Twenty Years of FRC Tunnel Segments Practice: Lessons Learnt and Proposed Design Principles

ITA Working Group 2
Research

N° ISBN: 978-2-970 1013-5-2

ITA REPORT N°16 / APRIL 2016

ASSOCIATION INTERNATIONALE DES TUNNELS ET DE L'ESPACE SOUTERRAIN
AITES
ITA
INTERNATIONAL TUNNELLING AND UNDERGROUND SPACE ASSOCIATION

ITA Report n° 16 - **Twenty years of FRC tunnel segments practice: lessons learnt and proposed design principles**
N°ISBN : 978-2-970 1013-5-2 / APRIL 2016 - Layout : Longrine – Avignon – France – www.longrine.fr

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1 >> Preface

The construction of underground infrastructures for transport purposes (roads, railways, and metro), for mountain, sea straits or rivers crossing, for water transportation (clean or sewage water), and for utilities, including multi-purpose tunnels has a key role in the development of modern society. In this context, the use of mechanised tunnelling faces continuously increasing challenges in terms of diameter, depth, machine power, adaptability to different geological context. Consequently, the conception of the lining - usually made of precast segments - has to evolve accordingly in terms of mechanised behaviour, bearing capacity, crack control and water-tightness.

In the last two decades, the use of Fibre Reinforced Concrete (FRC) progressed and was adopted in several tunnel projects. Among the benefits related to the inclusion of fibre reinforcement in the cementitious composites, the most important is the noticeable increase of the post-cracking tensile residual properties. In addition, the fibre reinforcement facilitates production process of the lining segments. The enhancement of the general structural behaviour together with the improvement of the industrialized production of precast tunnel segments are probably the two main key-factors of the continuously growth in using FRC in precast tunnel linings.

The aim of this report is to provide advances in the design of FRC tunnel lining in accordance with the objectives of the International Tunnelling Association (ITA) prescribed in Section II of the Statutes of the ITA (ITA, 1976). Standards and recommendations related to the design of general FRC elements are already available, but they do not provide details on specific requirements and loading conditions applicable in the case of tunnel lining segments. The scope of the report is hence to take advantage of twenty years of FRC practice in precast tunnel lining - including research and feedback from real cases – to provide additional design principles which complete the existing standards and recommendations for the specific case of tunnel lining.

This document was conceptually agreed during the meeting of ITA Working Group 2 (WG2) in Budapest, 2009. The first early draft document was presented in Vancouver, 2010. After several discussions and meetings within WG2 the current version was completed, it was reviewed by the WG2 members, and it went through the formal approval process of ITA.

1 >> Preface

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Annular gap :
space between the surrounding excavated ground and the outer surface of the segments;

Average tunnel segment aspect ratio :
the ratio of average segments length (based on the different length of segments, counter-key segments and the key-segment) to segment thickness;

Circumferential joint or ring joint :
joint between two adjacent segmental rings;

Connections:
fixed connecting elements (e. g. bolts, dowels) subjectable to tensile stress and shearing for temporary or permanent connecting two segments or segmental rings in the longitudinal and circunferential joints;

Conventional or traditional rebars :
steel reinforcing bar place into the segment mould before pouring the concrete (RC tunnel segments);

Cross-passage or cross-cut :
connecting structure between two tunnel tubes or between a tunnel tube and the ground surface or a shaft, with special transition structures with the main tube (usually connection in the side wall area);

D-region :
disturbed region where the typical beam theory cannot be applied, but a more general approach should be used. D-regions occur in tunnel segments during the TBM thrust phase where high concentrated forces exerted by TBM jacks determine stresses parallel and perpendicular to the axis of the applied forces (2D or 3D stresses condition);

Eccentricity :
in general is a misalignment of an applied force with respect to the centre of gravity of the resisting concrete cross section. Hence, it can be an eccentricity of the axial force (normal ring force) in the segment (e.g. final state) or it can be a possible irregularity in the position of TBM jacks with respect to tunnel rings. It can be caused by different factors;

Fire spalling of concrete :
a physical process of breaking up of surface layers of concrete which crumble into small pebble-like pieces in response to high temperatures exposure during a fire event;

Grouting process :
process of filling the annular gap with mortar or blowing pea gravel (or other composites as in case of two-component grouting) in order to produce a frictional connection between the subsoil and the segmental lining;

Internal ring forces :
actions arising in tunnel linings (N=axial force or normal force, V=shear force, M=bending moment) generally calculated by accepting, as a first approximation, the typical beam theory;

Longitudinal joint or radial joint :
joint between adjacent segments belonging to the same ring;

Post-cracking residual strengths :
post-cracking stresses that a cracked concrete sample under a uniaxial tensile force can still carry. Generally speaking, they are also intended as the nominal simplified strengths that can be obtained from tests of cracked concrete samples under flexure;

Ring model :
simplified numerical or analytical model, based on beam theory and using simplified approaches for simulating the surrounding ground, used to calculate the internal ring forces (N, V, M);

Segment width :
dimension of the segment ring in its centre axis in the longitudinal direction of the tunnel;

Segment length :
mean segment length measured along the segment mean curved plane;

Sealing system :
sealing strips (gaskets) installed on the perimeter of each individual segment which ensure permanent sealing of the tunnel tube against groundwater pressure;

Segment thickness :
distance between the inner and outer sides of the lining segment;

Spalling stresses :
local tensile stresses that arise in the lining in the region between the thrust jack plates (thrust shoes) because of the interaction of adjacent jacks or couple of jacks;

Splitting stresses :
local tensile stresses (also called bursting stresses) which arise during the jacks thrust in tunnel segments under their loading areas in transverse direction (perpendicular to the axis of the applied forces);

Structural characteristic length :
parameter used for easily connect continuous mechanics, governed by a stress-strain constitutive relationships, and fracture mechanics, governed by a stress-crack opening law;

Tail void :
annular space between the outside diameter of the shield and the extrados of the segmental lining;

Thrust jack configurations :
location of the hydraulic jacks along the segment length, which determines where the stress concentrations in tunnel segments take place;

Thrust jack or thrust ram :
hydraulic jack serving to transmit the thrust forces of the tunnel boring machine to the segmental ring;

Thrust jack phase :
temporary phase when the TBM pushes itself forward while excavating the ground and pushing through its hydraulic jacks on the already installed tunnel lining;

Toughness :
ability of a material to absorb energy and deform without rupturing. With regard to concrete tensile behaviour, toughness generally indicates the ability to resist internal crack propagation;

Tunnel boring machine (TBM) :
the machine used to excavate tunnels with generally a circular cross section;

Tunnel lining aspect ratio :
the ratio of tunnel lining ring diameter to ring thickness (segment thickness);

Tunnel overburden :
clear ground cover over the crown of the tunnel lining;

Tunnel segment aspect ratio :
the ratio of segment length (segment extension) to ring thickness (segment thickness);

Tunnel segment :
curved prefabricated element that composes the tunnel lining rings; depending upon the type of lining, the corresponding radial joints can be inclined or parallel with respect to the tunnel longitudinal axis;

Un-even supports or non-smooth supports :
referring to the TBM thrust phase, corresponds to the configuration when one or more supports of a tunnel element in a ring joint because of several possible reasons;

Universal ring :
tapered segmental ring that can be installed in a number of allowed positions permitting to face straight and curved alignment.

3 >> List of symbol and abreviations

A'_s	Cross-sectional area of conventional steel reinforcing bar in compression;
A_s	Cross-sectional area of conventional steel reinforcing bar in tension;
$B_{segment}$	segment width;
c	clear concrete cover;
CMOD	Crack Mouth Opening Displacement;
d	effective depth of the segment cross-section;
D_i	internal tunnel lining diameter;
D_i/h	tunnel aspect ratio;
E_c	concrete elastic modulus;
E_s	steel elastic modulus;
f_{cd}	design value of cylindrical compressive concrete strength;
f_{ck}	characteristic value of cylindrical compressive concrete strength;
$f_{ck,\ de\text{-}mould}$	characteristic value of cylindrical compressive concrete strength at de-moulding;
f_{cm}	mean value of cylindrical compressive concrete strength;
f_{ctd}	design value of tensile concrete strength;
f_{ctk}	characteristic value of tensile concrete strength;
$f_{ctk,0.05,de\text{-}mould}$	characteristic value of tensile concrete strength at de-moulding time;
f_{ctm}	mean value of tensile concrete strength;
f_{Fts}	serviceability residual strength (post-cracking strength for serviceability crack opening);
f_{Ftsd}	design value of post-cracking strength for serviceability crack opening;
f_{Ftsk}	characteristic value of post-cracking strength for serviceability crack opening;
f_{Ftu}	ultimate residual strength (post-cracking strength for ultimate crack opening);
f_{Ftud}	design value of post-cracking strength for ultimate crack opening;
f_{Ftuk}	characteristic value of post-cracking strength for ultimate crack opening;
f_L	limit of proportionality;
f_{Lm}	mean value of limit of proportionality;
$f_{R,j}$	residual flexural tensile strength of fibre reinforced concrete corresponding to CMOD=CMODj;
$f_{R,jd}$	design value of residual flexural tensile strength of fibre reinforced concrete corresponding to CMOD=CMODj;
$f_{R,jk}$	characteristic value of residual flexural tensile strength of fibre reinforced concrete corresponding to CMOD=CMODj;
$f_{R,jm}$	mean value of residual flexural tensile strength of fibre reinforced concrete corresponding to CMOD=CMODj;
$f_{t,fiber}$	mean value of ultimate tensile strength of fibres;
f_{yd}	design value of rebar yield strength;
f_{yk}	characteristic value of rebar yield strength;

3 >> List of symbol and abreviations

FRC	Fibre Reinforced Concrete;
h	Lining thickness;
$H_{overbunden,min,max}$:	tunnel overbunden (min-max);
L_f	fibre length;
L_f/ϕ_f	fibre aspect ratio;
$L_{segment}$	segment length;
l_{cs}	structural characteristic length;
M	applied internal bending;
M_{Ed}	design value of the applied internal bending moment;
M_{Rd}	design value of the bending resistance moment;
N	applied axial force (normal ring force);
N_{Ed}	design value of the applied axial force;
N_{jack}	total number of TBM jacks;
N_{key}	number of key segments;
N_{Rd}	design value of the axial resistance;
$N_{segments}$	number of segments;
NLFM	Non-Linear Fracture Mechanics approach;
R_{ck}	characteristic value of cubic compressive concrete strength;
R_{cm}	mean value of cubic compressive concrete strength;
RC	Reinforced Concrete (reinforced by traditional rebar);
SFRC	Steel Fibre Reinforced Concrete;
SLS	Serviceability Limit State;
SynFRC	Synthetic Fibre Reinforced Concrete;
s_{rm}	mean value of crack spacing;
TBM	Tunnel Boring Machine;
$T_{max,jack}$	maximum thrust exerted per jack;
$T_{max,total}$	total maximum thrust exerted by TBM jacks;
$T_{SC,jack}$	thrust exerted per jack, service condition;
ULS	Ultimate Limit State;
V	applied shear force;
V_{Ed}	design value of the applied shear force;
V_f	volume fraction of fibres;
V_{Rd}	design value of the shear resistance force;
w_u	maximum crack opening accepted in structural design;
ε_{SLS}	average tensile member strain at SLS, based on s_{rm};
ε_{ULS}	average tensile member strain at ULS, based on s_{rm};
γ_c	partial factor for concrete;
γ_f	partial factor for Fibre Reinforced Concrete;
ϕ_f	fibre diameter;
ρ_s	longitudinal reinforcing ratio (referred to steel reinforcing bars, steel rebars).

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5 >> Introduction

Tunnel segments are generally reinforced with conventional rebars that are placed in Reinforced Concrete (RC) elements to resist the tensile stresses both at Serviceability (SLS) and Ultimate Limit States (ULS).

In the last two decades, Fibre Reinforced Concrete (FRC) was used in many precast tunnel segments in combination or not with conventional rebars [1,2,3,4], as reported in Table 1. FRCs are composite materials with a cementitious matrix and a discontinuous reinforcement, the fibres, that may be made of metal, glass, synthetic or natural materials [5]. The reason for adding fibres to cementitious composites is principally to improve their tensile behaviour; for normal fibre dosages (corresponding to a volume fraction $V_f<1\%$) the concrete tensile strength is not affected by fibre addition, whereas the post-cracking residual strengths considerably enhance.

Table 1 collects case studies of tunnel linings made by FRC. Fourteen of these case histories are reported in more detail in Annex I. , Based on data collected in Table 1, possible general tendencies are analysed in Annex II.

FRC represents a competitive material for tunnel segmental lining for the following main reasons:

- fibres considerably improve the concrete post-cracking behaviour, defined as toughness in the following;
- fibre reinforcement enables a better crack control, especially in combination with traditional reinforcing bars. Hence, smaller crack openings are expected at SLS resulting in a considerable improvement of the durability of the structure;
- fibres give higher resistance to impact loading;
- the industrial production process is improved, since a partial or complete substitution of conventional rebars can be achieved, which means time reduction in handling and placing curved rebars. A considerable reduction or elimination of storage areas for traditional reinforcement can be obtained;
- fibre reinforcement is distributed everywhere in the segment, including the concrete cover which, in RC segments, often needs to be considerably thick for the fulfilment of the fire protection and durability requirements;
- durability is improved. The latter is generally associated to permeability, but it also depends on microcracks or cracks related to internal and external strain or deformations experienced by structures. In this regard, fibre reinforcement can considerably reduce the cracking phenomena and control permeability of concrete under stress;
- fibre reinforcement enhances a sustainable use of structural concrete, due to a low environmental impact and to the resulting mechanical performance of structures.

For structural purposes, Steel Fibre Reinforced Concrete (SFRC) is widely utilized, even though some types of structural synthetic fibres (macro fibres) are able to enhance concrete toughness in order to make it adequate for structural purposes [6,7]. In fact, any type of fibre that meets structural performance requirements with respect to Serviceability Limit State (SLS) and Ultimate Limit State (ULS) is considered suitable for structural applications.

In the case studies collected in Table 1, type of fibres and their contents are specified even if, as it will deeply explained in Sections 6 and 7, fibre dosage is not a complete information, since it does not provide any information on the post-crack residual performance required by the designer for a specific concrete application.

With the progressively growing use of fibre reinforcement in tunnel elements as well as in other structural elements (e.g., slabs at grade, track-slabs) several national standards were developed for the characterization of the post-cracking residual strengths exhibited by FRCs. Beside the quantification of toughness provided by fibre reinforcement, it is of paramount importance the definition of rules and recommendations for the design of FRC structural elements, since the classical design, based on the elastic approach, is acceptable only for ordinary RC structure. A proper design procedure that takes into account the significant residual tensile strength provided by fibres after cracking should be adopted in case of FRC structural elements. In particular, the cracking phenomena of concrete matrices containing fibres can be accurately represented by using an approach based on non-linear fracture mechanics (NLFM; Hillerborg et al., 1976, [8]). A similar simplified method is well described within the new fib Model Code 2010 [9]. The latter follows the publication of several guidelines concerning design of FRC elements (RILEM Guidelines, 2003, [10]; CNR-DT-204, 2006, [11]; DafStb Guideline, 2012 [12]).

The present document focuses on the structural behaviour of FRC precast tunnel linings and it was developed with a framework similar to that of the report titled “Guidelines for the design of shield tunnel lining”, Working Group 2 of ITA [13], which gives information regarding the design of precast tunnel elements reinforced by conventional rebars only.

The interest of the International Tunnelling Association (ITA) in developing this document is mainly to give specific recommendations which are adequate to FRC tunnel segmental lining and their specific design procedure. For this purpose, as it will be deeply explained in Section 6 and 7, among the existing standards regarding FRC elements, the standard more suitable for the design of FRC tunnel lining is certainly the recent Model Code 2010 [9]. Based on general approaches and criteria proposed by Model Code 2010 [9], the main goal of the present document is to offer general rules and procedures according to the particular requirements and loading conditions of segmental tunnel linings. Information or specific advices will be given with regard to analytical and numerical procedures necessary to quantify, during the design process, the beneficial effects due to fibre addition.

The scope of the document is to provide design principles which are specific for precast FRC tunnel lining, and which are based on the experiences acquired in the last two decades in various projects.

TUNNEL NAME	YEAR	COUNTRY	FUNCTION	DI (M)	H (M)	DI/H (-)	TYPE OF FRC	FIBRE CONTENT (KG/m³)	FIBRE VOLUME FRACTION (%)	REBARS USED
Metrosud	1982	Italy	Subway	5.8	0.30	19.3	SFRC	N.A.'	N.A.'	No
Fanaco	1989	Italy	Water Supply	3.0	0.20	15.0	SFRC	N.A.'	N.A.'	No
Heathrow Baggage Handling	1993	England	Service	4.5	0.15	30.0	SFRC	30	0.38%	No
Heathrow Express	1994	England	Railway	5.7	0.22	25.9	SFRC	30	0.38%	No
Napoli metro	1995	Italy	Subway	5.8	0.30	19.3	SFRC	40	0.51%	No
Lesotho Highlands	1995	South Africa	Water Supply	4.5	0.30	15.0	SFRC	50	0.64%	No
Hachinger	1998	Germany	Water Supply	2.2	0.18	12.2	SFRC	N.A.'	N.A.'	No
2nd Heinenoord	1999	Netherlands	Road	7.6	0.35	21.7	SFRC	N.A.'	N.A.'	No
Jubilee Line	1999	England	Subway	4.5	0.20	22.3	SFRC	30	0.38%	No
Trasvases Manabi (La Esperanza)	2001	Ecuador	Water Supply	3.5	0.20	17.5	SFRC	30	0.38%	No
Essen	2001	Germany	Subway	7.3	0.35	20.9	SFRC	N.A.'	N.A.'	No
Sorenberg	2002	Switzerland	Gas Pipeline	3.8	0.25	15.2	SFRC	40	0.51%	No
Canal de Navarra	2003	Spain	Water Supply	5.4	0.25	21.6	N.A.'	N.A.'	N.A.'	No
Oënzberg tunnel	2003	Switzerland	Railway	10.8	0.30	36.0	N.A.'	N.A.'	N.A.'	No
Oënzberg-TBM	2003	Switzerland	Railway	11.4	0.40	28.5	SFRC	30	0.38%	Yes
Oënzberg-Shield	2003	Switzerland	Railway	11.4	0.40	28.5	SFRC	60	0.76%	No
Barcelona Metro Lin 9 - Can Zam Stretch	2003	Spain	Subway	10.9	0.35	31.1	SFRC	60	0.76%	No
Channel Tunnel Rail Link (CTRL)	2004	England	Railway	7.2	0.35	20.4	SFRC	30	0.38%	No
Heathrow Express Extension (HexEx)	2005	England	Railway	5.7	0.22	25.9	SFRC	30	0.38%	No
Metropolitan Expressway Central Circular Shinjuku Route tunnel	2005	Japan	Road	10.9	0.45	24.2	SFRC	63	0.80%	Yes
San Vicente	2006	USA	Water Supply	3.2	0.18	17.8	SFRC	30	0.38%	No
Heathrow– SWOT	2006	England	Water Supply	2.9	0.20	14.5	SFRC	30	0.38%	No
Barcelona Metro Line 9 - Stretch I	2006	Spain	Subway	8.4	0.32	26.3	SFRC	30 and 25	0.38% and 0.32%	Yes
Lötschberg	2007	Switzerland	Temporary pilot	4.5	0.22	20.5	SFRC	N.A.'	N.A.'	No
Beacon Hill Tunnels	2007	USA	Road	6.7	0.30	22.3	N.A.'	N.A.'	N.A.'	No
Hofoldinger Stollen	2007	Germany	Water Supply	2.9	0.18	16.1	SFRC	40	0.51%	No
Madrid Metro	2007	Spain	Subway	8.4	0.30	28.0	SFRC	25	0.32%	Yes
Gold Coast Desalination Plant	2008	Australia	Water Supply	3.4	0.20	17.0	SFRC	35	0.45%	No
Big Walnut Sewer	2008	USA	Waste Water	3.7	0.23	16.1	SFRC	35	0.45%	Yes
Heathrow – PiccEx	2008	England	Subway	4.5	0.15	30.0	SFRC	30	0.38%	No
Heathrow Express Ext. Tunnel to T5	2008	England	Railway	5.7	0.22	25.9	SFRC	30	0.38%	No
Hobson Bay	2009	New Zealand	Waste Water	3.7	0.25	14.8	SFRC	40	0.51%	No
Sao Paulo Metro Line 4	2009	Brazil	Subway	8.4	0.35	24.0	SFRC	35	0.45%	No
Copenhagen District Heating Tunnel	2009	Denmark	Water Supply	4.2	0.30	14.0	SFRC	35	0.45%	No
Docklands Light Railway (DLR) Extension	2009	England	Railway	5.3	0.25	21.2	SFRC	N.A.'	N.A.'	No
Harefield Gas Tunnel	2009	England	Gas Pipeline	2.6	0.18	14.4	SynFRC	7	0.78%	No

TUNNEL NAME	YEAR	COUNTRY	FUNCTION	DI (M)	H (M)	DI/H (-)	TYPE OF FRC	FIBER CONTENT (KG/m^3)	FIBER VOLUME FRACTION (%)	REBARS USED
Malaga Rail Tunnel	2009	Spain	Railway	8.4	0.32	26.3	SynFRC	5	0.56%	Yes
Fontsanta–Trinitat Interconnection	2010	Spain	Water Supply	5.2	0.20	26.0	SFRC	25	0.32%	Yes
Clem Jones - Clem 7	2010	Australia	Road	11.2	0.40	28.0	SFRC	37	0.47%	Yes
Ems-Dollard Crossing	2010	Germany-Netherlands	Gas Pipeline	3.0	0.25	12.0	N.A.'	N.A.'	N.A.'	No
City West Cable Tunnel (CWCT)	2010	Australia	Power Cable	2.5	0.20	12.5	N.A.'	N.A.'	N.A.'	No
Adelaide Desalination Plant	2010	Australia	Water Supply	2.8	0.20	14.0	SFRC	35	0.45%	No
FGC Terrassa	2010	Spain	Railway	6.0	0.30	20.0	SFRC	25	0.32%	Yes
Keio line	2010	Japan	Railway	6.7	0.30	22.3	SFRC	63	0.80%	Yes
Brightwater East	2011	USA	Waste Water	5.1	0.26	19.6	SFRC	35	0.45%	No
Brightwater Central	2011	USA	Waste Water	4.7	0.33	14.2	SFRC	40	0.51%	No
Brightwater West	2011	USA	Waste Water	3.7	0.26	14.2	SFRC	35	0.45%	No
East side CSO	2011	USA	Waste Water	6.7	0.36	18.6	SFRC	32	0.40%	No
Izumi-Otsu	2011	Japan	Water Supply	1.8	0.13	14.4	SFRC	32	0.40%	Yes
Metropolitan Expressway	2011	Japan	Road	13.4	0.45	29.8	SFRC	47	0.60%	Yes
Victorian Desalination Plant	2011	Australia	Water Supply	4.0	0.23	17.4	N.A.'	N.A.'	N.A.'	No
Monte Lirio	2012	Panama	Water Supply	3.2	0.25	12.8	SFRC	40	0.51%	No
Pando	2012	Panama	Water Supply	3.0	0.25	12.0	SFRC	40	0.51%	No
Airport Link	2012	Australia	Road	11.4	0.40	28.4	SFRC	35	0.45%	No†
Midosuji Utility	2012	Japan	Utility	5.1	0.15	33.8	SFRC	32	0.40%	Yes
Sagami Line	2012	Japan	Road	11.8	0.50	23.6	SFRC	47	0.60%	Yes
El Alto	2013	Panama	Water Supply	5.8	0.35	16.6	SFRC	40	0.51%	No
Asada Trunk Line	2013	Japan	Sewage	4.6	0.20	22.8	SFRC	25	0.32%	Yes
Koishikawa Kasen	2013	Japan	Railway	6.7	0.30	22.3	SFRC	47	0.60%	Yes
Oi-Ariake Cable	2013	Japan	Power Cable	4.0	0.20	20	SFRC	32	0.40%	Yes
Wehrhahn	2014	Germany	Subway	8.3	0.30	27.7	SFRC	30	0.38%	No
STEP Abu Dhabi Lot T-02	2014	UAE	Waste Water	6.3	0.28	22.5	SFRC	30	0.38%	Yes
San Francisco Central Subway	2014	USA	Railway	5.4	0.28	19.3	SFRC	30	0.38%	Yes
Legacy Way	2015	Australia	Road	11.3	0.35	32.3	SFRC	40	0.51%	No†
Metropolitan Expressway	2015	Japan	Road	12.3	0.40	30.8	SFRC	47	0.60%	Yes
Abu Hamour	2016	Qatar	Water Drainage	3.7	0.25	14.8	SFRC	40	0.51%	No
Doha Metro Red North Line	2016	Qatar	Subway	6.17	0.30	20.6	SFRC	40	0.51%	No
Abatemarco	-	Italy	Water Supply	3.5	0.20	17.5	SFRC	40	0.51%	No
Public Sewage	-	Japan	Sewage	5.6	0.18	32.0	SFRC	43	0.55%	Yes
Lee Tunnel Sewer	U.C.*	England	Waste Water	7.2	0.35	20.6	N.A.'	N.A.'	N.A.'	No
Downtown Line 3	U.C.*	Singapore	Subway	5.8	0.28	21.1	SFRC	40	0.51%	No
Thomson Line	U.C.*	Singapore	Subway	5.8	0.28	21.1	SFRC	40	0.51%	No
Crossrail	U.C.*	England	Railway	6.2	0.30	20.7	SFRC	30-40	0.38% and 0.51%	No

' N.A.: Not available.
** U.C.: Under construction.*
† Fibre reinforcement solution used for the vast majority of the alignment. Hybrid solution used only in highly loaded tunnel sections and at the location of cross-passages.
Table 1. Tunnel excavated with TBM and supported by FRC precast segments.

