Numerical assessment of the potential of fibre reinforced shotcrete for structural strengthening of underground masonry tunnels

Mahsa Taheri a,*, Joaquim A.O. Barros b, Hamidreza Salehian a, António Ventura-Gouveia c

a ISISE, Department of Civil Engineering, University of Minho, Guimarães, Portugal
b ISISE, Institute of Science and Innovation for Bio-Sustainability (IB-S), Department of Civil Engineering, University of Minho, Guimarães, Portugal
c ISISE, Department of Civil Engineering, Polytechnic Institute of Viseu, Viseu, Portugal

1. Introduction

The railway network as the safest form of land transport with the lowest risk of death and serious injury (ERRAC, 2017), has a key role in the transport of passengers and the distribution of goods. Rail transport is also recognized as the most environmentally friendly, with much higher energy efficiency, lesser CO₂ emission, widespread use of electric traction, and relatively smaller lands-occupancy by its infrastructure (ERRAC, 2017). These advantages are much more pronounced in high-speed railways, due to the significant decrease in the journey time and enhancement of passengers’ comfort, which justifies the increasing investment in this sector in recent decades. In fact, in the period between 2000 and 2018, the EU provided 23.7 billion euros of co-funding to develop high-speed rail infrastructures (EN, 2018). As a matter of fact, developing high-speed rails is significantly expensive (up to 25 million euros per km), which applies for the development of economic and environmental competitive upgrading solutions of the existing railway networks, mainly for underground tunnels due to the remarkable portion of the project time and budget.

The underground tunnels of the Portuguese railway network were typically constructed between 1876 and 1911. Such aged structures, with over a century of uninterrupted service, are suffering from insufficient safety and serviceability indexes due to a variety of deterioration types. The loss of mortar between bricks or rocks of the masonry structure, cracking and deterioration of the tunnel interior face, and rock displacement are frequent issues that typically occur due to the steam of temperature, moisture, penetration of water, and change of the pore water pressure around the tunnel and hydrostatic pressure on the walls (Yoo, 2016).

This scenario is representative of what happens in several countries with...
relatively old railway networks.

Over the past years, the reinforced shotcrete technique has been applied for the stabilization and securing of rock and soil systems in underground tunnels (Kalhor and Bagherpour, 2017) in which concrete or mortar is pneumatically projected onto a steel reinforcement mesh pre-installed on the tunnel interior surface (Bernard, 2002; Colombo et al., 2009; Liu et al., 2020). Since the 90s, the use of fibre-reinforced shotcrete (FRS) has gradually increased in tunnel stabilization and rehabilitation. The FRS technique minimizes the time of intervention due to the elimination of steel mesh, with consequent economic benefits (Bernard and Thomas, 2020; Cengiz and Turanli, 2004). The discrete fibres can be pre-added during the production of the fibre-reinforced concrete (FRC), which is designated by wet FRS technology (Guler et al., 2021), or can be added at the shotcrete moment through the dry FRS technology (Armengaud et al., 2018). In both situations, the role of the fibres is to provide concrete with the enhanced post-cracking tensile capacity capable of assuring the structural safety for the serviceability and the ultimate limit state conditions (Taberi, 2021; Taberi et al., 2020). This post-cracking enhancement is a consequence of the resisting pull-out mechanisms provided by fibres bridging the cracks (Oraee et al., 2011), leading to a noticeable improvement in the toughness of the shotcrete and increasing its spalling resistance in fire and explosion incidents (Bernard and Thomas, 2020).

Mechanical properties of FRC are strongly dependent on its composition (in particular the type and content of fibres and the strength of the matrix) and fibre distribution and orientation. Steel fibres are the type regularly used in FRS (Yan et al., 2018), however, the use of macro-synthetic fibres has recently increased, mainly in underground applications of relatively high aggressive environmental conditions (Kaufmann et al., 2013; Yin et al., 2015). Two main classes of FRC are being used for attending to the structural demands of the corresponding applications: FRC of strain-softening (SSFRC) and strain-hardening (SHFRC) character. The distinction is assessed by performing direct tensile tests with unnotched specimens (Naaman, 2008). An SSFRC has a maximum tensile capacity that coincides with the load at the crack initiation, with the formation of a unique crack. An SHFRC, however, comprises an additional phase after crack initiation, where the tensile load carrying capacity continues increasing with the formation of several narrow cracks up to the moment when one of these micro-cracks degenerates in a macro-crack with the entrance of the tensile response in a softening stage (Pereira et al., 2012). The tensile deformation capacity of an SHFRC is considerably higher than an SSFRC, but the tensile capacity depends significantly on the strength of the matrix (Kim et al., 2012). By engineering the FRC composition, it is possible to have SHFRC of moderate tensile strength, but with very high tensile deformability and energy absorption capacity (Committee, 2003), or an SHFRC of relatively high tensile strength, deformability and energy absorption capacity (Hakman et al., 1992; Naaman, 1992).

Thanks to such a customisability character, FRCs are interesting materials for the strengthening and rehabilitation of existing structures. Barros and Sena-Cruz (2001) implemented a thin layer of SFRC on the compression zone of reinforced concrete (RC) slabs to increase their flexural capacity. Martínlola et al. (2010) successfully utilised a U-shape FRC jacket made of 2.5 % of steel fibres with 40 mm thickness to strengthen RC beams in bending. The strengthening performance of SHFRC in RC beams is also highlighted by Jeyasehar and Balamur-alkrishnan (2012) by using slurry infiltrated fibre concrete (SIFCON) and self-consolidated micro-concrete (SCMC) lamintes. A layer of ultra-high performance strain-hardening cemementitious composite reinforced with steel bars was employed by Hussein et al. (2012) for the flexural strengthening of RC beams and a noticeable increase in the load at yield initiation and at the ultimate stage was obtained. SHFRC plates reinforced with fibre reinforced polymer (FRP) laminates were successfully adopted for the strengthening of RC elements (Esmaeili and Barros, 2015). In an experimental study conducted by Maheri et al. (2004), a new high-performance fibre-reinforced cementitious composite, designated by CARDIFRC, was employed in the form of precast strips to enhance the seismc performance of both damaged and undamaged RC beams. The strengthening potential of SHFRC is also well-acknowledged for increasing the shear capacity of RC beams (Baghi and Barros, 2016; Maheri et al., 2004; Mostosi et al., 2011) and energy absorption and dissipation capacity of columns (Cassese et al., 2021; Dogan and Neven, 2003; Shannag et al., 2002). FRCs have been also utilised for strengthening masonry beams and vaults (Esmaeili et al., 2013; Gattesco et al., 2018; Hüller, 2010). In the experimental research carried out by Kamrava et al. (2021), an increase of 200 % in the in-plane strength and 50 % in structural stiffness was observed by strengthening with SHFRC for seismic loading conditions, unreinforced masonry walls with an opening. The aimed mechanical properties for the adopted SHFRC were attained by using a relatively large amount of fibres. The use of SHFRC in the shotcreting technology poses extra challenges, since the relatively high dosage of fibres may lead to high internal friction and a consequent decrease in its pumpability. Through a parallel control of micro-mechanical parameters- and rheology-based design, Kim et al. (2003) developed a sprayable engineered cementitious composite (ECC), whose strain-hardening behaviour in the hardened state was assessed through direct tensile tests. The tensile and flexural properties of this SHFRC shotcrete and those evaluated in cast SHFRC were demonstrated to be similar (Kanda et al., 2002; Kim et al., 2004). Nevertheless, SHFRCs are relatively expensive materials and their use in large-scale applications, like underground tunnel strengthening, must be well justified by using rational and reliable design methodologies.

