HAZARDS IN TUNNELS
STRUCTURAL INTEGRITY

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ABSTRACT

This paper contains a discussion of modelling hazards and using them as part of structural design. The paper takes starting point in the result of the work in DARTS WP4 and WP5 "Hazard Aspects" and "Integrated Design". The paper includes also the results of UPTUN task 4.1, where critical structural components are identified. As part of the paper is shown through a simplified example how the methodology works.

1. INTRODUCTION

It may be possible to design tunnels with very good durability characteristics or with good fire resistance but looking only at one aspect at the time will not result in optimal structures from an overall point of view. Other measures may be unreasonably expensive or may cause environmental problems. An integrated approach is called for. This has been the starting point for the DARTS (Durable and Reliable Tunnel Structures) project. DARTS includes hazard aspects, durability, environmental considerations and economy in an overall integrated design, which lead to optimal decisions. In the present paper the line of thinking concerning integration of risk is presented.

2. RISK AS A STRUCTURAL DESIGN PARAMETER

The studies of risks in tunnels have become common practice in connection with tunnels. It is normal, at least for larger tunnel projects, to estimate the risk of fires and other large accidental events during construction and operation.

The studies of hazard aspects should be part of the process aimed at optimising measures by minimising the effect of hazards on the primary functions of the tunnel. The input to the process depends on design options, geology, type of tunnel, end user requirements, etc. All types of hazards should be considered: fire, explosion, leakage of aggressive materials, toxic releases, water inundation, and earthquakes. The methods are developed with the aim of integrating design methods for the hazards into the complete design and re-design of the tunnel, incorporating the different life-cycle stages of the tunnel. Fire in tunnels has significant impact on tunnel safety and it has high priority to study causes and consequences of fire for evaluation of structural design and safety measures.

3. STRUCTURAL RESPONSE TO ACCIDENTAL ACTIONS / FIRE

The structural response to accidental actions is dependent on among others the type of structure and the material. Some tunnels are unlined and tunnels with steel lining and similar exist, but most tunnels are constructed of reinforced concrete and the present paper will focus on concrete tunnels. The most common types of tunnels are arch-shaped rock-tunnels, with in-situ lining, circular bored tunnels with segmental lining, box-shaped immersed tunnels and box- or arch-shaped cut-and-cover tunnels.

The action of fire comprises the direct and indirect thermal impact, i.e. heating of the structure and the reinforcement and the resulting loss of strength and stiffness as well as the internal stresses resulting of the heating and the strain and deformations occurring during and after the fire. Among the deformation aspects concerned with concrete should be mentioned the important mechanism of LITS (load induced thermal strain). For further details concerning the modelling of structural response of concrete is referred to fib (international federation of structural concrete), which will issue a guideline concerning this topic later in 2004[1].

Another important topic (which is also covered by the fib guideline) is spalling. Even though there has been a significant progress in understanding the phenomenon, there is still not a practical engineering model for spalling. Since spalling has the potential for severely damaging the structure and ultimately cause the collapse of the structure it is very important take this damage process into consideration. Depending on the structural system, the load and the surrounding ground the tunnel may be more or less sensitive towards spalling.

The ultimate consequence of fire is collapse and loss of structure but also local damage and subsequent closing of the tunnel can be a severe situation, finally the fire may have influence on the long-term behaviour and the durability of the structure

None of the very severe fires in Europe in the recent years have caused collapse of the tunnel main structure. However, the fire in the Channel Tunnel in 1996 resulted in damage which could have resulted in collapse and flooding, if it was not for favourable ground conditions at the particular location of the fire. The damage to the segmental lining was caused by spalling (see Figure 3.1), which in the worst affected location of segments near the fire continued through the entire thickness of the lining.

Many road tunnels have ventilation ducts over the traffic space. In case of a fire the ceiling may be damaged due to reductions of strength, deformations and spalling. A local collapse can occur as it was observed after the fire in the Gotthard tunnel in 2001(see Figure 3.1), and in the fire in the Tauern tunnel in 1999. This local damage does not compromise the overall integrity of the structure and can be repaired within relatively short time.