6 >> Scope

In the last two decades, the design of FRC tunnel linings collected in Table 1 was possible because of the development of national or international standards concerning principally two issues:

- the quantification of post-cracking residual tensile strength exhibited by FRC. This strength can be considered as a typical “fracture parameter”, which enable to quantify the toughness provided by FRC as a composite material made by concrete matrix and fibre reinforcement;
- the design of FRC structural elements based on post-cracking residual strength.

In Section 7, the current most important standards are listed. Even if several standards are available for the design of FRC elements, no specific rules or recommendations are addressed for the specific design of tunnel lining.

In fact, the current Model Code 2010 [9,14], which is well known as the reference international standard for the design of FRC structural elements, provides general design rules. The latter can be easily applied for typical structures such as beams or slabs, but they need to be contextualized to the specific issues concerning tunnel lining elements.

Tunnel curved elements are, for instance, characterized by a temporary loading condition during the excavation of the tunnel, where considerable stresses are introduced in the lining due to high concentrated forces exerted by the TBM’s hydraulic jack (TBM thrust phase). The noticeable enhancement of post-cracking residual strengths due to fibres addition can be exploited during this stage even if no specific recommendations are included in Model Code 2010 [9] for this purpose.

Moreover, the stresses developing in tunnel segments during several temporary conditions (TBM thrust phase, handling and storage) are quite different from those arising in the final state, where the lining is mainly loaded by the surrounding ground. This issue is typical of precast elements and required special attention particularly when tunnel segments are reinforced by fibre reinforcement only. Specific rules need to be suggested for this purpose, for instance, starting from the general criteria already well introduced by Model Code 2010. Finally, in Section 8, some examples of typical problems encountered by designers will be described. Some of these issues are related to specific loading conditions (e.g., introduction of irregularities during the TBM thrust phase) that cannot be easily covered by a general purpose standard such as MC 2010.

The Model Code 2010 [9] is a remarkable standard which introduces the fundamental concept of post-cracking residual flexural strength ratios as parameters for classifying FRC as a composite. The fibre type or content are not complete information for design purposes; the key-factor is the quantification of toughness that a given fibre content or type of fibre can provide in a specific concrete matrix. However, as mentioned before, this performance approach is only the basis of a necessary and specific design procedure for precast tunnel segments. This leads to develop specific basis of design, design principles and recommendations for covering the gaps that reasonably remains in a general purpose standard.

Among the already existing available specific recommendations for the design of FRC precast tunnel elements, the following documents can be mentioned:

- the recommendations of AFTES (Association Française des Tunnels et de l’Espace Souterrain), concerning the design, dimensioning and execution of precast steel fibre reinforced concrete arch segments [15];
- the recommendations of DBV (Deutscher Beton und Bautechnik-verein e.v., German society for concrete and construction technology) concerning design principles of steel fibre reinforced concrete for tunnelling works [16];
- the recommendations of DAUB [17], (Deutscher Ausschuss für unterirdisches Bauen e. V., German Tunnelling Committee), concerning the design, production and installation of segmental rings.
- the ACI report 544.7R-16 on FRC tunnel segments [18], that has been recently approved.

The AFTES recommendation [15] is focused on the use of steel fibres as reinforcement for segmental tunnel lining. The document reports some basic concepts of the current Model Code 2010 [9], but it does not go into sufficient details on aspects such as specific boundary conditions and irregularities to be considered in the design stage in order to properly evaluate the possibility of partially or totally substituting traditional reinforcement with fibres.

The recommendations of German society for concrete technology [16] are focused, on steel fibre reinforcement only. The more recent recommendations of German Tunnelling Committee [17] provide several useful information concerning the design of precast tunnel segments reinforced with traditional rebars; nevertheless, only few sections are dedicated to SFRC precast tunnel segments.

The ACI 544.7R-16 report will complete the current general report on FRC from ACI-544 [19]. It worthwhile noticing that also a fib WP 1.4.1 is currently working on FRC for use in tunnel segments with the main scope of providing a detailed design procedure for FRC precast linings based on on-going research or recently developed research.

Within this framework, the needed to shed some new light on general rules and procedures regarding FRC precast tunnel segments is considerable. The scope of this document is basically to take benefits from the already developed experiences or already well known research evidences for providing suggestions and advices useful for the community of designers/contractors of FRC linings. This ITA document is not overlapping the work of ACI and fib groups, which are mainly focused on detailed design procedures (both) and on research issues still under discussion (fib WP 1.4.1).

7 >> Identification of applicable standards and guidelines

The main standards applicable for the evaluation of FRC post-cracking residual strengths are reported in the following :

(a) EN-14651, "Test method for metallic fibre concrete - Measuring the flexural tensile strength (limit of proportionally (LOP), residual)", European Committee for Standardization EN 14651 [20];

(b) ASTM C1609/C1609M, "Standard Test Method for Flexural Performance of Fibre Reinforced concrete (Using Beam With Third-Point Loading)", American Standard [21];

(c) ASTM C1399/C1399M, "Standard Test Method for Obtaining Average Residual Strength of Fibre Reinforced Concrete", American Standard [22];

(d) ASTM C1550/1550M, "Standard Test Method for Flexural Toughness of Fibre Reinforced Concrete (Using Centrally Loaded Round Panel)", American Standard [23];

(e) JCI-SF4, "Method of Tests for Flexural Strength and Flexural Toughness of Fibre Reinforced Concrete", Japanese Standard [24];

(f) DIN 1045-2, Concrete, reinforced and prestressed concrete structures - Part 2: Concrete - Specification, properties, production and conformity - Application rules for DIN EN 206-1, ANNEX O in conjunction with [12], German Standard [25].

A reference standard for toughness characterization of FRCs is the international European standard EN 14651 [20], since the new fib Model Code 2010 [9] requires that nominal values of the material properties (residual post-cracking strengths, see Section 10.1) can be determined by performing a 3-point bending test on a notched beam according to [20]. However, other tests can be accepted if correlation factors with the parameters of EN 14651 [20] are proven. Accordingly, the standard test proposed by ASTM (American Standard Test and Methods, [22], [23], [24]) are well known and acceptable, even if correlation factors are necessary for using them in the design process of FRC elements according to Model Code 2010 [9]. A similar well known standard is proposed by the Japanese Concrete Institute [24]. Finally, the annex dedicated to SFRC of German standard, DIN-1045-2 [25] is cited, since fracture parameters proposed by this document are adopted by the correspondent German standard for the design of FRC elements [12].

The recommended standard for the design of FRC structures is, as already mentioned, the current new fib Model Code 2010 [9], which has introduced post-cracking residual flexural strength ratios as key parameters for classifying FRCs (see Section 10.1). Nevertheless, within the European countries, other acceptable standards are the following:

(a) RILEM TC 162-TDF, "Test and design methods for steel fibre reinforced concrete. Design with σ-ε method", first international recommendations about the design of FRC elements [10];

(b) CNR-DT-204, "Guidelines for design, construction and production control of fibre reinforced concrete structures", Italian Standard [11];

(c) DafStb Guideline "Steel fibre reinforced concrete", Design and construction; specification, performance, production and conformity; execution of structures, German standard [12].

RILEM TC 162-TDF [10] is the first important document concerning the design of FRC elements. The CNR-DT-204 [11] was one of the first national standard based on a performance approach similar to that of current Model Code. The German standard [12] was cited since it is a recent national document about the design of FRC structures based on performance approach. It concerns steel fibre reinforcement only and requires that the FRC's post-cracking properties should be determined by performing a 4-point bending test on a beam without notch.

An annex on design of FRC elements is under preparation within Eurocode 2 [26].

The reinforcement used in precast tunnel segments can be of three types:

- RC: segment reinforced only by conventional rebars;
- FRC: segment reinforced only by fibre reinforcement;
- RC+FRC: segment reinforced by a combination of fibre and traditional reinforcement (also called hybrid-solution).

The use of Fibre Reinforced Concrete in tunnel segments allows for several advantages such as the improvement of post-cracking tensile behaviour and the correspondent better control of flexural cracks. Fibres enable a stable development of splitting cracks, a possible reduction of stirrups placed in the regions under the thrust jacks [27,28] and a possible complete replacement of shear reinforcement [7,29].

In FRC tunnel elements both localized and diffused stresses are generally present. Localized stresses are better resisted by localized reinforcement such as traditional steel reinforcing bars (rebars), while diffused stresses (e.g.: splitting stresses) are better resisted by spread reinforcement such as fibres.

The stress level is generally high during the application of TBM's thrust jacks. Regarding the local-behaviour under TBM thrust shoes, it was proven that, since the cracks due to splitting stresses remain relatively small, also FRCs having post-cracking properties corresponding to class "2e" (according to Model Code 2010, see Section 10.1) can probably be satisfactory in terms of local splitting behaviour under the loading areas [28]. Nevertheless, high localized stresses (e.g., bending stresses) can occur if some irregularities take place during the tunnel construction process (e.g., eccentricity of jacks, gaps between rings, etc.). The probability of occurrence and the extent of these irregularities directly govern the amount of localized stresses in segments and, consequently, the possible complete or partial substitution of traditional rebars by means of fibre reinforcement.

One of the first example of this possibility was the Barcelona Metro Line 9 project (see Annex A.I.1); a total of thirty rings reinforced only by steel fibres were constructed and installed. The ratio of internal diameter over thickness (D_i/h, tunnel aspect ratio, see Table 1) of this lining was 31.1 and its internal diameter D_i, equal to 10.9 m (note that the average segment aspect ratio is 13.88, see Annex A.I.1). During the construction of these rings some splitting cracks and local failures appeared. By means of numerical simulations [47], it was proven that some irregularities (e.g., eccentricities) probably occurred during this phase. In fact, by considering an ideal loading condition (with jacks and bearing pads perfectly placed) the solution with fibre reinforcement only could be generally satisfactory; on the contrary, by considering irregularities, cracks were expected. On the other hand, it can be noticed that the case study of Legacy Way (see Annex A.I.10, D_i/h=32.3) presents geometrical dimensions similar to Barcelona; nevertheless, in the major part of the alignment a solution with fibre reinforcement only was used. A hybrid reinforcement solution (RC+SFRC) was used only in highly loaded sections of tunnel and also at the cross-passage locations. This possible design approach will be briefly introduced in Section 9.

Irregularities during the TBM thrust phase occur also in RC tunnel segments but it has been understood that traditional rebars can cope with the corresponding higher localized stresses. In other words, it seems that FRC tunnel segments are probably more vulnerable to these conditions.

Traditional RC precast tunnel segments are generally characterized by a rather low longitudinal steel ratio (ρ_s ≅0.2-0.3% whereas in typical beam elements ρ_s is around 1%) since, at final stage (lining embedded in the ground), the lining is loaded by a favorable axial compressive normal force and it is supported by the ground. The soil-structure system interacts in a highly redundant manner. Hence, possible design solution of segments with fibre reinforcement only, generally presents a satisfactory resistant bending moment at ULS, even if it is rather lower than that of traditional RC configuration. Accordingly, it is generally fundamental to control the tunnel flexural demand during temporary phases (such as de-moulding, storage and positioning by erector or general handling to the project site) as well as in the final stage. For instance, Gettu et al. [30], [31] suggest to pay more attention to the position of the segments on supports during storage by reducing eccentricity between supports, especially in presence of segments reinforced by fibres only. In fact, these segments, under bending conditions (such as storage), after cracking present a softening branch and not a hardening behaviour, as occurs in RC segments; hence, unfavourable situations characterized by misalignment of supports should be limited. Similarely, recommendations are included in recent ACI 544.7R-16 report [18], where a typical maximum acceptable eccentricity between supports is also reported. Noteworthy is that a reduction of bending moments in tunnel segments (in presence of temporary and permanent loading conditions) can be directly achieved by controlling the segment aspect ratio.

The need of a more precise estimation of the flexural demand required to tunnel segments (basically the ratio between the applied bending moment and axial force) according to the specific loading conditions of a given tunnel project clearly emerges from experiences reported in Annex I (A.I.4). Among the three possible solutions previously mentioned, Dobashi et al. [32,33] successfully adopted a hybrid solution (RC+FRC) for a lining having a tunnel aspect ratio (D_i/h) of 24.2, with a large internal diameter of 10.9 m. The original RC tunnel segment solution was heavily reinforced, probably because of a high flexural demand. On the contrary, Caratelli et al. [34] fruitfully use only fibre reinforcement for a lining having a much smaller internal diameter (only 3.2 m) and a tunnel lining aspect ratio of 12.8 (see Annex A.I.8, Monte Lirio tunnel).

Based on several experiences already available in literature for typical structural elements (e.g., beams or slabs, see [14]), the fibre content is not a complete information from a structural point of view, as well emphasised by the performance approach reported in the Model Code 2010 [9]. The higher toughness provided by fibre addition in a concrete matrix is described by residual post-cracking strengths [9]. Referring to

FRC precast tunnel segments, an early experience based on this performance approach is reported in Annex I (see Annex A.I.8, Monte Lirio tunnel lining designed according to Model Code 2010, having an average segment aspect ratio of 7.75 and having a minimum required design FRC class of “4c”, according to Model Code 2010).

Even if it is not the purpose of this document, which is oriented on design aspects, some technological aspects related to the need to consider FRC as a composite material can be briefly mentioned. FRC is not simply produced by adding dispersed fibre reinforcement to a concrete matrix: a specific mix-design should be adopted by taking into account workability issues as well as the opportune system of concrete vibration. Referring to the segment production, the experiences of Kasper et al. [35] and King et al. [36] reported in A.I.5 and A.I.6, respectively, can be quoted. In both cases, a solution with fibre reinforcement only is adopted and, in both projects, initial problems concerning early production occurred (e.g., small cracks or tendency of SFRC to show higher risk to honeycomb and blow hole formation compared to conventional concrete). By means of a progressive optimization of the production technique [35] or mix design [36], FRC tunnel segments were satisfactory produced and FRC properties were exploited by achieving a general improvement of durability.

The following main basic criteria should be considered:

- Fibres considerably improve the concrete post-cracking tensile behaviour, providing significant residual strength, generally defined as toughness.
- The ability to resist internal crack propagation (generally defined as toughness) is a property of FRC considered as a composite in order to allow a performance–based-design. Model Code 2010 [9] and some other relevant codes [10, 11, 12] provide classes based on FRC post-cracking residual strengths. This concept will be introduced in details in Section 10 with reference to Model Code 2010.
- Similar to traditional RC structures, designers can start to develop their calculations by assuming a certain FRC class which it will become the minimum requirement to be checked in the production of FRC segments. The required FRC class can be verified by means of preliminary tests on small specimens (according to EN 14651, [20]). Then, conformity tests should be properly planned during the production of tunnel segments.
- The post-cracking residual strength exhibited by FRC should be included in the design approach by one of the following two non-linear methods: - analytical approaches for evaluating the behaviour at ULS and SLS of a tunnel segment in terms of typical beam actions (N, M, V, Section 11.2); - numerical approaches for evaluating the local and global behaviour of tunnel segments during crucial loading conditions such as the introduction of local forces exerted by TBM rams (Section 11.1 and 12.5).

Experimental tests on full-scale tunnel elements (flexural and point load tests) are useful tools for proving the design approach.

- Particular attention should be devoted to the evaluation of lining internal actions during annular gap grouting and at the final stage, as well as during demoulding and storage, in order to properly determine the maximum bending moment generated in the lining. In fact, the flexural demand in tunnel segments is a key-point for evaluating the possibility to completely substitute traditional rebars with fibre reinforcement. For this purpose, the role of modelling is fundamental as described in Section 12.7 and 12.8.
- Particular attention should be given to modelling the force exerted by TBM during the construction process. As already mentioned, if some irregularities occur, a design solution with fibre reinforcement only cannot be satisfactory. This loading condition should be analysed at local and global scale. Designers should be aware that some special loading conditions should be considered in the design process by using the methods mentioned above for including FRCs post-cracking properties. In this way, by assuming certain irregularities, designers can prescribe the necessary boundary conditions (e.g., maximum admissible eccentricity) associated to a certain segment configuration (FRC only or RC+FRC). The respect of these requirements conditions are important for the use of a specific reinforcement configuration of tunnel segments.
- Attention should be given to the frequency of occurrence of the mentioned irregularities, which can be associated to particular conditions of steering the TBM (e.g., minimum curvature radius, certain mixed face conditions, etc.). If, together with contractors, designers are able to accurately predict the most probable frequency of occurrence, they can provide appropriate design solutions (e.g., higher amount of fibre reinforcement, combination with steel reinforcing bars, …). For instance, a specific hybrid solution could be used in specific alignment sections, while using fibre reinforcement only in the rest of the alignment.
- In order to prove the beneficial effects of fibres in terms of crack control (hence, improvement of durability of the structure at SLS) proper approaches should be applied (Section 12.8); otherwise one of the main advantages of using fibres in tunnel lining cannot be demonstrated and used. Eventually it is worthwhile noticing that only fibers that satisfy the following requirements are generally suitable for use in concrete in conjunction with Model Code 2010 [9]:

Finally, only fibres that satisfy the following requirements are generally suitable for use in concrete, in conjunction with Model Code 2010 [9]:

- The fibre material shall not deteriorate in concrete over time nor shall the material show time dependent or thermo-hygrometrical phenomena (see Model Code 2010, chapter 5.6.1), both in uncracked and cracked state. The fibre material shall not deteriorate under storing conditions recommended by the supplier;
- The material properties of the final composite shall not suffer from negative effects induced by the added fibres unless these effects are acceptable for the intended use and unless these effects are especially accounted for in the design and construction of the structural element. This requirement applies to the properties of fresh concrete (workability, air content, homogenity of materials, etc.), hardened concrete strength and deformation properties (compressive strength, splitting tensile strength, static modulus of elasticity, creep and shrinkage behaviour, bond to reinforcing steel, etc.), and durability properties (carbonation and ion/water-transport properties determining steel and concrete corrosion risk, frost resistance, abrasion);
- The fibre material itself or additional substances applied to or with the fibres (e.g. spin finish, glues) shall not affect the corrosion resistance of steel being present in the concrete (reinforcing steel, tendons or – in case of fibre cocktails – steel fibres);
- The fibres have to resist mixing, transport and processing without a significant change of their properties;
- The fibre material must be suitable for the intended climatic conditions;
- The fibre material and the manufacturing process must be suitable to achieve tolerances on the fibres' properties which are acceptable for structural uses;
- Fibres shall satisfy the existing requirements on environmental compatibility and physiological harmlessness. In addition, safety risks shall be reduced to the minimum possible.

10 >> FRC PROPERTIES

In Sections 6 and 7, it was already underlined that concrete is characterized by a brittle behaviour in tension. Consequently, the main reason for adding fibres to cementitious matrices is to enhance the tensile behaviour after cracking, in terms of increase of residual tensile strength and ductility. In fact, after concrete cracking, fibres bridge the crack and allow to transmit higher forces between the crack planes.

FRC is a composite material and not the simple addition of fibres to a concrete matrix.

Post-cracking concrete residual properties (toughness) depend on fibre characteristics (such as material, shape, aspect ratio), quantity (usually expressed by the volume percentage), orientation as well as on the properties of the cementitious matrix surrounding the fibres.

Fibre reinforcement has little effect on the elasticity modulus, Poisson's ratio, compressive strength, electrical conductivity and porosity. Therefore, the concrete behaviour before cracking is not significantly modified by fibres.

The contribution of fibres lasts when they are either pulled out or broken. Fibre pull-out is the mechanism that is looked for in the post cracking phase because it allows higher energy dissipation and better post-cracking residual properties. Consequently, the fibre strength should be selected taking into consideration the concrete strength in order to favour fibre pull-out rather than rupture and to achieve the best possible fibre contribution and performance.

In order to determine the post-cracking behaviour of FRC in tension, various test methods are possible.

Uniaxial tensile tests enable to directly evaluate the post-cracking tensile behaviour (softening or hardening) but are quite difficult to perform. Design recommendations are usually based on bending tests that are able to analyse the flexural response of cementitious composites after cracking. As stated by several authors [37,38], after reaching the initial crack, a flexural softening or hardening behaviour will occur, depending on the type and amount of fibre reinforcement, as well as on the concrete matrix. Figure 1 schematically shows the enhanced post-cracking bending behaviour of FRC due to the addition of fibres.

As already mentioned in Section 7, materials having a noticeable post-cracking uniaxial tensile behaviour can globally present a flexural hardening. It is obvious that concretes presenting a uniaxial tensile hardening will exhibit a flexural hardening. In the latter case, the toughness exhibited by the composite is so high that, after reaching the tensile strength (f_{ctm}), a micro-cracking process occurs corresponding to tensile stresses higher than f_{ctm}. For higher applied loads, a macro-crack form. Generally, for these high performance cementitious composites it is recommended to develop bending tests on un-notched specimens in order to better capture also the micro-cracking stage [39].

Since FRCs for use in tunnel segments generally present a uniaxial tensile softening behaviour, applicable standards for the characterization of the FRC's toughness can be based on notched specimens or un-notched specimens under bending (see Section 7).

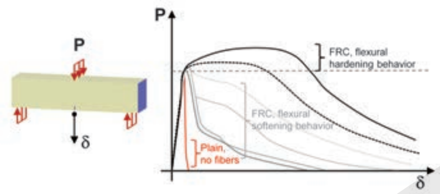


Figure 1. Bending performance of plain concrete and FRC.

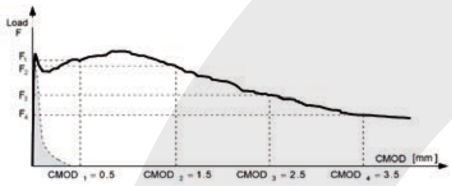


Figure 2. Typical load F – CMOD curve for FRC (EN 14651, 2005).

Among several standards, EN 14651 [20] defines the beam testing standard that the Model Code 2010 [9] refers to, where nominal values of the material properties are determined by performing a 3-point bending test on a notched beam.

Figure 2 shows a typical diagram of the applied force (F) versus the deformation which is expressed as Crack Mouth Opening Displacement (CMOD).

Parameters, f_{Rj}, representing the residual flexural tensile strength (fracture nominal parameters), are evaluated from the F-CMOD relationship, as follows:

$$f_{R,j} = \frac{3F_j l}{2bh_{sp}^2} \quad \text{(Eq. 1)}$$

where:

- f_{Rj} [MPa] is the residual flexural tensile strength corresponding to CMOD = CMODj;
- F_j [N] is the load corresponding to CMOD = CMODj;
- l [mm] is the span length;
- b [mm] is the specimen width;
- h_{sp} [mm] is the distance between the notch tip and the top of the specimen (125 mm).

Other tests can be accepted if correlation factors with the parameters of EN 14651 [20] are proven; among them, the most commonly used and acceptable are reported in Section 7.

