ECCOMAS Thematic Conference on
Computational Methods in Tunnelling (EURO:TUN 2007)
J. Eberhardsteiner et.al. (eds.)
Vienna, Austria, August 27-29, 2007

# FIRE SAFETY OF THE UNTERINNTALSTRECKE – NONLINEAR FE ANALYSES AS A TOOL FOR ULTIMATE LIMIT STATE DESIGN

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**Keywords:** Fire Safety, Ultimate Limit State Design, Non-linear Finite Element Analysis, Structural Fire Design, Tunnel.

**Abstract:** *Existing standards regulating structural fire design procedures of concrete structures originate from a background of rising structures, but few refer to underground infrastructures such tunnels. In this paper, an example of the structural fire design of a cut-and-cover railway tunnel is presented. Two analysis methods are applied: a non-linear thermo-mechanical analysis and a simplified approach using linear beam analysis. Comparison of these two approaches indicates that in the case of highly stressed structures, the latter yields more conservative, yet rather uneconomic results, whereas the first accounts for overall structural bearing capacities. Although standard software equipment does not always feature non-linear analyses, they appear to be the more adequate choice for obtaining a both safe and economic design in the fields of underground infrastructure.*

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## 1 INTRODUCTION

Fire safety is a major concern in the design process of railway lines with large underground stretches. Although there are some standards defining fire loads and safety requirements [2], few standards specify analysis methods which are appropriate to the conditions of massive and highly stressed underground structures.

Common methods of structural fire design usually require a two-step approach with application of equivalent sectional forces and design analysis on a cross-sectional level. For the benefit of a more economic design an integrated one-step approach is presented here. The method takes into account the overall capacity and load redistribution possibilities of the entire structure instead of looking at cross sections of selected structural members.

The results are then compared to those obtained by a simplified approach suggested in Eurocode 2 (ÖNORM ENV 1992-1-2) [1].

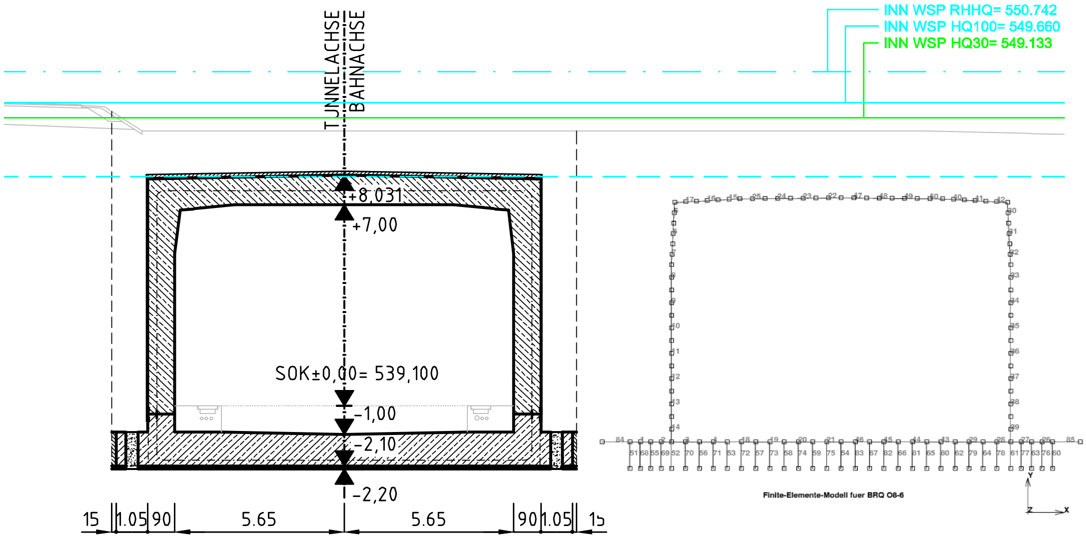


Figure 1: Analysed cross section and FEA-model

## 2 FIRE SAFETY REQUIREMENTS

Depending on the hazard potential in terms of danger both to passengers and to users of adjacent infrastructure, and depending on the construction technique, fire safety requirements were defined and classified for the Unterinntal railway line. These requirement classes were allocated along the alignment.