In this paper, a novel strengthening approach is developed for the structural rehabilitation of tunnels with the technology of FRS in which the potential of combining SSFRC with SHFRC is assessed by performing numerical simulations. It is demonstrated that by selecting FRC with proper properties and disposed in the correct places by suitable FRS technology, significant economic and strengthening benefits can be obtained. For this purpose, material nonlinear analysis with a curved shell layered approach based on the finite element method (FEM) is performed. The reliability of the adopted material nonlinear analysis is firstly demonstrated by simulating experimental tests with curved segments and masonry prototypes flexurally strengthened with SSFRC and SHFRC. Then, the potential of the SSFRC/SHFRC curved shell layer approach is explored for the strengthening of a typical Portuguese underground tunnel. The adopted design methodology, however, can be extended to any type of tunnel requiring strengthening intervention.

2. Short description of the adopted FEM-based material nonlinear approach

The numerical simulations were carried out with the FEM-based FEMIX 4.0 software (Azevedo et al., 2003; Barros, 2016). FEMIX 4.0 is based on the displacement method and includes a large library of finite element (FE) types, namely 3D frames and trusses, plane stress elements, flat or curved elements for shells, and 3D solid elements. This element library is complemented with a set of point, line and surface springs that model the contact with the supports and surrounding medium, and also several types of interface finite elements to model inter-element contact. FEMIX 4.0 includes several types of constitutive models for the material nonlinear behaviour of structures formed by cement, polymer- and metallic-based materials, under static and dynamic loading conditions, being also possible to simulate geometric nonlinear effects and thermohygro-mechanical phenomena like concrete maturattion and fire. In the same analysis, several types of material nonlinear effects (like concrete maturattion, shrinkage, creep and cracking) can be combined using the concept of a pattern of behaviours (Valente et al., 2019). The software comprises advanced numerical techniques such as the Newton-Raphson method combined with arc-length techniques and path dependent or independent algorithms (Sena-Cruz, 2004; Ventura-Gouveia, 2011).
2.1. Constitutive model

The elasto-plastic multi-fixed smeared crack model developed for plane stress state structures (Edalat Behbahani et al., 2015) is extended in the present work to the simulation of curved shells, due to the necessity of using the layered concept for modelling the potential of disposing FRC of different properties in distinct layers. In the present section, a brief description of the model is provided for understanding its relevant features and the material properties to be supplied for the material characterization of the adopted FRS. The description is provided at the integration point (IP) level of a generic FE of a plane stress problem, while the layered concept applied to curve shells is described elsewhere (Ahmad et al., 1970).

When the maximum principal stress in an IP attains the tensile strength of the cement-based material (fct), a crack is formed orthogonally to this principal stress. The incremental strain vector (Δε) of a cracked IP is decomposed into an incremental strain vector due to the opening and sliding of the smeared cracks (Δεcr) formed in a length representative of the IP’s area, and an incremental concrete strain vector (Δεct) (Sen-Cruz, 2004). This last term, besides the elastic strain part (Δεel), can also include the plastic strain vector (Δεpl) when the nonlinear behaviour of concrete in compression is significant and is intended to be considered, resulting:

\[
\Delta \varepsilon = \Delta \varepsilon_{el} - \Delta \varepsilon_{pl} - \Delta \varepsilon_{cr}
\]

where Δεel is correlated to the incremental stress vector (Δσ) through:

\[
\Delta \sigma = D^\varepsilon \Delta \varepsilon
\]

being \(D^\varepsilon\) the constitutive matrix of the intact material that is dependent on the Young’s modulus (Ecr) and Poisson coefficient (υcr) of the material. In Eq. (1):

\[
\Delta \varepsilon_{cr} = (T^c)^t \Delta \varepsilon_{cr}^T
\]

where \(T^c\) is a transformation matrix (dependent on the orientation of the crack), and \(\Delta \varepsilon_{cr}^T\) is the incremental crack strain vector with components in the crack local coordinate system.

The opening and sliding behaviour of the smeared cracks are governed by the following material constitutive relationship between the incremental crack stresses (\(\Delta \sigma_{cr}^T\), \(\Delta \sigma_{cr}^n\)) and the incremental crack strains (\(\Delta \varepsilon_{cr}^T\), \(\Delta \varepsilon_{cr}^n\)) in the crack local coordinate system (see Fig. 1):

\[
\begin{bmatrix}
\Delta \sigma_{cr}^T \\
\Delta \sigma_{cr}^n \\
\end{bmatrix}_{\Delta \varepsilon_{cr}^T} = \begin{bmatrix}
D_{cr}^{CT} & 0 \\
0 & D_{cr}^{CN} \\
\end{bmatrix}_{\Delta \varepsilon_{cr}^T}
\]

where \(D_{cr}^{CT}\) and \(D_{cr}^{CN}\) define the opening and sliding process, according to the fracture mode I and mode II, respectively, considered mutually independent (diagonal matrix). More than one set of smeared cracks can be formed, depending on the adopted criterion for opening a new crack (stress criterion and threshold angle between existing cracks and a potential new crack), therefore the interaction between cracks are considered in the present approach.

