarding potential damage. Several factors should be considered:

A) Segmentation Ratio, is the ratio between the arc length "L" of a segment and the segment width "W". Segments with a ratio less than 2.5 are preferred and have less tendency for cracking during handling and tunnel drives. A ratio greater than 2.5 may lead to faster ring builds, but may result in a lower quality liner.

B) Segment thickness: In most cases the final tunnel liner loading condition is not the driving load case. Grout pressure for backfilling, handling of the segments, or the TBM push forces should be considered and checked. The segmentation of a ring may drive the amount of installed push rams or push ram shoes. The load of the push ram should be variable and is not uniformly distributed around the ring. Driving curves and the dead load of the TBM may result in higher forces in the lower third of the ring, as well as on the sides, depending on the curve type. The design should have allowances to prevent applying push forces onto the gasket. On the other hand, the interior of the segments have a similar need. The thrust shoe pad should be in a distance of greater than 3 cm (1") from the edge of the segment. An additional provision should be to prevent thrusting on joints. Any imperfection on the plane of the two adjacent segments is likely to be "corrected" by the thrust force, resulting in significant stresses and cracks. To mitigate this problem, the segment thickness should be designed to allow for a proper load distribution from the shoes.

C) Damage caused by large segment taper: Curve drives or a correction alignment in straight tunnels lead to the segments being designed with sufficient taper. The minimum radius curve of the alignment is critical data for the TBM and segment design. To be able to catch up with the theoretical alignment of the tunnel, segment length and ring taper have to be designed for a correction radius smaller than the minimum theoretical radius of the alignment. Segment length and ring taper are a combination of parameters depending on:

   - Outer Diameter of the lining (OD)
   - Segment Length (L)
   - Ring taper (t)

   A rough estimate of the radius (R) a lining can accommodate can be obtained with the following formula: R_{lining} = L x OD/t. This formula is affected by the fact that not all key positions are allowed in order to stagger the ring joints. Placing limitations on ring taper can avoid the condition known as iron bond where the rings touch the tail skin of the TBM. An iron bound condition can result in cracks in the segments that can affect the water tightness of the segment. Developing an appropriate limitation is a potential topic of future research.

   Handling during ring erection: Large diameter tunnels, may use a vacuum system to erect rings. The cones in the segment should be designed to carry the full dead load of a segment (shear force) in case of vacuum failure. This might result in additional ring reinforcement around the cones depending on the segment weight.

Dowels per Segment

There are certain geologic and/or construction conditions that make slightly flexible or slightly more rigid rings desirable. One way to vary the flexibility of the ring is varying number of dowels per segment. If a ring is made up of a series of smaller (in arc) segments and two dowels are used per segment, and each successively constructed ring is rotated so the next segment spans a longitudinal joint, the ring has slightly more flexibility. An alternate case is there are three dowels per segment on a larger arc. With three dowels per segment engaging the previous ring, there are typically two dowels from the leading segment mating with two ports in the previous segment. Since these two anchored into the previous segment, the third dowel mates up with a different segment. But the two fixed into a previous segment generates more rigidity as it fixes the location of the third dowel location. This added rigidity can induce more segment cracking. The rigidity, however, may be desirable with segmental linings in rock with two component grouts, whereas the more flexible lining may be desirable in soft ground applications.

Damage Types

a) Spalling
b) Cracks
c) Structural Damage
d) Joint dislocation / deformation of joint

Repair procedures depend on the specified groundwater inflow and leakage criteria. If reinforcement is exposed or the structural integrity of the segments are at risk, repair procedures should be employed regardless of water inflow criteria.

Tunnel Alignment

Experience has shown that large diameter TBMs have difficulty negotiating changes in alignment. The articulation of the shield and cutterhead adjustment can be difficult to change abruptly when crossing a point of curvature or a point of tangent. Alignment problems with the TBM versus the design tunnel alignment can be aggravated through the use of reverse curves that cause an immediate contortion of shield operating parameters from one extreme to another. Experience in large diameter sewer tunnels has shown the incorporation of spirals to transition into and out of curves has been successful in mitigating deviations from the design tunnel alignment, and promotes reduced segment lining cracking from tail shield and thrust ram interactions. The use of transition spirals into and out of curves mimics the physical abilities of a large diameter TBM to more gradually adjust operating parameters into and out of curves. Reverse curves should be avoided altogether, with a minimum distance between two adjacent, reversed curves of at least one shield length (typically around 40 feet).

Roadway geometry, however, generally does not incorporate the use of spirals. Some method of transitioning from a tangent to a curve should be investigated. Investigating the use of tangent separation between reverse curves would also be useful.

Accessories and Planes of Weakness'

Coordination between the design and pre-casting teams that avoids aligning too many segment features along the same longitudinal axis through a segment could eliminate planes of weakness. The most common mistake is to have the dowels, a grout plug, and the shear lifting cones all in a line – usually through the mid-point of a segment. Individually they can have a minimal effect as far as the overall segment bending strength, but when lined up they remove a significant portion of the cross section. Experience has shown hairline cracks develop along this plane of weakness.

Steam Curing Considerations

There are two basic types of steam generators for use in steam curing. One is a traditional boiler where combustion is in one chamber and the boiling water in another. The steam is piped off from the boiler and used in the cure process.

Another type uses a combined chamber where natural gas is jetted into a nozzle, ignited, and later in the nozzle water is introduced which is quickly vaporized.

Understanding the difference between the two in pre-casting is important. The steam cure chamber (as in a carousel type plant) or the individually tarped segment forms (as in a static plant) can fill with bad gasses (carbon dioxide and carbon monoxide) because in the latter model of steam generator described, the steam is mixed with the byproducts of combustion. The quantities of CO and CO2 are generally not life-threatening, but air quality monitors tuned to these gases should be installed around the plant. Personnel should be trained to not enter the cure chamber or under cure tarps without a permit to enter process.

BIBLIOGRAPHY

Abbas, S. and Nedhi, M. (2016). “Structural Strength,” Tunnels and Tunneling 2016, 32-37.

Abbas, S., Soliman, A. and Nehdi, M. (2014). “Mechanical performance of reinforced concrete and steel fiber-reinforced concrete precast tunnel lining segments: A case study,” ACI Material Journal 11 (5), 501-510.

Airport Link, Brisbane (APL) – WSP Project files.

Alaskan Way Viaduct Replacement Program – WSP Project files.

Almong, E. and Mangione, M. (2015). ”Ground Relaxation in Segmental Lining Design Using the Convergence-Confinement Method”, In Underground Design and Construction Conference 2015 Proceedings.

American Association of State Highway and Transportation Officials (AASHTO) (2018). “A Policy on Geometric Design of Highways and Streets” AASHTO-GDHS-7, Washington, DC.

American Association of State Highway and Transportation Officials (AASHTO) (2017). “LRFD Road Tunnel Design and Construction Guide Specifications First Edition”, AASHTO-LRFDTUN-1, Washington, DC.

American Association of State Highway and Transportation Officials (AASHTO) (2011). “Roadside Design Guide”, AASHTO- RSDG-4, Washington, DC.

American Association of State Highway and Transportation Officials (AASHTO) (2010). “Technical Manual for Design and Construction of Road Tunnels – Civil Elements”, AASHTO-DCRT-1, Washington, DC.

American Association of State Highway and Transportation Officials (AASHTO) (2008). “AASHTO LRFD Bridge Design Specifications 4th Edition”, Washington, DC.

American Concrete Institute (ACI) (1995). “Specification for Materials, Proportioning, and Application of Shotcrete” ACI-506.2-95, Farmington Hills, MI.

American Concrete Institute (ACI) (2006). “Code Requirements for Environmental Engineering Concrete Structures and Commentary” ACI 350-06, Farmington Hills, MI.

American Concrete Institute (ACI) (2010). “Report on the Physical Properties and Durability of Fiber-Reinforced Concrete” ACI 544-5R-10, Farmington Hills, MI.

American Concrete Institute (ACI) (2014). “Building Code Requirements for Structural Concrete and Commentary” ACI 318-14 and ACI 318R-14, Farmington Hills, MI.