Subject to these requirements, ultimate limit state analyses of the cut-and-cover tunnels of Unterinntal section H7 were performed (Fig. 1). For the calculations presented here, the requirements for the respective class are defined as follows:

- $\leq$ 30 min of fire: reserve of global bearing capacity $\geq$ 35%
- 30 min < T $\leq$ 180 min: reserve of global bearing capacity $\geq$ 0%
- after fire event (cooled down state): partial safety factors on loads and resistance as specified by the Eurocodes for persistent and transient situations.

Along most parts of the alignment, the tunnel runs below ground water table, giving the design the additional demands of watertightness during regular operation and minimised risk of flooding during and after fire.

Depending on adjacent surface infrastructure, other classes show more stringent safety requirements for the cooled down state after the fire event. By reason of higher safety factors and reduced residual material strengths, the cooled down state often showed to be critical and should hence be equally scrutinised.

The design fire event was given as the so-called BEG-1 curve. Maximum temperature level is reached within 10 minutes, cooling down starts after 120 minutes, resulting in a transient temperature distribution over the cross section (Fig. 2) [3].

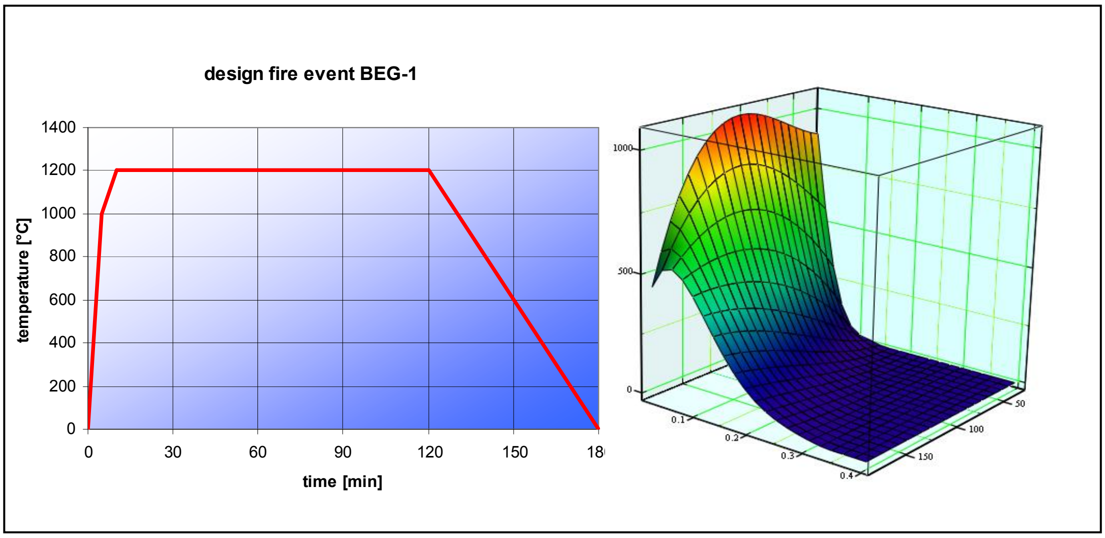


Figure 2: Design fire and temperature propagation over cross section

## 3 IMPLICIT DESIGN APPROACH

### 3.1 Material Modelling and FEM-implementation

Non-linear constitutive models for concrete and reinforcement were adapted and extended in order to account for cracking, softening and crushing of concrete and yielding of rebar steel. Ultimate stresses and strains for concrete and steel are all temperature-dependent and result in temperature-dependent stress-strain-relations and stiffness (see Fig. 3). Reduced strength for the cooled-down state was also considered, and stress-strain relations were extended in order to capture the behaviour during cooling down. Temperature-dependent relations for ultimate stresses, stress-strain relations, ultimate stresses for the cooled-down state and the coefficient of thermal elongation are based on project specifications and ÖNORM ENV 1992-1-2 [1]. Following the findings of Kusterle et al [2], spalling of concrete is reduced, if not prevented, by a polypropylene-fibre content of 2.0 kg/m³.

Integration over the cross section of the structural member is performed by means of layered beam elements, with the different layers representing either concrete or reinforcement.