10.1 CLASSIFICATION

The residual post-cracking strength represents, for designers, the performance parameters that should be satisfied. During the production of precast FRC tunnel segments, this parameter should be continuously verified through conformity tests. In order to classify the post-cracking strength of FRC, the characteristic residual strength significant for serviceability (f_{R1k}) and ultimate (f_{R3k}) conditions is considered; in particular, two parameters, namely:

- the strength interval f_{R1k} defined by two subsequent numbers in the series: 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0 [MPa];
- the residual strength ratio f_{R3k}/f_{R1k}, represented by a letter (a, b, c, d or e) as follows:

a if $0.5 \leq f_{R3k}/f_{R1k} \leq 0.7$;

b if $0.7 \leq f_{R3k}/f_{R1k} \leq 0.9$;

c if $0.9 \leq f_{R3k}/f_{R1k} \leq 1.1$;

d if $1.1 \leq f_{R3k}/f_{R1k} \leq 1.3$;

e if $1.3 \leq f_{R3k}/f_{R1k}$.

The designer has to specify the residual strength class, the ratio f_{R3k}/f_{R1k} as well as the material of the fibre (Figure 3).

According to fib Model Code 2010 [9], fibre reinforcement can substitute (also partially) conventional reinforcement at ultimate limit state, if the following relationships are satisfied:

$f_{R1k}/f_{Lk} > 0.4$; (Eq. 2)

$f_{R3k}/f_{R1k} > 0.5$. (Eq. 3)

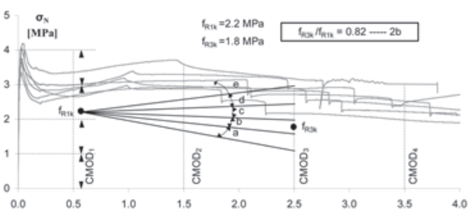


Figure 3. Typical load F – CMOD curve for FRC with the material classification.

10.2 Constitutive laws at Ultimate Limit State (ULS)

A stress-crack opening law in uniaxial tension is defined for the post-cracking behaviour of FRC.

Two simplified stress-crack opening constitutive laws, adequate for calculations at ULS, may be determined from the bending test results. The first concerns a plastic rigid behaviour, while the second proposes a linear post-cracking behaviour (hardening or softening), as schematically shown in Figure 4, where f_{Fts} represents the serviceability residual strength, defined as the post-cracking strength for serviceability crack openings, and f_{Ftu} represents the ultimate residual strength [9].

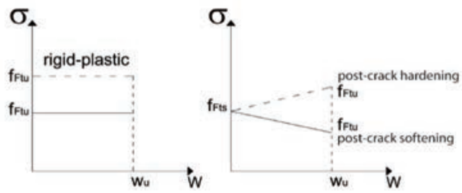


Figure 4. Simplified post-cracking constitutive laws: stress-crack opening (continuous and dashed lines refer to softening and hardening post-cracking behaviour respectively, MC 2010, 2012).

Rigid-plastic post-cracking model

The rigid-plastic model identifies an unique reference value, f_{Ftu}, based on the ultimate behaviour. Such a value is determined as:

$$f_{Ftu} = \frac{f_{R3}}{3} \Rightarrow f_{Ftu,d} = \frac{f_{R3,d}}{3} \quad \text{(Eq. 4)}$$

Linear post-cracking model

The linear model identifies two reference values, namely f_{Fts} and f_{Ftu}. They are defined through residual values of flexural strength by using the following equations:

$$f_{Fts} = 0.45 f_{R1} \quad \text{(Eq. 5)}$$

$$f_{Ftu} = f_{Fts} - \frac{w_u}{CMOD_3}(f_{Fts} - 0.5 f_{R3} + 0.2 f_{R1}) \geq 0$$

(Eq. 6)

where w_u is the maximum crack opening accepted in structural design; its value depends on the ductility requirement. Based on the characteristic values of residual strengths (f_{R3k} and f_{R1k}), $f_{Fts,k}$ and $f_{Ftu,k}$ can be determined: the corresponding design values are obtained by diving by a safety factor (γ_f).

10.3 Constitutive laws at Serviceability Limit State (SLS)

The study of the tunnel lining behaviour at SLS can be developed, according to Model Code 2010 [9], by adopting the law schematically depicted in Figure 5. This law refers to the characteristic FRC fracture parameters. For numerical analyses, more advanced constitutive laws are recommended. Further details on the design rules for FRC elements can be found in the new fib Model Code [9].

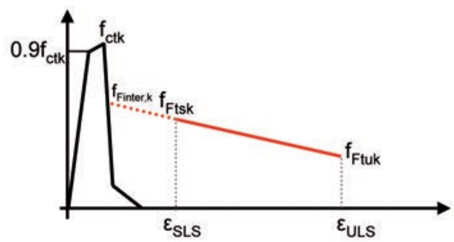


Figure 5. Constitutive law at SLS proposed by MC 2010.

The uniaxial tensile constitutive laws (both, ULS and SLS) should be expressed in terms of stress (σ) vs. strain (ε) by introducing the “structural characteristic length, l_{cs}” of the structural element [9, 39], which is a parameter used for easily connect continuous mechanics, governed by a stress-strain constitutive relationships, and fracture mechanics, governed by a stress-crack opening law. In elements with longitudinal rebars (RC or RC+FRC), l_{cs} corresponds to the “mean crack spacing, s_{rm}”, namely the most probable distance between two consecutive cracks according to current formulations available [9, 10]. It can be noticed that s_{rm} is used when typical plane sectional approaches are adopted in structural analyses [9, 39]. In case of FRC structures (fibre reinforced only), within the framework of simplified sectional analyses, MC 2010 [9] suggested to use, as a first approximation, the beam depth as a kind of reference l_{cs}.

When using finite element models (FEMs), it is necessary to introduce a specific internal length, which depends on the cracking model implemented in the program.

The design process of segmental tunnel lining is principally governed by several load conditions which occur during segment manufacturing, transportation, tunnel excavation, lining construction and final service stage. These loading conditions can be summarized as follows (in chronological order according to the construction process):

(1) de-moulding of tunnel segments;

(2) storage of segments;

(3) transportation of segments;

(4) positioning of segments by erector;

(5) thrust jack forces from TBM;

(6) introduction of normal ring force in longitudinal joint;

(7) ring behaviour of the tunnel lining during grouting process;

(8) ring behaviour of the tunnel lining embedded in the ground;

(9) ring behaviour during special event such as fire, explosion, earthquake (fire event only will briefly discussed in this document).

It is fundamental to observe that loading conditions (1)→(4) and (7)→(9) can be well analysed by means of ring models and bedded-beam models, respectively; hence, the internal lining actions can be described by typical beam actions: bending moment (M), shear force (V) and axial normal ring force (N). On the other hand, loading conditions (5) and (6) require more complicated numerical or analytical approaches. In Section 11.1 some general design aspects and issues concerning FRC tunnel linings will be introduced, especially with regard to the thrust jack phase. In Section 11.2, a similar overview will be briefly reported for all the other loading conditions. Finally, in Section 12, a possible general design procedure of FRC segmental lining will be proposed by analysing step-by-step the previously mentioned basic loading conditions.

11.1 GENERAL DESIGN ASPECTS CONCERNING THRUST JACK PHASE

Although the application of the thrust jack forces is a temporary loading condition during construction, it may govern the structural size as well as the final stage.

The TBM is pushed forward by thrust jacks acting on the last placed lining ring to compensate the friction forces on the shield and the ground and water pressure in the front and around the boring machine. Generally, designers study this temporary stage by referring to an ideal loading condition where the thrust jacks are perfectly placed and also the support pads on the face of the segments are perfectly located. Since different thrust jack configurations can be used (see French, German and Japanese standards / recommendations), the stress concentrations in the tunnel lining may take place differently.

The analysis of this load condition should be developed according to local and global scale. In the study of the global behaviour, it is recommended to consider the actual arrangement of rams of the selected TBM and to consider the effect of tolerances on those positions, and to conduct the study according to the following principles:

- study of the possible boundary conditions that could interest the tunnel segments during this phase;
- choose a realistic load scheme;
- choose of an appropriate resistant mechanism;

On the other hand, the local behavior under the loading thrust jacks should be carefully taken into account.

As far as the global behaviour of the segment under the forces exerted by TBM thrust jacks is concerned, simplified 2D models can be adopted. They are generally based on the following hypotheses:

- uniform distributed supports on the tunnel segment (ideal smooth support);
- bi-dimensional analysis;
- simplified resistant mechanism: strut-tie model, as shown in Figure 6 (that is generally quite conservative).

By using Finite Element Analyses (FEA), it is possible to study the segment global behaviour in more details (Figure 7). In fact, nonlinear FEA enables:

- to consider the concrete behaviour after the first cracking;
- to estimate the stresses redistribution in statically undetermined structure;
- to estimate the resistace contribution coming from FRC. This aspect can be considered only if non-linear material properties are included in the analyses (NLFM, Non Linear Fracture Mechanics);
- to investigate the real tri-dimensional behaviour of the tunnel segment as as a curved element.

FEA models based on NLFM allow to predict the maximum bearing capacity of the FRC segments as well as the cracking behaviour under service loads. Nowadays, several finite element programs enable to analyse concrete behaviour after cracking by means of typical local approaches such as the smeared cracking. These programs also enable to estimate the crack widths expected in tunnel segments during the TBM thrust phase. Nevertheless, specific knowledge is required for properly and fully exploit the opportunities offered by such programs.

FRCs typically used in tunnel linings present a post-cracking softening behaviour, which means that generally they tend to localize cracking phenomena occurring in a certain region in a single crack. Hence, a crack width estimation can be done by integrating the smeared cracking deformation along a reference length, which is representative of the cracked region.

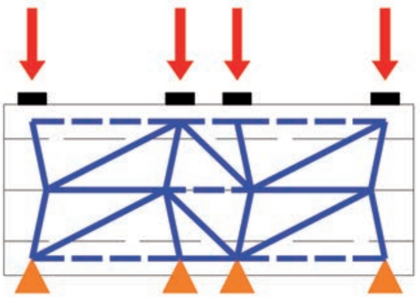


Figure 6. Scheme of a possible simplified load strut-tie model for the design of tunnel segments [27].

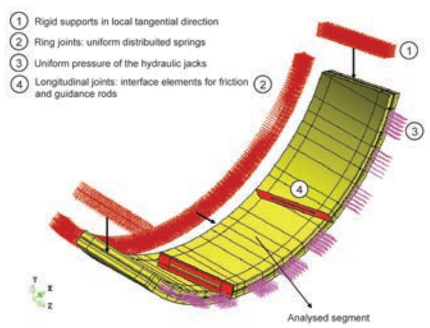


Figure 7. Example of a 3D model, simulating one segment and its interaction with the adjacent ones during the thrust jack phase [40].

As far as the local behaviour is concerned, the region of the segment under the jacks can be considered as a disturbed region (D-region). Transverse tensile and compressive stresses appear since a bi-axial or tri-axial stress condition occurs over a certain diffusive length (also called introduction length). These stresses are generally defined as splitting or bursting stresses. Moreover, each tunnel segment, depending to the segment configuration adopted, are loaded by two or more thrust shoes; the combined effect of the TBM high-concentrated loads determine, between the corresponding loading areas, tensile stresses in the local tangential direction, generally defined as spalling stresses [41].

In order to evaluate the maximum tensile stresses due to this local behaviour, simplified analytical models are available in literature [42,43]. Generally, the maximum tensile stresses are compared with the concrete tensile strength and local stirrups are placed under the jacks.

The local and global mechanisms, previously described, are analysed with particular reference to the enhancement of concrete post-cracking properties due to fibres addition. During the thrust jack phase, cracks in the segments can occur, and they can normally be accepted if they develop in a stable way, without exhibiting a brittle behaviour. In other words, if fibre reinforcement or conventional rebars are used, small cracks are expected, which are generally tolerated, particularly as the axial compressive force in the ring generated by the ground and grout loads tend to close these cracks. The spalling off of the concrete cover along the edges of the segments, or the development of longitudinal large cracks along the width of the ring are not tolerated; possible causes of these cracks could be the eccentricity or the inclination of the thrust jacks with respect to the longitudinal tunnel axis (by considering these possible irregularities, a different local splitting and spalling behaviour is expected).

In the real operational conditions, segments belonging to the same ring are hardly assembled in a perfect plane because of the small installation irregularities that are normally present (Figure 8). In practice, the tunnel segments are not supported uniformly by the back ring and a bending moment may arise due to un-even support of the segments on the ring joint (Figure 8). Depending on such irregularities, large undesired cracks can appear, which require reparation works.

Cracking phenomena in tunnel design could be limited by strictly controlling the jack position and alignment as well as the ring planarity; since this is not always easy to achieve in practice, cracks can be reduced by using a proper combination of FRC and conventional reinforcement localized in the critical regions [44,45].

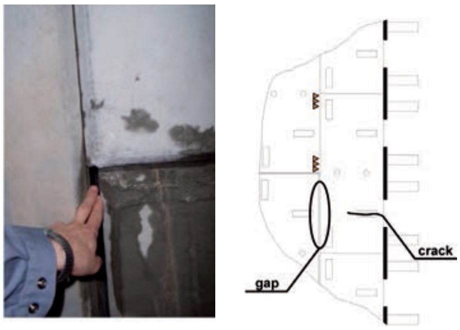


Figure 8. Possible gap between subsequent rings due to a no-perfect installation process; possible un-even support configuration.

An eccentric placement of the thrust jack in the radial direction may represent a significant loading condition. In particular, the presence of an eccentricity on the outward part of the lining can cause a rotation of the tunnel segment (Figure 9). Consequently, cracks may occur in the middle of the segment because of the superposition of the tensile stresses due to a bending moment generated by the segment rotation (Figure 9). Furthermore, splitting cracks under the loading areas can occur since the eccentricity determines an applied higher pressure on the outward part of the segment [46].

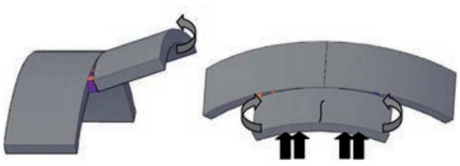


Figure 9. 3D scheme of the dominant mechanism in case of outward eccentricity; the lining behaviour tends to be governed by bending moments.

One of the most severe irregular configurations with respect to the already installed ring corresponds to the presence of localised contact on the corner of the segment, as shown in Figure 10 [40].

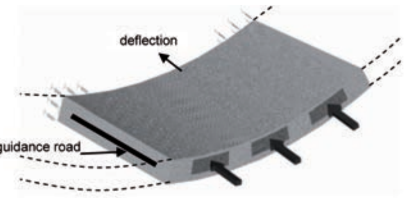


Figure 10. Possible un-even support configuration: localised contact on the corner of the segment with respect to the already installed ring

Research works clearly evidenced the beneficial effects of FRC in presence of load concentrations and splitting phenomena that arise in tunnel segments because of the introduction of thrust jack forces [27, 28]. By performing NLFM analyses [8], it can be demonstrated that, as the load increases, the area where splitting stresses occur in the concrete under the jacks becomes larger, as necessary for the equilibrium. Due to the residual post cracking strength of FRC, in spite the area is micro-cracked, a residual strength is available. Such residual strength allows a progressive redistribution of stresses in deeper points inside the segment, since FRC enables a stable propagation of cracks compared to plain concrete. This allows achieving a local equilibrium configuration which permits the total load to increase [44].

This phenomenon can be qualitatively explained by means of the diagrams shown in Figure 11, that represent the radial-stresses under a TBM jack vs. the distance along the width of the segment. The diagrams are obtained from numerical non-linear analyses of tunnel segments reinforced only by steel fibres [47]. By progressively increasing the load applied by TBM thrust jacks, after an initial development of splitting cracks (Figure 11), the splitting stresses do not rapidly decrease as in the case of plain concrete. The post-cracking residual strength provided by fibres enables a re-distribution of stresses along the segment width without reduction of the global load applied, which continuously increase (Figure 11). As a result, by using a FRC having proper post-cracking toughness, designers can reduce or entirely replaced the stirrups locally placed in the region under the jacks. Nevertheless, no specific simplified relationships are currently available to appreciate this local phenomenon in FRC tunnel segments.

When referring to the global behaviour of the segment with high localized stresses due to geometrical and/or load irregularities, rebars represent an optimised solution. In presence of irregularities such as possible eccentricities or un-even supports, the segment behaves like a deep beam under flexure/shear. In this case, high localized stresses appear in the segments and traditional rebars in combination with fibre reinforcement are adequate to absorbe such stresses [44], [47]. Longitudinal rebars can be concentrated in chords placed along the two longer side of the segment (Figure 12). The segment behaves like a deep beam, where shear between the two chords can be transmitted through fibre reinforcement.

On the other hand, if particular attention is devoted during construction to maintain the plane face of the rings, and if this is combined with suitable grouting of the tail void, the amount of this irregularities can be significantly reduced; this condition is favourable to the use of fibre reinforcement only (see Annex II).

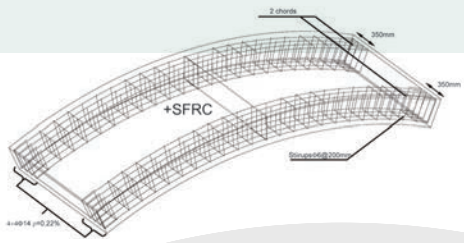


Figure 12. Possible reinforcement solution for precast segments: traditional rebars+fibers [44].

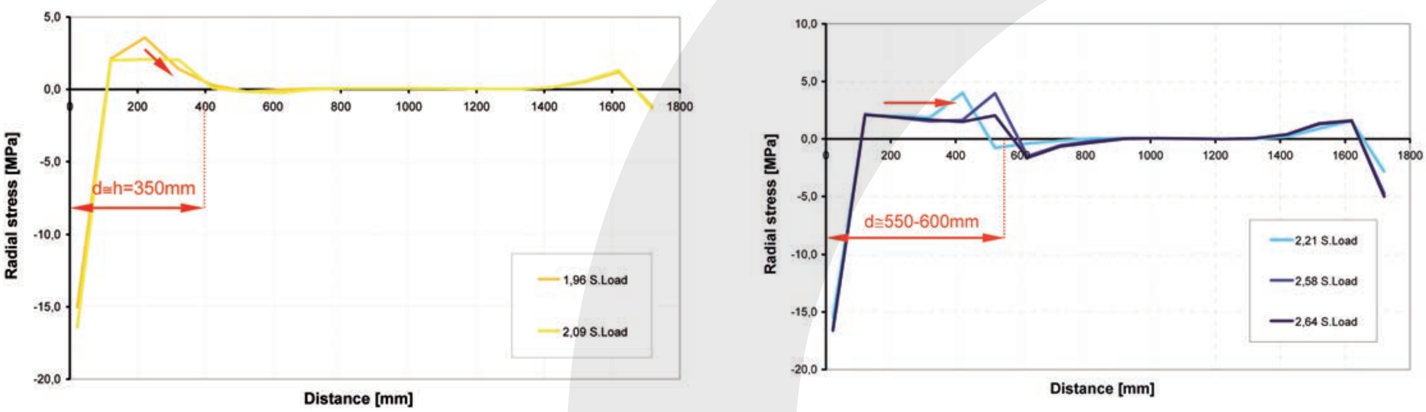


Figure 11. Distribution of radial stresses (splitting stress) along the segment width. Results obtained by means of numerical analyses [47].

11.2 GENERAL DESIGN ASPECTS RELATED TO SEGMENTAL LINING BEHAVIOUR DURING SERVICE CONDITIONS

This section reports a brief overview of the loads corresponding to service conditions (e.g., final stage or grouting process). The ring can be modelled through different types of models:

- Bedded-beam model;
- Finite element method (FEM);
- Elastic equation method;
- Schulze and Duddeck model;
- Muir Wood model.

When modelling a ring, special attention should be given to the way of properly simulating the longitudinal joints (between adjacent segments of the same ring), and the circunferential joints (between adjacent rings).

The behaviour of longitudinal joints is intermediate between a perfect continuity (of the ring) and a perfect hinge, and an proper rotational stiffness should be estimated. In the contact area of the longitudinal joint stresses and deformations are transferred, and deformations are determined by assuming a transfer zone over which the stresses are divided. Knowing the deformations on the contact area, it is possible to determine the rotation of the segment. A good estimate of the rotation can be simply determined by integrating the curvature over the depth of the contact area (Figure 13). The non-linear rotational behaviour of the longitudinal joint can be included in a rotational spring which connects the ends of two adjacent tunnel segments (e.g., in a bedded -beam model). The stiffness of this rotational spring can be determined by simply calculating the bending moment vs. rotation relationship of the equivalent concrete beam, as evidenced in Figure 13.

As regards circumferential joints, and referring again to relative simple bedded-beam models, the contact behaviour between rings can be modelled by means of local springs. As an example, a numerical model consisting of two rings laying on the same plane and connected by radial springs that simulate the forces transmitted through the ring joints, can be adopted [45,48]. This 2D model is able to consider in a simple way the influence of the "third dimension", as shown in Figure 14. This seems adequate for parametric studies or preliminary design.

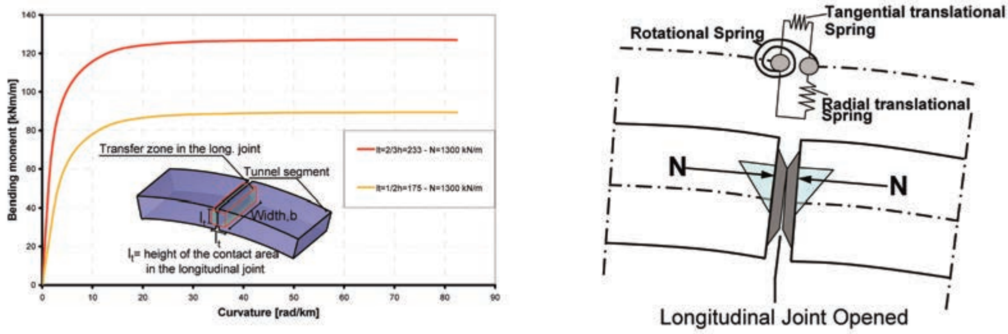


Figure 13. Possible scheme of the contact area for evaluating the stiffness of rotational spring between two adjacent segments.

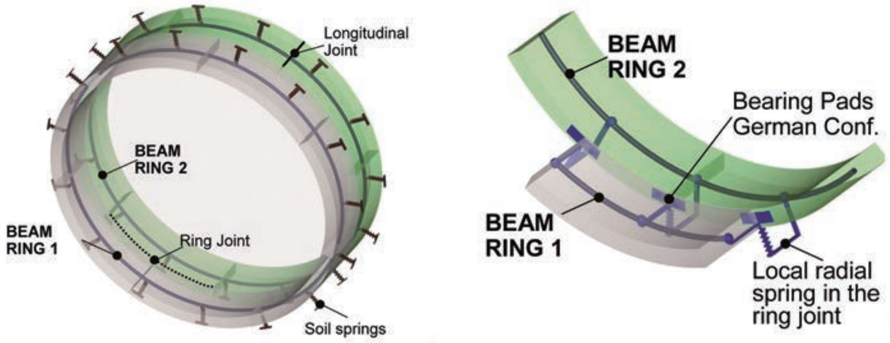


Figure 14. Model based on two adjacent rings with radial springs simulating the ground, and longitudinal and ring-to-ring joints simulating the interaction among segments and between rings. Scheme of the ring joint.

By properly considering the interaction between two adjacent rings, the distribution of the bending moments varies. Generally, an increment of the maximum bending moment occurs with respect to the results of the analysis of a single ring. This is important when considering segments reinforced by fibres only. In fact, RC tunnel segments generally exhibit high resistant bending moments, but this is not always the case of FRC segments.

It can be noted that build tolerances, such as ovalisation and steps between rings lead to irregularities similarly to those previously discussed in Section 11.1. Such aspects are difficult to be model accurately; a limited influence on the lining internal actions is expected, but a much important effect can be expected on the normal ring force transferred to the longitudinal joints.

11.2.1 Lining response at ULS

The residual strength fF_{tud} (Eq. 4, Section 10.2), introduced in the simplified rigid-plastic law suggested by Model Code 2010 [9], is expressed by scaling by the χ factor the tensile strength fctd, as shown in Figure 15 ($f_{ctd}=f_{Ftud}$, eq. 4, Section 10.2). The constitutive law reported in Figure 15 is expressed in terms of strain by considering the structural characteristic length (l_{cs}) as explained in Section 6.2.

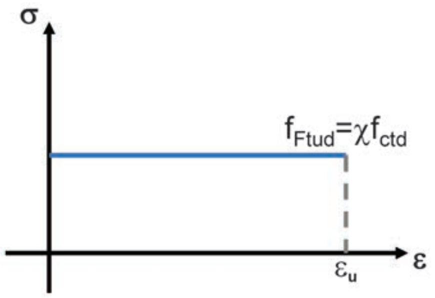


Figure 15. Constitutive law that can be adopted for SFRC under tension.

