Session 5

Numerical Modelling

ADAPTATION OF FEA CODES TO SIMULATE THE HEATING AND COOLING PROCESS OF TIMBER STRUCTURES EXPOSED TO FIRE

Mechanical Behaviour

Danny Hopkin [a/c], Jamal El-Rimawi [a], Vadim Silberschmidt [b], Tom Lennon [c]

[a] Loughborough University, Department of Civil and Building Engineering, Loughborough, UK
[b] Loughborough University, Wolfson School of Mechanical Engineering, Loughborough, UK
[c] BRE Global, Garston, Watford, Herts, WD25 9XX, UK (HopkinD@bre.co.uk)

INTRODUCTION

Computer analysis of timber structures exposed to fire is not a widely undertaken task. Typically, such modelling is limited to simplified sectional analysis tools based on spreadsheets and underpinned with basic code. In the case of timber, additional complexities related to its orthotropy and differences in tensile and compressive stiffnesses at elevated temperature mean that the analysis of timber structures in fire does not lend itself easily to more powerful generic finite element software packages, such as TNO DIANA and ABAQUS.

When using the properties of timber at increasing temperatures, an assumption of identical reduction of the modulus of elasticity (MOE) under both tensile and compressive stresses is not suitable; simple correlations between temperature and tensile and compressive strengths are also inadequate. In the case of the former, the compressive MOE of timber at increasing temperatures degrades at a much higher rate than the tensile MOE. As a result, in a uniformly heated symmetrical section, the neutral axis is not central as would be expected. The effect of temperature on the strength of timber is also unusual. Unlike other materials, timber undergoes a very noticeable physical change upon heating. At a temperature of approximately 300°C timber chars, leaving a friable weak residual mass. Therefore, characterising strength on the basis of temperature alone would result, in some instances, in an artificial strength recovery in char zones upon cooling. Clearly, the full temperature history of an element, together with its current temperatures, is important in the determination of any strength recovery upon cooling.

In this paper, the development of a number of sub-routines for characterising the behaviour of timber elements exposed to both heating and cooling is described using simple examples. Developed codes for the determination of MOE, compressive and tensile strength under both cooling and heating are discussed. The subroutines are coupled with DIANA total strain-based constitutive models, which describe yielding and fracture. The potential for these developments to be adopted in the fire design of large-section timber structures is also evaluated.

1 MODELLING FIRE-EXPOSED TIMBER STRUCTURES

The modelling of timber exposed to fires has largely focussed upon temperature development either in unprotected large-section or light-weight gypsum-lined structures. The mechanical modelling of timber exposed to fire is a recent endeavour instigated by Thomas (1997) followed subsequently by Konig & Walleij (2000). More recently, further investigations were undertaken by Schmid *et al.* (2010) who conducted simulations of cross-laminated timber (CLT) members exposed to fire. However, in almost all instances the modelling has been performed using in-house *ad-hoc* sectional analysis codes, which are only appropriate for single-member analyses. If buildings are to be designed in a performance-based manner, whereby full building interactions are considered, then modelling approaches need to be advanced and more powerful codes adopted.

The adoption of more general finite element packages, such as DIANA and ABAQUS, for modelling timber has yet to become common due to a number of complexities relating to the behaviour of timber. For example, it is brittle and fractures in tension, while being more ductile and plastic when subject to compression.

A more interesting behaviour of timber is that, with increasing temperature, the degradation in its constitutive behaviour in tension is different from that when it is in compression. As a result, its MOE depends on its state of stress. Therefore, a single MOE-temperature relationship cannot be defined. Finally, upon heating timber undergoes a phase change whereby wood becomes friable char. Upon cooling, char still has little or no strength or stiffness. Therefore, it is not appropriate to specify timber strength on the basis of temperature alone when cooling is to be considered; knowledge of the full temperature history is required. Most commercial finite element programs do not incorporate many of the above characteristics in a direct way. Therefore, it is necessary to adapt such codes to accommodate these behaviours. An approach for doing this is described in the remainder of this paper. Implementation of the approach in the FEA software TNO DIANA (Manie 2010) via FORTRAN user-supplied subroutines (USS) is also described.

2 USRYOU- A SUBROUTINE FOR DETERMINING MOE OF TIMBER

DIANA offers a number of subroutine options for customising the analyses performed. One such subroutine is the USRYOU option, which allows users to return MOE based upon a number of inputs, including integration point strain and temperature. The authors have developed a USRYOU USS for determining the MOE of timber exposed to both heating and cooling.

Firstly, integration point and element numbers are called from the program along with temperature at the given integration point. Temperature history of elements is recorded via a common block, which determines whether the temperature of the timber (a) exceeded the charring temperature of 300°C, (b) exceeded the moisture evaporation temperature of 100°C, or (c) below 100°C, that is, timber neither charred nor suffered moisture evaporation. Based on this, the temperature-history common block is updated incrementally, which allows state history to be recorded. The latter allows for a number of hypotheses to be investigated, which will be discussed below.