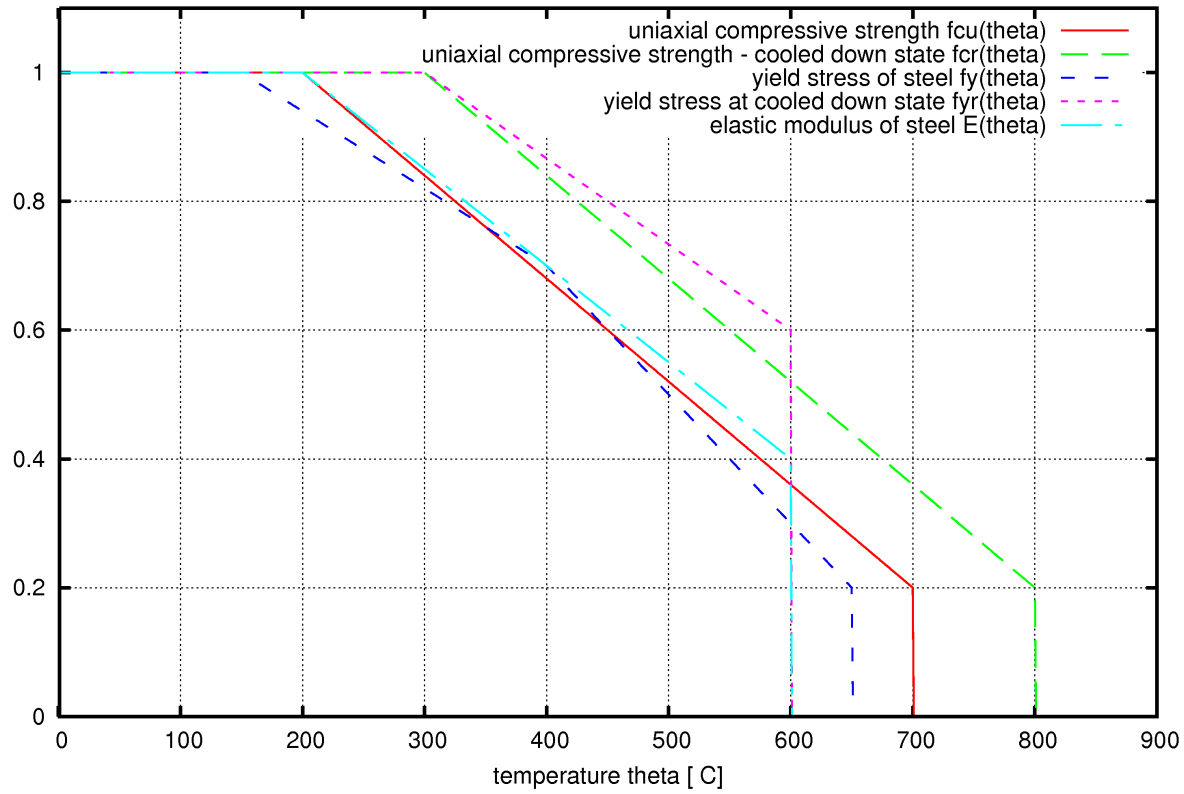


Figure 3: Temperature dependent material properties (ratios compared to cold state)

Creep and residual stresses for the cooled-down state were ignored in the analyses. Calculation, or even estimation these stresses is hardly possible. This is due to the fact that duration and temperature propagation of a real fire event, particularly during the cooling-down period, is not covered by design fire curves in a satisfactory manner. (The BEG-1-curve, for example, depicts only one of a multitude of possible cooling-down phases, whereas research shows that the initial phase is sufficiently realistic.).

### 3.2 Verification of bearing capacity - ULS-check

Partial safety factors for both resistance and loading can be derived from the safety requirement classes defined for the project. The constitutive laws applied allow an implicit design, as stresses in the concrete and reinforcement are automatically limited to design values bearable by the material at each level of loading and at each instant of time. Figures 4 and 5 show the development of concrete and steel stresses in the upper right corner. As can be seen from Fig. 4, normal stresses in layers close to the hot side augment up to strength values $f_c(\theta)$, due to constrained thermal expansion. The constraint results in an overall increase of negative bending moments (Fig. 7) and normal forces (Fig. 8), whereas shearing forces (Fig. 9) remain more or less unaffected. Compressive stresses in the concrete exhibit the expected decline due to reducing material strength $f_c(\theta)$ at high temperatures. They drop off from a peak further inside the cross section to zero on a growing zone of degradation on the hot side.