The crack opening modulus in the fracture mode I process (\(D_{cr}^{CT}\)) is determined from the quadrilinear diagram represented in Fig. 2, which is defined from the truss and ail parameters, and from the mode I fracture energy, \(G_{f1}\). For assuring the results are independent of the FE mesh refinement (Barros et al., 2021), this fracture energy is dissipated in a characteristic length dependent on the geometry of the FE, herein designated by crack bandwidth (\(l_b\)), which, in the present approach, was considered as being equal to the square root of the IP. The crack sliding moduli (\(D_{cr}^{CN}\)) in Eq. (4) is simulated by the following equation (Ventura-Gouveia, 2011):

\[
D_{cr}^{CN} = \frac{\beta}{1 - \beta} G_{f0}
\]

where \(G_{f0} = 0.5E_{cr}/(1 + \nu_{cr})\) is the elastic shear modulus of the cementitious material, and \(\beta\) is the shear retention factor that is obtained from the following equation:

\[
\beta = \left(1 - \frac{\epsilon_{cr}^{oc}}{\epsilon_{cr}^{oc} + \delta_{cr}}\right)^{P_i}
\]

where \(\epsilon_{cr}^{oc}\) is the ultimate crack normal strain (Fig. 2) and \(P_i\) exponent is a constant to define the decrease rate of \(\beta\) with the increase of \(\epsilon_{cr}^{oc}\) (Sen-Cruz, 2004).

The incremental plastic strain vector (\(\Delta \varepsilon_{pl}\)) comprises the irreversible strains of concrete in compression between cracks, determined from:

\[
\Delta \varepsilon_{pl} = \Delta \lambda \frac{\Delta q_p}{\mu}
\]

where \(\mu\) is the plastic potential function, and \(\Delta \lambda\) the non-negative plastic multiplier (Dunne and Petrinic, 2005; Edalat Behbahani et al., 2015). In the present study, \(\mu\) is assumed equal to the five-parameter William and Warnke (W-W) failure criterion defined in the effective stress space (Edalat Behbahani, 2017; William and Warnke, 1974):
\( f\left(\pi, \bar{\varepsilon}_i, \bar{\varepsilon}_i\right) = \left[\left(\frac{T_i}{\sqrt{2\pi c}} - \frac{\sqrt{2} T_i}{\sqrt{\pi} c}\right)\pi\left(\bar{\varepsilon}_i\right) - 2\alpha_1\right]^{1/2} - \pi\left(\bar{\varepsilon}_i\right) = 0 \)  \hspace{1cm} (8)

where \( \pi\left(\bar{\varepsilon}_i\right) \) represents the hardening function governed by the \( \sigma_i - \bar{\varepsilon}_i \) relationship depicted in Fig. 3 (Edalat Behbahani, 2017):

\[
\pi\left(\bar{\varepsilon}_i\right) = \begin{cases} 
E_i \left[\bar{\varepsilon}_i + \frac{\sigma_{c0}}{2E_i} - \frac{\sigma_{c0}}{E_i} + \lambda \right] & \text{for } \bar{\varepsilon}_i \leq \bar{\varepsilon}_{i1}
\end{cases}
\hspace{1cm} (9a)

\[
\lambda = \frac{\bar{\varepsilon}_{i1} - \bar{\varepsilon}_i}{4\sqrt{2}\sigma_{c0}} \left(\frac{E_i}{\bar{\varepsilon}_{i1} - \bar{\varepsilon}_i}\right)^{1/2} \hspace{1cm} (9b)
\]

\[
\kappa = f_i(a_i - 1) \left[\left(\frac{f_i}{E_c}\right)^2 - 2\alpha_0 f_i E_i \bar{\varepsilon}_i + \left(\bar{\varepsilon}_i\right)^2\right] \hspace{1cm} (9c)
\]

\[
\bar{\varepsilon}_{i,u} = \frac{3.4 G_{f,c}}{f_i} \frac{11}{4\varepsilon c} \hspace{1cm} (9d)
\]

The data to define this relationship is: \( a_0 \), which is a material constant obtained from uniaxial compression test and in the present study was taken equal to 0.4, \( \bar{\varepsilon}_c \), representing the normalized stress level above which the nonlinear behaviour in compression is considered; \( \bar{\varepsilon}_{i1} \), which is the equivalent strain at the compressive strength determined from:

\[
\bar{\varepsilon}_{i1} = \varepsilon_{ci} - f_i/E_c \hspace{1cm} (10)
\]

being \( \varepsilon_{ci} \), the total strain at compressive strength \( f_i \), can be determined considering the fib Model Code 2010 (2011) recommendations and \( G_{f,c} \), the compressive fracture energy; and \( l_i \) is the characteristic length where \( G_{f,c} \) is consumed, considered equal to the crack bandwidth \( (l_b) \) (Gernay et al., 2013; Lee and Fenves, 2000). The meaning of the other intervening parameters of Eq. (8) is indicated elsewhere (Edalat Behbahani, 2017).

2.2. Modelling the contact between different materials

For modelling the contact between two different materials in 2D problems, a line interface finite element (IFE) is adopted in a plane stress state, schematized in Fig. 4a, whose constitutive law is determined from the test setup depicted in Fig. 5a (Almeida et al., 2016). The \( \sigma_i - \bar{\varepsilon}_i \) relationship obtained by executing a direct tensile test, i.e., with \( F_i = 0 \), from which it is determined the normal stress at initial orthogonal separation of the edges of the interface \( (\sigma_{max}) \), as well as the fracture mode I softening modulus \( (D_n) \). The \( \sigma_i - \bar{\varepsilon}_i \) relationship can be also obtained with the same test setup, but now performing a direct shear test, i.e., with \( F_n = 0 \), from which it is determined the shear stress at initial sliding of the edges of the interface \( (\tau_{max}) \), as well as the fracture mode II softening modulus \( (D_t) \). By adopting different constant values of \( F_n \) and increasing \( F_i \) up to the rupture of the specimen, it is obtained the Mohr failure surface schematically represented in Fig. 5b. In the present study, since no sliding was observed in the experimental tests between mortar and bricks, a relatively large value, 100,000 N/mm², was adopted to \( D_t \) for assuring negligible sliding between the faces, while the \( D_n \) is simulated by the diagram represented in Fig. 4b.

3. Assessment of the predictive performance of the numerical model

3.1. Type of specimens, geometry, test setups and properties of the materials

The FEM-based constitutive models described in the previous section are developed to simulate the structural response of masonry underground tunnels strengthened with FRS, as will be described in Section 4. Due to the lack of a reliable comprehensive experiment on such a tunnel structure, the predictive performance of the FEM model was separately assessed by simulating experimental results presented in the literature, including tunnel segments (TS) made entirely with FRC (Fig. 6a) (Poh et al., 2009) - with some similarities to the functioning of some critical zones in a tunnel), and masonry beams strengthened (SMB) with SSFRC and SHFRC layer (Fig. 6b) (to simulate the behaviour of masonry substrate of tunnel strengthened with FRS). The geometric properties and types of the components are summarised in Table 1. The selected experiments for the model validation fairly comprise the main mechanisms that contribute significantly to the load-bearing mechanism of the aimed strengthened tunnel structure.