Using the recorded temperature history the stress state may be investigated. The strains in the local element co-ordinates are called from DIANA. The dominant integration point strain is determined, which is then used to evaluate MOE appropriate to temperature and the strain state. For example, if ε_{xx} is found to be the largest element strain and the strain is negative (that is compressive), the MOE is returned, based upon EN 1995-1-2 (BSI 2004) compression (-ve) reduction factors (K_{EC}) and element temperature. The converse case would be adopted if ε_{xx} was found to be positive (+ve). In this instance reduction factors are defined as K_{ET}. This process is shown diagrammatically in Figure 1.

3 USRCST/TST- A SUBROUTINE FOR DETERMINING COMPRESSIVE/ TENSILE STRENGTH OF TIMBER

Two more USSs, namely USRCST and USRTST, are available in DIANA for determining the tensile and compressive strength, respectively. These routines in particular are for implementation with Total Strain Based constitutive models, which are discussed in a supporting paper (Hopkin *et al*. 2011b). The routines utilise the temperature-history common block initialised in the above USRYOU routine to calculate tensile and compressive strength using EN 1995-1-2 reduction factors. The element number and integration point number are used to reference allocated memory slots, where temperature state variables are stored. Compressive and tensile strengths (limiting stresses) are passed to DIANA for implementation in the adopted Total Strain Based Constitutive model. This process is shown in a flow diagram in Figure 2.

4 THE USE OF TEMPERATURE STATE VARIABLES IN THE COOLING DOWN PHASE

When the temperature reaches a certain limit within a timber section its moisture content will be completely lost through evaporation. As the section's temperature increases, charring of certain parts may also occur. The charred timber will not contribute to the strength and stiffness of the section. Clearly, this damage is irreversible. However, little is known about the strength and stiffness of the remaining un-charred part of the section and how the section behaves during cooling down. To study this, the concept of temperature history state variables (SVs) has been introduced. This allows for a number of hypotheses to be investigated.

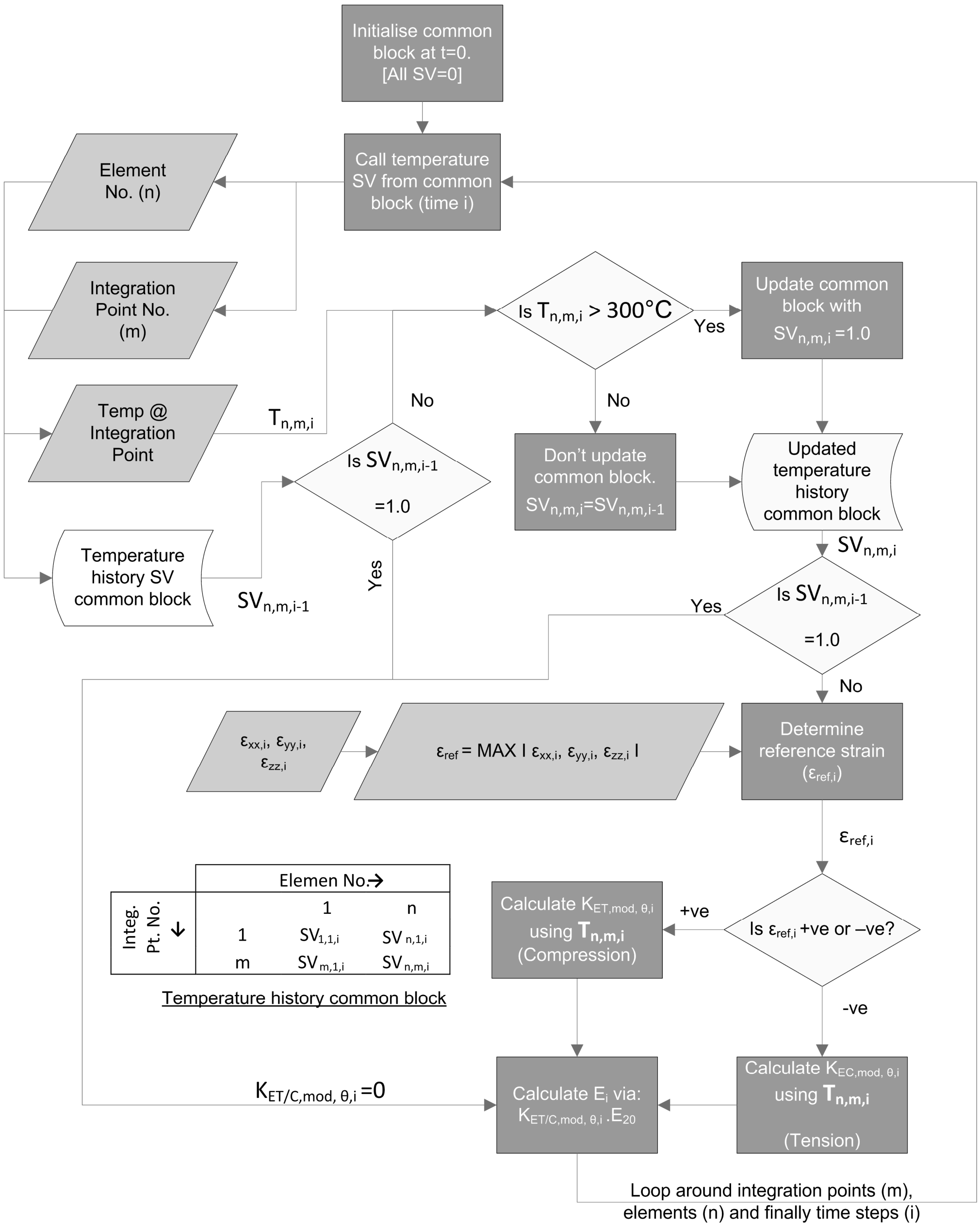


Fig. 1 Flow chart for USRYOU routine

The first hypothesis is based on the assumption that, during cooling down, undamaged timber recovers none of its strength or stiffness. This would be the case if temperature changes during the cooling phase are ignored. In this case, the maximum temperature reached during heating up governs the behaviour during cooling.