The fire in the Mont Blanc tunnel in 1999 caused relatively limited damage. The damage was caused by heating of the concrete and to some limited extent spalling (see Figure 3.2). Even though the main structure was damaged, the structural integrity was not severely compromised. The reasons were the ground conditions, the favourable arched shape of the lining as well as the thickness of the lining.

Figure 3.1 Fire damage to the tunnel structure. To the left: the local collapse of the ceiling after the fire in the Gotthard tunnel in 2001. To the right: the severe spalling of the segmental lining after the fire in 1996.

Figure 3.2 Structural damage to tunnels. To the left is shown the damage of the lining of the Mont Blanc Tunnel after the 1999 fire. To the right the severe spalling and detachment of the reinforcement after the fire in the Moorfleet Tunnel in 1968.

The structural damage and the critical components have been studied as part of UPTUN task 4.1[4], based on a review of actual fires and results of research studies.
New types of concrete, with high strength and low permeability have shown to be more prone to spalling than standard concrete with higher permeability. Hence, it appears that the requirement for durability and for fire safety may be in conflict. A well-balanced mix of the concrete fit for the purpose should be aimed for; reference is made to the results of fib Task Group 4.3[1] and to the methodology of the DARTS project[2].
However, for illustration of the fact that the spalling problem is not only concerned with new high-performance types of concrete, the damage to the Moorfleet tunnel after the fire in 1968 is shown in Figure 3.2.

4. MODELLING OF RISK

4.1 Logical Trees

In order to use risk as a design parameter and to find the well-balanced design with respect to fire risk, durability, construction costs and other important criteria, the risk will have to be systematically modelled.
The modelling of risk applies all available or achievable knowledge in terms of physical models, statistics, expert opinion and identified scenarios. The physical models may be a complicated science and reference is made to other sources, among others fib[1].
Design evaluations and decisions are required at different staged of a project. At the different stages more or less information may be available, and in general evaluations are made based on a lower level of detail in the first stages and increasing in detail during the project.
The point of risk analyses is to support decisions and the risk shall be modelled in a way respecting the level of detail, which is required and possible.
A common way to model risk is in terms of logical trees (see Figure 4.1). In practice the events leading to the fire are structures in terms of a fault tree, and the possible consequences of the fire event in an event tree. The outcome is the expected consequences, i.e. an integration of all possible consequences and their individual probability of occurrence.
Risk reducing measures are included in the model in terms of reduction factors for reducing the likelihood of the unwanted events or their consequences.