The two adopted SMB series of masonry bending beams are composed of handmade bricks bonded by a low-strength mortar. One of the series is strengthened by a thin layer of SSFRC (FRC3 and FRC4 in Table 3) (Häußer, 2010), while the other series was strengthened with an even thinner layer of a SHFCC (FRC5 in Table 3) (Esmaeeli et al., 2013). The SMB specimens were tested in a test setup schematically represented in Fig. 6b. It should be noted that in the SMB1, no part of the low strength mortar in the joints was replaced by the adopted strain-softening FRC3 (\( h_2 = 0 \), Fig. 6b and Table 1), while in the SMB2, 30 mm depth of the low-strength mortar was replaced by the adopted strain-softening FRC4. In the specimens strengthened with the same strain-hardening FRC5 (SMB3 and SMB4), the 20 mm depth of the low-strength mortar was replaced by this FRC5.

Table 2 identifies the FEM-based model parameters that are, or are not, available in the bibliographical sources.

For instance, the tensile strength \( (f_{t0}) \), the modulus of elasticity \( (E_i) \), and the post-cracking parameters \( (a_0, \xi) \) of FRC3 and FRC4 are all presented in Häußer, 2010 and hence directly used in the performed simulation. However, for the FRC1, FRC2 and FRC5 in Poh et al., 2009 and Esmaeeli et al., 2013, some of the values of the model parameters

\[
\frac{\Delta\sigma_x}{\Delta\sigma_y} = \begin{bmatrix} D_{xx} & 0 & D_{xy} \\ 0 & D_{yy} & 0 \\ D_{xy} & 0 & D_{zz} \end{bmatrix} \hspace{1cm} (11)
\]

where \( \Delta l \) and \( \Delta ul \) are the incremental stress (normal and shear stress components) and displacement (opening and sliding components) vectors, and \( D_{xx} \) and \( D_{yy} \) are the opening and sliding softening moduli, respectively, depicted in Fig. 4b and 4c. \( D_{xz} \) and \( D_{yz} \) can be evaluated from the test setup depicted in Fig. 5a (Almeida et al., 2016). The \( \sigma_i - \bar{\varepsilon}_i \) relationship is obtained by executing a direct tensile test, i.e., with \( F_i = 0 \), from which it is determined the normal stress at initial orthogonal separation of the edges of the interface \( (\sigma_{max}) \), as well as the fracture mode I softening modulus \( (D_n) \). The \( \sigma_i - \bar{\varepsilon}_i \) relationship can be also obtained with the same test setup, but now performing a direct shear test, i.e., with \( F_n = 0 \), from which it is determined the shear stress at initial sliding of the edges of the interface \( (\tau_{max}) \), as well as the fracture mode II softening modulus \( (D_t) \). By adopting different constant values of \( F_n \) and increasing \( F_i \) up to the rupture of the specimen, it is obtained the Mohr failure surface schematically represented in Fig. 5b. In the present study, since no sliding was observed in the experimental tests between mortar and bricks, a relatively large value, 100,000 N/mm², was adopted to \( D_t \) for assuring negligible sliding between the faces, while the \( D_n \) is simulated by the diagram represented in Fig. 4b.
where $f_{ck}$ and $f_{cm}$ are the characteristic and mean concrete compressive strength, respectively. fib Model Code 2010 (2011) recommends the linear stress-crack width response represented in Fig. 7a for modelling the post-cracking response of FRCs, whose defining parameters are determined from the following equations:

$$f_{c_{0.1}} = 0.30 f_{ck}^{0.3} \quad [\text{for concrete strength class } \leq C50 / 60]$$

$$f_{c_{0.3}} = 2.12 \ln (1 + f_{cm} / 10) \quad [\text{for concrete strength class } > C50 / 60]$$

$$E_c = 21.5 \left( \frac{f_{ck}}{10} \right)^{0.5} f_{cm} \text{ in MPa, } E_c \text{ in GPa}$$

$$f_{min} = 0.45 f_{ck}$$
Identification of the model parameters available in the bibliographical sources, and the ones needed to be estimated.

Table 2

<table>
<thead>
<tr>
<th>Designation</th>
<th>Author(s)</th>
<th>Availability</th>
<th>Determination/Estimation</th>
</tr>
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<tr>
<td></td>
<td></td>
<td>$f_l$</td>
<td>$f_n$</td>
</tr>
<tr>
<td>FRC1</td>
<td>Poh et al., 2009</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>FRC2</td>
<td>Poh et al., 2009</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
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<td>●</td>
</tr>
<tr>
<td>FRC4</td>
<td>Häßler, 2010</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>FRC5</td>
<td>Esmaeili et al., 2013</td>
<td>●</td>
<td>○</td>
</tr>
<tr>
<td>SSFRC</td>
<td>Amin et al., 2015</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
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<td>Løvgren et al., 2005</td>
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<tr>
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<td>Redaelli and Muttoni, 2007</td>
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<td>Esmaeili et al., 2013</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Brick1</td>
<td>Häßler, 2010</td>
<td>●</td>
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</tr>
<tr>
<td>Brick2</td>
<td>Esmaeili et al., 2013</td>
<td>●</td>
<td>○</td>
</tr>
</tbody>
</table>

- ● Available in the corresponding reference.
- ○ Not available in the corresponding reference.
- ✓ Determined.
- ○ Estimated.
- – Not applicable.

\[ f_{tu,n} = f_{tu,m} - \frac{w_{tu}^{th}}{CMOD_3} \left( f_{tu,m} - 0.5 f_{tu,m} + 0.2 f_{tu,m} \right) \geq 0 \]  \tag{15} \]

where $f_{tu,n}$ and $f_{tu,m}$ are the average residual flexural tensile strength of FRC at a CMOD1 = 0.5 mm and CMOD3 = 2.5 mm, respectively, evaluated from the force-CMOD relationship determined in three-point notched beam bending tests carried out according to the recommendations of fib Model Code 2010 (2011). In Equation (15) $w_{tu}^{th}$ is the ultimate crack width that depends on the level of required ductility. In the case of elements failing in bending, $w_{tu}^{th}$ is considered equal to 2.5 mm (fib Model Code 2010, 2011). Since the values of the residual flexural strength parameters are not available for FRC3, FRC4, and FRC5, they were estimated according to the following equations, whose adequate predictive performance was demonstrated elsewhere (Moraes-Neto et al., 2013):

\[ f_{tu,n} = 7.5 (V_f/l_f/d_f)^{0.8} \]  \tag{16} \]

\[ f_{tu,n} = 6.0 (V_f/l_f/d_f)^{0.7} \]  \tag{17} \]

being $V_f$, $l_f$ and $d_f$ the volume percentage, length and diameter of the used fibres, respectively.