American Concrete Institute (ACI) (2016). “Report on Design and Construction of Fiber-Reinforced Precast Concrete Tunnel Segments”, ACI- 544-7R-16, Farmington Hills, MI.

American Concrete Institute (ACI) (2018). “Guide to Design with Fiber-Reinforced Concrete”, ACI-544.4R-18, Farmington Hills, MI.

Anagnostou, G., Ehrbar, H., (2013) “Swiss underground construction and tunnel codes”, In: Tunneling Switzerland, Eds: Anagnostou, G., Ehrbar, Swiss Tunneling Society, ETH-Zurich, Switzerland.

ANIXTER, (2012). “Large Diameter Projects and Technical Solutions,” Presentation – Klug Associates European Tunneling Trip, Strasburg, France.

Armaghani, J., Taibi, Y. and Torrent, R. (2014). “Evaluation of Port of Miami Tunnel Segments”, ConcreteBeton, 136: 10-16.

Arnau, O. and Molins, C. (2011a). “Experimental and analytical study of the structural response of segmental tunnel linings based on an in situ loading test. Part 1: Test configuration and execution,” Tunneling and Underground Space Technology. 26 (6), 764-777.

Arnau, O. and Molins, C. (2011b). “Experimental and analytical study of the structural response of segmental tunnel linings based on an in situ loading test. Part 2: numerical simulation,” Tunneling and Underground Space Technology. 26 (6), 778–788.

Arnau, O. and Molins, C. (2012). "Three dimensional structural response of segmental tunnel linings." Engineering Structures 44 (2012): 210-221.

Asadollahi, P. and Kaneshiro, J.Y. (2014). “3D Analysis of Precast Segmental Liner and Induced Settlement for EPB Excavation”, in North American Tunneling Conference 2018 Proceedings.

Asche, H. R. and Ireland, T. J. (2013). "Segmental Lining Design for Large-Diameter Road Tunnels." In Rapid Excavation and Tunneling Conference 2013 Proceedings, pp. 866-877.

Association Française des Tunnels et de l'Espace Souterrain - French Tunnelling and Underground Space Association (AFTES) (2013) ”Design, Dimensioning and Execution of Precast Steel FRC Arch Segments”, GT38R1A1, Tunnels et Espace Souterrain, Vol 238.

Association Française des Tunnels et de l'Espace Souterrain - French Tunnelling and Underground Space Association (AFTES) (2005) ”The Design, Sizing and Construction of Precast Segments Installed at the Rear of a Tunnel Boring Machine (TBM)”, GT18R1A1, Tunnels et Ouvrages Souterrains, Vol 1.

Association Française des Tunnels et de l'Espace Souterrain - French Tunnelling and Underground Space Association (AFTES) (2005) ”Hydrophilic Swelling Gaskets for Tunnel Lining Segments - Gasket Assessment and Quality Control Procedures”, GT9R9A1, Tunnels et Ouvrages Souterrains, Vol 1.

Association Française des Tunnels et de l'Espace Souterrain - French Tunnelling and Underground Space Association (AFTES) (2005) ”Watertightness of Precast Concrete Lining Segments”, GT9R6A1, Tunnels et Ouvrages Souterrains, Vol 1.

Association Française des Tunnels et de l'Espace Souterrain - French Tunnelling and Underground Space Association (AFTES) (2005) ”Segmental Gaskets”, GT9R4A1, Tunnels et Ouvrages Souterrains, Vol 1.

Bakhshi, M. and Nasri, V. (2014). "Review of international practice on critical aspects of segmental tunnel lining design." In Proceedings: North American Tunneling Conference 2014, Los Angeles, CA.

Bakhshi, M. and Nasri, V. (2018) “Standard Practice and Latest Developments in Tunnel Segment Tolerances, Measurement, and Dimensional Control” Proceedings of WTC 2018, Dubai.

Bakhshi, M. and Nasri, V. (2018) “Tunnel segmental ring system and geometry - optimized design recommendations” Proceedings of WTC 2018, Dubai.

Bakhshi, M. and Nasri, V. (2016). “ACI Guideline on Design and Construction of Precast Concrete Tunnel Segmental Lining”. ITA-AITES, San Francisco, CA.

Bambridge, C., Wongkaew, M. and Kennedy, E. (2013). “Factors of Scale Planning Design and TBM Considerations for Large Diameter Bored Tunnels, In: Proceedings Rapid Excavation and Tunneling Conference, Washington DC, June 26, 2013.

Bäppler, K., (2014). “Constant demand on very large tunnel boring machine diameters for the construction of today's infrastructure systems”, In Proceedings: North American Tunneling Conference 2014, Los Angeles, CA.

Bäppler, K., (2016). “Large to very large Tunnel Boring Machine Diameters for today’s Infrastructure Systems”, In Proceedings: Tunneling Association of Canada Conference 2016, Ottawa, Ontario, Canada.

Bernard, E. (2016). “Crack Width Reduction in Reinforced Concrete Members Using Barchip Macro-Synthetic Fibers,” In: Proceedings ITA-AITES World Tunnel Congress 2016.

Blom, C. B. M (2002). “Design philosophy of concrete linings for tunnels in soft soils”. Doctoral Dissertation, Department of Civil Engineering, Delft University of Technology, The Netherlands.

Blom, C. B. M., Van der Horst, E. J. and Jovanovic, P. S. (1999). "Three-dimensional structural analyses of the shield-driven “Green Heart” tunnel of the high-speed line south." Tunnelling and Underground Space Technology 14, no. 2 (1999): 217-224.

Borghetti, F., Derudi, M, Gandini, P., Frassoldati, A., Tavelli, S. (2017). “Tunnel Fire Testing and Modeling - The Morgex North Tunnel Experiment”, Polytechnico di Milano, Springer.

Brinkgreve, R. B. and Post, M. (2015). “Geotechnical Ultimate Limit State Design Using Finite Elements”, In Geotechnical Risk and Safety V; 5th International Symposium on Geotechnical Safety and Risk; Rotterdam (The Netherlands), 13-16 Oct. 2015. IOS Press.

British Standards / British Tunnelling Society (BSI-BTS) (2016). “Tunnel design – Design of concrete segmental tunnel linings – Code of practice”, PAS 8810:2016, The British Standards Institution, London, United Kingdom.

British Tunnelling Society (BTS) and Institution of Civil Engineers (ICE) (2010). “Specification for Tunneling Third Edition”, Thomas Telford Ltd.

British Tunnelling Society (BTS) and Institution of Civil Engineers (ICE) (2004). "Tunnel Lining Design Guide", Thomas Telford Ltd.

Bull, A. "Stresses in the linings of shield-driven tunnels." Transactions of the American Society of Civil Engineers 111, no. 1 (1944): 443-474.

Bundesanstalt für Straßenwesen - Federal Highway Research Institute (BAST) (2010). “Additional Technical Contract Conditions and Guidelines for Civil Engineering Works - ZTV-ING - Part 5 Tunnel Construction - Section 3 Mechanical Shield-Driving Processes” (2018 English Version).

Bundesanstalt für Straßenwesen - Federal Highway Research Institute (BAST) (2010). “Technical Delivery Conditions and Technical Test Specifications for Sealing Profiles TLTP-ING Part 5 Section 3”.

Burger, W., Thewes, M. and Wehrmeyer, G., (2018). “Supersize TBM – Design aspects for very large tunnel boring machines (TBM), In: Tunnelbau 2018, Ernst and Sohn - German Geotechnical Society.

Caratelli, A., Maddaluno, G., Meda, A., Rinaldi, Z. and Spagnuolo, S. (2017). “Optimization of GFRP reinforcement in precast segments for metro tunnel lining”, Elsvevier LTD., Composite Structures, 181: 336-346.

Caratelli, A., Meda, A., Rinaldi, Z. and Romualdi, P., (2010). “Structural behavior of precast tunnel segments in fiber reinforced concrete,” Tunneling and Underground Space Technology 26 (2011) 284-291.

Caratelli, A., Meda, A., Rinaldi, Z. and Spagnuolo, S., (2016). “Precast tunnel segments with GFRP reinforcement,” Tunneling and Underground Space Technology 60 (2016) 10-20.