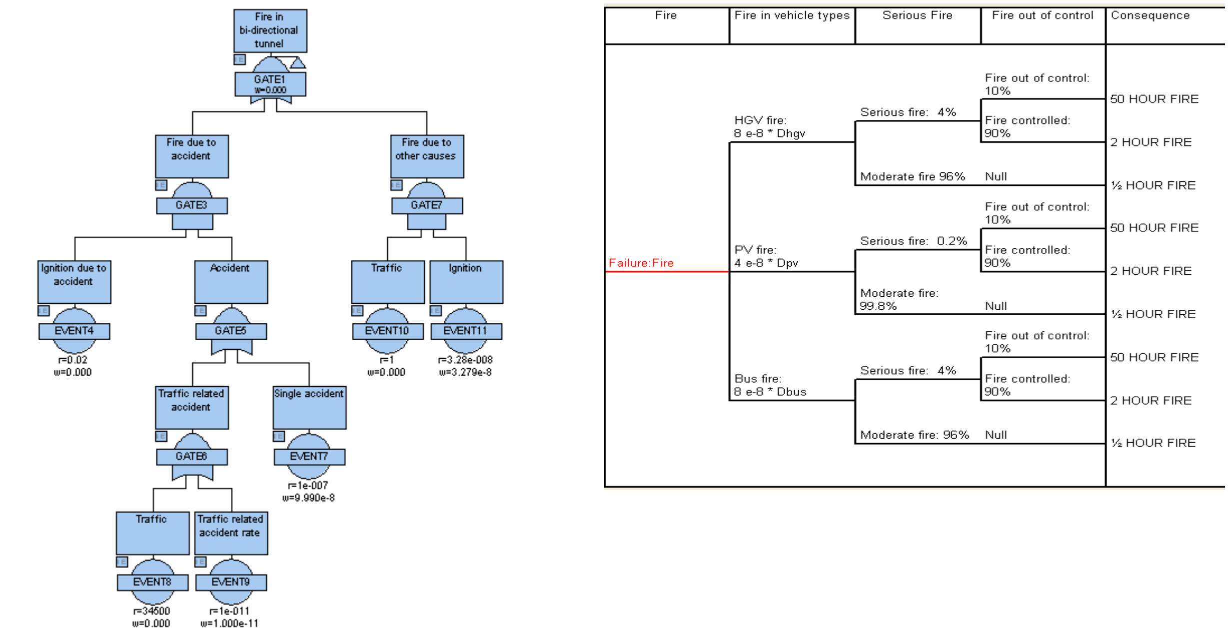

Figure 4.1 Logical trees. To the left: Simplified fault tree for illustration, indicating logical combinations (and / or gates) of events leading to the top even "Fire in tunnel". To the right: Part of a (simplified) event tree (for illustration only!), exploring the consequences of a fire.

Also other hazards are modelled in terms of fault trees and event trees. These logical trees can be more or less detailed depending on the available information and the decision problem at hand in the various design stages "necessity discussion, feasibility study, conceptual design, outline design, and detailed design" of the project.

Fault tree analysis: the evaluation of the so-called top event by a top-down analysis of identification of combinations of causes leading to the undesired event (e.g. fire in tunnel)
Event tree analysis: a logical diagram of success and failure combinations of events, leading to all possible consequences of a given initiating event (e.g. fire in tunnel)
Prevention measures can be formulated in terms of gates in the fault tree with a probability preventing events in developing into the top event. Mitigation measures can be modelled through the probability of success/failure in the event tree.
The final result is a set of consequences, which can be quantified in monetary units as present values and an associated set of probabilities. Weighting all consequences with their probabilities will lead to the expected hazard "costs". These costs can then be compared with construction costs and other relevant design parameters.

4.2 Uncertainty modelling

In risk modelling at a low level of detail it may be possible to assess some of the probabilities based on expert judgement, or simplified methods. All uncertainty is on this level modelled by the probabilities. At higher levels of detail the uncertainty may be modelled directly.
Structural reliability methods may be used for determining probabilities based on uncertain variables. This involves defining a limit state for which the probability is determined and specifying the distribution functions and parameters (mean, standard deviation, etc.) for each variable. The limit state may of the type $M = R_T - S_T$, where R_T is e.g. the strength during the fire and S_T is the load during the fire. The probability $p(M<0)$ corresponds to the probability of the unwanted event (see Figure 4.2). These considerations have also formed the basis for the EC 1 part 1-2[5] (fire action) and the load factors herein.
The distribution functions and the associated parameter values will have to be carefully selected based on for example experimental data and likelihood.

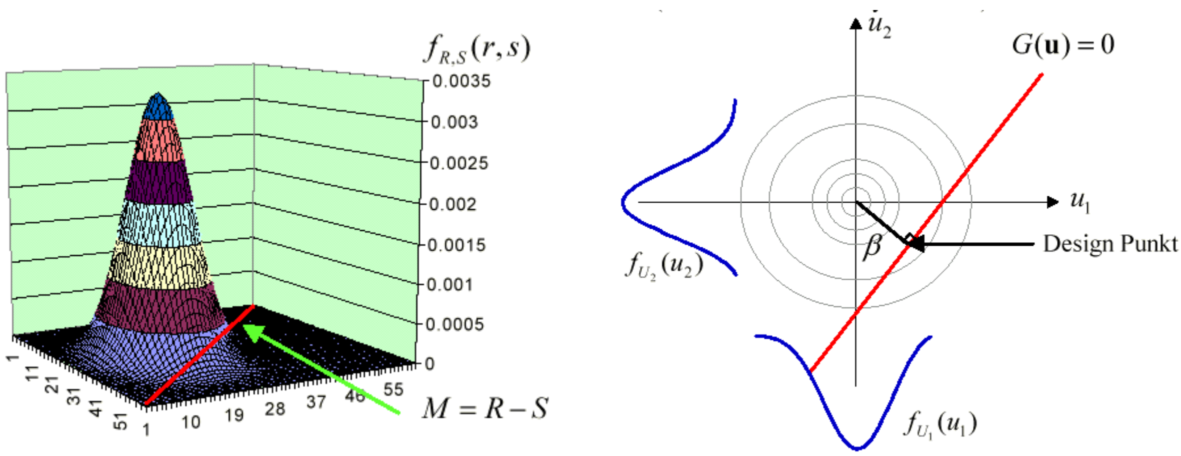

Figure 4.2 Determination of probability of exceeding limit state M = R - S and the associated β - value.