Consistent with the post-cracking constitutive law of Fig. 2, the stress-crack width response of Fig. 7a is idealised in a normalised form of Fig. 7b, in which a quite small value is assumed for $\xi_1$ to introduce the first post-cracking branch. This is in accordance with experimental evidence, since this branch is governed by the post-cracking tensile capacity of the matrix, reflected in an abrupt load decay up to a cracking opening that activates the reinforcement mechanisms of fibres bridging the crack. Correspondingly, the normalised stress ranges between 1 and $\alpha_1 = f_{tu,n}/f_{tu}$. The second branch of Fig. 7b comprises the constitutive law of Fig. 7a with $\alpha_2 = f_{tu,n}/f_{tu}$ and $\xi_2 = w_{tu}^{th}/w_{tu}^{fib}$, where $w_{tu}^{fib}$ is the ultimate crack width adopted in the FEM simulation, above which the tensile stress is null. $w_{tu}^{fib}$ influences the maximum deformability of the structure at its failure, but it has negligible effect on its maximum load-bearing capacity.

In this simulation $w_{tu}^{fib}$ of 5 mm was adopted for FRC1 and FRC2, two times the one recommended by fib Model Code 2010 (2011) (i.e. $w_{tu}^{fib}$), to attain the long tail of the structural softening responses of Fig. 9 registered experimentally. For the remaining simulations a value very close to the $w_{tu}^{fib}$ was adopted (2.55 mm). It should be noted that, according to the fib Model Code 2010 (Fig. 7a), the tensile capacity at $w_{tu}^{fib}$ is not necessary null, therefore, it is totally acceptable the adopted values for the $w_{tu}^{fib}$, as well as the strategy considered for simulating the post-cracking tensile capacity for crack width above $w_{tu}^{th}$ (Fig. 7b), defined not only by the $w_{tu}^{fib}$, as well as by the point $(\alpha_2, \xi_3)$. Considering $\alpha_2 = 0.2\alpha_2$ and $\xi_3 = 1.02\xi_2$, an abrupt reduction is imposed to the tensile capacity of the material beyond $w_{tu}^{th}$, aiming to simulate a significant loss of the fibre pull-out resisting mechanisms for this crack opening level. The determined values of material parameters are summarised in Table 3.

In the major part of the experimental tests with SMB specimens, some cracks were formed at the interface between the low-strength mortar joints and bricks. Therefore, the contact between these two
Table 3
Mechanical properties of the material constituents (Figs. 2 and 3).

<table>
<thead>
<tr>
<th>Material designation</th>
<th>$f_c$ (MPa)</th>
<th>$f_{ct}$ (MPa)</th>
<th>$E$ (GPa)</th>
<th>$a_0$ (µm)</th>
<th>$a_{c1}$ (N/mm)</th>
<th>$G_f$ (N/mm)</th>
<th>$V_f$ (%)</th>
<th>$b_f/d_f$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRC1</td>
<td>53.92</td>
<td>2.7</td>
<td>37.70</td>
<td>0.4</td>
<td>2.4</td>
<td>40</td>
<td>0.34</td>
<td>0.005</td>
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<td>27.10</td>
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Fig. 7. (a) Tensile stress vs. crack width diagram recommended by fib Model Code 2010 (2011) and (b) normalised post-cracking response considered in the performed FEM simulation.

Table 4
Constitutive laws considered for the IFEs (Fig. 4b).

<table>
<thead>
<tr>
<th>Interface</th>
<th>$\sigma_{max}$ (N/mm²)</th>
<th>$a_1$ (-)</th>
<th>$a_2$ (-)</th>
<th>$w_1$ (mm)</th>
<th>$w_2$ (mm)</th>
<th>$w_3$ (mm)</th>
<th>$G_f'$ (N/mm)</th>
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<td>0.3</td>
<td>0.4</td>
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<tr>
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<td>0.21</td>
<td>0.07</td>
<td>0.3</td>
<td>0.4</td>
<td>1.0</td>
<td>0.28</td>
</tr>
</tbody>
</table>
materials was simulated by adopting IFEs with constitutive laws, whose defining parameter values are indicated in Table 4. Since no experimental information was available regarding the stress-crack width response of the contact between mortar joint and brick, the values of $\alpha_1$, $\sigma_{\text{max}}$, and $\psi$ were estimated by inverse analysis in order to obtain the best compromise in terms of force-deflection relationship and the crack pattern observed experimentally.

3.2. Finite element meshes

The FE meshes used for simulating the tunnel segments (TS1 and TS2), the masonry beams strengthened with SSFRC layer (SMB1 and SMB2), and with SHCC layer (SMB3 and SMB4) are represented in Fig. 8a to 8c.

By taking advantage of its structural symmetry, only half of the SMT-type specimens is simulated by adopting the proper support conditions (Fig. 8b and 8c). The specimens of the TS series were modelled by 8-node Serendipity Ahmad shell FEs (Ahmad et al., 1970) with a $2 \times 2$ Gauss-Legendre integration scheme. Each shell FE was discretized in 10 layers in order to simulate the cracking progress through the thickness of the shell (Fig. 8a). This number is based on previous sensitivity analysis about the influence of the number of layers on the accuracy of the results versus computing time (Ventura-Gouveia et al., 2011). This element type is appropriate for very thin shells of composite materials by using appropriately the reduced and selective integration techniques for the evaluation of the stiffness matrix of the FEs (Figueiras, 1983), and also for RC shells of moderate thickness since the out-of-plane shear components are also considered. By using adequate constitutive laws for simulating the out-of-plane shear behaviour, complex failure modes like punching can be simulated with Ahmad and Reissner-Mindlin shell-type formulations (Ventura-Gouveia et al., 2011).

The SMB’s constituents (Brick, mortar, FRC) were simulated by 8-node Serendipity plain stress elements with a $2 \times 2$ Gauss-Legendre integration scheme. The debonding between brick and mortar was simulated by adopting 6-node linear interface FEs with Gauss–Lobatto integration scheme of $1 \times 3$ IP.

By using the FEM-based model described in Section 2, it was already demonstrated that the FE mesh refinement has negligible influence of the results, in the design context of structural elements (Barros et al., 2021), as long as a minimum refinement is exceeded, which was assured in the present simulations.