After the evaluation of the FRC post-cracking behaviour (Figure 15), it is possible to calculate the resistant domain M_{Rd}-N_{Ed} at ULS of a 1m long longitudinal section of tunnel lining. For this purpose, the constitutive law proposed by Eurocode 2 [26] for concrete under compression can be adopted (Figure 16); it can be considered a reasonable approximation because the addition of fibres has a little influence on the peak compressive strength. Similarly, the law suggested by Eurocode 2 [26] for steel under compression/tension can be adopted (Figure 16).

The following hypotheses are also taken:

- the plane section assumption remains valid under rotation; as a result, the strain distribution over the section is linear;
- a perfect-bond relationship is assumed between SFRC and longitudinal rebars.

The behaviour of the longitudinal tunnel lining section at ULS is shown in Figure 17.

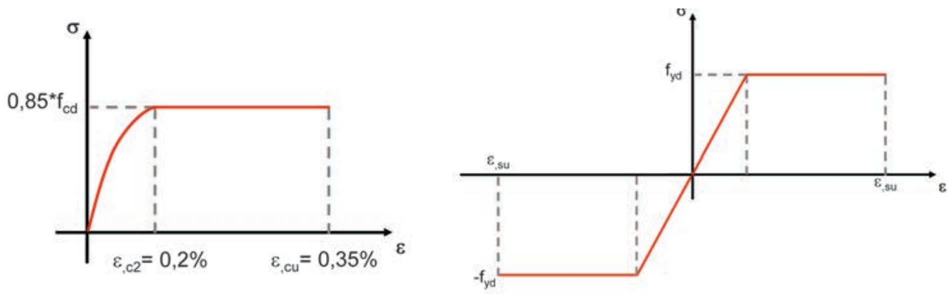


Figure 16. Constitutive law that can be adopted for concrete under compression (a) and for steel under tension/compression (b).

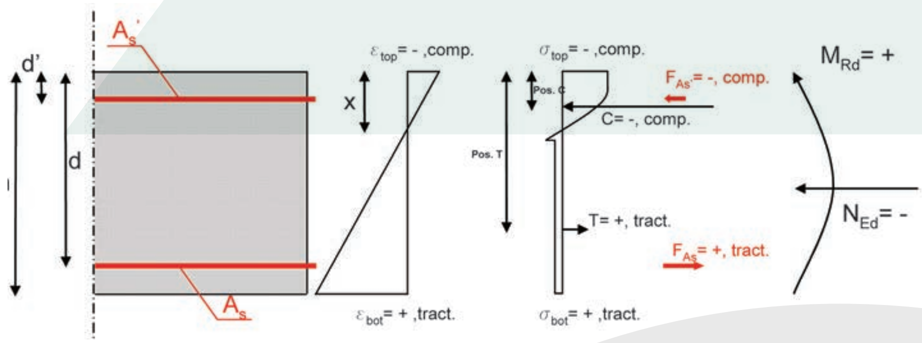


Figure 17. Scheme with evidenced the distribution of strain and stresses along the longitudinal section at ULS.

As an example, by referring to a longitudinal section of a tunnel lining having a width of 1 m and by assuming a post-cracking residual strength (parameter χ) equal to 25%, 50% and 75% of f_{ctd}, the domain represented in Figure 18 can be calculated (curves refer to a segment with a thickness of 350 mm). A reference concrete presenting a concrete strength class C 40/50 is considered [26]. The following mechanical properties are adopted: E_c=35000 MPa, f_{ck}=40 MPa, f_{ctk}=2.456 MPa, f_{cd}=22.67 MPa and f_{ctd}=1.637 MPa [45]. The lining is reinforced only with fibres.

Similarly, the domain can be calculated also for linings reinforced by means of a combination of traditional rebars and fibres. As an example, by adopting two layers of rebars B450C and a longitudinal steel ratio equal to ρ_s=0,20%, the diagrams shown in Figure 19 can be obtained (note that the concrete cover, measured from the rebar centroid, is 41 mm).

The diagrams clearly show that, because of the low longitudinal steel ratio, the increment of the resistant bending moment at ULS due to fibres can be noticeable when the compressive axial load is small (typical condition of segment storage or construction stage); the increment becomes negligible when the axial load increases. As previously explained, this aspect can be considered by means of simple calculation by designers in order to get the best design solution.

With regard to the shear capacity of the tunnel lining, the fibre resistant contribution V_{Rd},F, can be estimated by using the expressions suggested by standards concerning the design of FRC structure. The new fib Model Code 2010 [9], proposes a shear strength that is a quite simple adaptation of the current formulation included in the Eurocode 2 [26] for shear in members without conventional transverse reinforcement. The fibre contribution is included as a modification of the longitudinal reinforcement ratio, since fibres can be considered as an additional randomly distributed reinforcement all over the depth of the beam.

Shear forces are generally small in the final state; hence, the minimum shear reinforcement provided by fibres is generally sufficient in tunnel linings.

11.2.2 Lining sectional response at SLS

Besides the necessity to provide a certain required design bearing capacity of the lining at ULS, the Serviceability Limit State (SLS) of the structure should be taken into account. This means, for a concrete structure, that the designers should guarantee, at SLS, a minimum performance level (especially with reference to the crack control) in order to guarantee the necessary durability of the structure.

At Serviceability Limit State a lining in its final load configuration has to ensure durability with respect to a required nominal life of the tunnel, which very often is 100 years or more. For concrete structures, durability generally involves a proper choice of the concrete matrix and mix-design [49], but also involves a proper control of crack width. In this sense, for RC or RC+FRC tunnel segments, the reference maximum accepted crack widths suggested in Model Code 2010 [14] as a function of the expected environmental conditions can be used. The latter are reference values similar to that reported in other standards [10, 11, 12, 26] and they are generally based on criteria for preventing the corrosion of conventional steel rebars.

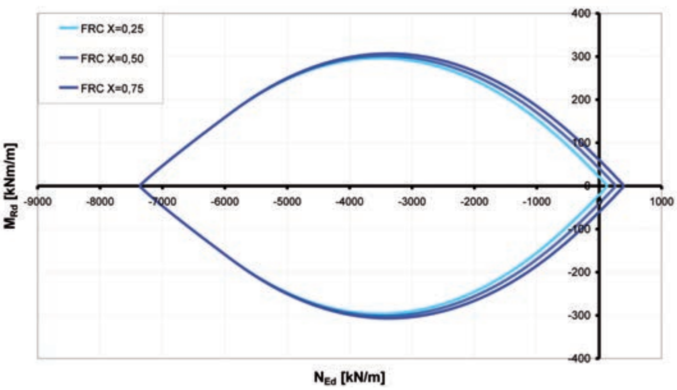


Figure 18. Examples of M_{Rd}-N domain at ULS, FRC segments.

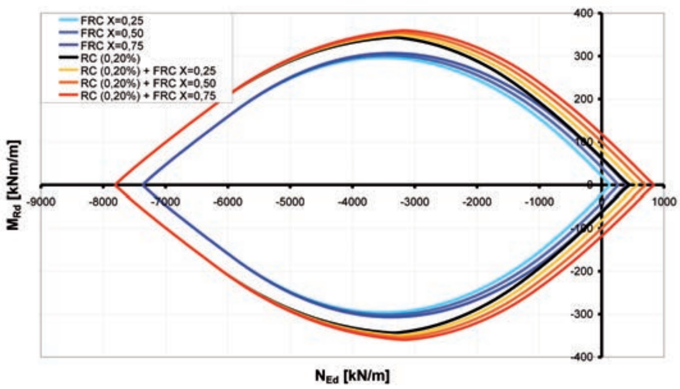


Figure 19. Comparison of the domain M_{Rd}-N_{Ed} calculated for different reinforcement combinations. Fibres only (FRC), traditional reinforcement (RC) and combination of these two types of reinforcement (RC+FRC).

Within the scientific community, the maximum accepted crack width in order to prevent the corrosion of steel fibres, especially in case of a SFRC tunnel segment solution (only fibre) is a matter of discussion. Early reference values (still under discussion) have been reported in the recommendation of AFTES [15], concerning the design, dimensioning and execution of precast steel fibre reinforced concrete arch segments. The following values are reported: design width of cracks w_k of 0.15 mm (exposure classes XS1, XD1, XS2, XD2, XA1 and XA2) or 0.2 mm (exposure classes XC1, XC2, XC3 and XC4). Synthetic fibres (SynFRC) are generally not affected by corrosion, and in case of combined use with conventional rebars, the maximum accepted crack width is governed by the deterioration of steel rebars.

In addition, independently by the possible deterioration of materials, in case of tunnel linings, the maximum accepted crack width should be determined by taking into account also the water-tightness requireents of the lining, further to the environmental conditions. Indeed, in particular situations the maximum accepted crack width can be very small.

Since FRC, due to its noticeable post-cracking tensile properties, principally allows a better crack control, its contribution at SLS can be remarkable. This additional advantage means a longer durability and lower maintenance costs of the tunnel lining.

In case of RC+FRC solution, it should be highlighted that tunnel linings are generally structure characterized by low reinforcement rebars ratios (ρ_s), which means that the crack opening limitation could have a relevant impact on the design. Eventually, the combination of regular reinforcement, consisting in longitudinal rebars, involve the necessity of using an opportune crack width criterion based on a proper study of the tension stiffening phenomena.

Among different methods proposed by the scientific community for evaluating crack width occurring in FRC or RC+FRC structural elements, a simplified method will be reported in Section 12.8. This method is particularly suitable for the final stage when the rings are embedded in the ground and small cracks are acceptable. On the other hand, as described in Sections 12.2, 12.3 and 12.4, at stages of de-moulding, storage and positioning of segments, it is recommended to avoid cracks as much as possible.

The evaluation of crack width at SLS in the final stage (lining embedded in ground) also involves another issue under discussion in the scientific community, that is, creep of concrete in tension. The latter depends on several factors especially the type of fibre reinforcement adopted. It is well known that concrete creep in tension can lead to substantial widening of cracks.

A further aspect to be considered with respect to crack width development at long term is the concrete natural capacity of autogenous crack healing. The latter can be the consequence of hydration of un-hydrated cement particles and dissolution and subsequent carbonation of calcium hydroxide. In addition to these mechanisms, swelling of the matrix and blocking of the crack due to debris present in the infiltrated water, or loose concrete particles resulting from cracking may also cause autogenous healing.

The overall contribution of the mechanisms in autogenous healing remains a matter of debate. On the other hand, researchers agree that autogenous healing can improve when the amount of reactive binder agents is increased, when additional water is present in the crack region (e.g., presence of moisture), or when crack widths are restricted [50]. The latter can be achieved by the addition of fibres.

Fibres improve autogenous healing by enhancing multiple crack formation and limiting the crack width. In fact, generally cracks narrower than 50 μm closed completely, while cracks with a width between 50 and 150 μm only show partial closure [51]. A review of the phenomena involved in autogenous healing in presence of micro-synthetic fibre reinforcement was recently proposed by Snoeck et al. [51], and research on this aspect is currently under development.

In Section 11 a general overview of the main load-conditions concerning the segmental lining was presented and the beneficial effects due to the addition of fibres as well as possible design solutions were briefly introduced and discussed.

The aim of this section is to describe the general design procedure for FRC segmental lining based on remarks introduced in Section 11. With this purpose, the following loading conditions will be described:

- de-moulding of tunnel segments;
- storage of segments;
- transportation and positioning of segments by erector;
- thrust jack forces from TBM;
- introduction of normal ring force in longitudinal joint;
- ring behaviour at the final stage;
- ring behaviour during fire event.

FFor each of this conditions, possible analytical methods for including the fibre resistant contribution is suggested, starting from general approaches already reported in the Model Code 2010 [9] for the design of FRC structures.

The loading conditions will be schematically described with respect to SLS and ULS by referring to a typical tunnel lining presenting the main characteristics summarized in Table 2, where each of the main parameters used in the following sections is introduced with its correspondent symbol.

In the following the design procedure will be referred to the three possible reinforcement combinations previously introduced in Section 8:

- RC: segment reinforced only by conventional rebars;
- FRC: segment reinforced only by fibre reinforcement;
- RC+FRC: segment reinforced by a combination of fibre and traditional reinforcement (hybrid solution).

TUNNEL CHARACTERISTIC	
Tunnel overburden (min-max)	$H_{overburden,min,max}$
Thickness	h
Internal diameter	D_i
N°. of segments	N (segments) +N_{key} (key segment; counter key segments are also specified)
Segment length/ width	$L_{segment}/B_{segment}$
Concrete class of strength	f_{ck}/R_{ck}
TBM DATA	
N. jacks	N_{jack}
Thrust (max)	$T_{max,total}$
Thrust (max) exerted by each jack	$T_{max,jack}$
Thrust (SC) exerted by each jack	$T_{sc,jack}$
REINFORCEMENT	
CONVENTIONAL REINFORCEMENT (REINFORCING BAR, REBAR)	
Longitudinal steel ratio	ρ_s
Characteristic yield strength	f_{yk}
Design yield strength	f_{cd}
Steel elastic modulus	E_s
FIBER REINFORCEMENT	
Mean residual flexural tensile strengths according to EN 14651 [20] as introduced in Section 10.	f_{Lm}
	$f_{R1,m}$
	$f_{R2,m}$
	$f_{R3,m}$
	$f_{R4,m}$
Characteristic residual flexural tensile strengths according to EN 14651 [20] as introduced in Section 10.	f_{Lk}
	$f_{R1,k}$
	$f_{R2,k}$
	$f_{R3,k}$
	$f_{R4,k}$

Table 2 : Basic parameters to be used in the design procedure of a FRC segmental lining.

12.1 FRC MECHANICAL PROPERTIES

The general mechanical properties of concrete such as:

- elastic modulus, E_c;
- compressive strength, f_c;
- tensile strength, f_{ct};

can be obtained by simply applying the general rules for conventional concrete (see Eurocode 2 [26], MC 2010 [9]) since these mechanical properties are not influenced by volume fractions of fibres (V_f) smaller than 1%, as are generally used in segmental tunnel lining (see Case Studies in Annex I).

As previously described in Section 11.2.1, the lining sectional response at ULS can be obtained by means of M_{Rd}-N_{Ed} domain. The post-cracking constitutive laws, which describes the residual strength provided by fibres, can be obtained by using the approach proposed by MC 2010 [9]. A rigid-plastic law can be used according to Eq. 4, Section 10.2, or by using a linear post-cracking law, which can be easily obtained according to Eq. 5 and Eq. 6. These equations refer to FRC fracture parameters summarized in Table 2, according to the post-cracking performance criteria adopted by MC 2010 [9]. In particular, in order to evaluate the lining behaviour at ULS, the reference parameters for design are the following:

$$f_{Fts,d} = \frac{f_{Fts,k}}{\gamma_f} \qquad \text{(Eq. 7)}$$

$$f_{Ftu,d} = \frac{f_{Ftu,k}}{\gamma_f} \qquad \text{(Eq. 8)}$$

12.2 DEMOULDING OF TUNNEL SEGMENTS

The main issue of this temporary phase is the early-age of concrete. Generally, a minimum concrete compressive strength ($f_{ck,de\text{-}mould}$) should be guaranteed when demoulding.

Demoulding is a typical temporary phase which should be analysed at SLS in order to evaluate crack development, since it is desirable to avoid cracking as much as possible during this stage. Nevertheless, this loading condition can be also considered at Ultimate Limit State in order to evaluate the minimum required bearing capacity that segments must provide for not collapsing. In fact, the safety of workers must always be guaranteed, and a verification at ULS is generally required (Figure 20).

It is recommended to evaluate the fracture parameters by testing standard specimens having the same curing time and curing procedure. Alternatively, as a rough first approximation, the fracture properties at a certain curing time (demoulding time) can be estimated to be proportional to the compressive strength, even if there are no currently specific relationships available for evaluating the post-cracking residual strengths at early age based on the one exhibited at 28 days.

Besides the evaluation of the flexural bearing capacity, it is necessary to calculate the corresponding resistant shear, V_{Rd}, based on the approach suggested by Model Code 2010 [9]: notice that this method is only suitable for RC and RC+FRC segment configurations. In case of FRC segments, a method was recently suggested by Coccia et al. [52].

Serviceability Limit State (SLS) is of paramount importance for this loading condition, with the main objective to avoid cracking. Based on the adopted static scheme (Figure 20), the segment internal force at SLS can be easily calculated (N, V, M). Since, it is expected that concrete remains in the un-cracked state, the maximum tensile stress should be less than the tensile strength ($f_{ctk,0.05}$) evaluated at demoulding time. Hence, the following should be verified:

$$\sigma_{1,2} \le f_{ctk,0.05}(de - moulding\ time) \qquad \text{(Eq. 9)}$$

In Equation 9 the term $\sigma_{1,2}$ represents the principal tensile stress in the most critical tunnel segment section calculated by means of Mohr's circle according to the combination of N, V, M and by assuming an un-cracked concrete section (as it is expected in this phase).

This verification is basically independent by fibre resistant contribution (since fibres act after cracking) and also traditional rebars are neglected in analytical calculations. If the tunnel segment does not remain in the un-cracked state, other possible specific calculations in order to evaluate the crack width can be developed with the simplified method described in Section 12.8, even if it is strictly recommended to avoid as much as possible cracking during demoulding, storage, transportation and positioning of segments.

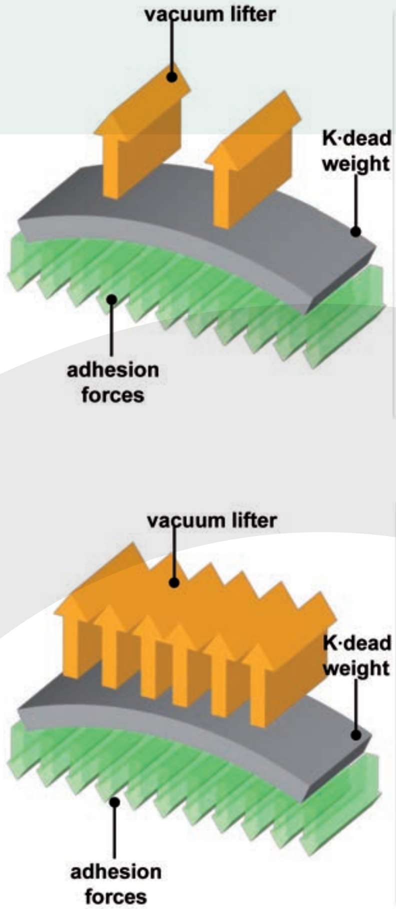


Figure 20 . Possible static schemes for evaluating the segment internal actions (N, V, M) during de-molding-stage.

12.3 STORAGE OF SEGMENTS

This temporary stage is characterized by the same concerns regarding the early-age concrete. Furthermore, similarly to demoulding, it is desirable to avoid cracking as much as possible during this stage; a minimum bearing capacity that segments must provide for not collapsing is also required.

Possible static schemes typically adopted are schematically depicted in Figure 21. Particular attention should be given to the misalignment of the supports of piled tunnel segments, which can introduce unwanted eccentricities and high bending moments which can be particularly severe for tunnel segments reinforced by fibres only.

The main steps of the design process for this phase are the following:

- evaluation of the actual compressive strength of concrete at the time of segments storage: ($f_{ck,storage}$);
- evaluation (or estimation) of FRC fracture parameters (f_{Ri}), at the time of storage (an experimental campaign on testing samples is recommended);
- based on assumed static scheme, analytical calculation of N , V, M at SLS and at ULS are performed;
- verification at SLS of the expected un-cracked state of concrete tunnel segments.

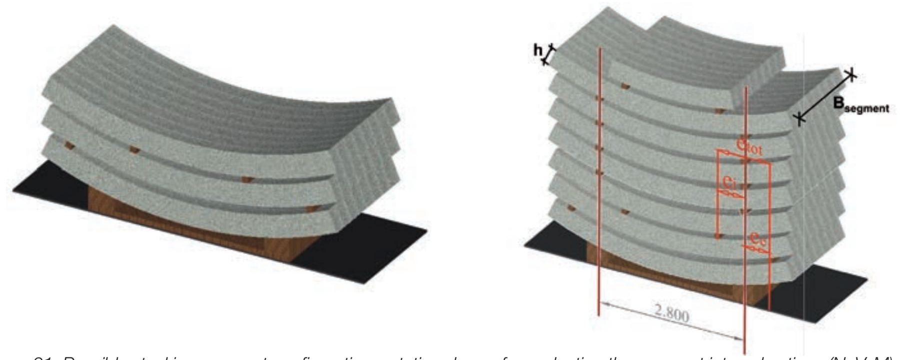


Figure 21. Possible stacking segment configurations: static scheme for evaluating the segment internal actions (N, V, M) during this phase should consider possible eccentricities.

12.4 TRANSPORTATION AND POSITIONING OF SEGMENTS BY ERECTOR

The segments need to be transported around the segment plant, to the project site, and down to the tunnel. A variety of lifting devices can be used such single point, side clamps, fork lift, vacuum lift, etc.

Then, the segments should be positioned by means of the segment erector. This particular temporary stage concerns the construction of a ring, and the concrete curing-time shall be well beyond 28-days; hence, no particular concerns arise with regard to mechanical properties.

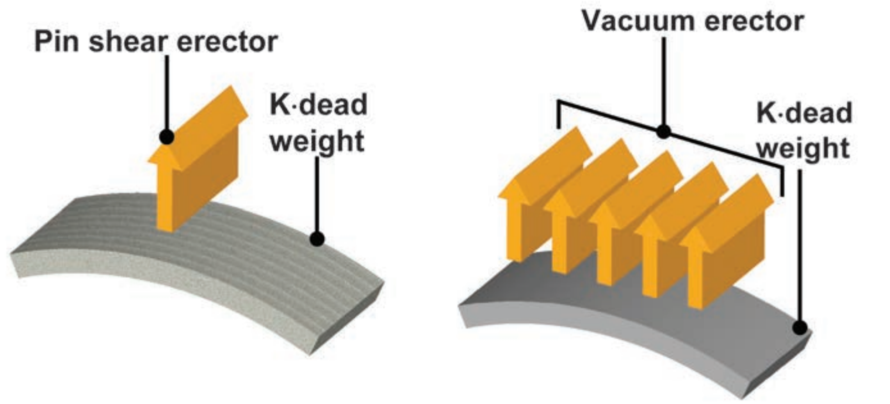


Figure 22. Possible static schemes for evaluating the segment internal actions (N, V, M) during the process of positioning the segment by means of pin-shear erector.

The same procedure previously described in Sections 12.3 and 12.4 can be applied by referring to concrete mechanical and fracture properties at 28 days (characteristic and design values, respectively for SLS and ULS). The static scheme used in this temporary stage strictly depends on the erection system. Possible configurations are reported in Figure 22.

12.5 THRUST JACKS PHASE

As mentioned in Section 11.1, the first step to assess the local stresses generated in the lining segments under the action of the TBM jacks is to evaluate the risk of cracking due to local splitting stresses based on typical relationships available in the bibliography [42, 43]. In order to take properly into account the fibre resistant contribution during this stage, two approaches are possible:

- full scale experimental tests on FRC or RC+FRC tunnel segments;
- non-linear numerical analyses based on non-linear fracture mechanics (NLFM, [8]), in order to take properly into account the post-cracking residual strength of FRCs.

These approaches allow to evaluate the safety-factor in terms of additional capacity supplied by fibre reinforcement with respect to the most probable forces exerted by TBM jacks during service condition.

The procedure and instrumentation for full-scale tests are generally expensive, and this approach does not enable a proper parametric study to evaluate different segment boundary conditions. On the other hand, full-scale tests allow to observe the actual segment behaviour.

In this section a possible two-steps numerically-based design procedure is briefly introduced, as a possible alternative approach to experimental full-scale tests.

The first step of a numerical approach is the creation of a simplified 2D numerical model in order to develop a parametric non-linear study.

A typical 2D model is presented in Figure 23. The segment length measured on the mid-tunnel segment plane ($L_{segment}$) is evidenced together with the segment width ($B_{segment}$). Particular attention should be given to the boundary conditions which, as a first approximation, can be described as rigid supports.

Based on a simplified 2D numerical model, several simulations can be carried out by varying:

- the total reinforcement adopted in the segment (RC, FRC, RC+FRC);
- the boundary conditions.

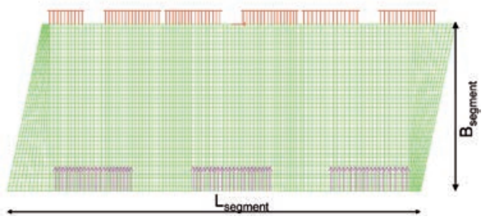


Figure 23. Example of typical 2D numerical model adequate for developing an initial parametric study.