Caratelli, A., Meda, A., Rinaldi, Z., Giuliani- Leonardi and S., Renault, F. (2018). “On the behavior of radial joints in segmental tunnel linings,” Tunneling and Underground Space Technology. 71, 180-192.

Cavalaro, S. H. P., Blom, C. B. M., Aguado, A., and Walraven, J. C. (2011). "New design method for the production tolerances of concrete tunnel segments." Journal of Performance of Constructed Facilities 26, No. 6: 824-834.

Chen, J. and Hai-hong M. (2008). "Mechanical behavior of segment rebar of shield tunnel in construction stage." Journal of Zhejiang University-Science A 9, no. 7 (2008): 888-899.

Chesapeake Bay Bridge and Tunnel District, (2018), http://www.cbbt.com/project-description/.

Cheung, K., West, K., Yeow, H., and Simpson, B. (2010). “Do Eurocodes make a difference?”, Geomechanics and Tunnelling, 3(1), 35-47.

Chiaia, B., Fantilli, A. P., and Vallini, P. (2009). “Combining Fiber-Reinforced Concrete with Traditional Reinforcement in Tunnel Linings”, Engineering Structures, 31(7), 1600-1606.

Chongqing Communications Research & Design Institute (2004) “Specifications for Design of Highway Tunnel” Industrial Standards of the People’s Republic of China JTGD70, Beijing, China.

Chuan H., Kun, F., Zongxian, S. (2011). “Development and application of loading test system of prototype structure for underwater shield tunnel with large cross-section,” Chinese Journal of Rock Mechanics and Engineering. 30, 2, 254-266.

Clarke-Hackston, N., Messing, M., Loh, D., and Lott, R. (2006). "Modern high precision high speed measurement of segments and moulds." Tunnelling and Underground Space Technology 21, No. 3: 258-258.

Clem7 Project (Brisbane) – WSP Project files.

Comis, E., Younis, M., and Goodfellow, R. (2016). “Concurrent Segment Lining and TBM Design: A Coordinated Approach for Tunneling Success” Proceedings of WTC 2016, San Francisco, CA.

Cross Island Line (Singapore) – WSP Project files.

Curtis, D. J., Mott, Hay and Anderson (1976). "Discussion: The circular tunnel in elastic ground." Geotechnique 26, no. 1 (1976): 231-237.

DB Netz AG (2014). ”Richtlinie 853 Eisenbahntunnel planen, bauen, und instand halten, Deutsche Bahn DB (Guideline 853 Railroad tunnel design, construction, and maintenance, German Railroad)“ Frankfurt/Main, Germany.

De la Fuente, A., Blanco, A., Cavalaro, S. and Aguado, A. (2016). “Sustainability assessment of precast concrete segments for TBM tunnels,” In: Proceedings II International Conference on Concrete Sustainability, Madrid, Spain.

De la Fuente, A., Lin, L., Cavalaro, S. and Aguado, A. (2014). “Fibre reinforced precast concrete segments: design and applications,” In: Proceedings Proceedings of the World Tunnel Congress 2014 – Tunnels for a better Life. Foz do Iguaçu, Brazil.

Deutscher Ausschuss für Stahlbeton eV. (DAfStb) - German Committee for Reinforced Concrete (2010). “Richtlinie Stahlfaserbeton” – “Steel Fiber Reinforced Concrete Guideline” Berlin, Germany.

Deutscher Ausschuss für unterirdisches Bauen e. V. (DAUB) German Tunnelling Committee (ITA-AITES) (2013). “Recommendations for the Design, Production and Installation of Segmental Rings” Working group "Lining Segment Design" Cologne, Germany.

Deutscher Ausschuss für unterirdisches Bauen e. V. (DAUB) German Tunnelling Committee (ITA-AITES) (2010). “Recommendations for the Selection of Tunneling Machines” Cologne, Germany.

Deutscher Ausschuss für unterirdisches Bauen e. V. (DAUB) German Tunnelling Committee (ITA-AITES) (2000). “Recommendations for Design and Operation of Shield Machines” Cologne, Germany.

Deutscher Ausschuss für unterirdisches Bauen e. V. (DAUB) German Tunnelling Committee (ITA-AITES) (1997). “Recommendations for Selecting and Evaluating Tunnel Boring Machines” Cologne, Germany.

Deutscher Beton- und Bautechnik-Verein E.V. - German Society for Concrete and Construction Technology (DBV) (2001). “Guide to Good Practice-Steel Fibre Concrete” Berlin, Germany.

Ding, W. Q., Yue, Z. Q., Tham L. G., Zhu, H. H., Lee, C. F. and Hashimoto, T. (2004). "Analysis of Shield Tunnel" International Journal for Numerical and Analytical Methods in Geomechanics, 28: 57-91.

Ding, W., Gong, C. and Mosalam, K.M. (2017). “Development and application of the integrated sealant test apparatus for sealing gaskets in tunnel segmental joints,” Tunneling and Underground Space Technology. 63, 54–68.

Do, N. A. (2014). “Numerical Analyses of Segmental Tunnel Lining Under Static and Dynamic Loads”, PhD Dissertation, L’Institut National Des Sciences Appliquees De Lyon.

Dobashi, H., Matsuda, M., Kondo, Y., and Fujii, A. (2007). “Development of steel-fiber reinforced high fluidity concrete segment and application to construction,” RETC Proceedings 2007, 1205-1223.

Duddeck, H., and Erdmann, J. (1982). "Structural design models for tunnels". Universitat Braunschweig, Germany.

Duddeck, H., and Erdmann, J. (1985). “On structural design models for tunnels in soft soil." Underground Space, 9(5-6), 246-253.

Einstein, H. and Schwartz, C. W. (1979). "Simplified analysis for tunnel supports." Journal of Geotechnical and Geoenvironmental Engineering 105, no. ASCE 14541 (1979).

Eurasia Project Report – WSP Project files.

Federal Highway Administration (FHWA) (2009). “Technical Manual for Design and Construction of Road Tunnels – Civil Elements”, FHWA-NHI-10-034, Washington, DC.

Federal Highway Administration (FHWA) (2015). “Specifications for the National Tunnel Inventory”, FHWA-HIF-15-006, Washington, DC.

Federal Highway Administration (FHWA) (2015). “Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual”, FHWA-HIF-15-005, Washington, DC.

Fédération Internationale du Béton (*fib*) (1999). “Structural Concrete, the Textbook on Behaviour, Design and Performance”, *fib* Bulletin 51.

Fédération Internationale du Béton (*fib*) (2006). “Model Code for Service Life Design”, *fib* Bulletin 34.

Fédération Internationale du Béton (*fib*) (2008). “Practitioner’s Guide to Finite Element Modeling of Reinforced Concrete Structures”, *fib* Bulletin 45.

Fédération Internationale du Béton (*fib*) (2010). “Structural Concrete: Textbook on Behavior, Design and Performance”, *fib* Bulletin 54.

Fédération Internationale du Béton (*fib*) (2012). “Model Code 2010”, *fib* Bulletin 65.

Fédération Internationale du Béton (*fib*) (2017). “Precast tunnel segments in fibre-reinforced concrete”, *fib* Bulletin 83.

Frew, B., Wong, K. F., Mok, C. K. and Du, F. (2008). "A review of shield tunnel lining design." The Shanghai Yangtze River Tunnel Theory, Design and Construction 5 (2008): 37-41.

Fukuchi, G. (1991). "The present and future of mechanized tunnel works in soft ground." Tunnelling and underground space technology 6, no. 2 (1991): 175-183.

Gall, V. E., Ahmed M., Mario S., Markus O., Peter M., and Günther M. (2018). "A holistic approach for the investigation of lining response to mechanized tunneling induced construction loadings", Underground Space 3, no. 1 (2018): 45-60.

Gehwolf, P., Walter, A., Galler, R., Wagner, H. (2016). “Shear tests on flexible shear dowels for use at cross passages with thin segmental linings,” Geomechanics and Tunnelling. 9, 234-242.