The inner reinforcement layer undergoes a temperature rise of more than 400°C (Fig. 5). Constraint of thermal expansion results in a significant growth of compressive stresses until yielding is reached after 120 minutes. From there on the stress equals yield strength $f_y(\theta)$ which keeps decreasing with temperatures growing.

As for the outer reinforcement on the cool side of the member, tensile stresses reach yielding level after 60 minutes. The initial build-up during the first hour is caused by an increase of bending moments due to constrained thermal expansion. From this point, no further increase of bending moments can be taken up by this cross section. The development of a plastic hinge causes overall load redistribution and mobilisation of further structural bearing capacity.

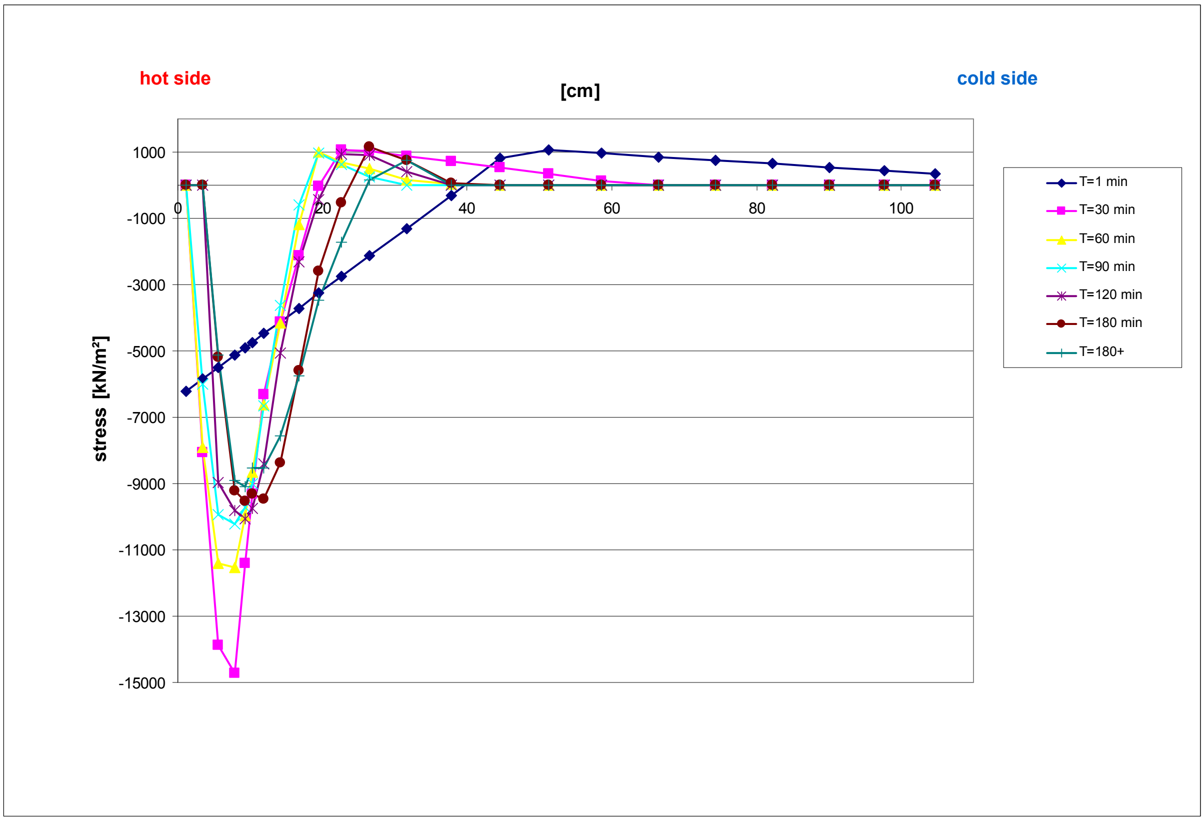


Figure 4: Development of normal stresses in concrete layers (top end of right wall)