According to what was noticed in the experimental program of the TS samples, when the maximum load was attained, a crack was localized under the applied load, and the force versus central deflection of the specimens entered in a deflection softening phase with the widening of this failure crack. In the numerical simulation of the TS samples, this crack localization was achieved by decreasing in 10% the tensile strength and the fracture energy of the elements located right below the applied load. A parametric study was executed to estimate what should be the minimum reduction of that properties, and a value of 10% was obtained for both TS1 and TS2 samples. This decrease has the purpose of simulating provable weakness in the zone of higher tensile strains in real specimens, promoting the localization of the failure crack.

3.3. Results of the performed FEM simulation

In Figs. 9 and 10, the force-central displacement responses obtained from the performed numerical simulations are compared with the experimental results of TSs and SMBs, respectively. This comparison shows that the adopted model is capable of capturing the load carrying capacity and deformation performance of these types of material systems with good accuracy, not only up to peak but also in the structural softening stage. This level of predictive performance was similar when using SSFRC and SHFRC. The crack pattern at failure was also captured properly (Fig. 11). During the crack propagation stage, a crack can be in the following status, and a colour is used to identify this status: opening (red), closing (green), closed (dark blue), reopening (light blue) and fully-open (purple).

In the case of TS1 and TS2, the failure crack was localized at the middle of the segment, which coincides with the loaded section, in similitude to what happened in the experiments. The SMB1 failed by the formation of a crack progressing at the interface between brick and mortar joint, in the pure bending region, which was very well captured by the numerical simulation, but the interface was not precisely the same due to the real heterogeneities in the specimen, which are not considered in the model. In the case of SMB2, the failure crack progressed through the brick, outside the pure bending region, and has inclined toward the brick and mortar joint. The model has captured very well the propagation of the crack through the brick, but inside the pure bending region, with almost vertical inclination, a difference assumed acceptable due to the material heterogeneities of these specimens.

Regarding the SMB3 and SMB4, a diffuse crack pattern was formed during the loading process, which was well captured numerically, and finally the specimens failed by the formation of a flexural crack in the pure bending zone. In the numerical simulations, due to difficulties in the convergence process after the peak load, the failure crack (crack in fully open status) was not able to represent, but the localization of the meso- and macro-cracks (with a crack opening higher than 0.1 mm), represented in Fig. 11, clearly indicate that failure would be in the imminence of occurring in the pure bending region of these specimens. It should be noted that the localization of the failure crack in SMB3 and SMB4 was not the same, which besides the reasons already exposed about natural heterogeneities in this type of specimens, their post-inspection (after testing) has shown different thickness of the FRC5 layer in the longitudinal direction of the specimens.

4. Development of an optimal strengthening configuration for tunnels

4.1. Relevant characteristics of the tunnel and strengthening strategy

The potential of FRS of strain-softening or strain-hardening nature is assessed for the strengthening of a Portuguese typical underground aged tunnel. The geometry and type of substrate of the adopted tunnel for the numerical simulations were provided by the institution that manages the Portuguese railway network, “Infraestruturas de Portugal”. The cross-sectional geometry of the tunnel and strengthening FRS layer are depicted in Fig. 12a and 12b, respectively. The computation time of the simulation increases significantly with the length of the tunnel. The optimum length of the tunnel was investigated in preliminary simulations, by comparing the internal nodal forces at the tunnel extremity ($F_{\text{N}}^{\text{ext}}$) with the ones at the tunnel centre ($F_{\text{N}}^{\text{cen}}$). It was verified that for a length of 4 m, $F_{\text{N}}^{\text{ext}}$ to $F_{\text{N}}^{\text{cen}}$ ratio is limited to 0.003%, which is small enough to assume negligible alteration on the relevant tunnel’s behaviour aspects if a larger depth of the tunnel is simulated. Therefore, for the parametric studies, a length of 4 m for the tunnel was adopted, since it was considered sufficient to capture the 3D strengthening effects provided by the FRS layer when occurring a loading event like the one
carving, the thickness of the FRS strengthening layer was limited to 100 mm. These relatively old tunnels were generally designed with the minimum thickness of the structural strength represented in Fig. 12 b. Due to costs or/and technical considerations, they were generally designed with the minimum thickness each, numbered from the interior to the exterior (in contact with the tunnel substrate). The load transferred to the FRS strengthening layer by this unstable block of rock is assumed distributed in a 500 × 500 mm² area (see Fig. 12b). For the strengthening of a tunnel with an FRS shell system, this loading scenario is one of the most critical since it can induce punching failure in the shell, which is a brittle failure mode. The considered dimension of the loaded area (500 × 500 mm²) is an idealised representation of the area of the bottom face of the rock blocks often used to build the substrate of the aged underground tunnels (Alexakis and Makris, 2013), such as the one adopted in the present research.

Fig. 8. FE meshes for simulating the (a) tunnel segments (TS), (b) masonry beams strengthened with SSFRC (SMB1 and SMB2), and (c) masonry beams strengthened with SHFRC (SMB3 and SMB4) (dimensions in mm).

The considered dimension of the loaded area (500 × 500 mm²) can induce punching failure in the shell, which is a brittle failure mode. The cantilever load transferred to the FRS strengthening layer is assumed distributed in a 500 × 500 mm² area (see Fig. 12b). For the strengthening of a tunnel with an FRS shell system, this loading scenario is one of the most critical since it can induce punching failure in the shell, which is a brittle failure mode. The considered dimension of the loaded area (500 × 500 mm²) is an idealised representation of the area of the bottom face of the rock blocks often used to build the substrate of the aged underground tunnels (Alexakis and Makris, 2013), such as the one adopted in the present research.

The contribution of the tunnel’s soil system to the stiffness of the strengthened structure is simulated by using surface springs continuously distributed orthogonally to the FRS shell FEs, as schematized in Fig. 13a. The contribution of the springs to the stiffness of the structure is mobilised only when the springs are in compression, otherwise, null stiffness is automatically assumed since no tensile capacity was considered for the soil. The stiffness matrix of the springs applied in a certain shell FE is determined from (Ventura-Gouveia, 2011):

$$K_s = \int_{-1}^{1} \int_{-1}^{1} T^T N^T k_s N T d s_1 d s_2$$

(18)

where $T$ is the transformation vector with the cosine of the angles formed by the direction of the spring (orthogonal to the shell) with the tangential coordinate system at the integration point (IP) of the shell FE, $N$ is the vector of the shape functions of the shell FE evaluated at the IP, $k_s$ is the spring stiffness, and $J$ is the determinant of the Jacobian at the IP. For the “normal” (not weakened) soil, $k_s = 0.2$ kN/mm² was assumed. The influence of soil deterioration is taken into consideration by adopting a weaker soil with $k_{sw} = 0.1k_s$ (10 % of the stiffness of the “normal” soil system) in the “critical” region of the tunnel cavity (Fig. 13b).