Gettu, R., Barragan, B. E., Garcia, T. Ortiz, J., and Justa, R. (2006). “Fiber Concrete Tunnel Lining – Construction of a subway line in Barcelona,” In: Concrete International, August 2006, pp.63-69.

Gijsberg, F. B. J., and D. A. Hordijk. (1997). "Experimenteel onderzoek naar het afschuifgedrag von ringvoegen." TNO-rapport COB K 111 (1997).

Gipperich, C., Schaab, A. and Otremba, H-J. (2010). “Material technology developments in shield tunnelling – annular gap mortar, high-strength concrete and fire-protection concrete,” Geomechanics and Tunneling. 3, 283-294.

Gong, C., Ding, W., Mosalam, K.M., Günay, S., Soga K. (2017). “Comparison of the structural behavior of reinforced concrete and steel fiber reinforced concrete tunnel segmental joints,” Tunneling and Underground Space Technology., 68, 38-57.

Gong, C., Ding, W., Soga, K., Mosalam, K.M., Tuo, Y. (2018). “Sealant behavior of gasketed segmental joints in shield tunnels: an experimental and numerical study,” Tunneling and Underground Space Technology., 77, 127-141.

Goodfellow, R.J.F. (2011). “Concrete for Underground Structures Guidelines for Design and Construction” Society for Mining, Metallurgy, and Exploration.

Groeneweg, T. (2007). "Shield driven tunnels in ultra high strength concrete: Reduction of the tunnel lining thickness." PhD diss., MSc Thesis, Delft University of Technology, The Netherlands.

Grueber, T. and Dienner, A., (2012). “Design Issues, References and New Technologies – Tunneling Gaskets for Precast Tunneling Linings,” Presentation – Klug Associates European Tunneling Trip Strasburg, France.

Grübl, F. (2006). "Modern design aspects of segmental lining." In CPT-ITA World Tunnel Congress - 2006.

Grübl, F. (2012). “Segmental Ring Design: New Challenges with High Tunnel Diameters”, AITES/ITA.

Guglielmetti, V., Piergiorgio, G., Ashraf, M., and Shulin, X. (2007). “Mechanized Tunnelling in Urban Areas: Design Methodology and Construction Control” Taylor & Francis Group, London, United Kingdom.

Handke, D. (2012). “High-Precision Segments Prerequisite for a high-quality monocoque Tunnel” Tunnel 8: 42.

Harding, A. and Francis, O. (2014). “Designing at the Limit: Brisbane Airport Link Segmental Lining,” In: Proceedings Transportation Research Board 93rd Annual Meeting, Washington DC.

Hartmann, F (1972). “Elastizitatstheorie des ausgekleideten kreisformigen Hohlraumes und des eingebohrten kreisformigen - Rohres. StraBe Briicke” Tunnel 22 (1970).

Hartmann, F. (1985). "Einfache berechnung überschütteter, kreisförmiger rohre von beliebiger steifigkeit, elastizitätstheorie des überschütteten rohres." Bautechnik 62, no. 7 (1985): 224-235.

Hashash, Y.M.A., Hook, J. J., Schmidt, B., John, I., and Yao, C. (2001). "Seismic design and analysis of underground structures." Tunnelling and underground space technology 16, No. 4: 247-293.

Herrenknecht, A., and Bäppler, K. (2003). "Segmental concrete lining design and installation." In Soft Ground and Hard Rock Mechanical Tunneling Technology Seminar.

Herrenknecht, M., Bäppler, K. (2012). “Newest technology and trends in mechanized tunneling”, Proceedings, Tunneling Association of Canada Conference 2012, Montreal, Canada.

Hordijk, D.A., Gijsbers, F.B.J. (1996). “Laboratorium proeven tunnel segmenten, CUR/COB,” Project bureau boor tunnels, June 1996.

Horichi, N., Hirashima, M., Matsushita, Y. and Ishi, T. "A numerical method for analysis of shield segment rings under consideration of longitudinal rigidity" Doboku Gakkai Ronbunshu 1989, no. 406 (1989): 195-204.

Huang, X., Zhu, Y., Zhang, Z., Zhu, Y., Wang, S., Zhuang, Q., (2018). “Mechanical behaviour of segmental lining of a sub-rectangular shield tunnel under self-weight,” Tunneling and Underground Space Technology. 74, 131-144.

Iftimie, T. (1996). "A contribution to the concept and structural analysis of precast circular linings for shield driven tunnels." Ph. D. Thesis, Bucharest Romania.

Ingason, H., zhen Li, Y. and Lönnermark, A., (2015). “Tunnel Fire Dynamics,” Springer.

International Tunneling Association (ITA – ITAtech) (2016). “ITAtech Guidance for Precast Fibre Reinforced Concrete Segments - Vol. 1: Design Aspects” ITAtech Report No 7.

International Tunneling Association (ITA) (2000). “Guidelines for the Design of Shield Tunnel Lining”, Tunneling and Underground Space Technology, Vol 15 (3), 303-331.

International Tunneling Association (ITA) (2016). “Twenty Years of FRC Tunnel Segments Practice: Lessons Learnt and Proposed Design Principles” ITA Report No 16.

International Tunneling Association (ITA) (2019). “Guidelines for the Design of Segmental Tunnel Lining” ITA Report No 22.

Ireland, T.J., Asche H.R, (2011). “Developments in Segmental Lining Design.” In proceedings of Rapid Excavation and Tunneling Conference, June 2011. pp 450–462.

Italian National Research Council (2007). CNR-DT204/2006 “Guide for the Design and Construction of Fiber-Reinforced Concrete Structures”.

Iyengar, K.T. and Sundara R. (1962). "Two-dimensional theories of anchorage zone stresses in post-tensioned prestressed beams." In Journal Proceedings, vol. 59, no. 10, pp. 1443-1466. 1962.

Janssen, P. (1983). ”Tragverhalten von tunnel ausbauten mit gelenk tubbings,” Ph.D. Thesis Technischen Universitat Carolo-Wilhelmina, Braunschweig, December 1983

Japan Society of Civil Engineers (JSCE) (1996). “Japanese Standard for Shield Tunneling” Working Group for Shield Tunneling; Subcommittee on English Edition of the Japanese Standard for Tunneling.

Japan Society of Civil Engineers (JSCE) (2007). “Standard Specifications for Tunneling-2006: Shield Tunnels” Working Group for Shield Tunnels; Subcommittee on the Standard Specifications for Tunneling-2006.

Jian, Y., Clark, G. and Wu, J. (2018). “Evolution and Challenges of Segmental Liner Design and Construction for SR 99 Tunnel”, in North American Tunneling Conference 2018 Proceedings.

Jian, Y., Swenson, M. and Moore, T. (2013). “Liner Segment Design of the Largest TBM Tunnel in the Worlds – Alaskan Way Tunnel in Seattle”, in Rapid Excavation & Tunneling Conference 2013 Proceedings.

Jin, Y.L., Ding, W.Q. and Yan, Z.G. (2017). “Experimental investigation of the nonlinear behavior of segmental joints in a water-conveyance tunnel,” Tunneling and Underground Space Technology., 68, 153-166.

Kim, J., Lee, D., Ozturk, M., Lee, J., Kim, T.K., Arioglu, B. (2016). “Istanbul Strait Road Tunnel Project— Seismic Joint and Passive Fire Protection in Double Deck TBM Tunnel.” In: Proceedings, ITA-AITES World Tunnel Congress 2016.

Klappers, C., F. Grübl, and B. Ostermeier. (2006). "Structural analyses of segmental lining-coupled beam and spring analyses versus 3 D-FEM calculations with shell elements." Tunnelling and Underground Space Technology 21, no. 3 (2006): 254-255.

Kolic, D., and Mayerhofer, A. (2009). “Segmental Lining tolerances and imperfections” Proceedings of ITA WTC 2009 Symposium, Budapest, May 23-28, 2009, pp.8.

Koyama, Y. and Nishimura, T. (1997) "The Design of Lining Segment of Shield Tunnel Using a Spring Model with Two Ring Beams." In Proceedings of Tunnel Engineering Conference, JSCE, vol. 7, pp. 279-284. Japan Society of Civil Engineers, 1997.