5. INTEGRATION OF RISK

In order to achieve the optimum, the decisions should be made taking into account all available information and with the goal of reaching the optimal solution seen in a life-cycle perspective. With the term "utility" describing the overall use or benefit of a certain activity, the optimum decision can be formulated as: *The* decision giving the highest expected "utility" among all possible decisions. If all decision criteria have been transformed into costs, the best decision is the one resulting in minimum costs in a life-cycle perspective.
It is therefore important to identify all relevant decision scenarios and to quantify all criteria in terms of costs and finally find the minimum.

6. EXAMPLE

6.1 Selection of main design

Considerations regarding risk analyses and durability have only limited value in themselves, but will be supportive in achieving decisions, which are optimal with respect to the goal of the decision-maker. This means the starting point is always a decision.
Taking the design situation, at an early stage of a tunnel project the decisions will have an impact on measures taken against deterioration, fire risks, construction costs and maintenance costs. Only the matters influencing the actual decision need to be modelled. In this case only fire risk is modelled since it is assumed that the choice of concrete will not have any impact on other types of hazard. It is stressed that the example is simplified in order to show the functionality of the integrated approach. The design consideration is concerned with concrete, reinforcement and fire protection in 8 combinations as shown in Table 6.1.

Aspect	Design option
Concrete	Conventional
	Porous
	Dense
Reinforcement	Conventional
	Stainless steel
Fire protection	Insulation
	No insulation

Case	Concrete	Reinforcement	Fire-protection
1	Conv.	Conv.	no
2	Conv.	Conv.	Protected
3	Dense	Conv.	no
4	Dense	Conv.	Protected
5	Porous	Conv.	no
6	Porous	Conv.	Protected
7	Porous	Stainless	no
8	Porous	Stainless	Protected

Table 6.1 Design options and combination in 8 cases

6.1.1 Assumptions and input

The basic assumptions for the example are given in Table 6.2. For the present example deterioration is modelled as probability of corrosion initiation (for details about the deterioration models is referred to papers concerning DARTS WP2). Failure probabilities for initiation of corrosion are shown in Figure 6.1 for conventional, dense and porous concrete with conventional reinforcement.

– Structural life perspective:		100 years (no corrosion)
– Maintenance and repair:		25 years interval (a fixed interval is chosen for simplicity of the presentation)
– Tunnel length:		3km
– Traffic (AADT):		50.000 Vh/day
– Heavy vehicles:		10%
– Increase in traffic,	Cars	2% pa
–	HGVs	4% pa
– The increase in traffic is assumed to stagnate after 30 years		
– Fires frequency,	Cars:	1 fire per 20 Million Vhkm
–	HGVs	1 fire per 10 Million Vhkm
– Repair is assumed to reinstate the original failure probability (corrosion initiation).		

Table 6.2 Basic assumptions

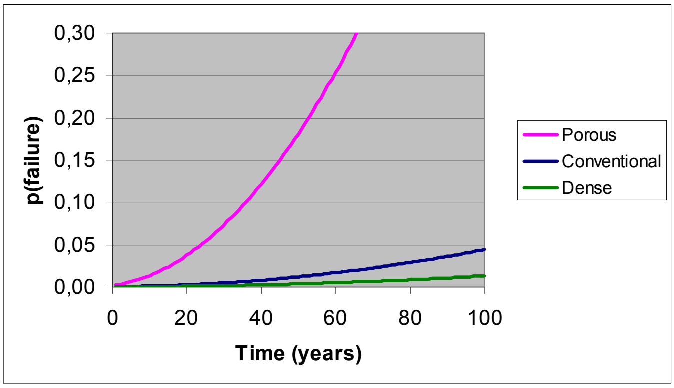

Figure 6.1 Probability of initiation of reinforcement corrosion

In the present example one risk reduction measure is considered, namely the measure of fire protection, and a structural fire protection has been assumed. The fire protection is assumed to have maximum effect for preventing structural failure and no effect on those fires giving minor damage, as there will always be some consequences associated with fires.