In the second hypothesis it could be argued that moisture lost during the heating phase cannot be regained during cooling down. This means that strength and stiffness of timber whose temperature, upon heating, did not exceed 100°C will be fully recovered to that appropriate to its temperature during cooling down. However, timber heated beyond 100°C, but not charred, may recover its strength and stiffness but only up to the maximum value applicable to dry moisture-free timber. The latter condition implies that reduction factors corresponding to 100°C should remain applicable even when timber temperature drops below this limit.

A third and final hypothesis is that, while cooling down, all non-charred timber will recover the entire strength and stiffness appropriate to its temperature. This implies that loss of moisture during heating had no effect on the recovered properties. Clearly, this approach may be implausible as it suggests the return of moisture into the timber during cooling. Although timber is a hydroscopic material the return of free moisture would take time much in excess of the decay phase of most fires. The implications for these three hypotheses can be observed through a number of simple numerical tests on single quad elements or simply supported beams. This testing process is discussed in the following section.

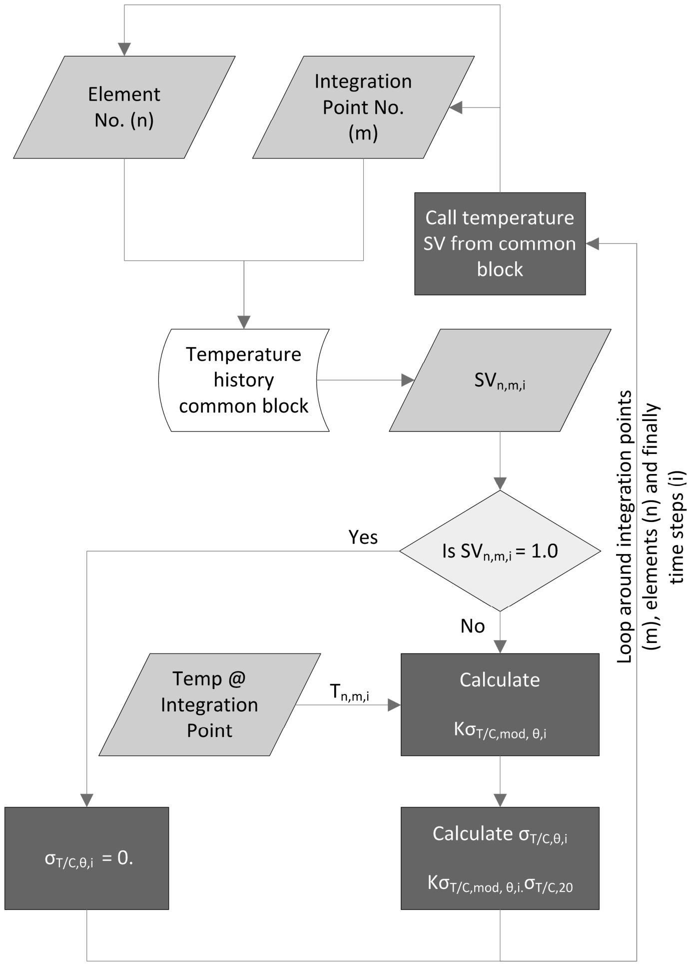


Fig. 2 Flow chart for USRCST/TST routine

5 SUBROUTINE TESTING AND IMPLEMENTATION

The testing of the USSs was conducted at a number of different scales. Firstly, trials of hypotheses 1–3 were conducted using single first-order quad elements uniformly heated and cooled down, and subject either to a compressive or tensile strain. In these trials, only the USRYOU subroutine is implemented so that non-linear elastic solutions can be sought without either cracking or plasticity. Resulting strain-temperature plots are shown in figure 3 for all three hypotheses. A constant load was applied throughout. In such trials it is not possible to indicate permanent charring damage as it would result in numerical instability. Thus, the maximum applied temperature was 210°C. Tensile and compressive loads of identical magnitude were applied to allow for the difference in MOE degradation with temperature for different strain states to be checked. However, only one set of results (compressive) is shown as the other indicated the same pattern.

The second element of USS testing is concerned with the behaviour of simply supported beams subject to a temperature gradient. A beam was modelled simply in DIANA using a number of first-order 2D beam elements. Temperatures were specified at 11 integration points through the cross section of beam elements. Integration point distribution was according to a Simpson integration scheme. The adopted temperature profiles are shown in Figure 4a. The legend indicates fire from below with 11 integration points numbered from the top down. The temperatures applied are fictitious temperatures and serve only to demonstrate implementation of the USS. The modelled beam is 4 m in length and has a 100 mm x 250 mm cross-section. The beam is subject to a nominal load of 5 kN/m. The development of deflection upon heating, followed by cooling can be seen in Figure 4b. In this case, the beam temperature developed beyond 300°C. Therefore, permanent deformation due to charring was apparent, including the case with full un-charred timber strength recovery (hypothesis 3).