Kramer, G. J., Gregor, E., Ghazi, T., M., and Herbert, C. D. (2003). "Segment Design for Tunneling Productivity." In: Proceedings Rapid Excavation and Tunneling Conference 2003.

Kunst D.J (2017). “Modelling Construction Phases of Bored Tunnels with respect to Internal Lining Forces”, Master Thesis, Delft University of Technology.

Lee, K.M., Hou, X.Y., Ge, X.W., Tang, Y., (2001). “An analytical solution for a jointed shield-driven tunnel lining,” Int. J. Numer. Anal. Meth. Geomech. 25 (4), 365–390.

Leonhardt, F, and Reimann, H. (1965). “Betongelenke: Versuchsbericht; Vorschläge zur Bemessung und konstruktiven Ausbildung - Kritische Spannungszustände des Betons bei mehrachsiger, ruhender Kurzzeitbelastung” Ernst, 1965.

Leonhardt, F. and Reimann, H. (1966). "Betongelenke." Der Bauingenieur 41 (1966): 49-56.

Li, X., Yan, Z., Wang, Z., Zhu, H., (2015). “Experimental and analytical study on longitudinal joint opening of concrete segmental lining,” Tunneling and Underground Space Technology. 46, 52–63.

Liu, X., Bai, Y., Yuan, Y., Mang, H.A. (2016). “Experimental investigation of the ultimate bearing capacity of continuously jointed segmental tunnel,” linings Struct. Infrastruct. Eng., 12 (10), 1364-1379.

Liu, X., Dong, Z.B., Bai, Y. (2017). “Investigation of the structural effect induced by stagger joints in segmental tunnel linings: first results from full-scale ring tests,” Tunneling and Underground Space Technology., 66, 1-18.

Liu, X., Zhang, C., Zhang, C., Yuan, Y. (2018). “Ultimate load-carrying capacity of the longitudinal joints in segmental tunnel linings: Illustrated with the typical segment joints of the Shanghai Metro rapid transit system,” Struct Concrete. 18, 693–709.

Lottman, B.B.G. (2007). “Fire in bored tunnels – Structural behavior, during fire conditions of bored tunnels made with a reinforced segmental lining”, Master Thesis, Delft University of Technology.

Lu. L., Lu, X., Fan, P. (2006). “Full-ring experimental study of the lining structure of Shanghai Changjiang tunnel,” The 4th International Conference on Earthquake Engineering, Taipei, Taiwan. 056, 1–8.

Luttikholt. A. (2007). “Ultimate Limit State Analysis of a Segmented Tunnel Lining - Results of full-scale tests compared to finite element analysis,” Ph.D. dissertation, Delft University of Technology.

Maidl, B., Herrenknecht, M., Anheuser, L. (2006). “Mechanised Shield Tunnelling” Wilhelm Ernst & Sohn, Berlin, Germany.

Maidl, B., Herrenknecht, M., Maidl, U., Wehrmeyer, G. (2012). “Mechanised Shield Tunnelling” Wilhelm Ernst & Sohn, Berlin, Germany.

Maidl, B., Thewes, M., Maidl, U. (2013). “Handbook of Tunnel Engineering Volume 1” Wilhelm Ernst & Sohn, Berlin, Germany.

Meda, A. and Rinaldi, Z., (2015). “Technical report: Tests on precast tunnel segment in concrete newly high tensile strength steel fibers Dramix 4D 80/60BG,” Tunneling Engineering Research Center, University of Rome Tor Vergata, 1-31.

Meda, A., Nanni, A., Rinaldi, A. and Spagnuolo, S. (2017) “Precast Concrete Tunnel Segments with GFRP Reinforcement”, American Society of Civil Engineers.

Miami Herald (2014), “Port Miami’s new tunnel”, Article. http://media.miamiherald.com/static/media/projects/2014/port-tunnel/.

Mobasher, B., Yao, Y. and Soranakom, C. (2015). “Analytical solutions for flexural design of hybrid steel fiber reinforced concrete beams,” Engineering Structures (100), pp: 164-177.

Morgan, H. D. (1961). "A contribution to the analysis of stress in a circular tunnel." Geotechnique 11, no. 1 (1961): 37-46.

Muir Wood, A M. (1975). “The circular tunnel in elastic ground”. Geotechnique, 1, 1975, 115-127

Naaman, A. E. (2018). “Fiber Reinforced Cement and Concrete Composites”, Techno Press, United States of America.

National Fire Protection Association (NFPA) (2017). “Standard for Road Tunnels, Bridges and Other Limited Access Highways” NFPA-502.

Neu, G.E., Gall, V. E., Freitag, S. and Meschke, G. (2018). “Robust design of hybrid steel fiber reinforced concrete tunnel lining segments”, In: Proceedings of the Conference on Computational Modelling of Concrete and Concrete Structures (EURO-C 2018), February 26 - March 1, 2018, Bad Hofgastein, Austria.

Österreichischen Vereinigung für Beton- und Bautechnik - The Austrian Society for Concrete and Construction Technology (ÖVBB) (2011). “Concrete Segmental Lining Systems”, Austrian Society for Concrete and Construction Technology.

Çimentepe, A.G. (2010). "Evaluation of structural analysis methods used for the design of TBM segmental linings." Master Thesis, Middle Eastern Technical University, Turkey.

Phan, L. T., 2008. “Pore Pressure and Explosive Spalling in Concrete,” Materials and Structures, Vol. 41, Issue No. 10, December 2008, pp. 1623–1632.

Pilotto, M., Jian, Y. (2012). “Design of the SR99 Bored Tunnel in Seattle, Washington”, in North American Tunneling Conference 2012 Proceedings.

Plizzari, G., Tiberti, G. (2007). “Structural behavior of SFRC tunnel segments”. In: Proceedings of the 6th International Conference on Fracture Mechanics of Concrete and Concrete Structures. 3. 1577-1584.

Poel, J. T. V. D., Hergarden, H. J. A. M. and Dekker H. R. E. (2006). "Soil load acting on shield tunnels: comparison between bedded beam model and finite element calculations." In Proceedings – Tunnelling - A Decade of Progess. GeoDelft 1995-2005 (2006): 195-201.

Port of Miami Partnership - FDOT (2011). http://www.portofmiamitunnel.com/press-room/fact-sheets-2/.

Port of Miami Tunnel – WSP Project files.

Promat. (2016). “Port of Miami tunnel fire protection - A post-fixed facetted system Promat-T”, firecladding product case study.

Réunion Internationale des Laboratoires et Experts des Matériaux, systèmes de construction et ouvrages (RILEM) (2016). “Performance-Based Specifications and Control of Concrete Durability”, State-of-the-Art Report RILEM TC 230-PSC, Springer.

Réunion Internationale des Laboratoires et Experts des Matériaux, systèmes de construction et ouvrages (RILEM) (2000). “Test and design methods for steel fibre reinforced concrete”, State-of-the-Art Report RILEM TC 162-TDF, Springer.

Sagüés, A. (2003). “Modeling the Effects of Corrosion on the Lifetime of Extended Reinforced Concrete Structures”, Corrosion, 59(10): 854-866.

Salemi, A., Esmaeili, M., Sereshki, F. (2015). “Normal and shear resistance of longitudinal contact surfaces of segmental tunnel linings,” International Journal of Rock Mechanics and Mining Sciences. 77. 328-338.

Schmid, H. (1926). “Statische Probleme des Tunnel-und Druckstollenbaues und ihre gegenseitigen Beziehungen: Gleichgewichtsverhältnisse im massiven und kreisförmig durchörterten Gebirge und deren Folgeerscheinungen. Spannungsverhältnisse unterirdischer Gewölbebauten” Springer.

Schreyer, J., Winselmann, D. (2000). “Suitability Tests for the Lining for the 4th Elbe Tunnel Tube – Results of large-scale Tests,” Tunnel, 1, 34-44.

Schulze, H., and Duddeck. H. (1964). "Spannungen in schildvorgetriebenen Tunneln." Beton-und Stahlbetonbau 59, no. 8 (1964): 169-175.