Maintenance and repair costs are also assumed based on realistic cost information. Repair costs include also contributions related to traffic disturbance and other consequences.

The failure costs will include both the actual costs concerned with a fatal accident, i.e. clearing, stabilisation, reconstruction, compensations and a monetary quantification of lost lives, injuries, traffic disturbance, and environmental impact. The failure costs may also include the decision-maker's risk attitude, i.e. his aversion against accidents.

Discount rates may have an important impact on life cycle considerations; her 4% is assumed.

6.1.2 Risk analysis

The risk is quantified by means of fault trees and event trees. The initiating events are defined as "Fire in car" and "Fire in HGV". In the present example an ultra simple event tree is applied. It has 4 branches: "Failure", "Serious damage", "Minor damage" and "No damage". The course of events into the branches is described by 4 conditional probabilities, ref. Table 6.3. It is assumed that the relative frequency of fires is constant in time. Hence the frequency of fires in the tunnel will increase with the traffic; the figures in Figure 6.2 are given for the initial year.

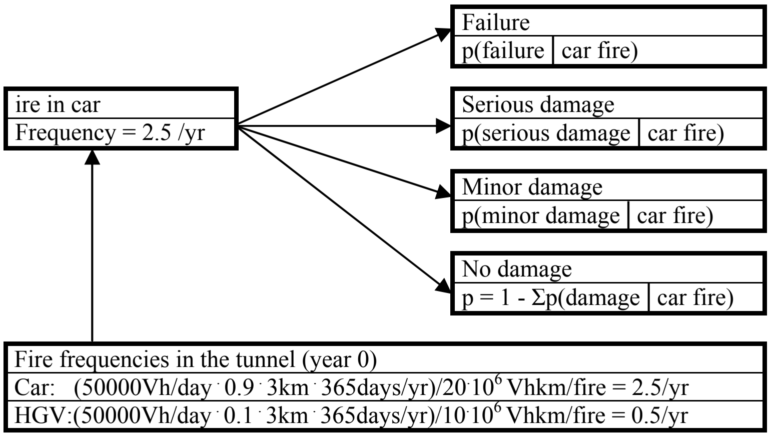

Figure 6.2 Simplistic logical tree for fire damage

Concrete	Conventional	Porous	Dense
p(failure │ car fire)	0.02%	0.01%	0.04%
p(serious damage │ car fire)	1%	0.7%	2%
p(minor damage │ car fire)	16%	16%	16%
p(failure │ HGV fire)	0.1%	0.05%	0.2%
p(serious damage │ HGV fire)	5%	2.5%	10%
p(minor damage │ HGV fire)	80%	80%	80%

Table 6.3 Risk, conditional probabilities assumed

Efficiency of fire protection. Reduction factors	
Damage resulting in failure:	***0.10***
Major damage:	***0.20***
Minor damage	***1.00***

Table 6.4 Reduction factor

The risk measures can be further explored by detailing of the analysis either in terms of detailing the event tree or a fault tree. The events "fire in car" respectively "fire in HGV" are denoted "initiating event in the event tree analysis (ETA) and the "top event" in the fault tree analysis (FTA). In the ETA the steps between the initiating and the final consequences is further detailed whereas the FTA investigates the logical causes leading to the top event. More detailed analyses will be necessary in order to evaluate the effects of various risk reducing measures.

By use of the above event trees and the conditional probabilities indicated in Table 6.3 the frequencies of the 3 degrees of damage can be estimated. The frequency of a damage degree is determined as the conditional probabilities of the damage multiplied with the respective frequency of fire. Contributions from car fires and HGV fires are added.