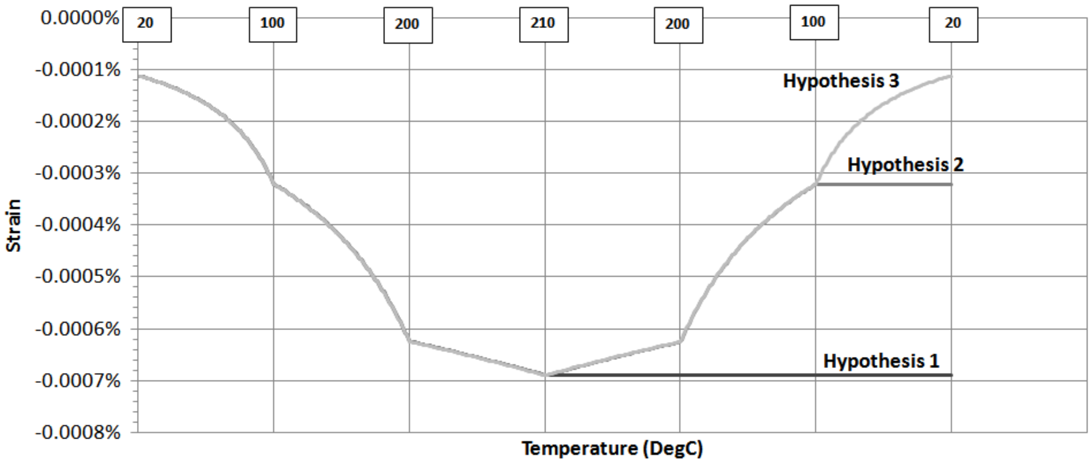
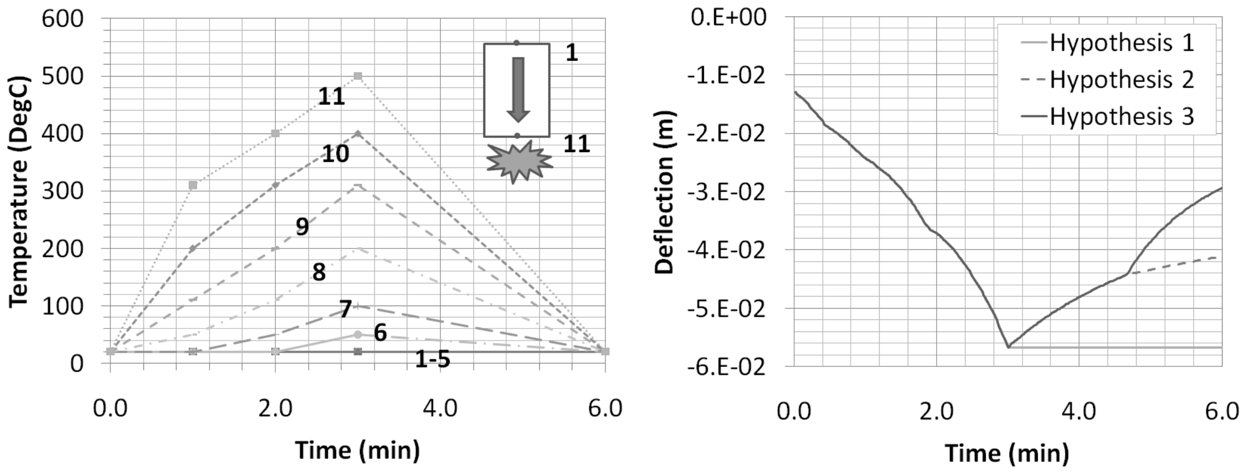


Fig. 3 Single-element implementation of USRYOU subroutine: temperature-strain plots for a constant nominal load



(a) (b)

Fig. 4 Beam implementation of USRYOU and USRCST/TST subroutines: (a) Temperature profiles at integration points (b) Deflection–time plots.

6 SUMMARY

A number of relatively simple modifications to a commercial finite element code have been presented. The modifications allowed for timber in fire to be modelled in potentially much more complex scenarios than previously considered. Until recently, the thermo-mechanical modelling of timber using commercial codes has not been widely undertaken. Where attempts have been made it has been done with simple *ad-hoc* codes having limited fields of application. If more advanced simulations of timber are to be conducted then either specialist codes need to be developed or commercial codes adapted.

The adaptation of a commercial code, such as DIANA, by means of FORTRAN user-supplied subroutines is desirable for a number of reasons. Firstly, with simple modifications to the stress relations, outlined above in the USSs presented, any number of element variations, from beam, through shell, to block elements, can be considered depending upon the problem encountered. Secondly, the powerful robust solvers, which are heavily tested in commercial codes, can be adopted with only the aspects of material behaviour which need to be appropriately represented by the USS, such as MOE, tensile and compressive strength with increasing (and decreasing) temperature.