The localization of the zone of the weaker soil (critical region) was based on the criterion of providing the smallest load carrying capacity for the tunnel strengthened with the FRS layer. For this purpose and in a preliminary analysis, five different possibilities for the localization of the critical region were investigated, by using the angles $\theta_1$ and $\theta_2$ represented in Fig. 13b, whose values are summarized in Table 5.

### 4.3. Material properties

In the numerical analysis, the SSFRC developed by Amin et al. (2015) is adopted for the FRS structure, whose values of the parameters of the constitutive law are provided in Table 3. This SSFRC of moderate strength class includes a volume fraction ($V_f$) of 0.4 % of hooked-end steel fibres, which is quite current in FRS tunnelling applications.
Regarding the SSFRC adopted in chapter 4, its tensile strength and elastic modulus values are the ones reported in Amin et al. (2015), while its post-cracking response was determined according to the fib Model Code 2010 (2011) recommendations, as explained in section 3.1.

4.4. Determination of the critical region for the weak soil

The force transmitted by the unstable block of rock to the FRS shell structure versus the central deflection of the tunnel for the scenarios indicated in Table 5 is represented in Fig. 14a, from which it can be concluded that the scenario IV for the localization of the weaker soil is the most unfavourable regarding the decay of the load carrying capacity (loss of 27% regarding the load carrying capacity when non-weaker soil is considered – the scenario I). The tunnel cross-section deformation for the scenario IV at peak load is shown in Fig. 14b, where it is visible that the weaker soil is subjected to a higher compressive deformability than

![Fig. 9. Comparison of the force-central displacement responses obtained from the test and numerical simulations of (a) TS1 and (b) TS2.](image-url)

![Fig. 10. Comparison of the force-central displacement responses obtained from the test and numerical simulations of (a) SMB1, (b) SMB2, (c) SMB3 and (d) SMB4.](image-url)
Fig. 11. Comparison of the crack patterns obtained from the tests and numerical simulations, where cracks with a width higher than 0.1 mm are represented for (a) TS1, (b) TS2, (c) SMB1, (d) SMB2, (e) SMB3, and (f) SMB4. (In the numerical crack patterns, the red and purple represent a crack in opening and fully-open status, respectively). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Fig. 12. (a) Digitalized geometry of the tunnel cross-section (b) strengthening FRS structure and loading localization considered in FEM simulations (dimensions in mm).
the remaining soil, causing higher damages in the FRS due to cracking in this zone, as well as in the zone just below the localization of the unstable block of rock. These two zones coincide, respectively, with the sector BD and DE represented in Fig. 15, and accordingly, the sector CE represented in Fig. 15 will be considered the critical region in the remaining numerical simulations of the present work.

### 4.5. Tunnel strengthening plans

With the aim of restoring the load carrying capacity of Scenario I when weaker soil exists in the critical region (Scenario IV), the five FRS-based strengthening plans (StrPlan) indicated in Table 6 were analysed. For these strengthening schemes, the three types of FRC shown in Table 3 were considered, one of strain-softening nature (SSFRC) and two of strain-hardening character (SHFRC1 and SHFRC2) whose values of the parameters of their constitutive laws are indicated in Table 3. Due to the lack of information, the constitutive law of the SHFRC1 (Loigren et al., 2005) was obtained by following the fib Model Code 2010 (2011) recommendations, as explained in section 3.1, while in the case of SHFRC2 it was defined from the information available elsewhere (Redaelli and Muttoni, 2007). It should be noted that the SHFRC1 in Table 3 is reinforced with steel fibres of 60 mm length, which are too long to be practically sprayable through the current shotcrete methods. Therefore, if an FRS of SHFRC1 properties is aimed to be adopted in shotcrete, the mix composition should be properly redesigned with fibres of length suitable in order for this FRS to be sprayable. The types of applied FRC, the strengthening sector (Fig. 15), and the section type of the strengthening intervention (Fig. 16) are indicated in Table 6. This table also includes the volume of FRC types applied in each strengthening plan. By having the costs for the application of each strengthening plan, the most cost-competitive can be determined by also taking into consideration the recovering level of load carrying capacity of Scenario I (without weak soil), whose information is also provided in Table 6 ($F_{\text{max}}/F_{\text{scenario}}$).

It should be noted that in the zones out of the sector AF, SSFRC was applied for all the strengthening plans. Therefore, in the StrPlan1 to StrPlan4 strengthening plans, the use of SHFRCs was optimized in the AF sector, while in the StrPlan5 it was limited to sector BD (Fig. 15).

Despite StrPlan1 to StrPlan4 have been applied in the same sector, they differ due to the cross-section assigned along the AF sector. In fact, in the StrPlan1 the SHFRC1 was applied according to the section Sec1 (Fig. 16a), which means that all layers of the FRS are formed by SHFRC1. In the StrPlan2 to StrPlan4 the SHFRC2 was used (of much better mechanical properties than SHFRC1, see Table 3), but the type of section was different in each of these strengthening plans: in StrPlan2, the SHFRC2 was applied according to section Sec2 (Fig. 16b), which means that all the layers of the FRS are formed by SHFRC2; in StrPlan3, the FRS was applied according to section Sec3 (Fig. 16c), which means that SHFRC2 was applied in the two inner and two outer layers, while in the remaining layers the SSFRC was applied; in StrPlan4, the FRS was applied according to section Sec4 (Fig. 16d), which means that SHFRC2 was applied in the inner and in the outer layers, while SSFRC was applied in the remaining layers. Finally, the optimized strengthening intervention StrPlan5 was limited to the sector BD (Fig. 15) by applying SHFRC2 according to the section Sec2 (Fig. 16b), which means that all the layers of the FRS are formed by SHFRC2. All these strengthening plans involve the application of FRS along with the longitudinal development of the representative portion of the tunnel (4 m, see Fig. 12b).

In the case of Sec4 (Fig. 16b), the two outer layers have a thickness of 10 mm, which is lower than the minimum thickness possible to apply with regular FRS technology. However, in the scope of the ongoing project, which this numerical research is part, is intended to develop SHFRC suitable to be applied according to the shotcrete technology, by considering the achievements already attained by some researchers in this area (Kim et al. 2003 and 2004) and enlarging this knowledge by using machine learning tools for the design of an FRS mix composition with aimed rheological and experimental properties. These SHFRC use relatively small aggregates and fibres, so the possibility of applying layers of a thickness between 10 and 20 mm could be a reality.