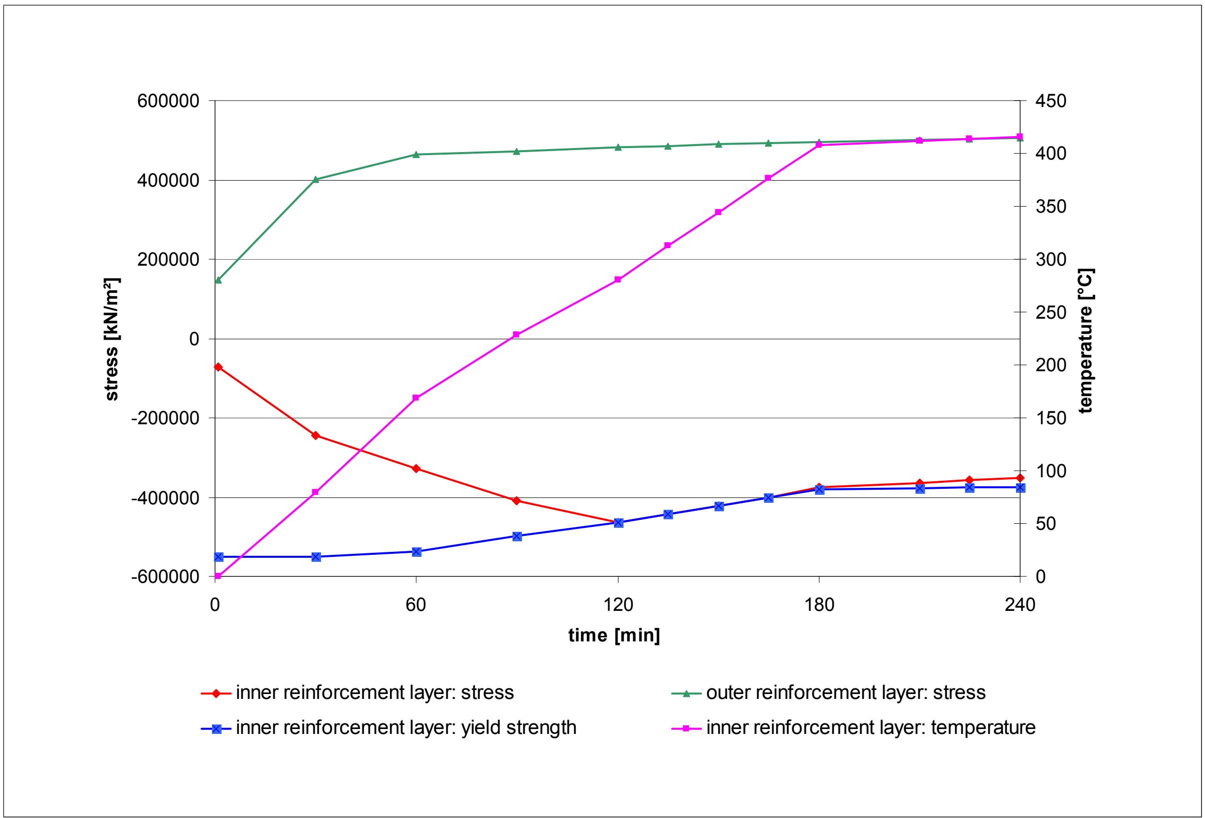


Figure 5: Development of normal stresses in reinforcement layers (top end of right wall)

## 4 SIMPLIFIED APPROACH

As non-linear thermo-mechanical analyses as presented above require features that are beyond the capablities of most standard beam analysis programs, they are usually not applicable in everyday engineering practice. Eurocode 2 [1] suggests a simplified analysis method for structures under fire loading. In this method the decline of stiffnesses and bearing capacities of structural members due to material degradation in the course of a fire event is considered by means of a reduction of the member cross sections $A_c$ by the factor $k_{c,m}$. To this end, the cross section is divided into a number of n zones. For each zone concrete strength decreases by a factor $k_c(\theta_i)$, where $\theta_i$ is the temperature at a certain time at the centre of the zone. The reduction factor $k_{c,m}$ is a mean value obtained by a weighted average of material reduction factors in each zone:

$$A_{c,red} = k_{c,m} \cdot A_c$$

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

where $d_i$ is the thickness of the each zone, $d_{tot}$ the thickness of the member cross section and n the number of zones.
The reduction of Young's modulus is calculated as:

$$E_{c,red} = (k_c(\theta_M))^2 \cdot E_{ck}(\theta = 20°C) \tag{1b}$$

where $k_c(\theta_M)$ is the reduction factor at the centre of the member cross section. For member cross section dimensions common in underground structures, the reduction factor will hardly

ever be considerably less than 1.0, since temperatures do not propagate as deeply during design fire events.
The reduced cross section is assumed to show homogenous material properties and standard concrete strength values $f_{ck}(\theta=20°C)$.

External loads including temperature propagation due to fire are then applied to the new structural system with reduced member cross sections. As linear beam analysis only allows for temperature gradients that are linear over cross sections, the thermal loads due to fire need to be substituted by equivalent linear temperature gradients ($\Delta T$ being defined as constant temperature increase and $\Delta\Delta T$ as temperature difference over cross section height).

To this end, the zones which are introduced for the calculation of the cross section reduction are considered as thermo-elements acc. to [1]. As temperatures rise, the thermo-elements undergo thermal expansion $[\Delta l(\theta)/l]_c$. In statically indeterminate structures, thermal expansion is constrained and hence results in additional bending moments and normal forces:

$$N_\theta = \sum_{i=1}^{n} [\Delta l(\theta_i)/l]_c \cdot E_c(\theta_i) \cdot d_i \tag{2}$$

$$M_\theta = \sum_{i=1}^{n} [\Delta l(\theta_i)/l]_c \cdot E_c(\theta_i) \cdot d_i \cdot z_i \tag{3}$$

with $z_i$ being the eccentricity of each zone.

For the computation of these constraint forces, stress is limited to actual concrete strength $f_c(\theta_i) = k_c(\theta_i) \cdot f_c(\theta=20°)$ and Young's modulus, $E_c(\theta_i)$, is adjusted according to the present temperature $\theta_I$ in each thermo-element. Constraint forces would be exorbitantly high if temperature dependence was neglected.

In the next step, equivalent linear temperatures are calculated as:

$$\Delta T = \frac{N_\theta}{[\Delta l(20°)/l]_c \cdot E_c A}, \quad \Delta\Delta T = \frac{M_\theta \cdot d_{tot}}{[\Delta l(20°)/l]_c \cdot E_c I} \tag{4}$$

and applied to the structure together with the other external loads. Sectional forces can now be computed using conventional linear beam analysis.

Finally, for the design check of relevant cross section, steel properties are considered with reduced values according to current temperature ($f_c(\theta)$ and $E_s(\theta)$). It is pointed out that, depending on the depth of the degradation zone, reinforcement layers on the hot side may be outside the reduced concrete cross section. This could evoke difficulties when using software tools that do not feature the definition of "negative" concrete covers.

## 5 COMPARISON OF MODELS

As can be seen from Table 1 and Fig. 6, the simplified approach generally results in considerably higher negative bending moments and even in a lack of positive bending moments in the roof span. This is due to the fact that cracking is not considered in the simplified approach. In the non-linear analysis, negative bending moments increase just until cracking and yielding initiate at the corners. Consequently, bending moments remain positive in the roof span during the entire fire. Load redistribution towards the span mobilises additional structural bearing capacities.

Table 1 also shows that the simplified analysis yields a considerably higher amount of flexural reinforcement in the corners. In the presented case, the non-linear analysis results in no further reinforcement in addition to the amount calculated for regular (i.e. "cold") design load cases. Even though a certain increase of the inner reinforcement in the roof span is necessary in order to benefit from higher overall structural capacity, the resulting total amount of flexural reinforcement is significantly lower compared to the simplified analysis.

| T=120min | non-linear analysis | simplified analysis |
|---|---|---|
| bending moment | 1285 kNm/m | 1913 kNm/m |
| required reinforcement | 20 cm²/m | 34 cm²/m |

Table 1: Comparison of results for T=120 min.