In relation to the strength and stiffness recovery of timber upon cooling, experimental evidence suggests that the first two hypotheses may be more realistic (Lennon *et al*. 2010 & Hopkin *et al*. 2010). In the many experiments conducted by BRE on timber structures over the last decade, there appears to be little evidence to suggest any strength or stiffness recovery in timber structures exposed to fire, upon cooling.

Our papers presented at this event (Hopkin *et al*. 2011a/b), coupled with the developments proposed in this paper, give an integrated procedure for modelling large-scale timber assemblies exposed to fire. The modified conductivity model (MCM) proposed in a supporting paper (Hopkin *et al*. 2011a) provides a means of determining transient reduced cross section during parametric design fires, which can be adopted with the USS presented herein in thermo-mechanical analyses. A further study (Hopkin *et al*. 2011b) discusses implementation of the constitutive behaviour of timber in FEA models using relatively simple Total Strain Based models, with appropriate fracture energies. All the approaches presented will be developed further. However, initial findings appear to give consistent results with many empirical methods present in EN 1995-1-2 for isolated structural members. Thanks to the growth of timber constructions market share, both in the UK and continental Europe, it is becoming increasingly important to develop tools appropriate for structural fire design. As timber buildings become increasingly larger and more complex, numerical simulations would become a valuable resource.

7 ACKNOWLEDGEMENTS

The authors would like to thank EPSRC and BRE Global for their support.

REFERENCES

BRITISH STANDARDS INSTITUTION, 2004. Eurocode 5: Design of timber structures – Part 1-2: General – Structural fire design. BS EN 1995-1-2. London: BSI.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., LENNON, T., A numerically derived modified conductivity model for softwood exposed to parametric design fires. In: Proceedings of applications in structural fire engineering, 29th April 2011, CTU, Prague.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., LENNON, T., The impact of assumed fracture energy on the fire performance of timber beams. In: Proceedings of applications in structural fire engineering, 29th April 2011, CTU, Prague.

HOPKIN, D.J., LENNON, T., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. Failure mechanisms of structural insulated panel (SIP) systems under natural fire conditions, V.R. KODUR and J.M. FRANSSEN, eds. In: Proceedings of the sixth international conference on structures in fire (SiF), 2-4th June 2010, DEStech pp520-527.

KONIG, J. and WALLEIJ, L., 2000. Timber frame assemblies exposed to standard and parametric fires part 2: A design model for standard fire exposure. I0001001. Stockholm: SP Tratek.

LENNON, T., HOPKIN, D.J., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. Large scale natural fire tests on protected engineered timber floor systems. Fire safety journal, **45**(3), 168–182.

MANIE, J., 2010. DIANA User's Manual. 9.4 edn. Delft: TNO.

THOMAS, G., 1997. Fire resistance of light timber framed walls and floors. PhD Thesis. New Zealand: University of Canterbury Press.

SCHMID, J., KONIG, J. and KOHLER, J., 2010. Design model for fire exposed cross-laminated timber, V.R. KODUR and J.M. FRANSSEN, eds. In: Proceedings of the sixth international conference on structures in fire, 2-4th June 2010, DEStech pp511-519.

SIMULATION OF THE STRUCTURAL BEHAVIOR OF STEEL-FRAMED BUILDINGS IN FIRE

Filippo Gentili[a],Luisa Giuliani[b]

[a]Sapienza University of Roma, Structural and Geotechnical Engineering Department, Rome, Italy
[b]Technical University of Denmark, Department of Civil Engineering (DTU-BYG), Lyngby, Denmark