Schweiger, H. F. (2014). “Influence of EC7 Design Approaches on the Design of Deep Excavations with FEM. geotechnik, 37(3), 169-176.

Schweiger, H. F., Marcher, T., and Nasekhian, A. (2010). “Nonlinear FE-analysis of Tunnel Excavation–Comparison of EC7 Design Approaches. Geomechanics and Tunneling, 3(1), 61-67.

Shalabi, F. I., Cording, E. J., and Paul, S. L. (2016). "Sealant behavior of gasketed segmental tunnel lining–Conceptual model. "Geomechanics and Tunnelling 9, No. 4: 345-355.

Shalabi, F.I., (2001). “Behavior of gasketed segmental concrete tunnel lining,” Ph.D. Dissertation, University of Illinois at Urbana-Champaign, Urbana, IL.

Shalabi, F.I., Cording, E.J., Paul, S.L., (2007). “Sealant behavior of gasketed segmental concrete tunnel lining,” 4th International Conference on Earthquake Geotechnical Engineering. Paper No.1636.

Shalabi, F.I., Cording, E.J., Paul, S.L., (2012). “Concrete segment tunnel lining sealant performance under earthquake loading,” Tunneling and Underground Space Technology. 31, 51–60.

Smarslik, M, Putke, T., Marwan, A., Gall, V. E., Meschke, G. and Mark, P. (2017). "Berechnungsmodelle für Bau-und Endzustände von Tübbingtunneln." Taschenbuch für den Tunnelbau 2018: Kompendium der Tunnelbautechnologie Planungshilfe für den Tunnelbau 42 (2017): 101-135.

Spagnuolo, S., Meda, A., Rinaldi, Z. and Nanni, A., (2018). “Precast concrete tunnel segments with GFRP reinforcement” Journal of Composite Structures 21 (5).

Städing, A. and Krocker, T. (2010). “Application of the Partial Safety Factor Concept for the Structural Design of Tunnels in Germany. Geomechanics and Tunnelling, 3(1), 48-58.

STUVA (editor) (1996). “Eignungsprüfungen 4. Elbröhre Elbtunnel, Reibungsversuche”, http://www.stuvatec.de/tubbing_ergebnisse.htm.

STUVAtec Research Association for Tunnel and Transportation Facilities (STUVA) (2006). “STUVA Recommendations for the use of gaskets for sealing segmental linings” TUNNEL-GUTERSLOH-2: p28.

STUVAtec Research Association for Tunnel and Transportation Facilities (STUVA) (2005). “STUVA Recommendations for Testing and Application of Sealing Gaskets in Segmental Linings” TUNNEL-GUTERSLOH-8: 8.

STUVAtec Research Association for Tunnel and Transportation Facilities (STUVA) (2019). “Recommendation for Gasket Frames in Segmental Tunnel Linings” Forschung und Praxis 54

Susetyo, J., Shobayry, R., Gregor, T. (2014). “Modeling of Precast Concrete Tunnel Lining Subjected to Fire Event”, in North American Tunneling Conference 2014 Proceedings.

Swiss Society of Engineers and Architects (2004): SIA 197:2004 “Design of tunnels – Basic principles”.

Swiss Society of Engineers and Architects (2004): SIA 197/2:2004 “Design of tunnels – Road tunnels”.

Swiss Society of Engineers and Architects (2004): SIA 198:2003 “Underground Structures - Execution”.

Swiss Society of Engineers and Architects (2004): SIA 260:2003 “Basis of structural design”.

Tamjima, H., Kishida, M., Kanada, T. (2006). “RC segment model experiment on the behavior of TBM shield tunnel lining under high temperature in fire,” Doboku Gakkai Ronbunshuu E. 62, 4, 844-854. (In Japanese).

Tarada, F. and King, M. (2009). “Structural fire protection of railway tunnels,” In: Proceedings Railway Engineering Conference, University of Westminster, United Kingdom, June, 2009.

Tiberti, G., Plizzari, G. and Chiriotti, E. (2018), “Structural use of fibre reinforced concrete in precast segments – Lessons from twenty years of application” ITA WG2 Presentation, 24 May 2018, Zurich, Switzerland.

Tirpitz, R.R., Hestermann, U. (1999). ‘4th tube of the tunnel under the river Elbe in Hamburg- On the load carrying behavior of the segmental lining,” Geotechnical Engineering for Transpiration Infrastructure. 2147-2153.

Tiberti, G., Plizzari, G., Walraven, J.C., Blom, C. B. M. (2008). “Concrete tunnel segments with combined traditional and fiber reinforcement” In: Tailor Made Concrete Structures: New Solutions for our Society. Eds: Walraven & Stoelhorst, Taylor & Francis Group.

TunnelTalk.com (March 7, 2019), “Tracking the World’s Mega-TBM’s (https://www.tunneltalk.com/Discussion-Forum-Tracking-the-mega-TBMs-of-the-world.php).

TunnelTalk.com (March 7, 2019), “ST Petersburg Neva River Highway Revival” (https://www.tunneltalk.com/Russia-04Mar2019-Russia-plans-to-build-replacement-to-Orlovsky-tunnel-in-St-Petersburg.php)

Tvede-Jensen, Bo, Faurschou, M., and Kasper, T. (2017). "A modelling approach for joint rotations of segmental concrete tunnel linings." Tunneling and Underground Space Technology 67 (2017): 61-67.

US Department of Transportation (1983). ”Design Recommendations for Concrete Tunnel Linings Volume II: Summary of Research and Proposed Recommendations”, DOT-TSC-UMTA-83-16, Washington, DC.

Vazaios, I. and Vlachopoulos, N. (2015). “The Influence of Segmental Lining in Deep TBM Tunneling”, in GEOQuebec 2015 Proceedings.

Vervuurt, A.H.J.M., den Uijl, J.A., Gijsbers, F.B.J., van der Veen, C. (2003). “Aanvullende roeven in de tunnelproefopstelling: Constructiegedrag onder gebruiksbelastingen en het effect van plaatsingsonnauwkeurigheden. Deel 2: Resultaten van serie B,” Delft Cluster.

Vigl, A., Walter, A., Lemmerer, J., Atzl, G., and Handke, D. (2016). "TBM research project–Tolerances segmental lining/TVM-Forschungsprojekt–Toleranzen Tübbingausbau." Geomechanics and Tunnelling 9, No. 3: 243-259.

Walter, H. (2010). “Safety Concepts in Geotechnical Engineering then and Now – How Do the Eurocodes Affect Tunneling?”, Geomechanics and Tunneling, 3(1), 24-33.

Wang, J. (1993). Seismic Design of Tunnels – A Simple State-of-the-Art Design Approach, Parsons Brinckerhoff Monograph No. 7.

Waterview Connection Project (Auckland) – WSP Project files.

Windels, R (1966). "Spannungstheorie zweiter Ordnung für den teilweise gebetteten Kreisring." Die Bautechnik 43, no. 8 (1966): 265ff.

Windels, R. (1967). "Kreisring im elastischen Kontinuum." Der Bauingenieur Bd 42 (1967): 429.

WSDOT, (2012) “Alaskan Way Viaduct Replacement Program”, Presentation – Klug Associates European Tunneling Trip, Strasburg, France.

Yan, Z., Shen, Y., Zhu, H., Lu, Y. (2016). “Experimental study of tunnel segmental joints subjected to elevated temperature,” Tunneling and Underground Space Technology. 53, 46–60.

Yan, Z., Zhu, H., J. Ju, W., Ding, W. (2012). “Full-scale fire tests of RC metro shield TBM tunnel linings,” Construction and Building Materials. 36, 484-494.

Yan, Z., Zhu, H., Ju, W. (2015). “Experimental investigation of reinforced concrete and hybrid fiber reinforced concrete shield tunnel segments subjected to elevated temperature,” Fire and Safety journal. 71, 86-99.

Yan, Z.G., Zhu, H.H., Ju, J.W. (2013). “Behavior of reinforced concrete and steel fiber reinforced concrete shield TBM tunnel linings exposed to high temperatures,” Construction Building Materials, 38, 610–618.