A typical crack pattern in the most interior and most exterior layer is depicted in Fig. 17a and b, respectively, according to which and regarding the yield line theory (Johansen, 1962): for the adopted load configuration, an almost circular crack band is propagated in the top surface (layer 10), while in the bottom surface (layer 1) radial cracks are...
developed from the loaded area.

Fig. 18a compares the force–deflection for the Scenario I and Scenario IV (without and with weak soil, respectively) with the two strengthening plans, StrPlan1 and StrPlan2. It verified that the StrPlan2 was capable of exceeding both the load carrying capacity and the stiffness of the Scenario I, which did not happen in the StrPlan1. Therefore, the SHFRC2 was the adopted one for the remaining strengthening strategies. Fig. 18b compares the force-deflection for the Scenario I and Scenario IV with the ones obtained for the StrPlan3, StrPlan4 and StrPlan5. It is verified that, although these three strengthening scenarios were capable of recuperating the load carrying capacity of Scenario I, the stiffness was only recovered in the StrPlan3 and StrPlan5. Considering the prices of each FRC ($150 \, \text{€}/\text{m}^3$ for the SSFRC and $500 \, \text{€}/\text{m}^3$ for the SHFRC2), the most cost-competitive strengthening solution is the StrPlan5 ($1013 \, \text{€}$ - StrPlan5 versus $1176 \, \text{€}$ - StrPlan3 for the 4 m tunnel length, exclusively for the FRC).

The relationship between the load versus the crack width registered in layer 1 (the most internal one) and layer 10 (bonded to the tunnel’s substrate) is represented in Fig. 19a and Fig. 19b, respectively, for the Scenario I, Scenario IV, and the strengthening plans StrPlan3 and StrPlan5. The crack width is determined by the following equation:

$$w_{\text{max}} = l_b \varepsilon_{\text{cr},\text{max}},$$

where $\varepsilon_{\text{cr},\text{max}}$ is the maximum tensile strain normal to the crack in integration points and $l_b$ is the crack bandwidth, assumed equal to the square root of the area of the integration point. Comparing the results of Fig. 19a and 19b, it is verified that the crack width is much higher in the interior layer (number 1) than in the external layer (number 10). Assuming a crack width limit of 0.4 mm due to durability concerns, the maximum load at serviceability limit state conditions is 533, 364, 619 and 751 kN for the Scenario I, Scenario IV, StrPlan3 and StrPlan5, respectively, which demonstrates that regarding the Scenario IV, an improvement of 70% and 106% in terms of crack width limitation is assured by the StrPlan3 and StrPlan5, respectively. For these load levels, the crack width in the external layer (Fig. 19b) in both strengthening plans is lower than the limit one. Besides being more cost-competitive, StrPlan5 also assures smaller maximum crack width.
5. Conclusions

In the present paper, the potential of fibre reinforced shotcrete (FRS) was evaluated for the strengthening and rehabilitation of underground tunnels, by performing FEM-based numerical simulations with constitutive models capable of capturing the relevant features of fibre reinforcement. The reliability of these models was assessed by simulating experimental tests reported in the literature, including tunnel segments made by strain-softening fibre reinforced concrete (SSFRC), and masonry flexural elements strengthened with SSFRC and strain-hardening fibre reinforced concrete (SHFRC). These constitutive models were then adopted to simulate the behaviour of a relatively thin FRS shell (100 mm) for the strengthening of a typical Portuguese underground tunnel, by numerically investigating possible combinations of SSFRC and SHFRC along the depth of the FRS shell and in different sectors of the tunnel cross-section, with the final aim of maximizing the load carrying capacity and stiffness of the strengthened tunnel with the minimum costs in terms of applied FRC. By placing SHFRC of proper mechanical properties in the external layers of the FRS shell and in the regions of the tunnel where the most tensile and compressive strain fields are developed regarding the installed damages on the tunnel, the load carrying capacity and the stiffness of the undamaged tunnel conditions can be recuperated with a relatively low amount of FRS. These optimized shotcrete configurations with proper FRCs require, however, the use of a new generation of robot systems capable of shotcreting in a certain X,Y,Z position of the tunnel as well as the type(s) and content(s) of fibres forming the aimed FRC for that zone, in strict respect with the design outputs similar to the ones obtained in the present work. Integrating damage detection in tunnels, advanced numerical simulations and automation of robotics that include a new generation of shotcreting nozzle are the next research challenges in the vision of exploring, in an optimized context, the potential of fibre reinforcement for the

\[ F_{\text{max}} \] maximum load carrying capacity of Scenario I, 1054 kN (Fig. 14a).

<table>
<thead>
<tr>
<th>Strengthening plan</th>
<th>Section</th>
<th>Sector</th>
<th>( F_{\text{max}} ) (%)</th>
<th>Total consumed material</th>
<th>SSFRC (m³)</th>
<th>SHFRC1 (m³)</th>
<th>SHFRC2 (m³)</th>
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</thead>
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<td>86</td>
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<td>2.39</td>
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</tr>
<tr>
<td>StrPlan5</td>
<td>Sec2</td>
<td>BD</td>
<td>139</td>
<td>5.09</td>
<td>–</td>
<td>0.50</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 16. Cross-sections considered in the strengthening plans of the strengthened sectors, (a) Sec1, (b) Sec2, (c) Sec3, and (d) Sec4.
Fig. 17. Typical crack pattern of the FRS structure in (a) layer 1 (most interior) and (b) layer 10 (most exterior), (the red, green, dark blue, light blue, and purple represent a crack in opening, closing, closed, reopening, and fully-open status, respectively). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Fig. 18. Force-central displacement responses of: (a) Scenario I, Scenario IV, StrPlan1 and StrPlan2, and (b) Scenario I, Scenario IV, StrPlan3, StrPlan4 and StrPlan5.

Fig. 19. Force versus maximum crack width in the layer (a) 1 (most interior) and (b) 10 (most exterior).
strengthening and rehabilitation of underground infrastructure.

CRediT authorship contribution statement

Mahsa Taheri: Methodology, Investigation, Validation, Writing – original draft, Visualization. Joaquim A.O. Barros: Conceptualization, Supervision, Software, Writing – original draft. Hamidreza Salehian: Visualization, Writing – review & editing. Antonio Ventura-Gouveia: Software.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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