The boundary conditions can be varied, for instance, by adopting the schemes shown in Figure 24, which introduce the irregularities of support in the jacks, starting from a regular configuration (UNI).

Based on a 2D parametric study, initial choices can be made with respect to the amount of fibre and traditional reinforcement to be used. The numerical analyses are generally carried out by applying $T_{sc,jack}$, which is the service load for each pushing arm and by verifying the cracks arising in the lining in terms of the cracked region dimensions and maximum crack width. The load can be progressively increase up to $T_{max,jack}$ (the maximum load applicable by each jack). In this way, both SLS and ULS can be investigated.

The second step consist of 3D numerical analyses can be carried out to evaluate the additional effects of possible eccentricities of the pushing arms. A typical numerical 3D model of a tunnel segment is reported in Figure 25. Advanced modelling techniques can be used like, for instance, interface elements (Figure 25). A 3D model enables to better capture local and global mechanisms of the segment during this phase even though these analyses are time-consuming. Nevertheless, if these models are used after a preliminary 2D study, the analyses can be used to refine final evaluations regarding the best reinforcement design solution.

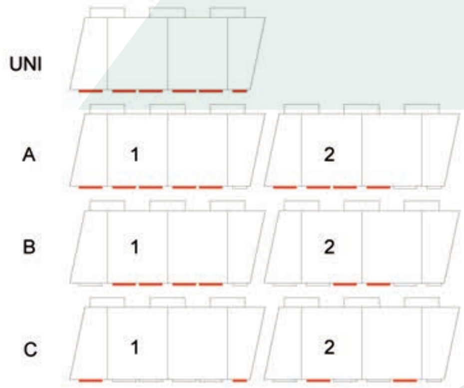


Figure 24. Scheme of possible boundary conditions of tunnel segment during the thrust phase.

12.6 RING BEHAVIOUR OF THE TUNNEL LINING DURING GROUTING PROCESS

The most serious source of ring bending in segmental linings is the pressure of grout injected to fill the anular void [53]. If the grouting process is not carried out carefully, the lining could sink in the invert under its own weight plus that of the tunnel construction equipment. This could cause an eccentric annular void, larger at the crown than at the invert. In case of incomplete grouting condition, a local increase of the bending moment arise in the lining, which can be particularly crucial for FRC tunnel segments with fibre reinforcement only.

The available bibliography shows that complex 3D modelling exists, trying to simulate the overall behaviour of the construction process of a shield driven tunnel, including the grouting phase. Nevertheless, for the scope of lining design, it is recommended to use simpler models, which are able to reasonably capture the bending moments arising in the lining during the grouting process [48]. Such models allow determining the internal lining actions (N, M, V) during the grouting process, and comparing them to the resistant actions at ULS as previously described for other loading conditions. The study of this temporary phase at ULS is particularly meaningful for the "incomplete grouting condition", since it represents the effects of a not-well controlled grouting process and it consists in a kind of ultimate condition.

Referring to SLS, if the tunnel segment, as expected, does not remain in the un-cracked state, other possible specific calculations in order to evaluate the crack width can be developed with the simplified method described in Section 12.8.

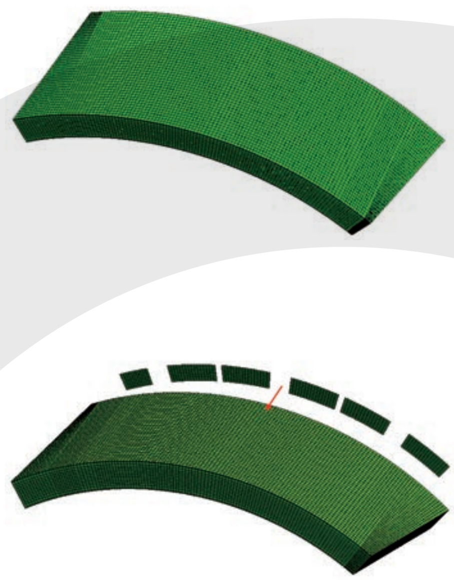


Figure 25. Example of typical 3D numerical model adequate for final evaluations concerning the segment behavior during the thrust phase Scheme with evidenced interface elements.

12.7 INTRODUCTION OF NORMAL RING FORCE IN LONGITUDINAL JOINTS

The longitudinal joints are those located between adjacent segments of the same ring. These joints can be characterized by a simple contact between the flat surfaces of adjacent segments, shear keys or guidance rods. Hence, the normal hooping force, arising in the lining because of the ground pressure, should be transmitted along a contact segment-to-segment surface, which generally is reduced according to the joint systems.

A local traditional reinforcement could be placed in order to cope with local bursting stresses arising in the segment because of the local introduction of high axial compressive forces (normal ring forces) along small surfaces. The amount of this local reinforcement, when necessary, is based on simplified relationships available [42, 43]. In order to evaluate this local mechanism with respect to possible use of fibre reinforcement, linear or non-linear numerical analyses can be carried out by using simple 2D models, and also by including possible local eccentricities, as shown in Figure 26.

Linear elastic analyses as well as analytical elastic formulations can be used for evaluating if the local bursting stresses exceed the concrete tensile strength [42, 43]. On the other hand, in order to capture the post-cracking FRC contribution, non-linear numerical analyses should be carried out. These analyses enables to evidence the maximum crack width at service condition (if the bursting stresses are higher than f_{ctk}) and enables to evaluate the safety factor, guaranteed by fibre reinforcement in terms of both bearing capacity and local ductility.

12.8 RING BEHAVIOUR AT THE FINAL STAGE

The evaluation of the lining internal actions at the final stage, when the lining is embedded in the surrounding ground, can be developed by means of several available analytical or numerical/analytical approaches, as generally described in Section 11.2.

A design procedure similar to that previously described in Sections 12.3-12.4 can be used on analytical study of the lining behaviour at ULS and SLS. It is worthwhile noticing that, during the final stage, the Ultimate Limit State condition is more significant than in temporary phases such as demoulding or positioning of segments, since the final stage is investigated in a long term condition. The ULS verification can also represent a particular situation along the tunnel where unexpected ground conditions occur.

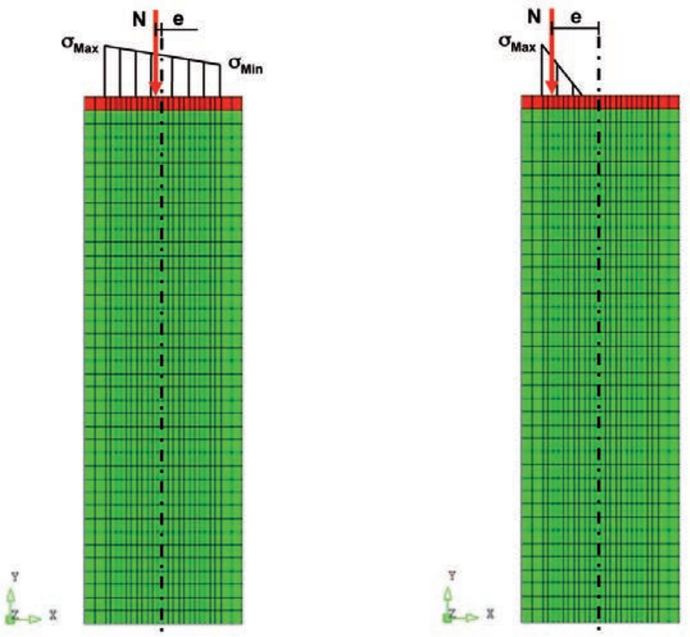


Figure 26. Example of typical 2D numerical model adequate for evaluating the introduction of normal ring force in longitudinal joint.

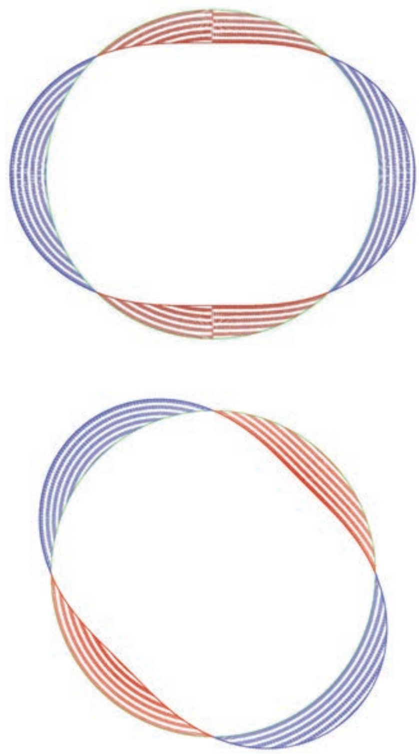


Figure 27. Typical distribution of bending moments M and shear forces V of a segmental lining ring embedded in soil.

Typical diagrams representing the distribution of bending moments and shear forces along a tunnel lining are reported in Figure 27 for a continuous ring having a width of 1 m, embedded in soft soil. As previously mentioned in Section 11.2, these diagrams can be further developed by considering the interaction through adjacent rings as well as the influence of longitudinal joints according to proper approaches available in the bibliography. The ring is subjected to compressive force in the hoop direction, which means that each segment transverse section is subjected to a certain applied normal ring force.

Based on the lining internal actions at SLS, the corresponding actions at ULS (M_{Ed}, N_{Ed}, V_{Ed}) can be estimated by using a multiplying factor (taken from the relevant design code used for the project or based on the risk of possible unfavorable ground conditions).

For RC, FRC, RC+FRC, the domain M_{Rd} vs. N_{Ed} can be defined with the general rules previously described (Section 11.2.1).

At the serviceability limit state attention is given to a proper choice of the concrete matrix and mix-design, and to a proper control of crack width. Among different methods available for evaluating crack width occurring in FRC or RC+FRC structural elements [45, 54 55, 56, 57], a simplified procedure is proposed starting from the constitutive law proposed by Model Code 2010 [9]. By using a layer-by-layer plane-sectional model as introduced by Hordijk [58] and developed by Kooiman [37] for FRC members, the sectional response of a generic RC, FRC or RC+FRC members (in terms of bending moment vs. curvature) can be plotted. In case of precast tunnel lining, the diagram M vs. curvature should be calculated in presence of a certain axial compressive force, corresponding to the normal ring force of a given lining section under investigation. An example of a diagram M vs. curvature is reported in Figure 28. The approach is a kind of sectional smeared approach and can be developed by referring to FRC characteristic properties ($f_{Ri,k}$, see Table 2). The post-cracking constitutive law shown in Figure 5 can be applied in order to consider the FRC resistant contribution at SLS (Section 11.2.2).

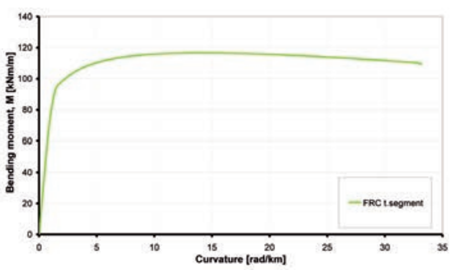


Figure 28. Typical moment vs. curvature diagram of 1 m width FRC tunnel lining section in presence of a given applied normal ring force.

For a given tunnel lining section (thus, a given normal ring force), it is possible to enter in the diagram with the bending moment acting in the section at SLS (M_{SC}). For each point of the diagram, the distribution of strains and stresses along the tunnel lining thickness is known. Consequently, the maximum tensile strain exhibited along the lining thickness can be obtained for the applied bending moment. If this maximum tensile strain exceeds that corresponding to the tensile strength, the lining is cracked.

In presence of a cracked lining section the crack width can be estimated by multiplying the maximum tensile strain (retrieved by the strain distribution along the lining thickness) arising in this section by the the structural characteristic length, l_{cs}. In the recent fib Model Code 2010 [9], relationships for evaluating l_{cs} are given for RC, FRC and RC+FRC members.

In summary, for a given tunnel lining cross-section under an expected axial force (N), based on the constitutive law reported in Figure 5, a moment vs. curvature diagram can be calculated (Figure 28). Afterwards, based on l_{cs}, similar diagram can be obtained in terms of M vs. average crack opening. By entering in this diagram with the expected bending moment in service condition, the corresponding estimation of average crack width can be obtained and compared to the maximum allowable crack width.

The procedure can be repeated for different axial forces and a given allowable maximum crack width, resulting in a domain M vs. N for a given crack width [45].

12.9 RING BEHAVIOUR DURING FIRE EVENT

This section aims to give some general advices concerning the fire resistance of tunnel segments with the awareness that this issue is, in general, a matter of strong debate in the scientific community especially with reference to spalling phenomena, and to the influence of fibres addition on these aspects.

Three possible types of concrete spalling are kown: surface spalling, corner break-off and explosive spalling; the latter is probably the most crucial phenomenon.

The possible causes for explosive spalling are the following [59]:

- thermal processes (thermal stresses, elastic incompatibilities due to different elongations between aggregates and hardened cement paste);
- thermo-hydraulic processes (vapor pressure). The pore water in the concrete evaporates instantly when concrete is exposed to fire and causes spalling of large chunks of concrete.

Referring to tunnel linings, Dehh [60] underlined that, in case of a tunnel fire, destructive spalling occurs at supporting concrete structures. This is mainly caused by the water content of the tunnel concrete which is mostly higher than in other concrete structures, and to the temperature increase rate, which is especially high in tunnel fires.

Spalling generally occurs within the first 5 to 30 minutes of a fully developed fire. Other parameters which can affect the spalling behaviour include the base materials, particularly aggregates, and the internal stress conditions due to the non-linear temperature along the cross-sectional area [60]. Some studies have shown that the occurrence of spalling depends on the following factors: maximum fire temperature, moisture content of concrete, temperature increase rate, concrete density and strength, arrangement of the reinforcement, geometry of the structure, pore volume and mean pore size, pressure stress within the structure, mineralogical/petrographical composition of the aggregates [60]. For instance, for high-performance concrete the saturated zone is located nearer to the surface exposed to fire because of the denser structure of the concrete. This causes a quicker heating-up of the condensed water and a quicker rise of the vapor pressure [59].

An effective reduction of explosive spalling can be achieved through a pore system which guarantees sufficient reduction of the high steam pressure. This can be achieved, for example, by adding micro-synthetic fibres, which melt when exposed to fire and open micro-pore passages and form a porous transition zone between the matrix and fibre through which the steam pressure can be released at an early stage.

Dehn et al. [59], based on a micro analytical investigation on the effect of polypropylene fibres in fire exposed high-performance concrete, stated that the addition of synthetic fibres may lead to the following effects:

- improvement of permeability due to formation of capillary pores which are composed during melting and burning of fibres. This effect is generally mentioned to describe the efficiency of fibres;

- improvement of permeability due to creation of transition zones (TZ) which are open for diffusion. Considering the structure in TZ between aggregates and cement matrix additional pores are formed. The thickness of these TZ are mainly influenced by w/c ratio, cement type, and additives. By addition of fibres, TZ will be connected. The system will get open for filtration and permeability;

- improvement of permeability due to additional micro pores which are the effect of structural aeration while mixing fibres in the concrete;

- formation of micro-cracks as result of the melting process.

In several other research works, the beneficial effects of micro-synthetic fibres in terms of controlling spalling in tunnel lining was demonstrated [61, 62, 63]. The use of monofilament polypropylene (PP) micro-fibres is well known as a good solution for considerably reducing the effect of fire spalling in order to achieve a passive fire-protection of structural elements such tunnel linings.

Fire spalling is a complex phenomenon, and predicting the risk of fire-spalling or the effect of a certain dosage/type of fibre reinforcement on spalling is difficult. In fact, even if the beneficial effect of the addition of micro synthetic fibres to concrete (SynFRC) is well known, there is a general consensus within the scientific community that the accurate determination of fibre dosage, to provide the required explosive spalling resistance, should be evaluated on specific experimental fire tests on tunnel segments. The latter are probably the only current available method for deeply and clearly investigating this issue. This tendency is also confirmed by current standards, which usually provide a rather limited number of specific rules or recommendations with regard to spalling [64].

Referring to the main international available standards, in Eurocode 2 (Part 1-2) [65] the fire design of concrete structures is presented for conventionally reinforced and pre-stressed concrete. By means of analytical simplified approaches or numerical analyses, the development of temperature within a tunnel lining ring in presence of a standard fire event can be reproduced. Based on the temperature distribution along the lining thickness (h), a proper domain M vs. N can be evaluated for the lining section according to the isothermal 500° Eurocode 2 (Part 1-2), Annex B1 [65]. The reliability of this method with respect to more complicated approaches is well discussed in [66], with particular reference to its application to FRC exposed to high temperature by considering the decay of FRC mechanical properties according to the applied temperatures [67,70]. In Eurocode2 (Part 1-2) [65], no recommendations are given for the design of FRC elements. With respect to explosive concrete spalling, methods are mentioned which result in a reduction of spalling for high-strength concrete (use of 2.0 kg/m^3 of monofilament synthetic fibres for concrete grades ranging between C80/95 and C90/105 according to the strength class definition of EN 206-1 [49]).

The fib Model Code 2010 [9] contains information about principles of fire design (§7.5.1.2) for concrete with conventional reinforcement. In fib Bulletin 38 [69], the influence of fire on concrete structures is reported. The use of synthetic fibres is characterized to reduce pore pressure and risk of spalling (§6.4). Nevertheless, the bulletin includes no information regarding fire design of FRC. Moreover, in fib Bulletin 46 [70], it is shown that fibres are conducive to better mechanical properties after exposure to high temperatures.

The evaluation of properties of FRC exposed at high temperatures is also of main interest for estimating the tunnel lining behaviour during and after a fire event [66]. The main mechanical properties of plain concrete as well as of FRC during and after exposure to high temperatures can be evaluated on small specimens. In case of FRC, the evaluation of post-cracking residual strengths is of main interest [67, 68]. Furthermore, in the Italian guideline [11], it is described that a low percentage of steel fibres does not influence thermal diffusivity. However, steel fibres improve residual strengths for concrete exposed to temperatures higher than 600 °C. Additionally, a damage coefficient is introduced. The damage coefficient is equal to the residual strength (compression or direct tension test) tested after exposing the specimens to high temperature normalized to the strength tested with specimens at room temperature.

13 >> Concluding remarks

The document has been developed to provide recommendations and guidelines specifically adequate to FRC tunnel segmental lining with special emphasis to their design procedure.

This work is based on twenty years of FRC tunnel segments practice that show how the use of FRC in tunnel segments allows for several advantages such as the improvement of post-cracking tensile behaviour and the consequent better control of flexural cracks. Fibres enable a stable development of splitting cracks and a possible reduction of stirrups placed in the regions under the thrust jacks as well as a possible complete replacement of shear reinforcement.

The document proposes a brief overview on current available applicable standards for the characterization of FRC's fracture properties and for the design of FRC elements. Among them, the new fib Model Code 2010 is mentioned, which refers to EN 14651 for determining the main significant residual post-cracking strengths.

Based on the performance criteria and approaches provided by fib Model Code 2010 and based on experiences already developed on FRC tunnel segments, design rules and procedures according to the particular requirements and loading conditions of segmental lining are suggested. The analytical and numerical procedures useful to quantify during design the main advantages due to fibre reinforcement are also discussed. In fact, the current fib Model Code 2010 can be easily applied for structures such as beams or slabs, but needs to be contextualized to the specific issues concerning tunnel lining elements.

In the document special attention is devoted to some particular loading conditions that, based on experience, should be considered since they can be very severe for a tunnel segment reinforcement solution based on fibres only. As a general criterion, it has been emphasized that localized stresses are better resisted by localized reinforcement such as traditional steel reinforcing bars (steel rebars), while diffused stresses are better resisted by spread reinforcement, such as fibres. Hence, depending on FRC properties, lining geometry and ground-conditions, optimized solutions can be based on fibres only (FRC) or fibres plus traditional rebars (hybrid solution, RC+FRC).

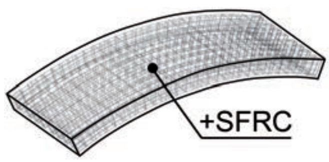
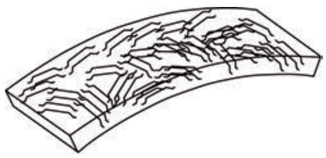
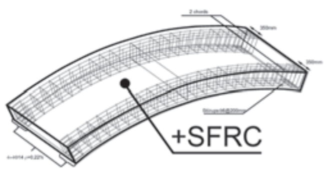
In this section, fourteen case studies mentioned in Table 1 are presented in more detail, based on information reported in published papers or in international journals. In some of these case studies, FRC or RC+FRC tunnel segments were not used along the whole tunnel alignment.

Out of the fourteen case, two are feasibility studies regarding FRC use in major tunnel projects. The first feasibility study is based on a non-linear numerical simulation, whereas the second is based on full-scale experimental tests. They represent the two currently available methods for investigating the FRC tunnel segments behaviour during the thrust phase, as addressed in Section 12.5.

A.I.1 BARCELONA METRO LINE 9

PROJECT PARTICULARS		
Location	Barcelona (SPAIN)	
Construction period	Started in 2003, under construction.	
Owner	GISA, Gestio d'infrastructures, s.a.	
Designer(s)	-	
Contractor(s)	UTE Linia 9 (Line 9 joint venture)	
Engineer(s)	PAYMACotas	
GENERAL PROJECT DESCRIPTION		
The TBM-excavated tunnel of Barcelona has an internal diameter of about 10.9 m, a length of more than 41.4 km and is located from 30 to 70 meters below the surface. The tracks and platforms will be accommodated in a double deck configuration (metro line). The metro line has 43 stations and 15 interchanges. The lining will be an important connection to the airport, the law district, the Barcelona fair and the high speed railway.		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	41.4 km	
Boring diameter	12.1 m	
Overburden (min-max)	30-70 m	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.35 m	
Internal diameter D_i	10.9 m	
Tunnel aspect ratio (D_i/h)	31.14	
Average segment aspect ratio	13.88	Gettu et al., 2004 [30].
N°. of segments	7 segments +1 key segment	
Segment length/width	4.7 m/1.8 m	
Connectors	Bolts in inclined socket.	
CONCRETE MIX DESIGN		
Concrete class of strength	C 50/60	Gettu et al., 2006 [31]
Cement CEM I 52.5R	400 kg/m³	
Sand: 0÷5 mm	745 kg/m³	
Coarse aggregate: 5÷14 mm	558 kg/m³	
Coarse aggregate: 12÷22 mm	559 kg/m³	
Water	132 kg/m³	
Admixtures	4.8 kg/m³ (HRWRA, high range water-reducing admixture)	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
Stretches which present tunnel overburdens ranging from 60-90 m are in prevalent rock with dominant granite (granodiorite). All the other stretches are principally characterized by soil.		

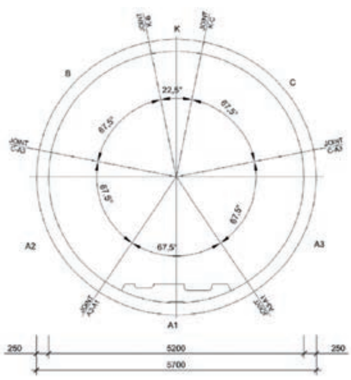
METHOD OF EXCAVATION	
Mechanised tunnelling method by means of a Tunnel Boring Machine with shield.	
TBM DATA	
Manufacturer	-
TBM Type	EPB-Type Earth Pressure Balanced Tunnel Boring Machine (also a DUAL EPB/HR TBM was used)
N. jacks	30 (2 jacks act on each steel thrust shoes)
$T_{max,total}$	140 MN
$T_{max,jack}$	4.67 MN
$T_{sc,jack}$	3.0 MN (typical load in service condition)
Back-up length	-

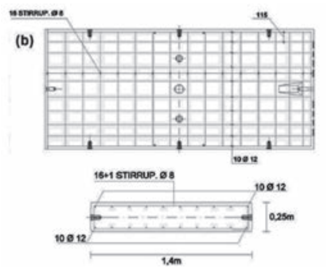
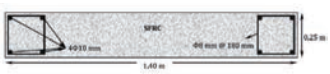
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution (structural contribution of fibres not considered)		97	25 SFRC 50/1.0, L_f/ϕ_f=50, $f_{t,fiber}$=1100 MPa	122	Reported in: Gettu et al. 2004 [30]
Solution 01		-	60 SFRC 50/1.0, L_f/ϕ_f=50, $f_{t,fiber}$=1100 MPa	60	Proposed by: Gettu et al. 2004 [30]
Solution 02		46 (in two chords along the longer segment sides).	25 SFRC 50/0.75, $Lf/L_f/\phi_f$=66.67, $f_{t,fiber}$=1100 MPa $f_{lf,m}$=5.095 MPa $feq_{(0-0.6),m}$=3.943 MPa $f_{eq(0.6-3),m}$=3.474 MPa $f_{lf,k}$=4.323 MPa $f_{eq(0-0.6),k}$=3.024 MPa $f_{eq(0.6-3),k}$=2.395 MPa [71]	71	Proposed by: Plizzari et al. 2005 [44]
Solution adopted	The original design solution was adopted.				Reported in: De la Fuente et al. 2012 [1]

REMARKS CONCERNING THE SOLUTION PROPOSED
The solution 01 was proposed by Gettu et al. [30], based on experimental campaign on SFRC tunnel segment. The experimental research includes the mechanical characterization of FRC, full scale bending tests and local splitting tests. The solution 02 was proposed by Plizzari et al. [44] [47], based on the mechanical characterization of FRC, analytical approaches and non-linear numerical analyses for evaluating the thrust-phase temporary phase (TBM thrust jacks acting on the lining).