Concrete type	Conventional		Porous		Dense	
	year 0	year 30	year 0	year 30	year 0	year 30
Failure	0.0010	0.0027	0.0005	0.0013	0.0020	0.0053
Serious damage	0.052	0.133	0.031	0.076	0.104	0.267
Minor damage	0.83	2.13	0.83	2.13	0.83	2.13
Fire, no damage	2.13	3.97	2.15	4.03	2.07	3.83

Table 6.5 Estimated annual frequencies of damage due to fire for 3 concrete types and 2 points of time

6.1.3 Durability, Maintenance and Repair

In order to simplify the example the maintenance is assumed to take place at equidistant intervals of 25 years where repair brings the reliability back to the initial condition. The annual probability of failure (corrosion initiation), within the 100 years lifetime is illustrated in Figure 6.3 for conventional concrete.
In order to compare the risk of failure with the costs, the present value of failure consequences must be determined, which requires an estimation of the annual probability of failure for each year and the assumed failure costs. For the example presented no limits in terms of minimum reliability level (respectively maximum failure probability) has been introduced. The safety is regulated by the specified failure cost.

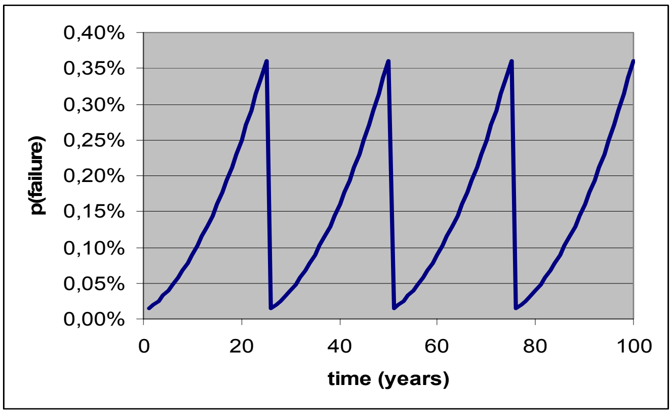

Figure 6.3 Corrosion probabilities of conventional concrete with repair every 25 years

6.1.4 Results

For the assumptions given the "total expected present values" for each of the 8 alternatives appear from Figure 6.4. The expectation value is the mean value of an uncertain consequence or simply the probability multiplied with the consequence of an event.
The components of the expected present value are illustrated in Figure 4.4. It appears that the initial costs are the major part of the expected present value. The consequences of fire result in lost competitiveness for the unprotected solutions with conventional and dense concrete. The expected consequences of failure due to corrosion contribute significantly to the solutions with porous concrete.
The uncertainties of the input are explored by a sensitivity study. With the given assumptions it appears that the best solution is to choose conventional concrete, with conventional reinforcement and supply the structure with fire protection. The stainless steel solutions have a high initial cost, which in the example by far cannot be outbalanced by their advantages with respect to maintenance.

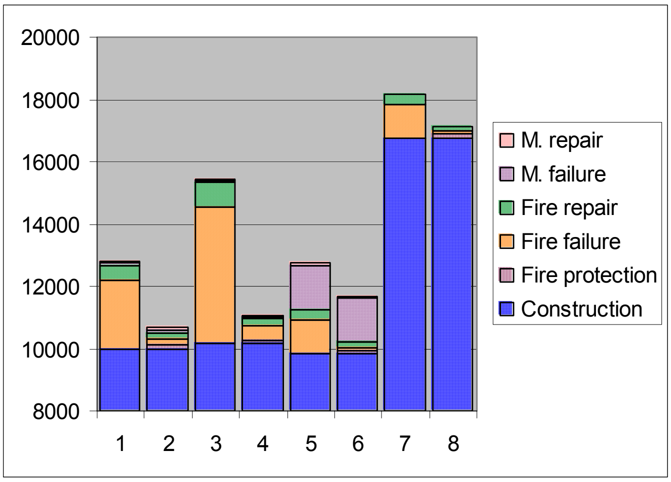

Figure 6.4 *Components of the expected present value for 8 cases. Observe: truncated second axis!*

Case	Concrete	Reinforcement	Fire-protection	Expected present value
1	Conv.	Conv.	no	12806
2	Conv.	Conv.	Protected	**10663**
3	Dense	Conv.	no	15447
4	Dense	Conv.	Protected	11058
5	Porous	Conv.	no	12738
6	Porous	Conv.	Protected	11691
7	Porous	Stainless	no	18186
8	Porous	Stainless	Protected	17139

Expected present value includes: construction, maintenance, repair, failure due to corrosion and risk of fire

Table 6.6 Expected present values for the 8 alternatives (expressed in terms of cost units).