Yao, Y., Bakhshi, M., Nasri, V., Mobasher, B. (2018). “Interaction diagrams for design of hybrid fiber-reinforced tunnel segments,” Materials and Structures 51:35.

Zhang, Z., Zhu, Y., Huang, X., Zhu, Y., Liu, W. (2019). “Standing full-scale loading tests on the mechanical behavior of a special-shape shield lining under shallowly-buried conditions,” Tunneling and Underground Space Technology, 86, 34-50.

Zhao, C., Lavasan, A. A., Barciaga, T., Kämper, C., Mark, P. and Schanz, T. "Prediction of tunnel lining forces and deformations using analytical and numerical solutions." Tunnelling and Underground Space Technology 64 (2017): 164-176.

Zhao, Y., Chu, C., Yi, Y. (2016). “Study on an engineering measure to improve internal explosion resistance capacity of segmental tunnel lining structures,” Journal of Vibroengineering. 18. 5 2997-3009.

Zhu, Y., Zhang, Z., Huang, X. (2018). “Prototype Loading Tests on Full-Ring Segmental Lining of Rectangular Shield Tunnel,” Journal of Shanghai Jiaotong Univ. (Sci.) 23, 746.

APPENDIX A: LARGE DIAMETER TBM PROJECTS

#	COUNTRY	PROJECT NAME	CONSTR. YEAR	LENGTH	TBM DIAMETER	TBM	BORES AND ROADWAY LAYOUT	FUNCTION	SEGMENT LAYOUT	SEGMENT MATERIAL TYPE	LINING INTERNAL DIAMETER	LINING THICKNESS	LINING SLENDERNESS RATIO	SEGMENT WIDTH	RING VOLUME	TBM TORQUE	TBM THRUST	THRUST TO EXC. AREA	CUTTERHEAD POWER	SEGMENT WEIGHT	DESIGN LIFE	WATER PRESSURE (bar)	GASKET
				(m)	(m)						(m)	(m)		(m)	(m^3)	(kN-m)	(kN)		(kW)	(tn)	(years)	(bar)	
1	Australia	Melbourne West Gate Highway Tunnel	2018	2800 East and 4000 West	15.6	Herrenknecht x 2 EPB	twin tube, three lane	roadway	9+1, universal ring, rectangular	FRC	14.1	0.5	1/28.2	2.4	55.0	68000	193961	1014.788031	#N/A	#N/A	#N/A	#N/A	cast-in, EPD
2	Japan	Tokyo Outer Ring Road - Gaikan	2017	9155 - Tomei North Section	16.1	1 Kawasaki and 3 JIM, EPB	twin tube,three lane	roadway	12+1	steel rebar and hybrid - steel rebar and steel plate reinforced	14.5	0.65	1/22.3	1.6	49.5	100400	294000	1444.128028	9900	#N/A	#N/A	#N/A	#N/A
3	Italy	Santa Lucia Highway Tunnel, Autostrada A1 Barberino di Mugello - Firenze Nord	2016	7528	15.87	1 Herrenknecht EPBM (S-900)	one main bored tunnel 3-lanes, and one smaller 4.3 m diam. emergency tunnel	roadway	9 + 0 / universal rings	steel rebar	14.3	0.55	1/26	2.2	56.4	101296	#N/A	#N/A	8750	16	100-150	6	cast-in
4	Hong Kong	Tuen Mun - Chek Lap Kok, subsea highway link (1st bore)	2015	650 @ 17.6m and 4030 @ 12.4m	17.6 and 14	1 Herrenknecht Mixshield later modified from 17.6 to 14m SPB (S880 and S881)	Twin tubes, 2-lane	roadway	11+1 rectangular for the 17.6 diameter	#N/A	15.6	0.7	1/22.3	1.7	60.9	27722	160800	660.9533794	5600	#N/A	#N/A	<6	#N/A
4	Hong Kong	Tuen Mun - Chek Lap Kok, subsea highway link (2nd bore)	2015	4200	14	Herrenknecht Slurry TBM S882	2-lane	roadway	8	#N/A	12.4	0.55	1/22.5	2.2	49.2	#N/A	118800	771.739071	#N/A	#N/A	#N/A	<6	#N/A
5	China	Wuhan Metro road/metro river crossing	2015	2590	15.76	2 Herrenknecht Mixshields	twin tube, double function, upper deck 3-lane, lower deck rail	roadway and rail	10 (7 standard, 2 counter-key and 1 key segment)	#N/A	13.9	0.65	1/21.4	2	59.4	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	5.3	#N/A

#	COUNTRY	PROJECT NAME	CONSTR. YEAR	LENGTH	TBM DIAMETER	TBM	BORES AND ROADWAY LAYOUT	FUNCTION	SEGMENT LAYOUT	SEGMENT MATERIAL TYPE	LINING INTERNAL DIAMETER	LINING THICKNESS	LINING SLENDERNESS RATIO	SEGMENT WIDTH	RING VOLUME	TBM TORQUE	TBM THRUST	THRUST TO EXC. AREA	CUTTERHEAD POWER	SEGMENT WEIGHT	DESIGN LIFE	WATER PRESSURE (bar)	GASKET
				(m)	(m)						(m)	(m)		(m)	(m^3)	(kN-m)	(kN)		(kW)	(tn)	(years)	(bar)	
6	**Turkey**	**Eurasia Tunnel (Istanbul Straight Crossing)**	2014	3340	13.7	Herrenknecht Mixshield	single tube, twin deck, 4-lane	roadway	6+2 + 1 (universal ring)	rebar reinforced and polypropylene fiber	12.0	0.6	1/20	2	47.5	23290	247300	1677.618091	10330	15-16	100 (127 estimated)	11	twin Datwyler gaskets
7	**Italy**	**Caltanissetta highway tunnel, Sicily**	2013	3880 and 3925	15.08	1 NFM Technologies, EPB	twin-tube, 2-lane	roadway	8 +1, trapezoidal universal type	#N/A	13.45	0.6	1/22.4	2	53.0	73300	235665	1319.479123	7656	16	#N/A	typ. 6 and 7.5 bar extreme	#N/A
8	**New Zealand**	**Waterview highway connection, Auckland**	2013	2400	14.41	1 Herrenknecht EPBM (S-764)	twin tube, 3-lane	roadway	9 + 1	hybrid	13.1	0.45	1/29.1	2	38.3	82546	199504	1223.302222	8400	#N/A	#N/A	6	glued Phoenix M385 87A 'Groene Hart'
9	**USA**	**Port of Miami Tunnel**	2013	1280	12.89	Herrenknecht EPB S-600	twin tubes, 2-lane	roadway	8 (5 standard, 2 counterkey, 1 key)	steel rebar	11.9	0.61	1/19.5	1.7	40.7	37211	#N/A	#N/A	6300	12.2	150	#N/A	glued on. Phoenix M385 87a
10	**Australia**	**Airport Link, Brisbane**	2012	5100	12.48	2 x Herrenknecht EPBM	twin tube, two lane, two deck	roadway	9 + 1, rectangular, tapered L/R	FRC and in areas RCC	11.34	0.4	1/28.4	2	29.5	20400	89300	730.0160471	4900	#N/A	#N/A	5	glued on
11	**China**	**Shanghai West Changjiang Yangtze River Road Tunnel (Huangpu river)**	2011	3100	15.43	1 Herrenknecht Mixshield, Ex-Shanghai Changjiang highway tunnel Project (S-569 ex. S-318)	twin tube	roadway	7+2+1	#N/A	13.7	0.65	1/21.1	2	58.6	39941	203066	1085.96413	3750	#N/A	#N/A	#N/A	