REMARKS CONCERNING THE SOLUTION ADOPTED
The stretch Bon-Pastor to Cam-Zam was one of first pilot project for the application of SFRC in precast segments. A total thirty rings with 60 kg/m³ of steel fibres were constructed (solution 01). Three of them were instrumented in order to carry out a loading test for simulating soil pressure in the field conditions by means of jacks (Molins et al., 2009, [72]). The working condition was adverse (descending stretch and water leaks with a temperature up to 60°C). As a matter of fact, some splitting cracks and local failures appeared, which also took place in the stretches with traditional reinforcement. Even considering the problems the results from loading were satisfactory (Molins et al., 2009, [72]). The solution was not generalized in the whole tunnel for several reasons, many of them bearing non relation to the technical reasons [72].

A.I.2 FONTSANTA-TRINITAT TUNNEL

PROJECT PARTICULARS		
Location	Barcelona (SPAIN)	
Construction period	-	
Owner	-	
Designer(s)	-	
Contractor(s)	-	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
It consists of a tunnel for transportation of desalinated water from Llobregat River toward the Ter river. For this purpose, a main station was built in Fontsanta (Santa Just d'Esvern, Barcelona). The tunnel was excavated under the Collserola mountain (Barcelona) simultaneously from two opposite fronts (Riera de Sant Just and La Trinitad) using two double-shielded TBMs (for more details, see de La Fuente et al., 2012 [1]).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	-	
Boring diameter	6 m	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.25 m	
Internal diameter D_i	5.2 m	
Tunnel aspect ratio (D_i/h)	20.8	
Average segment aspect ratio	16.49	
N°. of segments	5 segments +1 key segment	
Segment length/width	3.2 m/1.4 m	
Connectors	-	
CONCRETE MIX DESIGN		
Concrete class of strength	C 50/60 (was used in stretches with poor quality of the rock mass as well as the presence of the phreatic level expectable in few stretches according to the geotechnical analyses).	De La Fuente et al., 2012 [1]
Cement CEM I 52.5R	420 kg/m^3	
Sand: 0÷5 mm	746 kg/m^3	
Coarse aggregate: 5÷14 mm	558 kg/m^3	
Coarse aggregate: 12÷22 mm	559 kg/m^3	
Water	145 kg/m^3	
Admixtures	3.12 kg/m^3 (superplasticizer)	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
The layout gets across heterogeneous soil with several different geological formations. The predominant materials are schist and phyllite (35% of the length), slate and quartzite slate (15%) being the latter ones the worst concerning its geotechnical quality. On the contrary, the rest of the formations are rocks with medium high quality (granodiorite, hornfels, porphyry, quartzite and metamorphic rocks, for more details, see de La Fuente et al., 2012 [1]).		

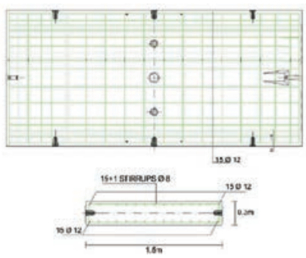
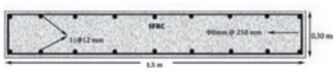
METHOD OF EXCAVATION					
Mechanised tunnelling method by means of two double-shielded TBMs.					
TBM DATA					
Manufacturer	-				
TBM Type	-				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution		68 (n.d) (estimated)	-	68 (n.d.) (estim.)	Reported in: De La Fuente et al., 2012 [1]
Solution 01		19 (in two chords along the longer segment side). (estimated)	25 SFRC 60/0.75, L_f/ϕ_f=50, $f_{t,fiber}$=1100MPa $f_{R1,m}$=3.537 MPa $f_{R4,m}$=3.275 MPa [20]	44	Proposed by: De La Fuente et al., 2012 [1]
Solution adopted	The rings RC+SFRC (solution 01, C50-60) were used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
The solution 01 was proposed by de La Fuentes et al. [1], based on the experience of Barcelona Metro Line 9 and based on analytical approach for evaluating resistant bending moments of RC+SFRC tunnel lining section. The internal lining actions (M, V, N) were evaluated by means of numerical analyses.					
REMARKS CONCERNING THE SOLUTION ADOPTED					
Ten rings made by RC+SFRC segments (solution 01, two edge chords presenting 4+4ϕ10 and stirrups ϕ8@180 mm) were manufactured. None of the rings presented cracks during the assembling operations neither service loads.					

A.I.3 TERRASSA TUNNEL

PROJECT PARTICULARS		
Location	Barcelona (SPAIN)	
Construction period	-	
Owner	-	
Designer(s)	-	
Contractor(s)	-	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
It consists in two parallel rail tunnels in the urban area of Terrassa as the extension of the Ferrocarriles de la Generalitat de Catalunya (FGC)for more details, see de La Fuente et al., 2012 [1]).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	4.51 km (each tunnel)	
Boring diameter	6.90 m	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.30 m	
Internal diameter D_i	6 m	
Tunnel aspect ratio (D_i/h)	20	
Average segment aspect ratio	10.64	
No. of segments	6 segments +1 key segment	
Segment length/width	3 m/1.5 m	
Connectors	-	
CONCRETE MIX DESIGN		
Concrete class of strength	C 30/37	De La Fuente et al., 2012 [1]
Cement CEM I 52.5R	335 kg/m^3	
Sand: 0÷5 mm	565 kg/m^3	
Coarse aggregate: 5÷14 mm	650 kg/m^3	
Coarse aggregate: 12÷22 mm	500 kg/m^3	
Water	133 kg/m^3	
Admixtures	5.40 kg/m^3 (superplasticizer)	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
The analysed stretch for evaluating an optimal tunnel segment reinforcement passes through a soil formed by quartenary clay, with a 24.5 m cover and with a presence of the phreatic level 9.5 m above the tunnel crown (for more details, see de La Fuente et al., 2012 [1]).		

METHOD OF EXCAVATION	
Mechanised tunnelling method by means of a TBM with a diameter section of 6.90 m. The existing gap between the segmental ring and drilled soil was 15 cm wide and it was filled with grout.	

TBM DATA	
Manufacturer	-
TBM Type	-
N. jacks	-
$T_{max,total}$	-
$T_{max,jack}$	-
$T_{sc,jack}$	-
Back-up length	-

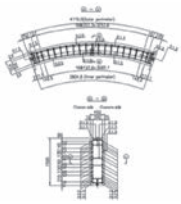
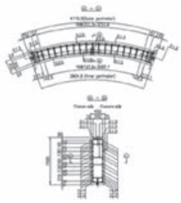
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution		76 (n.d.) (estimated)	25 SFRC	101 (n.d.) (estim.)	Reported in: De La Fuente et al., 2012 [1]
Solution 01		55 (estimated)	25 SFRC 60/0.75, L_f/ϕ_f=50, $f_{t,fiber}$=1100MPa $f_{R1,m}$=3.065 MPa $f_{R4,m}$=2.838 MPa [20]	80 (n.d.) (estim.)	Proposed by: De La Fuente et al., 2012 [1]
Solution adopted	The original design solution was used.				

REMARKS CONCERNING THE SOLUTION PROPOSED
The solution 01 was proposed by de La Fuente et al. [1], based on the experience of Barcelona Metro Line 9 and based on analytical approach for evaluating resistant bending moments of RC+SFRC tunnel lining section. The internal lining actions (M, V, N) were evaluated by means of numerical analyses. The solution 01 reduces the amount of longitudinal curved rebars with respect to the original solution (11+11 ϕ12 vs. 15+15 ϕ12). Furthermore, the amount of stirrups is reduced (ϕ 8mm@250 mm).

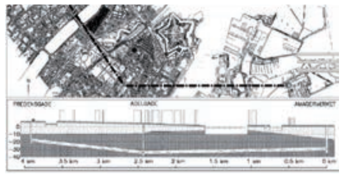
REMARKS CONCERNING THE SOLUTION ADOPTED
The original design solution (for which the contribution of the steel fibres was not considered) was finally used. The obtained results were excellent at a production level (reducing times and risks during the manufacture) as well as in efficiency rates (decreasing significantly the problems to impact and the thrust of jacks) thanks to the use of fibres (de La Fuente et al., 2012 [1]).

A.I.4 METROPOLITAN EXPRESSWAY CENTRAL CIRCULAR SHINJUKU ROUTE TUNNEL

PROJECT PARTICULARS		
Location	Tokyo (Japan)	 Dobashi et al., 2007 [32]
Construction period	March 14, 2002 – October 27, 2005	
Owner	Metropolitan Expressway Company Limited	
Designer(s)	-	
Contractor(s)	-	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
The Metropolitan Expressway Central Circular Route with a total length of approximately 47 km is located outside the Inner Circular Route of Tokyo (at a radius of approximately 8 km from the city center). In the Center Circular Shinjuku Route, a section with approximately 11 km in west side, RC+SFRC tunnel segments were used. These segments have applied to outbound sections SJ51 through SJ53 (for more details, see Dobashi et al., 2007 [32]).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	11 km With 60 rings made by RC+SFRC segments (work sections through SJ51-SJ53)	Dobashi et al., 2007 [32]
Boring diameter	12.02 m	
Overburden (min-max)	7.3 m to 23.4 m	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.45 m	
Internal diameter D_i	10.9 m	
Tunnel aspect ratio (D_i/h)	24.2	
Average segment aspect ratio	9.69	
No. of segments	9 segments	
Segment length/width	3.96 m/1.5 m	
Connectors	Shear key	
CONCRETE MIX DESIGN		
Concrete class of strength	f_{ck}=48.1 MPa (self compacting highly flowable concrete)	Dobashi et al., 2007 [32]
Cement CEM I 52.5R	310 kg/m³	
Sand: 0÷5 mm	310 kg/m³	
Coarse aggregate: 5÷14 mm	1037 kg/m³	
Coarse aggregate: 12÷22 mm	501 kg/m³	
Water	180 kg/m³	
Admixtures	- (superplasticizer)	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
Tunnel in soil: Musashino gravel layer, Tokyo layer, Tokyo gravel layer, Kasusa group [33]. Hence, the soil profile consists of: sand and silty sand (diluvial layer), clay and silt (diluvial layer), sand gravel, clayey silt (diluvial layer) [73].		

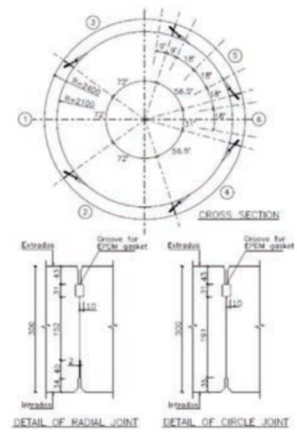
METHOD OF EXCAVATION					
Mechanised tunnelling method by means of a TBM having the largest mud shield in the world [33].					
TBM DATA					
Manufacturer	-				
TBM Type	-				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution Longitudinal curved rebars: 14 ϕ25 (tunnel intrados); Longitudinal curved rebars: 14 ϕ22 (tunnel extrados); Transverse reinf.: ϕ16 and ϕ13, an amount of 167.8		204 (n.d.) (estimated)	-	204 (n.d.) (estim.)	Reported in: Dobashi et al., 2007 [32] Dobashi et al., 2006 [33]
Solution 01 Longitudinal curved rebars: 8 ϕ25+6 ϕ22 (tunnel intrados); Longitudinal curved rebars: 8 ϕ22+6 ϕ19 (tunnel extrados); Transverse reinf.: ϕ13, an amount of 22.2 kg/segment. Total amount of reinf. : 360.7 kg/segment.		135 (estimated)	63 SFRC 30/0.6, L_f/ϕ_f=50	198 (n.d.) (estim.)	Proposed by: Dobashi et al., 2007 [32] Dobashi et al., 2006 [33]
Solution adopted	The solution 01 was adopted in 60 rings along the Center Circular Shinjuku Route.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
The solution 01 was proposed Dobashi et al., 2007 [32] in order to improve the performance and durability of segments by means of the addition of steel fibres, which enables to guarantee a better crack control and a considerable reduction of the transverse reinforcement. These advantages were well demonstrated by Dobashi et al. by means of full scale experimental tests on RC+SFRC segments. The following tests were carried out: bending tests, tests for verifying the distribution of stress of main reinforcement, flexural compression failure test, verification of distribution of steel fibres, fire resistance tests and measurements in the field.					
REMARKS CONCERNING THE SOLUTION ADOPTED					
The solution 01 was adopted in 60 rings along the Center Circular Shinjuku Route. Visual inspection of the surface of RC+SFRC after the assembly of a segmental ring found that concrete was well placed throughout the segment and neither the loss of concrete in corners nor cracking occurred. Measurements of stresses in the reinforcement of RC+SFRC segments in the field confirm that SFRC segments can provide a performance equivalent or higher than that of conventional RC segments with a reduction of the total conventional reinforcement of about 34% (Dobashi et al., 2007 [32]).					

A.I.5 DISTRICT HEATING TUNNEL IN COPENHAGEN

PROJECT PARTICULARS		
Location	Copenhagen (Denmark)	
Construction period	June 2006 – March 2009	
Owner	-	
Designer(s)	-	
Contractor(s)	-	
Engineer(s)	-	Kasper et al., 2008 [35]

GENERAL PROJECT DESCRIPTION

The 3.9 km long district heating tunnel carries heating from a power plant on the island Amager into the centre of Copenhagen. A shaft on Amager and two shafts in the city centre provide access to the tunnel, which is located the Copenhagen limestone at a depth between 25 and 38 m below the ground surface. The tunnel is excavated with an earth pressure balance (EPB) shield machine and is lined with steel fibre reinforced concrete (SFRC) segments without any conventional steel bar reinforcement. The tunnel has an internal diameter of 4.2 m and carries two steam pipes, two hot water pipes and two condensation pipes.

TUNNEL CHARACTERISTICS		
Total Tunnel Length	3.9 km	
Boring diameter	4.5 m	
Overburden (min-max)	from 25 to 38 m	
Lining type	Segmental	
Ring type	Universal ring with a taper of 2 cm to allow for alignments correction during construction	
Thickness	0.30 m	
Internal diameter D_i	4.2 m	
Tunnel aspect ratio (D_i/h)	14	
Average segment aspect ratio	10.08	
N°. of segments	3 standard segments, two counter-key segments and one-key segment. (segments installed with a vacuum pad erector in combination with a 110 mm diameter pin shear erector).	
Segment length/width	2.83 m (standard segment)/1.5 m	Kasper et al., 2008 [35]
Connectors	Bolts in inclined socket. The lining has flat concrete-to-concrete radial joints.	

CONCRETE MIX DESIGN		
Concrete class of strength	C 50/60	
Cement CEM I 52.5R	24 kN/m^3	
Sand: 0÷5 mm	36800 MPa	
Coarse aggregate: 5÷14 mm	0.17	Kasper et al., 2008 [35]
Coarse aggregate: 12÷22 mm	50 MPa	
Water	1.4 MPa	
Admixtures	0.6 MPa	

ENVIRONMENTAL AND GEOLOGICAL CONDITIONS

The tunnel is bored with an EPB shield machine through Copenhagen limestone. This type of TBM was successfully used in the Copenhagen metro project. The geological conditions along the alignment of tunnel are characterised by quarternary soft soil deposits down to depths between 14 and 18 m below the ground surface, followed by the Copenhagen limestone with different degrees of induration. The design groundwater levels range between 0 and 8 below the ground surface (for more details see [35]).

METHOD OF EXCAVATION					
Mechanised tunnelling method by means of a TBM EPB shield machine.					
TBM DATA					
Manufacturer	-				
TBM Type	EPB shield machine				
N. jacks	20 push arms equally distributed over the circumference acting on 45 cm long and 25 cm wide ram shoes.				
$T_{max,total}$	26.6 MN				
$T_{max,jack}$	1330 kN				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution	-	-	35 SFRC 47/0.80, L_f/ϕ_f=58, $f_{t,fiber}$=n.d.	55	Proposed by: Kasper et a. 2008 [35]
Solution adopted	The original design solution was used.				

REMARKS CONCERNING THE SOLUTION PROPOSED

It was decided to design the tunnel lining for the bored tunnel with SFRC segments without conventional steel bar reinforcement, using the German SFRC design guideline (DBV, 2001) as the design basis. The specified service life of the tunnel is 100 years.
It is worthwhile noticing that the grout (15 cm thick annular gap) for the district heating is a two-component grout based on cement, bentonite, water and stabilizer as component A and water and sodium silicate (water glass) as component B (grout having short gelling and setting time). This ensures a quick stabilization of the tunnel lining, avoids buoyancy induced uplift of the rings, settlements due to the loads from the back-up trailer and the resulting shear displacements between the segments.
The solution was proposed Kasper et al. [35], based on previously experiences concerning SFRC in tunnel linings reported in literature and based of opportune analytical/numerical calculations. Linear elastic numerical analyses were carried in order to evaluate the loading by the push rams. The failure of the push rams during installation due to mistake withdrawn from the segment has been verified by hand calculations and testing on the bolt connections. Linear elastic numerical analyses were carried for evaluating the permanent loading condition without/with heating of the tunnel.

REMARKS CONCERNING THE SOLUTION ADOPTED

The original design solution consisting in SFRC segments without conventional reinforcement was applied [35]. The experiences during the segment production and tunnel construction can be summarized as it follows. Mainly at the beginning of the segment production, some small cracks were observed after de-moulding at the inner edges of the gasket grooves around the segment corners. In fact, SFRC has a higher risk of honeycomb and blow hole formation compared to conventional concrete. Limited defects around the gasket grooves were reworked after de-moulding. After optimization of the production technique and production parameters, the SFRC segments have not shown a larger size of blow holes and honey-comb defects compared to traditionally reinforced segments. No formation of fibre clusters (balling) has been detected by visual inspection or petrographic analyses. In the first 2.4 km long section of the tunnel drive, cracks at the corners of key segments have occurred in a larger number of rings. The number of these damages has significantly decreased in the second section of the tunnel drive. In general, the segments have shown a high quality and have been easy to handle and install in the tunnel, which is deemed to be one of the reasons for high production rates of up to 49.5 m per day [35].

A.I.6 SAN VICENTE AQUEDUCT PIPELINE TUNNEL

PROJECT PARTICULARS		
Location	San Diego (United States)	
Construction period	2005 – 2010	
Owner	San Diego County Water Authority	
Designer(s)	Halcrow Group Ltd.	King et al., 2007 [36]
Contractor(s)	Traylor Bros. Inc. JF Shea Company (TSJV)	
Engineer(s)	Jacobs Associates	
GENERAL PROJECT DESCRIPTION		
The San Vicente Aqueduct Pipeline consists of a water transmission pipeline running from San Vicente Pump Station to the Rancho Penasquitos Pressure Control and Hydroeletric Facility. The segment manufacturer for the San Vicente to Second Aqueduct Pipeline decided to make use of simplified precasting methods for the production of over 66000 segments to be used for the primary lining of the tunnels. The methods examined and adopted included the use of steel fibres to replace traditional reinforcement bars, vertical molds to minimize surface finishing, and self compacting concrete to minimize vibration and compaction effort. The SVP is constructed predominantly in tunnel with a total length of approximately 17.4 km, constructed mainly through sedimentary rocks (for more details, see King et al., 2007 [36]).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	17.4 km (segmental lining for 2.9+10.27 km, Reach 2 and Reach 4, respectively)	
Boring diameter	-	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring with a taper of 20mm across the diameter	
Thickness	0.178 m	
Internal diameter D_i	3.2 m	
Tunnel aspect ratio (D_i/h)	17.98	King et al., 2007 [36]
Average segment aspect ratio	10.36	
Nº. of segments	6 segments trapezoidal	
Segment length/width	1.7 m /1.22 m	
Connectors	Two spear bolts are used across each longitudinal joint, and the ring has a total of 12 dowels across the circumferential joint. Blind end grout sockets are cast into the centre of each segment	
CONCRETE MIX DESIGN		
Concrete class of strength	Required f_{ck}=41.3 MPa	
Cement	420 kg/m³	
Fly Ash – Class F	140 kg/m³	
Steel fibres	30 kg/m³	
Wash Concrete Sand	715 kg/m³	
3/8" Aggregate	860 kg/m³	King et al., 2007 [36]
HWRA, High Water Reducing Admixiture	4.7 cc/kg	
Compressive strength 7-days	40.8 MPa	
Compressive strength 28-days	50.7 MPa	
Compressive strength 56-days	64.9 MPa	
SFRC flexural strength at 1st crack	8.0 MPa	
SFRC residual flexural strength	3.7 MPa	
Tensile strength	5.4 MPa	King et al.,2007 [36]

ENVIRONMENTAL AND GEOLOGICAL CONDITIONS
The tunnel is bored mainly in sedimentary rocks. The ground traversed within these sections is predominantly weaker Sedimentary formations and Conglomerates between higher strength Granitic rocks either side of the two Reaches (for more details, see King et al., 2007 [36]).

METHOD OF EXCAVATION
Mechanised tunnelling method by means of CTS Tunnel Shield Machines (TSMs), which were one-faced rippers with enhanced features intended to engage the anticipated unique geology of the San Vicente Pipeline Project. The machines can be described in three main shield sections along with the backup and ancillary equipment.

TBM DATA	
Manufacturer	Jesse Engineering facilities in Tacoma, WA
TBM Type	CTS Tunnel Shield Machines
N. jacks	-
$T_{max,total}$	-
$T_{max,jack}$	-
$T_{sc,jack}$	-
Back-up length	-

REINFORCEMENT DESIGN SOLUTION PROPOSED

Sketch of the reinforcement		Conventional reinf. [kg/m³]	Fibre reinf. [kg/m³]	Total reinf. [kg/m³]	Ref.#
Original solution	-	-	30 SFRC 60/0.75, L_f/ϕ_f=80, $f_{t,fiber}$=1050 MPa	30	Proposed by: King et al.., 2007 [36]
Solution adopted	The original design solution was used.				

REMARKS CONCERNING THE SOLUTION PROPOSED
The solution based on only fibre reinforced tunnel segments (SFRC tunnel segment) was finally used. The SFRC segments were produced on a two shift per day basis, five days per week. The average daily production was 48 rings. King et al. 2007 [36] quote the principal advantages and disadvantages related to the use of vertical molds, SCC and SFRC. Referring to vertical molds, it implies the ability to cast two molds per cycle, doubling output with a decrease in cost of molds. The main disadvantage is the top edge finishing, which is not a completely smooth surface since is hand-finished. Depending on the project, this could have a negative effect on the sealing performance of a gasketed segment, and could cause point loading along this edge. Regarding to use of SCC, the vibration was not completely eliminated but significantly reduced since the use of long fibres required the use of vibration in order to reduce the air voids. It was believed that the fibres tended to impede the flow of air bubbles up and out of the mold. SCC enables a quicker concrete placement, less wear and tear on the molds and batch plant, less noise, better consolidation around mold details. The main disadvantage is that the use of SCC can require more time to develop the desired mix with respect to a traditional mix and SCC can be less forgiving than traditional concrete, depending on the local materials and the use of other admixtures. No remarks are reported concerning the fibre orientation in presence of vertical mold. Referring to SFRC, King et al. underlined that SFRC is cost saving with respect of traditional reinforcement and makes possible an automatic dosing and record keeping. The reinforcement is provided in all the area of the segment, no need for special reinforcement around bolt pockets, grout sockets. SFRC enables a general improvement of durability and corrosion resistance. On the other hand, SFRC requires new equipment for accurate and efficient dosing and additional set of performance testing. From the structural point of view, SFRC requires careful analysis of all segment handling, from demoulding to segment erection and produces a segment “weaker” than a fully reinforced section.

