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Practical design, testing & verification guidelines for pre-cast segmental tunnel linings subjected to fire loading

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ABSTRACT

This paper presents a practical 5-stage fire design procedure for bored pre-cast segmental tunnel linings subjected to an accidental fire load. A case study is presented with each stage discussed together with references to design assumptions, structural analysis, concrete mix design, fire test specifications and guidance on the review and interpretation of fire test results. Finally, the effects of progressive local deep spalling are investigated with a non-linear transient thermal-mechanical coupled finite element analysis. The spalling behaviour is modelled with the removal of elements during the analysis to match those recorded during fire tests.

1. Introduction

Segmental tunnel linings that have been subjected to extreme fire loads such as the Channel Tunnel, Mont Blanc and Tauern all resulted in significant damages to the lining (Lönnermark, 2005). The designer must provide as a minimum criteria adequate strength to support the ground loading following the occurrence of a fire. This ensures safe egress for passengers and emergency services and will also provide a safe environment during subsequent tunnel lining repair operations. As there are no design guidelines presented in current standards to aid tunnel designers through the process of a segmental tunnel fire design, this paper presents a 5-Stage fire design procedure, Fig. 1, with recommendations for each step.

The final stage of the design is the interpretation of the fire test results. This stage has been given considerable effort by including a non-linear transient thermal-mechanical coupled numerical analysis. Mechanisms such as explosive and surface spalling, segmental tunnel lining joints and the cracking and crushing of concrete are all included. The case study presented in this paper adopts a tunnel lining with an internal diameter of 8.5 m, a lining thickness of 400 mm and the depth measured to the tunnel crown is 18 mbgl.

2. Stage 1 – Input parameters

The tunnel fire design is often undertaken prior to any fire testing, meaning the first step is an assumption regarding the behaviour of concrete, particularly the depth the spalling will be limited to. A reasonable assumption is between 15 mm and 40 mm. The second step is a heat transfer analysis to determine the temperature distribution through the tunnel segment. The fire load adopted is the RABT ZTV German requirement for train tunnels shown in Fig. 2.

The commercially available finite element software package LUSAS (2016) has been adopted to calculate the temperature distribution of the fire curve though the tunnel segments. The temperature dependant properties are in accordance with BS EN 1992-1-2:2004 (Fig. 3) and the results are presented in Fig. 4.

3. Stage 2 structural analysis

3.1. Design inputs

BS EN 1992-2-1-2 provides two simplified methods for determining the fire resistance of reinforced concrete sections, i.e. the reference isotherm method and the zone method. The zone method is more suitable for sections subjected to combined bending and axial actions, making the method particularly suitable for tunnel linings. In addition, the isotherm method can overestimate the load bearing capacity of high strength concrete (Meda et al., 2002) whereas the zone method provides greater accuracy (Naranayan and Beeby, 2005), for these reasons the zone method has been adopted.

In the zone method, the decline in the bearing capacity of the structural member due to material degradation is considered by means of a reduction in the member cross section by the damage factor $K_{c,m}$,

$$k_{c,m} = \left(1 - \frac{0.2}{n}\right) \sum_{i=1}^{n} k_c(\theta_i).\ \frac{d_i}{d_{tot}} \tag{1}$$

where n is the number of zones, $k_c(\theta_i)$ is the concrete strength reduction at reference temperature, d_i is the zone thickness and d_{tot} is the total depth.

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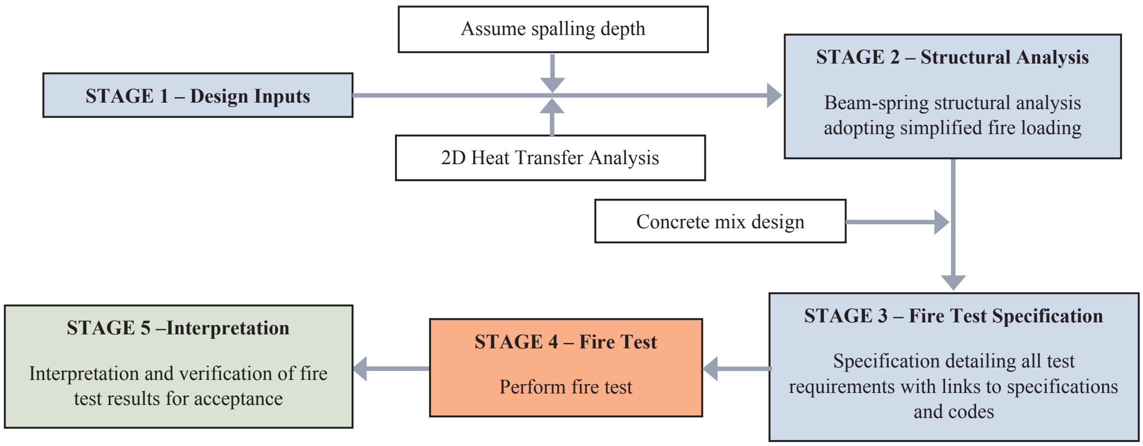


Fig. 1. 5-Stage fire design procedure.