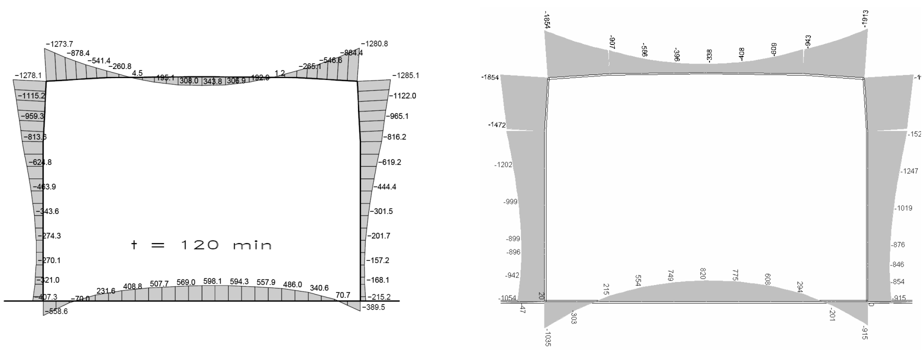


Figure 6: Comparison of resulting bending moments for T=120 min;
left: non-linear analysis; right: simplified analysis

While the simplified approach suggested by Eurocode 2 is more conservative, it yields less economic results in the case of structural fire design of tunnels.

The authors consider that with highly stressed structures, the effects of cracking and load redistribution in conjunction with the development of plastic hinges outweighs the influence of the accuracy of temperature dependent material properties.

Neither model provides a satisfactory solution for shear design. Anchoring of shear reinforcement on the hot side of the cross section is crucial. As a first approximation, the required shearing reinforcement can hence be calculated using steel properties of the weakest (i.e. hottest) point of a stirrup on the one hand and a reduced concrete cross section according to concrete degradation on the other.

## 6 CONCLUSIONS AND OUTLOOK

The thermo-mechanical multi-layer analysis procedure allows an economic design: All fire relevant specifications are integrated either as part of the material model or by means of the temperature loading definition.

- Integrated single-step analysis for typical external load configuration plus fire loading
- Non-linear analysis (plastic design including cracked state), allowing for the redistribution over the entire structure

- Economic design: overall structure capacity reserves can be mobilised, rather than capacity reserves on the cross-sectional level only.
- Implicit design check due to limiting of stresses of concrete and reinforcement
- For a realistic assessment of residual stresses after an actual fire event, the model could be enhanced by high temperature creep terms.

The simplified analysis suggested in ÖNORM ENV 1992-1-2 can be performed without the need of advanced software. This convenience may be of advantage in everyday practice, where quick assessments and conservative solutions are required. However, in the case of highly stressed structures such as cut-and-cover tunnels, the chief disadvantage of a purely linear beam analysis may be a uneconomic design, since overall structural bearing capacity is not always accounted for.

Infrastructure projects with large stretches such as cut-and-cover tunnels hold a high potential of economical optimisation of reinforcement design. Therefore, the use of non-linear models appears to be most advisable.