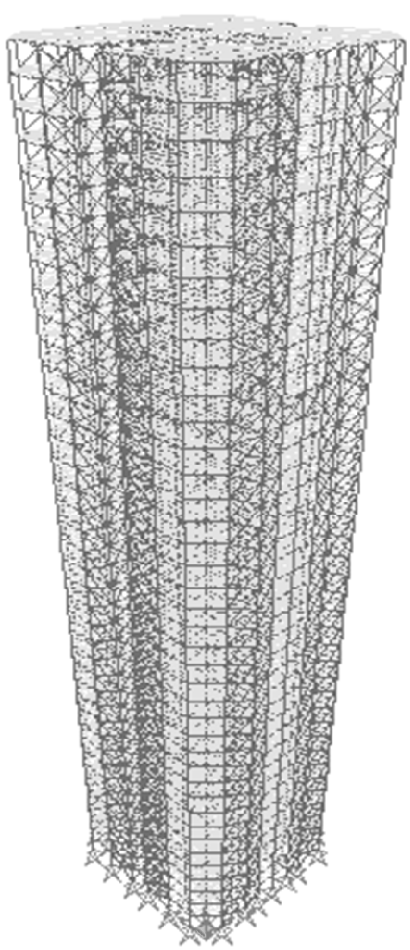
INTRODUCTION

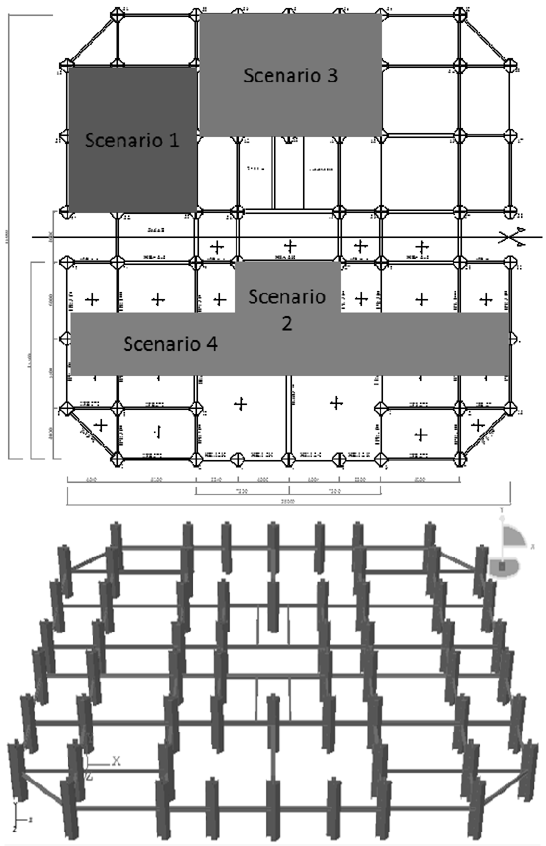
Structural integrity of buildings and safety of people in urban areas have been often endangered in the past by malevolent or accidental fires. Among all buildings in particular, high-rise structures pose particular design challenges with respect of fire safety for a number of reasons (Craighead, 2003), which refer not only to enhanced difficulties in evacuating the building, but also to the developing of the fire (i.e. the characteristic of the action)and to the response of the buildings (i.e. the characteristic of the structural system).
From the point of view of the fire action, either it is difficult for the fire-fighters to extinguish the fire, due to reduced accessibility and the limited length of ladders and fire hoses, and the fire might easily propagates upwards and spread to higher floors through the internal façade ducts or externally through windows as a consequence of the buoyancy effect on the thermal plume: Andraus Building fire of Sao Paulo in 1972, the First Interstate Bank fire in 1988 and the One Meridian Plaza fire of Philadelphia in 1991, are some of the worst cases where external fire spread was identified as a significant factor (Crooke, 2002).In order to reduce the increased risk due to the high elevation (BS 5588-5, 2004; NFPA 101, 2009), building codes of several countries nowadays require special fire safety systems for high-rise building, including, smoke venting, sprinkler systems and special fire alarms for an early detection of the fire and a prompt intervention of the fire-fighters. External shelters and window protrusions can also be effective for avoiding external fire propagation(Galea et al., 1996). Nevertheless, the design of active fire safety measures proved to be insufficient in several cases: some examples are the Parque Central Tower fire of Caracas in 2004, where the sprinkler system was not in working condition due to lack of maintenance, and the Mandarin Oriental Hotel fire of Beijing in 2009, where the sprinkler system was not installed yet, since the building was under construction.
Under those circumstances, the passive fire resistance of structural elements (MSB-6, 2008) and the intrinsic robustness of the system are the only measures to rely on, in order to maintain the structural integrity of the building during and after the fire and avoid major economic losses and additional casualties due to collapsing members.High-rise buildings of different typologieshave been susceptible to progressive collapses induced by fire, such as the Windsor Tower fire of Madrid in 2005 (steel and concrete), the Architecture Faculty Building fire of Delft in 2008(concrete) and the Odd Fellow Palace fire of Copenhagen in 2010 (wood and masonry).The first and the last ones in particular seem to have experienced pancake-type collapses (Starossek, 2009) that originate when the an initial the loss of vertical load bearing members cause the failure and separation of the horizontal slabs, which, impacting the slabs below, would propagates the collapse downwards. A different possible collapse mechanism have been instead recently highlighted (Usmani et al., 2003) for the case of the WTC collapse of New York in 2001 (steel), where the initial buckling of the heated floor slabs induced by hindered thermal expansion would have subsequently caused the buckling of the columns, as a consequence of the loss of horizontal restrains of the floors. It seems plausible to extend this type of mechanism to other type of steel framed buildings too, in particular those with a strong column - slender beam system, such as the one considered here as case study.

1 CASE STUDY

In this paper, a 3-dimensional substructure representing one floor of a high rise building has been investigated under different fire scenarios, with the avail of a finite element commercial code.

Particular attention is devoted to the description of the i) methodology followed for building the model, ii) setting the analyses and iii) interpreting the results.





Floor model properties	Node number	2738
	Element number	1144
	Total D.O.F	9556

Fig.1Model of the high-rise building   Fig. 2 floor plan FEM model and fire scenarios

1.1 Description of the structure

The building considered as case study is a multi-storey building, whose premises are devoted to offices and residential use and has been designed on the basis of the geometry and characteristic of a building recently built up in Latina, Italy.

The building has a steel framed structural system. A vertical bracing system provides stiffness against horizontal actions, while no horizontal bracings are present in the floor planes: the stiffness within the floor planes is achieved by means of bidirectional concrete floor slabs, which maintain the biaxial symmetry of the floors and are lightened by spherical hollows. The slab characteristics allow for long spans and slender beams, which can be contained within the height of the slabs.