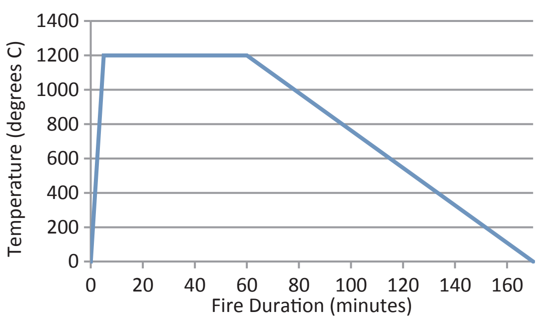


Fig. 2. EUREKA fire curve.

The reduction in the stiffness can be accounted for by a reduction in the Young's Modulus as follows,

$$E_{c,\text{red}} = (k_c(\theta_M))^2.\ E_{ck} \tag{2}$$

where $k_c(\theta_M)$ is the reduction factor at the centre of the member cross-section, however, this is unlikely to be of any significant value.

It is not usually possible to input temperature curve gradients shown in Fig. 4b) onto beam elements in a numerical analysis and for this reason, an equivalent load must be calculated. According to Grunicke and Walter (2007) the equivalent load will contain a uniform temperature (ΔT) and a temperature difference across the section height (ΔΔT). These maybe calculated by dividing the section into zones with the sum equating to the normal force (N_θ) and moment (M_θ). Similar techniques have been applied by Zia et al. (1995) and Caner et al. (2005). This procedure should be completed over a section reduced by the assumed spalling depth.

$$N_\theta = \sum_{i=1}^{n} \left[\frac{\Delta l(\theta_i)}{l}\right]_c .\ E_c(\theta_i).\ d_i \tag{3}$$

$$M_\theta = \sum_{i=1}^{n} \left[\frac{\Delta l(\theta_i)}{l}\right]_c .\ E_c(\theta_i).\ d_i.\ z_i \tag{4}$$

where n is the number of zones, $\left[\frac{\Delta l(\theta_i)}{l}\right]_c$ is the thermal expansion at the reference temperature, $E_c(\theta_i)$ is the young's modulus at the reference temperature, d_i is the zone depth and z_i the lever arm to the center of the zone. The material properties shall be in accordance with BS EN 1992-1-2:2004.

The stress in each zone shall not exceed the allowable concrete stress or Young's modulus at the reference temperature, otherwise excesses stresses will occur. Fig. 5 presents the thermal stress at time step t = 80 min with the corresponding allowable stress profile in accordance with BS EN 1992-1-2:2006.

With the section divided into zones the equivalent linear temperatures maybe be calculated from;

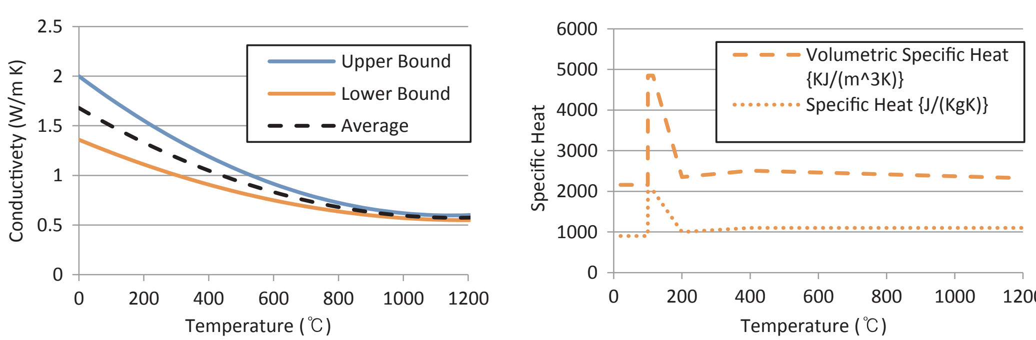
$$\Delta T = \frac{N_\theta}{\left[\frac{\Delta l(20\text{Â°})}{l}\right]_c .\ E_c A} \tag{5}$$

$$\Delta\Delta T = \frac{M_\theta .d_{tot}}{\left[\frac{\Delta l(20\text{Â°})}{l}\right]_c .\ E_c I} \tag{6}$$

where $\left[\frac{\Delta l(20\text{Â°})}{l}\right]_c$ is the coefficient of thermal expansion at 20°, $E_c$is the young's modulus at 20°, A is the area, $d_{tot}$is the total depth and I is the second moment of area.

3.2. Beam-spring analysis

The individual segments of the tunnel lining have been modelled in



(a) Concrete conductivity (b) Concrete specific & volumetric heat

Fig. 3. Thermal properties at elevated temperatures. Figure notes. (a) BS EN 1992-1-2:2004 provides upper and lower bound thermal conductivity; the average between the two has been adopted. (b) A moisture content of 3% has been adopted. The concrete heat capacity has a peak between 100 and 115 °C, representing the evaporation of water during heating (Wang et al., 2012).