Figure 7: Development of bending moments during fire event

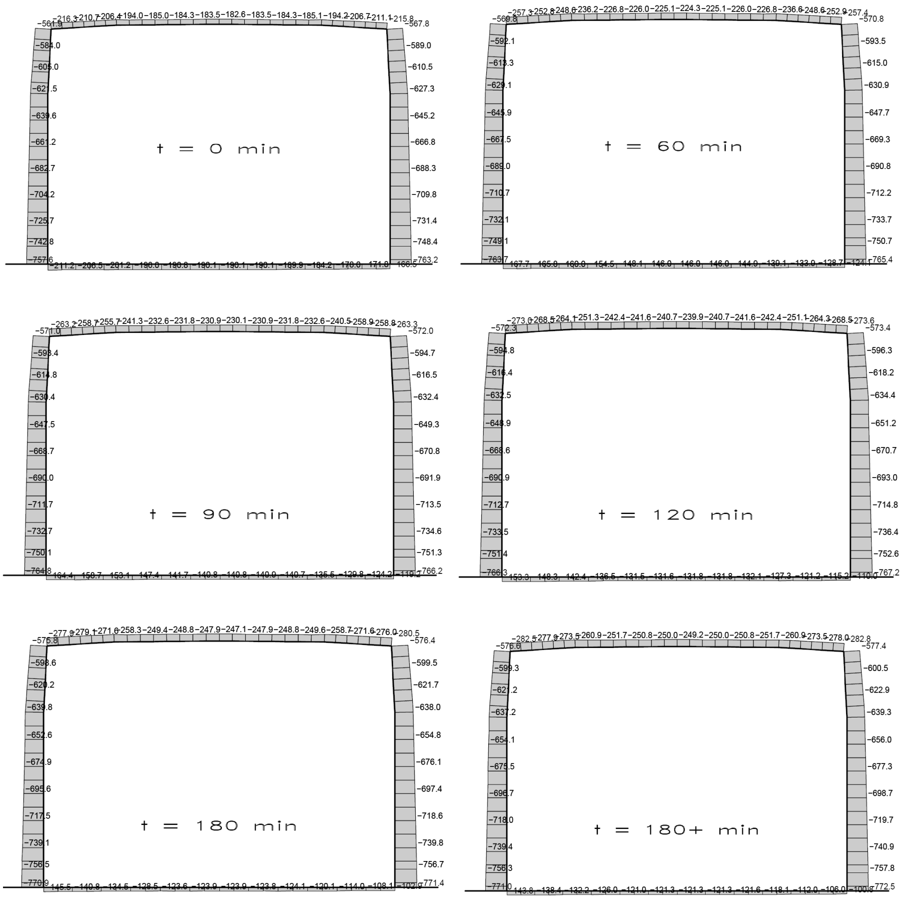


Figure 8: Development of normal forces during fire event

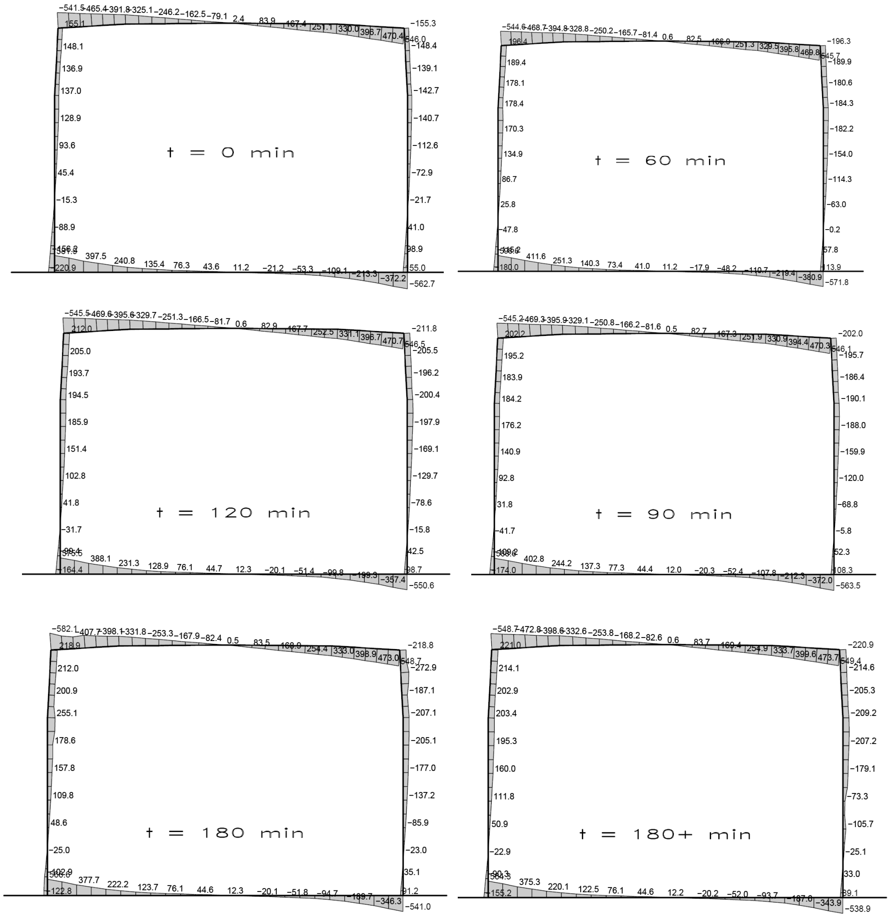


Figure 9: Development of shearing forces during fire event

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