1.2 Fire scenarios and thermal model

Some relevant fire scenarios can be pragmatically identified on the structure (Bontempi, 2010): for this study it is assumed that the vertical compartmentalization of the building remains intact and the fire originates and spreadsin one floor only. In principle, different fire scenarios along the building height should be considered, since floors have different elements and loads and vertical propagation of the failure can be different. However, in the following, only four fire scenarios will be investigated, which refer to fire on the 5th floor of the building and are reported in Fig.1.

The temperature-time curve considered for the fire is the ISO 834, while the heating curves of the beams involved in each fire scenario have been calculated under the assumption of uniformtemperature in the element, according to the Eurocodes formula for unprotected steel, using a convective coefficient α = 25 W/(m^{2}K) anda totalemissivity ε = 0.5 (no shadow effect considered).

1.3 Structural model

The modelling of a complex construction requires formulating some assumptions and starting with the investigations of simpler substructures, where the collapse mechanisms can be identified more easily and the validity assumptions can be checked. Judging by the results of this preliminary investigation, the model can be refined in a later stage and bigger substructures can be considered.
In the following investigations therefore, only beams have been preliminarily assumed to be directly involved in the fire.It is expected that under these conditions, the primary initial effects of the fire will be localised to the elements of the floor, so basic preliminary investigations are carried on a substructure, which represents the 5th floor of the building (Fig.2 bottom).This condition can be representative of a construction stage, where the beams, which are not insulated, are left unprotected by the absence of the floors. It can also be significant for understanding the effect of beam failures on the adjacent elements and highlight a possible propagation of the failures to zones of the structure not directly involved in the fire.
As a matter of fact, a particular dangerous situation for high-rise buildings under fire is represented by an indirect involvement of the columns, which are either pushed outside by the horizontal thermal expansion of the beam, or pulled inwards by the vertical runaway of the beams (Song et al., 2009). This mechanism is particularly relevant for frames where the stiffness of the beam is significant and comparable with the flexural stiffnessof the columns (EUR 24222 EN, 2010), while for strong column – slender beam system, the involvement of the column can be due to the stress redistribution and loss of lateral restrain consequent to the buckling of the horizontal members, as mentioned above.In both cases, the collapse would not remain localized and would propagate downwards through failures of columns and other floors.
In order to see a possible influence of the fire effects on the columns and to model with a sufficient accuracy the translational and rotational capability of the beam end nodes, columns are included in the model to the extent of half-length of the columns pertinent to the 5th floor (below the floor level) and half length of the columns pertinent to the 6th (above the floor level). The columns are continuous and restrained by hinges at bottom, and by vertical sliding support at the top and are considered to be unloaded in this preliminary investigation. In case a possible significant overloading of the column is evidenced in the analysis, a refined model should be considered, where more floor planes have to be modelled and a more realistic loading condition should be considered for the columns.

1.4 Analysis and collapse criterion

The analyses take into account thermo-plastic material and geometric nonlinearities. Dead and live loads pertinent to beams are applied as line forces and considered in a first static analysis step, together with self-weight, while in a second load step the temperatures of the calculated steel heating curves of the beams are applied to the beam nodes and an implicit dynamic solver has been used in order to overcome convergence problems due to local mechanisms and be able to follow the propagation of failures.
Depending on the safety level considered and on the design objectives of the investigation, a nominal collapse condition can be assumed when displacements overcome a given limit, which in this work has been taken equal to L/20, where L is the beam length. Another possibility is to consider the runaway of the beam, with this term meaning the accelerating and irreversible downward displacement (Usmani et al. 2003) of the beam mid-span node. In the following the time resistance of the structure is evaluated and compared for the four fire scenarios considered, with reference to both the above mentioned collapse criteria.

2 RESULTS

In the following, the main outcomes are presented and discussed, with reference to each fire scenario. The first fire scenario is analysed in detail,with the aim of outlining a methodology for the interpretation of the results in this kind of investigations.The resistance times corresponding to each fire scenarios is evaluated and compared on the basis of the collapse criteria discussed above.

2.1 First fire scenario

The results of the analysis on the first scenarios highlight the following sequence of failures in the area involved in the fire:

1. After 2 min of fire, an out of plane buckling mechanism triggers, involving the three beams that converge in the middle external column (i.e. the two external transversal beams and the longitudinal beam between them). Almost contemporarily also the most internal beam on the left (indicated as InB on the left bottom image of Fig.3) buckles out of plane.
   The early failure of those beams is only due to the eigenstresses induced by the hindered thermal expansion of the beams (dotted and continuous lines in the top graphs at the right of Fig.3), consequent to the strong column- slender beam frame type and also due to the absence of the restrain provided by the slab: specifically, the three beams have IPE270 profiles, while the columns adjacent to them have HEM1000* profiles.As a consequence, thebeam failures trigger when the temperatures are very low (around 100 °C) and no degradation of the steel mechanical properties, which typically plays a determinant role in fire-induced collapses, has occurred yet.
2. Shortly after the first four beams, the two transversal beams in the middle, which have a slightly bigger profile (IPE300), buckle out of plane too.
3. At about 10 and 15 minutes of fire respectively, also the last two beams directly involved in the fire buckle out of plane. The higher resistance of those beams is due to the different profile of the sections, which is a HEA240.
4. At this point the temperature are quite high (ca. 600 °C) and the internal beams, which carry higher load than the external ones, experience a runaway and exceed the maximum acceptable displacement considered as nominal collapse criterion (right top graph of Fig.3). It is the material degradation which is responsible for the runaway, which determines the overcoming of the collapse condition and the lack of convergence of the analysis at about 20 min of fire.