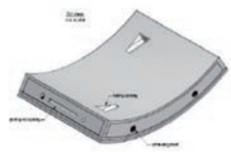
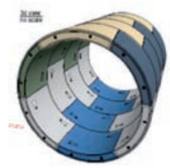
REMARKS CONCERNING THE SOLUTION ADOPTED
The original design solution was used in Reach 2 and Reach 4 as initially planned.

A.I.7 EAST SIDE COMBINED SEWER OVERFLOW TUNNEL

PROJECT PARTICULARS		
Location	Portland Oregon (United States)	-
Construction period	May 2007°– December 2011	
Owner	City of Portland, Bureau of Environmental Services	
Designer(s)	-	
Contractor(s)	Joint venture Kiewit Construction Co. and Bilfinger Berger	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
The East Side Combined Sewer Overflow (ESCSO) tunnel is 9.7 km long tunnel along the east bank of the Willamette River in Portland Oregon for the city of Portland, Bureau of Environmental Services. The tunnel is from 18.3 m to 48.8 m below ground surface and in water bearing alluvium deposits. The SFRC segmental rings are the tunnel lining for approximately 85% of the alignment where the tunnel is located within highly competent Troutdate formation (for more details see Cook et al., 2007 [74).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	9.7 km	-
Boring diameter	-	
Overburden (min-max)	from 18.3 to 48.8 m	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.356 m	
Internal diameter D_i	6.7 m	
Tunnel aspect ratio (D_i/h)	18.84	
Average segment aspect ratio	8.63	
N°. of segments	7 trapezoidal segments and 1 smaller trapezoidal key segment with skewed longitudinal joint of 5 degrees	
Segment length/width	3.53 m (standard segment)/1.5 m	
Connectors	dowels provide connections between segments on the circumferential joints. Each dowel comprises a steel shank within a plastic sleeve with specified pull-out strength. Pullout bolts are placed along skewed longitudinal joints	
CONCRETE PROPERTIES (SFRC REQUIREMENTS)		
Compressive strength, f'_c at 28 days:	41.4 MPa	
Young's modulus (E) at 28 days:	30500 MPa	
Splitting tensile strength at 28 days:	4.76 MPa	
First peak strength, f_1 at 28 days:	4.55 MPa	
Average residual strength, $f'_{cr.3.0}$ at 28 days:	2 MPa	Cook et al., 2007 [74]
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
The tunnel is excavated in Troutdale formation.		

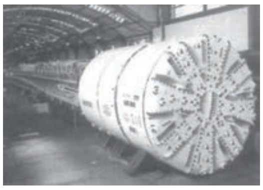
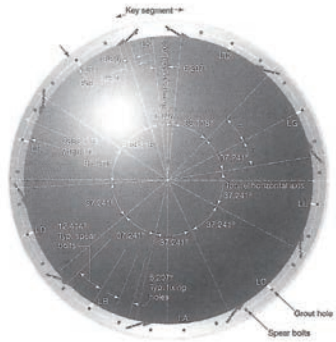
METHOD OF EXCAVATION					
Mechanised tunnelling method by means of a TBM.					
TBM DATA					
Manufacturer	-				
TBM Type	-				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution	-	-	32.6 SFRC	32.6	Proposed by: Cook et al., 2007 [74]
Solution adopted	The original design solution was used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
The solution proposed for the 85% rings of the tunnel alignment consists of only fibre reinforced segments (SFRC segments). The solution was designed by referring to the following permanent loading conditions: external ground and groundwater loads, ring distortion resulting from its erection, seismicity based on an earthquake and transient internal water pressures. These loading conditions were analysed by means of analytical solution and linear numerical analyses. The corresponding internal lining forces were compared to the resisting ones by calculating a domain moment vs. axial force diagram. The principal issues related to tunnel temporary loading conditions (removal of segments from molds, handling and stacking in the storage yard, transportation, lining erection, TBM thrust loads, backfill grouting) were also checked. For instance, for installation underground, the segments were checked in terms of bursting under the action of the loads from the thrust arms of the tunnel boring machine. The final design choice was to ensure that the segments remain elastic and un-cracked under temporary construction loading conditions, which means that the stresses are required to be maintained below the average residual flexural strength as well as below the compressive and tensile strengths of the SFRC (Cook et al., 2007 [74]). Regarding durability issue, to meet the sulphate resistant needs cement type II with fly ash was specified. To further protect against corrosion, the segment thickness was increased by 1.9 centimetres to provide a sacrificial zone on the tension side of the section. Cook et al., 2007 [74] in order to prove the validity of the design solution proposed carried out experimental tests concerning the evaluation of post-cracking strength of SFRC (three types of fibre were considered). Furthermore, tests in compression at full-scale for evaluating the capacity of the actual joint between segments were done. In addition, dowels were tested for tensile and shear capacity in the embedded condition, into SFRC concrete blocks.					
REMARKS CONCERNING THE SOLUTION ADOPTED					
The original SFRC design solution was adopted for 85% of the tunnel lining.					

A.I.8 MONTE LIRIO

PROJECT PARTICULARS		
Location	Monte Lirio (PANAMA Republic)	 [72]
Construction period	March 2010-Under construction	
Owner	Electron Investment SA Panama City	
Designer(s)	-	
Contractor(s)	Constructora SELI Panama SA	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
The Monte Lirio (Panama) hydraulic tunnel is 7878 m long with an external diameter equal to 3.7 m. The thickness of the lining, made with precast segment is equal to 250 mm, thus the internal diameter of the tunnel is 3.2 m (for more details see Caratelli et al., 2012, [34]).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	7.878 km	  Caratelli et al., 2012, [34]
Boring diameter	3.92 m	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.25 m	
Internal diameter D_i	3.20 m	
Tunnel aspect ratio (D_i/h)	12.8	
Average segment aspect ratio	7.75	
N°. of segments	6 segments	
Segment length/width	1.84 m/1.2 m	
Connectors	Bolts in inclined socket (segments of the same ring). Plastic sockets for the connectors between rings.	
CONCRETE MIX DESIGN		
Concrete class of strength	C 55/67	
Cement CEM I 52.5R	-	
Sand	-	
Coarse aggregate	-	
Coarse aggregate	-	
Water	-	
Admixtures	-	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
-		

METHOD OF EXCAVATION					
Mechanised tunnelling method by means of a EPB-TBM with a boring diameter of 3.92 m and a maximum cutterhead thrust of 13560 kN, [75].					
TBM DATA					
Manufacturer	SELI				
TBM Type	EPB				
N. jacks	12				
$T_{max,total}$	13.56 MN				
$T_{max,jack}$	1.13 MN				
$T_{sc,jack}$	0.785 MN				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution	-	-	40 SFRC 30/0.35, L_f/ϕ_f=86, $f_{t,fiber}$=- $f_{L,m}$=4.55 MPa $f_{R1,m}$=4.66 MPa $f_{R2,m}$=4.97 MPa $f_{R3,m}$=4.63 MPa $f_{R4,m}$=3.95 MPa (n.d., estim.) [20] $f_{R1,m}$=6.15 MPa $f_{R3,m}$=6.13 MPa (data from precast tunnel segments production process) [76]	40	Proposed in: Caratelli et al., 2012 [34]
Solution adopted	The original design solution was used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
The design of the tunnel segment was performed in according to the fib Model Code 2010 [9] prescription for FRC, assisted by full scale tests. Two different kinds of experimental tests have been performed: bending test and thrust test. The latter was carried out for reproduce the TBM action on the segment during the excavation process. The full-scale tests were developed only on FRC tunnel segments without any conventional reinforcement. Preliminary 3PBT tests for FRC characterization were carried out on specimens having the same concrete matrix, same fibre content (40 kg/m³) but made with hooked fibres having different geometries (see Caratelli et al., 2012, [34]). The results showed a good behaviour of the elements that satisfy the prescription when 40 kg/m³ of steel fibres having a length of 30 mm and a diameter of 0.35 mm were adopted. It is worthwhile mentioning that, more recently, Di Prisco et al. [6] carried out full-scale tests on tunnel segments having the same geometry of those adopted in Monte Lirio tunnel. However, the segments were reinforced by a combination of macro-synthetic fibres and conventional rebars. The results were satisfactory and have confirmed the opportunities offered by a hybrid solution RC+SynFRC.					
REMARKS CONCERNING THE SOLUTION ADOPTED					
The excavation of the Monte Lirio tunnel is now in progress; in several circumstances the TBM had to push up its loading capacity and no problems with the lining were detected.					

A.I.9 CHANNEL TUNNEL RAIL LINK

PROJECT PARTICULARS		
Location	Cheriton-St Pancras (London)	
Construction period	2004	
Owner	-	
Designer(s)	-	
Contractor(s)	-	
Engineer(s)	Hayden Davies. Eddie Woods	[77]
GENERAL PROJECT DESCRIPTION		
The Channel Tunnel Rail Link (CTRL) is composed by two sections: Section 1 consists of 74 km constructed above ground; Section 2 consists of 38.5 km under the River Thames. CTRL required an internal diameter of 7.15 m. The thickness of the lining, made with precast segment is equal to 350 mm.		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	112.5 km	
Boring diameter	-	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.35 m	
Internal diameter D_i	7.15 m	
Tunnel aspect ratio (D_i/h)	20.4	
Average segment aspect ratio	7.46	
N°. of segments	9+1 segments	
Segment length/width	1.5 m/1.2 m	[77]
Connectors	Straight bolts into plastic insert.	
CONCRETE MIX DESIGN		
Concrete class of strength	C 60/75	
Cement CEM I 52.5R	-	
Sand	-	
Coarse aggregate	-	
Coarse aggregate	-	
Water	-	
Admixtures	-	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
The ground conditions imposed the use of closed face TBMs and a one pass precast gasketted bolted concrete lining. In Section 2, a presence of saline water is caused by the presence of the river Thames.		

METHOD OF EXCAVATION					
Mechanised tunnelling method by means of a EPB-TBM.					
TBM DATA					
Manufacturer	Kawasaki Heavy Industries				
TBM Type	EPBM shields				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution	-	-	30 SFRC 80/0.75, L_f/ϕ_f=107, $f_{t,fiber}$=1050 MPa	30	Proposed in: [77]
	-	-	1 SynFRC (Synthetic Fibre Reinforced Concrete, micro-fibres)	1	
Solution adopted	The original design solution was used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
A 60 MPa concrete mix was selected, with low permeability specified to improve durability and minimize water ingress. A series of tests were carried out to confirm the load bearing capacity of the SFRC segments. The concrete mix has strengths averaging 70 MPa at 28 days rising to 90-100 MPa with age. The inclusion of 1 kg/m³ of polypropylene fibres in the high strength, low permeable mixes tested significantly reduced the risk of explosive spalling.					
REMARKS CONCERNING THE SOLUTION ADOPTED					
The segments are put under significant loads during handling, erection and TBM thrust forces.					

A.I.10 LEGACY WAY

PROJECT PARTICULARS		
Location	Brisbane, Queensland (Australia)	
Construction period	April 2011-2015	
Owner	Brisbane City Council	
Designer(s)	-	
Contractor(s)	Brisbane-based BMD, Italian company Ghella and Spanish business Acciona Infrastructure	
Engineer(s)	-	[78]
GENERAL PROJECT DESCRIPTION		
Legacy Way is a 4.6 kilometer twin-bore toll road tunnel that will connect the Western Freeway at Toowong with the Inner City Bypass (ICB) at Kelvin Grove. It includes two parallel twin-lane tunnels linked by cross passages every 120 metres. The thickness of the lining, made with precast segment is equal to 350 mm, thus the internal diameter of the tunnel is 11.2 m.		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	4.6 km	
Boring diameter	12.4 m	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.35 m	
Internal diameter D_i	11.30 m	
Tunnel aspect ratio (D_i/h)	32.3	
Average segment aspect ratio	12.10	
N°. of segments	9	[78]
Segment length/width	4.2 m/2 m	
Connectors	-	
CONCRETE MIX DESIGN		
Concrete class of strength	C50/60	
Cement CEM I 52.5R	-	
Sand	-	
Coarse aggregate	-	
Coarse aggregate	-	
Water	-	
Admixtures	-	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
During tunnelling, the TBMs will travel through a range of underground geology, most predominantly rock types Bunya Phyllite and Neranleigh Fernvale, which are common in Brisbane's western suburbs. In unstable ground conditions, the gripper shoes are not used. In this instance, the TBM moves forward using parts of the TBM shield to push off against the previously installed concrete segments.) [79].		

METHOD OF EXCAVATION					
In order to excavate the hard rock mass, Transcity undertook drill and blast activities. Mechanized tunnelling method by means of two 12.4 m diameter double shield Tunnel Boring Machines (TBMs) [79].					
TBM DATA					
Manufacturer	Herrenknecht				
TBM Type	Two double shields TBM (Ø 12.34 meters)				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution	-	No (Hybrid solution used only in highly loaded length of tunnel and also in cross passage locations)	40	-	[80]
Solution adopted	The original design solution was used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
-					
REMARKS CONCERNING THE SOLUTION ADOPTED					
The universal type ring with an 8+1 division has been designed with trapezoidal/rhomboidal shaped segments, waterproof gaskets, shear/tension connectors in the ring joint, spare bolts and guiding rods in the longitudinal joints. All segments were fibre reinforced with steel and PP fibres and additional rebar reinforcement along the longitudinal joints to cope with the splitting force. For grouting the annulus gap was developed a cement based two component grout that has been injected through the tail shield in the lower section of the tunnel during the regripping of the TBM and in the upper section through special cast in grout ports in the precast segments during excavation [80].					

A.I.11 HAREFIELD GAS TUNNEL

PROJECT PARTICULARS	
Location	Harefield (United Kingdom)
Construction period	2009
Owner	National grid
Designer(s)	-
Contractor(s)	-
Engineer(s)	-

GENERAL PROJECT DESCRIPTION

The Harefield to Southall contract is a new 18 km steel gas transmission pipeline to meet energy demand in west London which links existing AGIs. Some 2.6 km of pipeline was installed in a pre-constructed tunnel and two thirds in an urban environment to avoid traffic sensitive roads.

TUNNEL CHARACTERISTICS	
Total Tunnel Length	1.0 km
Boring diameter	2.95 m
Overburden (min-max)	-
Lining type	Segmental
Ring type	Universal ring
Thickness	0.18 m
Internal diameter D_i	2.59 m
Tunnel aspect ratio (D_i/h)	14.39
Average segment aspect ratio	7.01
N°. of segments	7 segments +1 key segment
Segment length/width	1.26/1
Connectors	-

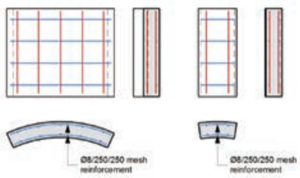
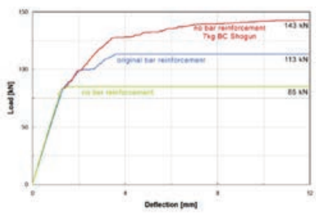
[81]

CONCRETE MIX DESIGN	
Concrete class of strength	C45/55
Cement CEM I 52.5R	-
Sand	-
Coarse aggregate	-
Coarse aggregate	-
Water	-
Admixtures	-

ENVIRONMENTAL AND GEOLOGICAL CONDITIONS

London Blue Clay

METHOD OF EXCAVATION	
Mechanised tunnelling method by means of a EPB-TBM.	
TBM DATA	
Manufacturer	-
TBM Type	-
N. jacks	-
$T_{max,total}$	-
$T_{max,jack}$	-
$T_{sc,jack}$	-
Back-up length	-
REINFORCEMENT DESIGN SOLUTION PROPOSED	

Sketch of the reinforcement		Conventional reinf. [kg/m³]	Fibre reinf. [kg/m³]	Total reinf. [kg/m³]	Ref.#
Original solution		-	-	-	[81]
Solution adopted		-	7 SynFRC (with macro-fibres) L_f = 42 mm, $f_{t,fiber}$=550 MPa	-	[81]

REMARKS CONCERNING THE SOLUTION PROPOSED
-
REMARKS CONCERNING THE SOLUTION ADOPTED
The lining was comprehensively designed and checked using an Finite Element Analysis software. Low-stressed steel bars within the section were replaced with 7 kg/m³ of macro-synthetic fibre. [81].

A.I.12 MALAGA RAIL TUNNEL

PROJECT PARTICULARS		
Location	Malaga-Fuengirola (Spain)	
Construction period	August 2009	
Owner	Aeropuertos Españoles y Navegación Aérea (AENA)	
Designer(s)	-	
Contractor(s)	Acciona Infrastructures and Sando Construcciones	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
The precast concrete lining comprises of concrete rings made up of seven segments giving a finished tunnel diameter of 9 m. The lining was designed with hybrid reinforcement comprising steel reinforcement cage and structural macro synthetic fibre.		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	2.9 km	 [82]
Boring diameter	9.40 m	
Overburden (min-max)	-	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.32 m	
Internal diameter D_i	8.43 m	
Tunnel aspect ratio (D_i/h)	26.3	
Average segment aspect ratio	13.70	
N°. of segments	6 segments+1 key segment	
Segment length/width	4.38/1.5	
Connectors	-	
CONCRETE MIX DESIGN		
Concrete class of strength	C40/50	 [82]
Cement CEM I 52.5R	-	
Sand	-	
Coarse aggregate	-	
Coarse aggregate	-	
Water	-	
Admixtures	-	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
TBM passing beneath the river, the existing suburban rail tracks and the land where the second runway is being buit.		

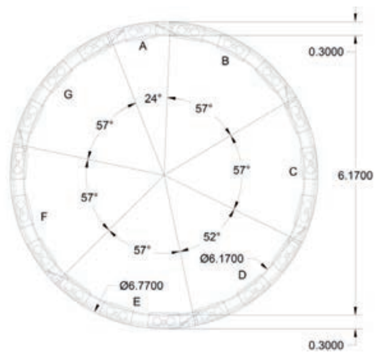
METHOD OF EXCAVATION					
Mechanised tunnelling method by means of a TBM.					
TBM DATA					
Manufacturer	Herrenknecht				
TBM Type	Earth pressure balance TBM “Biznaga”				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution		Yes	5 SynFRC (macro-fibres) L_f=48 mm $f_{t,fiber}$=550 MPa	-	[82]
Solution adopted	The original design solution was used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
-					
REMARKS CONCERNING THE SOLUTION ADOPTED					
Reinforcement options were carefully evaluated; it was decided to use 5 kg/m³ of macro-synthethic fibers in combination with traditional rebars.					

A.I.13 ABU HAMOUR

PROJECT PARTICULARS		
Location	Abu Hamour Drainage Tunnel, Qatar	 [83]
Construction period	2015-2016	
Owner	-	
Designer(s)	Design and build Contract: Impregilo-COWI	
Contractor(s)	Salini-Impregilo	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
Abu Hamour (Musaimeer) Surface & Ground Water Drainage Tunnel is 9.5 km long. Integral to the project, 21 shafts will be built in order to facilitate inflows along its route. This project was the first TBM mined tunnel with FRC in Doha - Qatar.		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	9.5 km	[83]
Boring diameter	-	
Overburden (min-max)	7-12 m	
Lining type	Segmental	
Ring type	-	
Thickness	0.25 m	
Internal diameter D_i	3.70 m	
Tunnel aspect ratio (D_i/h)	14.8	
Average segment aspect ratio	7.6	
N°. of segments	6 segments +1 key segment	
Segment length/width	1.90 m/1.3 m	
CONCRETE MIX DESIGN		
Concrete class of strength	C 45/55	[83]
Special mix consisting of a triple blend concrete made by ground granulated blast-furnace slag, ordinary portland cement and fly ash (GGBS + OPC + FA) suitable for harsh ground and groundwater conditions (chlorinated environment and sulfate aggression).	-	
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
Tunnelling challenges include the first ever tunnelling in Rus Formation (Chalky/Dolomitic Limestone and Chert) and about 5 km of mixed face condition (Limestone from Simsima Formation and Midra Shale). The Abu Hamour Tunnel is located in a hot humid environment with high concentrations of chlorides and sulphates which are present in the ground and groundwater, as well as in the storm water and dewatering the will flow through the tunnel. Chloride induced corrosion and sulphate attack are considered to be the principal deterioration mechanisms that may influence the long term durability of concrete structures. Both chloride penetration and sulphate accumulation are accelerated by the cyclic wetting and drying of the structural concrete.		

METHOD OF EXCAVATION					
The main tunnels will be constructed by two Herrenknecht EPB machines to be assembled in a launching shaft located in the middle of the tunnel alignment and will proceed in opposite directions to receiving shafts at the western and eastern end of the tunnel alignment.					
TBM DATA					
Manufacturer	Herrenknecht				
TBM Type	EPB				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	-				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution		Yes	40 SFRC 60/0.75, Lf/ϕf=80, $f_{t,fibre}$>1500 MPa	40	[83]
Solution adopted	The original design solution was used.				
REMARKS CONCERNING THE SOLUTION PROPOSED					
The dimensional tolerances of the steel fibres comply with the European Standard EN 14889-1. A minimum design FRC class of "4c" was required, according to Model Code 2010. The original conceptual design achieved a 100-years design life and 50 years maintenance-free performance for the tunnel by applying HDPE liner on the intrados and epoxy coating on the extrados. The design-build team proposed an alternative utilizing, among other things, steel fibre reinforcement, dosage rate 40 kg/mc, and special mix consisting of a triple blend concrete made by ground granulated blast-furnace slag, ordinary portland cement and fly ash (GGBS + OPC + FA) suitable for harsh ground and groundwater conditions (chlorinated environment and sulfate aggression).					
REMARKS CONCERNING THE SOLUTION ADOPTED					
The excavation of the Abu Hamour Drainage Tunnel is now completed. A design solution with segment reinforced by steel fibres only was successfully used.					

A.I.14 DOHA METRO RED NORTH LINE

PROJECT PARTICULARS		
Location	Doha Metro Red North Line, Qatar	[84]
Construction period	2013 - 2018	
Owner	Qatar Railway	
Designer(s)	CIO JV (COWI-ITALFERR-ONEWORKS)	
Contractor(s)	ISG JV (Salini Impregilo – SK – Galfar)	
Engineer(s)	Jacobs	
GENERAL PROJECT DESCRIPTION		
The “Doha Metro Red Line North Underground” Project represents a portion of a Master Plan for a large mobility system promoted by Qatar under the scope of the National Development Plan for 2030. The Red Line North starts from Msheireb station in the South to the Golf Course in the North. The contract includes the Design and Construction of the following structures: - 2 parallel tunnels having an approximate length of 11.4 km with an internal diameter of 6.17 m. In particular, the precast segments are made of Steel Fibre Reinforced Concrete only (class 4c acc.to CEB-FIP Model Code 2010), except for the special rings where stainless steel cages have been used. The C40/50 concrete mix is enriched with polypropylene fibres in order to achieve the prescribed fire resistance properties. The Tunnels will be provided with a cast in situ invert and side walls to accommodate the metro track system. - 22 Tunnel Cross Passages. These are excavated in NATM managing the groundwater with an extensive pre-grouting system. The structures are constructed with a cast in situ final lining enveloped in compartmented injectable waterproofing system. - Underground Stations including the shallow areas for the ventilation shafts and the relevant access structures. 1 Cut & Cover Structure, 2 Emergency Exits and 1 Tunnel Ventilation Shaft. These have been excavated with various temporary support methodologies such us shotcrete lining, diaphragm walls in the areas next to skyscrapers, secant and tangent piles and kingposts. Challenging dewatering activities have been implemented in all the excavation works requiring in some case the use of plastic walls to create a water cut off. All the structures are provided with a compartmented post injectable waterproofing system and are mainly constituted by cast in situ concrete with high durability and fire resistance requirements.		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	22.832 km	[84]
Boring diameter	7.10 m	
Overburden (min-max)	10.21 m – 37 m	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.3 m	
Internal diameter D_i	6.17 m	
Tunnel aspect ratio (D_i/h)	20.6	
Average segment aspect ratio	10.7	
N°. of segments	6+1 (1 trapezoidal key segment + 5 parallelogram segments + 1trapezoidal counter key segment)	
Segment length/width	3.20 m/1.5 m	
CONCRETE MIX DESIGN		
Concrete class of strength	C 45/55	[84]
Cement CEM I 52.5R	OPC Type I (30% -132kg/m^3), GGBS (65% - 286kg/m^3), Microsilica (5% - 22kg/m^3)	
Sand	Washed Natural Sand (800 kg/m^3)	
Coarse aggregate	20mm (577 kg/m^3)	
Coarse aggregate	10mm (484 kg/m^3)	
Water	158 kg/m^3	
Admixtures	HRWR (PCE) S/P Type F (4.4kg/m^3)	

ENVIRONMENTAL AND GEOLOGICAL CONDITIONS

The Red North Line is located in an environment with high concentrations of chlorides and sulphates (up to 6000mg/l) present in the soil and groundwater. The requirements to the concrete mix i.e. max. water/cement ratio and min. cementitious content has been determined to ensure a sufficient sulphate resistant concrete as tested in accordance with the German SVA method (max expansion of 0.5mm/m at 90days). The exposed geological formations in Qatar are all of Tertiary to Quaternary age. Local geological records a 20m to 25m thickness of Simsima Limestone overlying shales of the Midra Shale unit, clays and limestones of the Rus Formation. The Simsima Limestone is found from 0m to 9m below ground level, and in thickness up to 32m. The Simsima Limestone has been sub-divided into three sub-groups described by the degree of weathering. Weathered Simsima Limestone: matrix supported crystalline dolomitic limestone (< 50%) embedded into very porous limestone and uncemented carbonate sand, silt and clay, moderately to highly weathered. Less weathered Simsima limestone: bimodal grain supported crystalline dolomitic limestone (> 50%) with larger pockets and patches of porous limestone and uncemented carbonate sand, silt and clay, slightly to moderately weathered. Simsima Limestone: (generally) moderately strong, fresh to moderately weathered bimodal crystalline dolomitic limestone with pockets of softer material and some solution vugs. The Midra Shale is found with top at depths varying from 12m to 32m below ground level and average thickness of 3.7m. A weak to moderately strong, slightly to highly weathered shale interbedded with dolomitic limestone. Russ Formation: A (generally) very weak to moderately weak, slightly to highly weathered chalky and marly dolomitic limestone. Found with top at depths varying from 16m to 35m below ground level and extending to great depth (up to 100m). This solid geology is covered by a layer of unconsolidated overburden of natural marine silts, sands with calcarenite horizons which in turn have been recently (in the early 1970's) covered by a variable thickness of hydraulically placed fill material. Karstic features are common in Qatar [84].