6.2 Detailed design of fire protection

In a later stage in the design process the selected "case" of conventional concrete with fire protection will have to be designed in more detail. The damage caused by fire is assumed to be described by physical models for heating of concrete and reinforcement and the associated impact on material strength and deformation as well as spalling. The design measures to prevent heating of the concrete and reinforcement are a fire protection board and the concrete cover to the reinforcement.
Failure may be formulated in terms of the temperature reached in reinforcement and concrete combined with spalling. For simplicity only the temperature in the reinforcement is illustrated in the following and only fire protection is regarded as a design parameter.
The mechanism of failure is assumed to be yield in the reinforcement and the critical temperature depends on the utilisation of the bars at the individual location. Often the utilisation of reinforcement in tunnel is rather low and some reduction of the strength corresponding to some heating of the reinforcement can be acceptable. In this example two limits states are applied in order to illustrate the range of results: $T_{crit}1 = 750^oC$, where the remaining strength is very limited and $T_{crit}2 = 380^oC$, where strength is reduced and structures with high utilisation may have problems.
The limit state function for heating of reinforcement is formulated as Z:

$$Z = T_{crit} - T_{concrete}(x_{reinforce}, t)$$

where t is the time after the start of the fire, $x_{reinforce}$ is the position of the reinforcement, T_{crit} is the critical temperature.

$$T_{concrete}(x,t) = T_s - (T_s - T_0)\sum_{k=1}^{\infty} \frac{2\sin\omega_k}{\omega_k + \sin\omega_k \cos\omega_k} \cos\left(\frac{\omega_k(d-x)}{d}\right) \exp\left(-\frac{\omega_k^2 a_c}{d^2} t\right)$$

where x the distance to the concrete surface, d the thickness of the concrete wall, T_s the maximum tunnel surface temperature, T_0 the temperature before the fire starts, d the thickness of the concrete wall. ω_k are the eigen values of the series and a_c the thermal diffusivity:

$$a_c = \lambda_c / (\rho_c \cdot c_{p,c})$$

$$\varpi \cdot \tan\varpi = \lambda_1 \cdot d_c / (\lambda_c \cdot d_1)$$

T_s is in reality a function of development of a fire as is dependent on time. For simplicity T_s is set to the maximum temperature in the fire curve specified in the Netherlands (RWS curve).

	Description	Distribution	Mean	Unit	V
$T_{crit}1$	Critical temperature in reinforcement, high value.	lognormal	750	^oC	0.1
$T_{crit}2$	Critical temperature in reinforcement, low value	lognormal	380	^oC	0.1
T_o	Initial temperature of structure	lognormal	23	^oC	0.2
T_s	Surface temperature	lognormal	1300	^oC	0.1
d_c	Concrete cover	lognormal	0.035	m	0.3
ρ_c	Density of concrete	lognormal	2400	kg/m3	0.03
λ_c	Thermal conductivity concrete	lognormal	2.6	W/Km	0.30
c_c	Specific heat of concrete	lognormal	1100	J/kgK	0.15
d_l	Thickness protection board	lognormal	Variable	m	0.1
ρ_l	Density fire protection*	lognormal	870	kg/m3	0.03
λ_l	Thermal conductivity protection*	lognormal	0.175	W/Km	0.15
c	Specific heat protection*	lognormal	1130	J/kgK	0.15