#	COUNTRY	PROJECT NAME	CONSTR. YEAR	LENGTH	TBM DIAMETER	TBM	BORES AND ROADWAY LAYOUT	FUNCTION	SEGMENT LAYOUT	SEGMENT MATERIAL TYPE	LINING INTERNAL DIAMETER	LINING THICKNESS	LINING SLENDERNESS RATIO	SEGMENT WIDTH	RING VOLUME	TBM TORQUE	TBM THRUST	THRUST TO EXC. AREA	CUTTERHEAD POWER	SEGMENT WEIGHT	DESIGN LIFE	WATER PRESSURE (bar)	GASKET
				(m)	(m)						(m)	(m)		(m)	(m³)	(kN-m)	(kN)		(kW)	(tn)	(years)	(bar)	
12	**USA**	**SR-99 / Alaskan Way Highway Replacement Tunnel**	2011	2700	17.48	1 Hitachi Zosen EPBM	single tube, twin deck, 4-lane total	roadway	10 (7 - standard, 2 counter key, 1 key) / universal ring	steel rebar	15.85	0.6	1/26.4	1.98	61.4	147400	392000	1633.478148	13440	17.5	100 yr	<6	glued on, guide rods
13	**China**	**Weisan Road Tunnel, Nanjing**	2011	North = 3540, South = 4140	14.93	2 IHI/Mitsubishi/CCCC slurry TBMs	twin tube, double decker, 4-lane	roadway	9 + 1	#N/A	13.3	0.6	1/22.2	2	52.4	43588	278400	1590.229238	3780	#N/A	#N/A	7.7	#N/A
14	**Italy**	**A1 Sparvo highway tunnel**	2011	2600	15.55	1 Herrenknecht EPBM	twin tubes, 2-lane	roadway	9 +1 with guidebar	steel rebar	13.6	0.7	1/19.4	2	62.9	94800	315880	1663.303346	12000	17	#N/A	#N/A	Fama UG019A glued on
15	**France**	**Paris A86 East Duplex Tunnel (SOCATOP)**	2011	10000	11.6	Herrenknecht Mixshield Slurry and EPB (convertible)	single tube, double deck, two lanes+shoulder	roadway, lightweight vehicles	7+1	steel rebar	10.37	0.42	1/24.7	2	28.5	16400	#N/A	#N/A	4000	11	#N/A	3.5	#N/A
16	**Spain**	**Seville SE-40 Highway Tunnels**	2010	2 x 1900 and 2 x 2180	14	2 NFM Technologies EPBMs	four tubes, three lanes per tunnel	#N/A	#N/A	#N/A	12.6	0.5	1/25.2	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
17	**Russia**	**Orlovsky Tunnel, Saint Petersburg (original large diameter concept)**	2009	1000	19.25	1 Herrenknecht Mixshield	two decks, 3-lanes per deck	#N/A	12 +1	#N/A	17.25	0.7	1/24.6	2.2	86.8	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

#	COUNTRY	PROJECT NAME	CONSTR. YEAR	LENGTH	TBM DIAMETER	TBM	BORES AND ROADWAY LAYOUT	FUNCTION	SEGMENT LAYOUT	SEGMENT MATERIAL TYPE	LINING INTERNAL DIAMETER	LINING THICKNESS	LINING SLENDERNESS RATIO	SEGMENT WIDTH	RING VOLUME	TBM TORQUE	TBM THRUST	THRUST TO EXC. AREA	CUTTERHEAD POWER	SEGMENT WEIGHT	DESIGN LIFE	WATER PRESSURE (bar)	GASKET
				(m)	(m)						(m)	(m)		(m)	(m³)	(kN-m)	(kN)		(kW)	(tn)	(years)	(bar)	
18	**China**	**(1st Nanjing) Nanjing Yangtze River Tunnel**	2008	3837	14.93	2 Herrenknecht Mixshields	twin tube, 3-lane	roadway	10	#N/A	12.75	0.6	1/21.3	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
19	**China**	**Bund Tunnel, Shanghai**	2007	1098	14.27	1 Mitsubishi EPBM	#N/A	#N/A	#N/A	#N/A	12.75	0.76	1/16.8	2	64.5	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
20	**Malaysia**	**SMART Tunnel, Kuala Lumpur**	2007	5500 north, 4200 south	13.2	Herrenknecht x2 Slurry/ Mixshield S252/ S 253	twin tube, two lane, two deck	roadway and stormwater	8 + 1, rectangular, tapered L/R	rebar	11.83	0.5	1/23.7	1.7	32.9	24400	94500	690.5483068	4000	10	#N/A	#N/A	glued on EPDM
21	**China**	**Shanghai Changjiang under river highway tunnel**	2006	7470	15.43	2 Herrenknecht Mixshields S-317 and S-318	twin tube, 3-lane	roadway	10 (7 standard + 2 counterkey + 1 key). Tapered ring	steel rebar	13.7	0.65	1/21.1	2	58.6	39941	203066	1085.96413	3500	#N/A	#N/A	<6.5	glued on and hydrophilic gasket
22	**Spain**	**Madrid Calle 30 Highway Tunnels**	2005	South tunnel, south bypass = 7200, North bypass tunnel=4180	15.2	2 machines, 1 Herrenknecht S-300, 1 Mitsubishi (15.2 and 15)	twin tube, 3-lane	roadway	9 + 1	#N/A	13.45	0.7	1/19.2	2	62.2	125000	315880	1740.784745	10000 - 14000	#N/A	#N/A	6	#N/A
23	**China**	**Shangzhong Road Subacqueous Tunnel, Shanghai**	2004	1300	14.87	1 NFM Technologies, Ex-Groenehart machine	twin tube, twin deck, 4-lane	roadway	Segmentation unknown, universal segment with staggered joints per ITA	#N/A	13.3	0.6	1/22.2	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	6	#N/A

#	COUNTRY	PROJECT NAME	CONSTR. YEAR	LENGTH	TBM DIAMETER	TBM	BORES AND ROADWAY LAYOUT	FUNCTION	SEGMENT LAYOUT	SEGMENT MATERIAL TYPE	LINING INTERNAL DIAMETER	LINING THICKNESS	LINING SLENDERNESS RATIO	SEGMENT WIDTH	RING VOLUME	TBM TORQUE	TBM THRUST	THRUST TO EXC. AREA	CUTTERHEAD POWER	SEGMENT WEIGHT	DESIGN LIFE	WATER PRESSURE (bar)	GASKET
				(m)	(m)						(m)	(m)		(m)	(m³)	(kN-m)	(kN)		(kW)	(tn)	(years)	(bar)	
24	**Russia**	**Moscow Lefortovo Highway Tunnel**	2001	2200	14.22	1 Herrenknecht Mixshield, Ex-4th Elbe project machine	single tube, 3-lane	roadway	8 + 1	steel rebar	12.35	0.7	1/17.6	2	57.4	26598	120000	755.6003874	3200	#N/A	#N/A	3.5	#N/A
25	**Netherlands**	**Groenehart tunnel**	2000	7200	14.87	1 NFM Technologies	single tunne, two rail	railroad	10	#N/A	13.3	0.6	1/22.2	2	52.4	136000	184300	1061.23977	9540	#N/A	#N/A	4	single Phoenix style with water-stop strip
26	**Germany**	**Hamburg 4th Elbe River Highway Tunnel**	1997	2600	14.2	1 Herrenknecht Mixshield (S-108)	single tube, two lanes	roadway	8 + 1 (6 standard, 2 counter, 1 key)	rebar. The lining is reinforced with 100kg/m3 of steel and the eight main segments weigh 20 tonne each	12.35	0.7	1/17.6	2	57.4	25780	176520	1114.621327	3400	18	#N/A	5	double gasket PhoenixM385.65)
27	**Japan**	**Trans Tokyo Bay Highway Tunnel**	1994	9600	14.14	8 machines	twin tube, 2-lane	roadway	11 + 1	The 650mm thick segment is reinforced with 200-280kg of steel/m3 and weighs about 10 tonne.	11.9	0.65	1/18.3	1.5	38.4	31850	235360	1498.800934	#N/A	10	#N/A	#N/A	single water swelling gasket
28	**Australia**	**Clem Jones Tunnel (CLEM 7)**	2010	4300	12.34	2 x Herrenknecht Double shield	twin tube, 2-lane	roadway	8+1, rectangular, tapered L/R	FRC and steel rebar	11.2	0.4	1/28	2	29.2	18000	94000	785.9730463	4200	9	#N/A	#N/A	single EPDM, glued