METHOD OF EXCAVATION

Each of the TBMs for the Red Line North measures 7.1 m in diameter and 110 m in length with a max thrust pressure of 57.78MN.

TBM DATA

Manufacturer	Herrenknecht
TBM Type	EPB
N. jacks	38 (2 jacks act on each steel thrust shoes)
$T_{max,total}$	57.78 MN
$T_{max,jack}$	1.52 MN
$T_{sc,jack}$	-
Back-up length	110 m

REINFORCEMENT DESIGN SOLUTION PROPOSED

Sketch of the reinforcement		Conventional reinf. [kg/m³]	Fibre reinf. [kg/m³]	Total reinf. [kg/m³]	Ref.#
Original solution			40 SFRC 50/0.75, L_f/ϕ_f=67, $f_{t,fibre}$>1500 MPa $f_{L,m}$=6.34 MPa $f_{R1,m}$=7.85 MPa $f_{R3,m}$=9.0 MPa. [20]	-	[84]
Solution 01			2 SynFRC (Synthetic Fibre Reinforced Concrete, micro-fibres)	-	[84]
Solution adopted	The original design solution was used.				

REMARKS CONCERNING THE SOLUTION PROPOSED

The dimensional tolerances of the steel fibres comply with the European Standard EN 14889-1. The steel fibres reinforced concrete shall be minimum type 4.0 c according to CEB – FIP Model Code 2010 classification. Stainless Steel Reinforcement (SSR) 160kg/m3 used only at cross passages and sensitive locations. With regards to the corrosion, it is assumed that, for concrete reinforced with steel fibres, this may occur e.g. due to carbonation of the intrados surface which is limited to the near-surface area of the segments. This localized corrosion of the steel fibres does not cause damage (cracking and/or spalling) of concrete and it is consequently not considered problematic for the entire required service life of 120 years. On the basis of the above, no specific durability design is necessary with regard to corrosion of fibres. For the special segments the risk of reinforcement corrosion is eliminated by using SSR as reinforcement. Thus, the durability design of the segmental lining is mainly determined by the risk of sulphate attack [84].

REMARKS CONCERNING THE SOLUTION ADOPTED

The excavation of the Red Line North Tunnels is now completed. A design solution with segment reinforced by steel fibres only was successfully used.

A.I.15 VALENCIA METRO LINE 1: FEASIBILITY STUDY

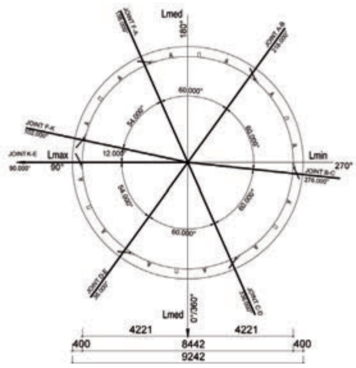
PROJECT PARTICULARS	
Location	Valencia (VENEZUELA)
Construction period	2001-2006
Owner	-
Designer(s)	GEODATA
Contractor(s)	Ghella
Engineer(s)	-

Perri, 2004 [85]

GENERAL PROJECT DESCRIPTION

Line 1 of the Valencia Metro starts in the southern part from the Monumental station and proceeds northward under the Avenue Bolívar up to the Miranda station at the city center. The total length is approx. 5 km, completely underground, with a small part built as an artificial tunnel, approx 0.5km long between the Monumental and Las Ferias stations and the major part, from the north part of Las Ferias station, being a tunnel built with an EPB TBM, excavation diameter of 9.52 m. The configuration of the running tunnel is single-tube, double-track. The project belongs to a mass transit system, an underground metro, with a volume of passengers of 35.000 per hour in each direction. The design paid particular attention to the correct dimensioning of the electro-mechanical installations and the entrance–exit of the stations in order to have an easy construction and functionality for the passengers in relation to the expected traffic volume. 7 stations between the Las Ferias and Miranda stations the Palotal – Santa Rosa – Michelena – Lara – Cedeño were built before the tunnel excavation, with an inverted cut and cover method. Line 1 will be completed with a second stage with 5 more stations connecting Guaparo town in the metropolitan area and the Universidad area. The project includes there-modelling of the Avenue above the metro line (from Plaza de Toros to Avenue Bolívar Sur and to the city centre). [86].

TUNNEL CHARACTERISTICS	
Total Tunnel Length	5 km
Boring diameter	9.52 m
Overburden (min-max)	7-17 m
Lining type	Segmental
Ring type	Universal ring
Thickness	0.40 m
Internal diameter D_i	8.44 m
Tunnel aspect ratio (D/h)	21.1
Average segment aspect ratio	12.04
N°. of segments	4 segments+2 counter key segments, 1 key segment
Segment length/width	4.6 m/1.5 m
Connectors	Bolts in inclined socket.



Perri, 2004 [85]

CONCRETE MIX DESIGN	
Concrete class of strength	C40/50
Cement CEM I 52.5R	-
Sand	-
Coarse aggregate	-
Coarse aggregate	-
Water	-
Admixtures	-

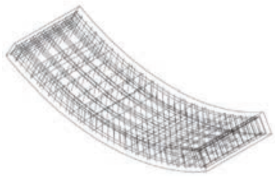
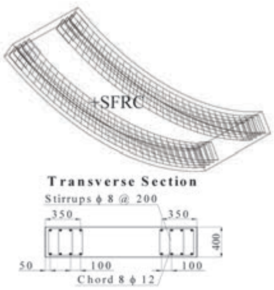
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS

Tunnel in soil, which profile principally consists of of silty clay and clayey sand.

METHOD OF EXCAVATION
The tunnel is excavated with an EPB TBM, excavation diameter of 9.52 m.

TBM DATA	
Manufacturer	LOVAT
TBM Type	EPB-Type Earth Pressure Balanced Tunnel Boring Machine
N. jacks	30
$T_{max,total}$	-
$T_{max,jack}$	-
$T_{sc,jack}$	2.365 MN (thrust shoes 710 x 110 mm)
Back-up length	-

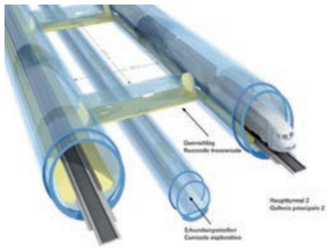
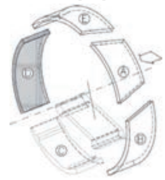
REINFORCEMENT DESIGN SOLUTION PROPOSED

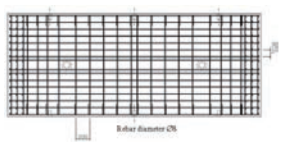
Sketch of the reinforcement		Conventional reinf. [kg/m³]	Fibre reinf. [kg/m³]	Total reinf. [kg/m³]	Ref.#
Original solution		76	-	76	Reported in: Perri, 2004 [85]
Solution 1		38 (in two chords along the longer segment sides).	40 SFRC 50/0.75, L_f/ϕ_f=67, $f_{t,fiber}$=1100 MPa $f_{lf,m}$=6.152 MPa $f_{eq(0-0.6),m}$=6.098 MPa $f_{eq(0.6-3),m}$=5.289 MPa $f_{lf,k}$=3.448 MPa $f_{eq(0-0.6),k}$=2.041 MPa $f_{eq(0.6-3),k}$=1.516 MPa [71]	78	Proposed by: Plizzari et al. 2006 [87]
Solution adopted	The original design solution was used.				

REMARKS CONCERNING THE SOLUTION PROPOSED
The solution 01 was proposed by Plizzari et al., 2006 [87] based on non-linear numerical analyses of thrust-phase (when the TBM pushing arms are acting of the last constructed ring in order to guarantee the drill process). Several analyses were carried out by considering several reinforcement combinations (SFRC only, RC+SFRC RC only). Furthermore, numerical analyses of the final stage (ground pressure acting on the lining) were carried out by also properly modelling the longitudinal joints. The final solution proposed was a combination of traditional rebars concentrated in two chords and amount of 40 kg/m³ of steel fibres. The research was developed when the construction process of the tunnel was already at an advanced stage. Hence, it was not possible to adopt this design solution.

REMARKS CONCERNING THE SOLUTION ADOPTED
The original design solution was adopted.

A.I.16 BRENNER BASE TUNNEL: FEASIBILITY STUDY

PROJECT PARTICULARS		
Location	Italy-Austria, Base tunnel from Innsbruck to Fortezza	 [88]
Construction period	Work is currently underway on the exploratory tunnel which is being built as a preliminary measure for geological and hydrogeological prospecting. This preliminary research will help to minimize both construction costs and risks. Excavation of the pilot hole between chainage 0+000 and chainage 10+400 to be bored by double shield TBM having an excavation diameter of 6,30 metres. The first section of the pilot hole (150 m) to be performed by drill and blast method.	
Owner	BBT Brenner Basistunnel BBT SE - Innsbruck (Austria)	
Designer(s)	-	
Contractor(s)	Joint Venture made of Pizzarotti (Leader), Condotte, SELI, Collini (Italy), Bilfinger Beger, Alpine Mayreder, Jaeger Bau, Beton Und Monierbau (Austria).	
Engineer(s)	-	
GENERAL PROJECT DESCRIPTION		
The Brenner Base Tunnel is a tunnel 55 km long between Italy and Austria. The project consists of two parallel railway tunnels and a central pilot hole. The latter was designed for the application in mechanised tunnelling with a TBM doubles shield machine between chainage 0.000 and chainage 10+400 (for more details see Caratelli et al., 2011, [89]).		
TUNNEL CHARACTERISTICS		
Total Tunnel Length	55 km	 [75]  [89]
Boring diameter	6.3 m (pilot hole)	
Overburden (min-max)	≈1300 m	
Lining type	Segmental	
Ring type	Universal ring	
Thickness	0.2 m	
Internal diameter D_i	5.6 m	
Tunnel aspect ratio (D_i/h)	28	
Average segment aspect ratio	17.67	
N°. of segments	3 segments+1 key segment + 2 counter-key segments	
Segment length/width	3.64 m/1.5 m	
Connectors	No longitudinal devices are present between the segments of the ring.	
CONCRETE MECHANICAL PROPERTIES		
RC segment RC segments present an average compressive strength of 50 MPa.		
FRC segment The concrete matrix of SFRC was opportunely designed for enhancing the performance of the adopted steel fibres, characterised by diameter of 0.35 mm and a length of 30 mm and whose dosage was equal to about 40 kg/m^3. SFRC segments present an average compressive strength of 75 MPa. The equivalent strength, adopted in the design process, was defined in agreement with the CNR-DT 204 [11] as the average stress evaluated for crack openings ranging between 0.6 and 3 mm. This equivalent flexural post-cracking strength was 6.5 MPa.		
ENVIRONMENTAL AND GEOLOGICAL CONDITIONS		
The tunnel layout gets across ordinary granite (specific load γ=26.5 kN/m^3, cohesion c=4000 kPa, friction angle ϕ=31°, Young modulus E=14 GPa, poisson ratio υ=0.3), [89].		

METHOD OF EXCAVATION					
Mechanised tunnelling method with a TBM doubled shield machine, constructor SELI, Italy [89-75].					
TBM DATA					
Manufacturer	SELI				
TBM Type	TBM doubled shield machine				
N. jacks	-				
$T_{max,total}$	-				
$T_{max,jack}$	1.579 MN				
$T_{sc,jack}$	-				
Back-up length	-				
REINFORCEMENT DESIGN SOLUTION PROPOSED					
Sketch of the reinforcement		**Conventional reinf. [kg/m³]**	**Fibre reinf. [kg/m³]**	**Total reinf. [kg/m³]**	**Ref.#**
Original solution		67 (n.d.)	-	67	Reported in: Caratelli et al. 2011, [89]
Solution 1		-	40 SFRC 30/0.35, L_f/ϕ_f=86, $f_{t,fiber}$=- $f_{eq(0-0.6),m}$=6.61 MPa $f_{eq(0.6-3),m}$=6.29 MPa (n.d., estim) [71]	40	Proposed by: Caratelli et al. 2011, [89]
Solution adopted	-				
REMARKS CONCERNING THE SOLUTION PROPOSED					
The solution 01 was proposed by Caratelli et al., 2011 [89] based on experimental research. Full scale tests (segment type “D”) on both traditional reinforced concrete and fibre reinforced elements have been performed. Bending tests were carried out in order to compare the behaviour of the segments under flexural actions, while point load tests were developed with the aim of simulating the thrust force induced by the TBM, and the effect of load concentration and splitting phenomena. The tests results showed that, in this peculiar application, the fibre reinforced concrete can substitute the traditional reinforcement. In fact, the FRC elements tested under flexure exhibited a better behaviour in terms of cracking control with respect to traditional RC segment; the load related to the first crack formation was higher, and the cracks opening were reduced.					
REMARKS CONCERNING THE SOLUTION ADOPTED					
-					

The case studies of FRC precast tunnel segments in combination or not with conventional rebars, previously reported in Table 1, have been briefly analysed in order to draw possible tendencies resulting from twenty years of FRC practice in tunnel linings.

Based on the data reported in Table 1, the tunnel linings reinforced with fibre reinforcement only are about 71%, and for the remaining 29% a hybrid solution (fibres + conventional rebars) is used (Figure 29).

In Figure 30, the internal diameter D_i of tunnel linings reinforced by fibre only and by fibre and conventional rebars are shown. Based on the collected case studies, the solution with fibres only has been used with diameter less than 8 m (Figure 30). The hybrid solution has generally not been used for lining having very small diameter (less than 4 m, Figure 30). In Figure 31 and in Figure 32 the distribution of Di is reported for each considered project. In the case of Legacy Way, Airport link and Oënzberg-Shield fibre reinforcement only was used for large diameter tunnels.

In Figure 33 the tunnel lining aspect ratio (D_i/h) of the reported case studies was briefly analysed. As a reference value, the typical tunnel aspect ratio suggested by many recommendations [90], which is equal to 20, was assumed. It seems that the design hybrid solution was used in presence of high slenderness (>20, Figure 33). In Figure 34 and in Figure 35 the distribution of D_i/h is reported for each considered project. It is worthwhile noticing that D_i/h is a simplified parameter and it does not provide any information on issues such as, for instance, the TBM ram loads or the expected ground conditions.

Finally, it is highlighted that no tendencies concerning fibre content were analyses, and this was done on purpose because, as strongly underlined in the document, fibre dosage is not a meaningful information for design purposes, since it does not provide any information on the post-cracking residual strengths required by designers for a specific tunnel lining.

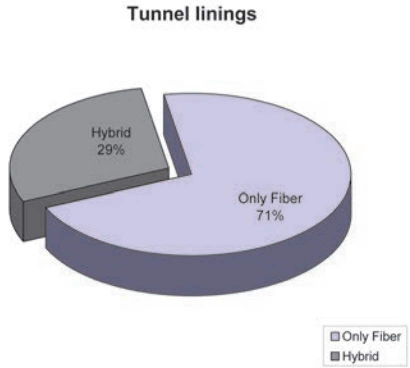


Figure 29 : Distribution of the reinforcement solutions adopted in the tunnel lining case studies reported in Table 1.

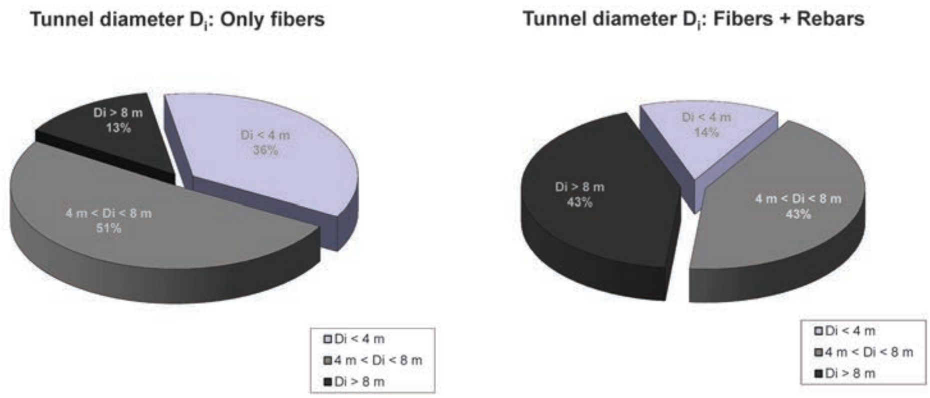


Figure 30 : Distribution of the internal diameter (D) in the tunnel lining case studies reported in Table 1.

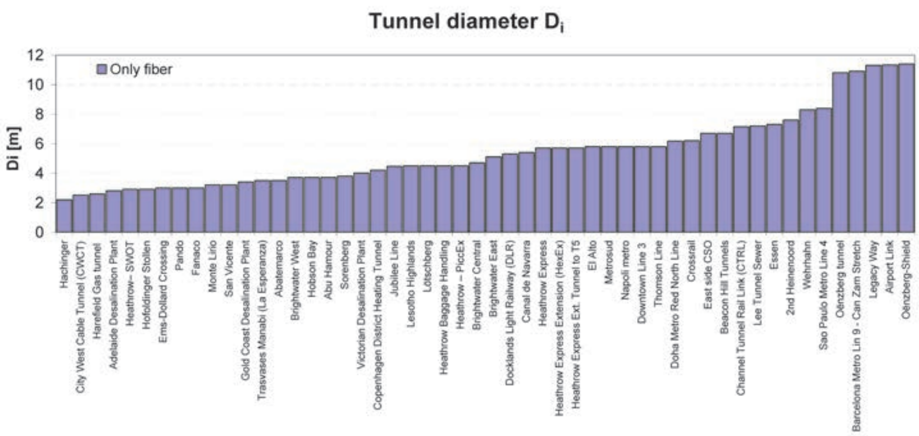


Figure 31 : Distribution of the internal diameter (D) in FRC tunnel lining case studies reported in Table 1

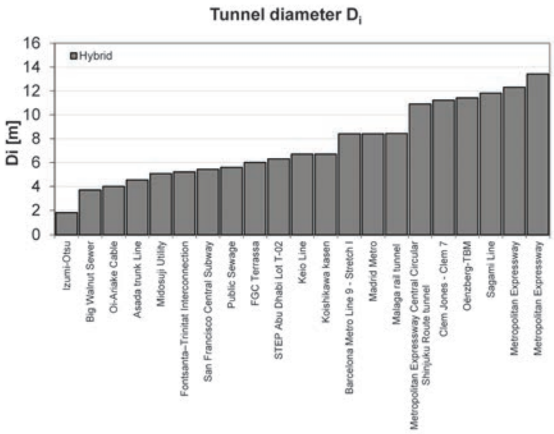


Figure 32 : Distribution of the internal diameter (D) in RC+FRC (hybrid, fibres+rebars) tunnel lining case studies reported in Table 1.

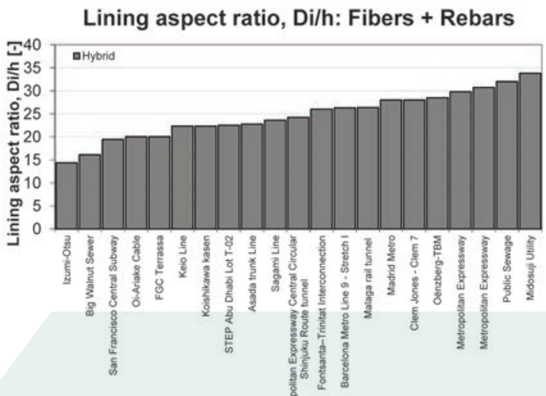


Figure 35. Distribution of the tunnel aspect ratio (D/h) in RC+FRC (hybrid, fibres+rebars) tunnel lining case studies reported in Table 1.

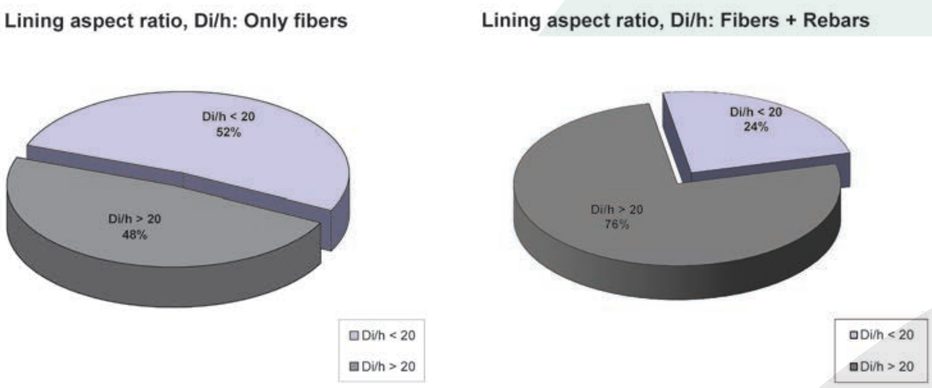


Figure 33 : Distribution of the tunnel aspect ratio (D/h) in the tunnel lining case studies reported in Table 1.

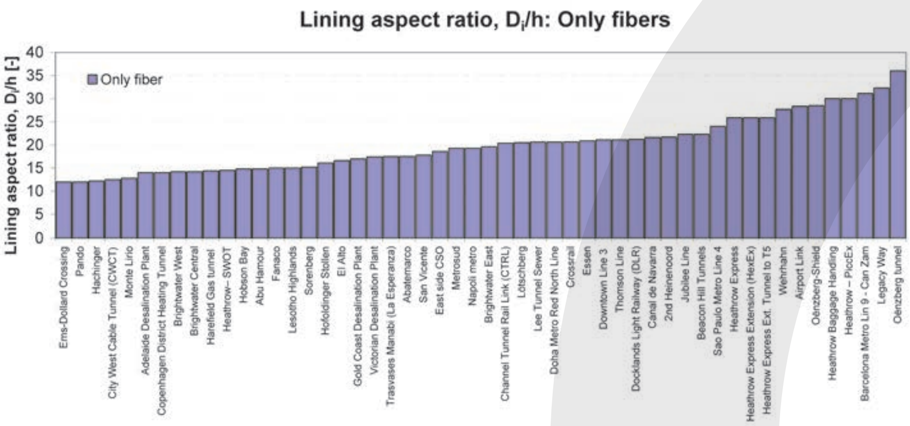


Figure 34. Distribution of the tunnel aspect ratio (Di/h) in FRC tunnel lining case studies reported in Table 1

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