Table 6.7 Input data for probability estimation

The calculation of the probability of exceeding the limit state (and the associated reliability index β, ref. Figure 4.2) is carried out using the so-call FORM (first order reliability method) by the German structural reliability programme STRUREL. In Figure 6.5 the probability of failure given a RWS fire is illustrated in terms of the relationship between the reliability-index, β and the thickness of fire protection board (for $T_{crit}1$ and $T_{crit}2$).
The probability can be used together with the probability of occurrence of the fire and the cost of the measure to determine the thickness of the fire protection.
Finally the achieved safety can be compared with the requirements of the applicable codes. The Eurocode specifies a target value of probability of failure due to fire of less than 7.23 10^{-5} during the structural lifetime.
Assuming the probability of a severe fire (corresponding to the RWS data) is in the magnitude 0.1 during the lifetime of the tunnel, the target value for the probability of failure given this fire is 7.23 10-4, which corresponds to a reliability index, β_T of 3.2. Comparing this value with the reliability indices in Figure 6.5, it appears that a protection layer in a thickness in the range 5 - 25 mm is required in this example in order to fulfil the requirements of the Eurocode, depending on the utilisation of the reinforcement. The optimum thickness of the fire protection may be higher and can be found based on a study of the specific consequences of failure and the costs of the fire protection relative to the reduction of repair at all fires.

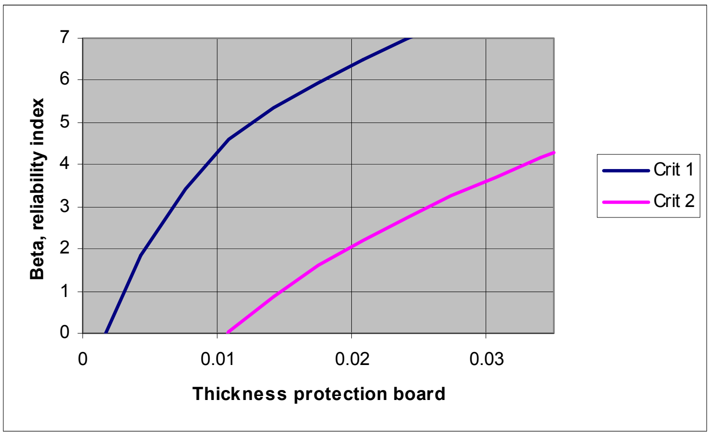

Figure 6.5 Reliability-index, β as function of thickness of fire protection board indicated for two critical reinforcement temperatures: $T_{crit}1 = 750^oC$ and $T_{crit}2 = 380^oC$, given a fire with temperature 1300^oC.

7. CONCLUSION

With the methodology developed as part of the DARTS project and presented in the paper, it is possible to have tailor made fire protection, which takes into account all information about the material and structural behaviour of the specific tunnel and the risk of fire based on the actual traffic. It is illustrated how this can be used as part of integrated design including also economic, environmental and durability considerations.

8. REFERENCES

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PRESENT-DAY DESIGN FIRE SCENARIOS AND COMPARISON WITH TEST RESULTS AND REAL FIRES: STRUCTURES & EQUIPMENT

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ABSTRACT

This paper discusses present-day design fire scenarios and comparison with test results and real fires. Use has been made of various sources, collected in the frame of the FIT, DARTS and UPTUN project. The purpose of the paper is to demonstrate the spate in current design fire scenarios through the member states and hence the need to harmonise the approach towards design fire scenarios in Europe.

1. INTRODUCTION

A fire could be defined as an unwanted and unforeseen fire as regards: place, size and time of occurrence, with extreme heat and excessive hot and/or toxic smoke development and spread. A scenario in this respect is an assumed course of events, following the ignition of the fire. A design fire scenario thus represents a possible outcome of a fire incident, based upon a number of governing conditions, for example the quantity and characteristics of combustible material, the arrangement of materials, tunnel geometry, fire compartment size, availability of ventilation, position of the fire in the tunnel, location of the fire on the vehicle/rolling stock (e.g. underneath the train, overheated breaks, …).

2. DESIGN FIRE SCENARIOS

A design fire scenario might concentrate on the pre-flashover stage only, when occupants are evacuating the train fire compartment, on post-flashover, when the impact on the tunnel structure becomes important, or on both stages. The pre-flashover stage is associated with a growth rate, e.g. slow, medium, fast or ultrafast.
When considering fire scenarios mainly two kinds of fire scenario curves are important, rate of **heat release curves** inside the train (the RHR curves are used for zone modeling and CFD) and **temperature time curves** (T-t) outside the train (the T-t curves are used for fire testing and analysis of impact of fire on the structure.