It seems relevant to highlight the fact that the same design characteristic that is responsible of the early failure of the beams, i.e. the strong column – slender beam system, ensures on the other side a compartmentalization of the collapsing sections of the structure and avoid the propagation of the collapse to the vertical elements, which are only slightly overloaded (last two row of graphs on the right of Fig 3) by the stress redistribution consequent to the beam failures and would therefore be hardly involved in the collapse mechanism.
The results in term of deformed configurations and element displacements and forces are summarized in Fig. 3. In the left part of the figure the deformed configurations at 14 (top) and 20 min of fire (bottom) are represented. In the top figure, the progression of beam failure is indicated by numbers from 1 to 4, which correspond to the steps illustrated above. In the right part of the figure the result history is shown with reference to one external and one internal beam (indicated as ExB and InB in the left figure). The first row shows the horizontal and vertical displacement of the beam mid-spans, while the second row shows the axial stresses of the two beams: in order to highlight the effect of material degradation, the displacement and axial forces obtained without consideration of material degradation are reported in the graphs as dotted lines. The last two rows show instead the overloading of the columns adjacent to the monitored beams (indicated as ExLfC, ExRgC, InLfC and InRgC in the bottom left image of Fig.3), in term of axial and shear forces at the bottom node.

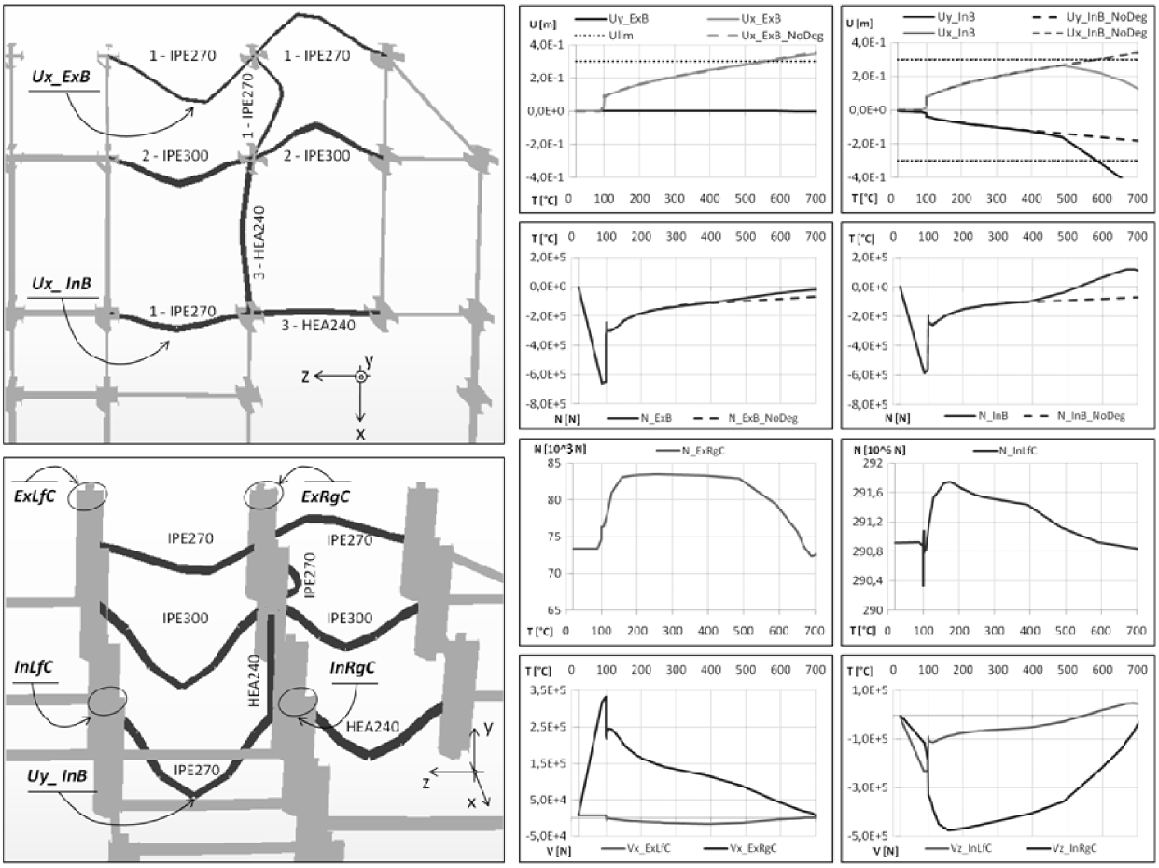


Fig. 3Results for fire scenario 1, in term of deformed configurations, forces and displacements.

Other fire scenarios

The deformed configurations for the other 3 scenarios are reported in Fig. 4, together with the horizontal and vertical displacements of the nodes, identified by the points A, B, C in the figures.

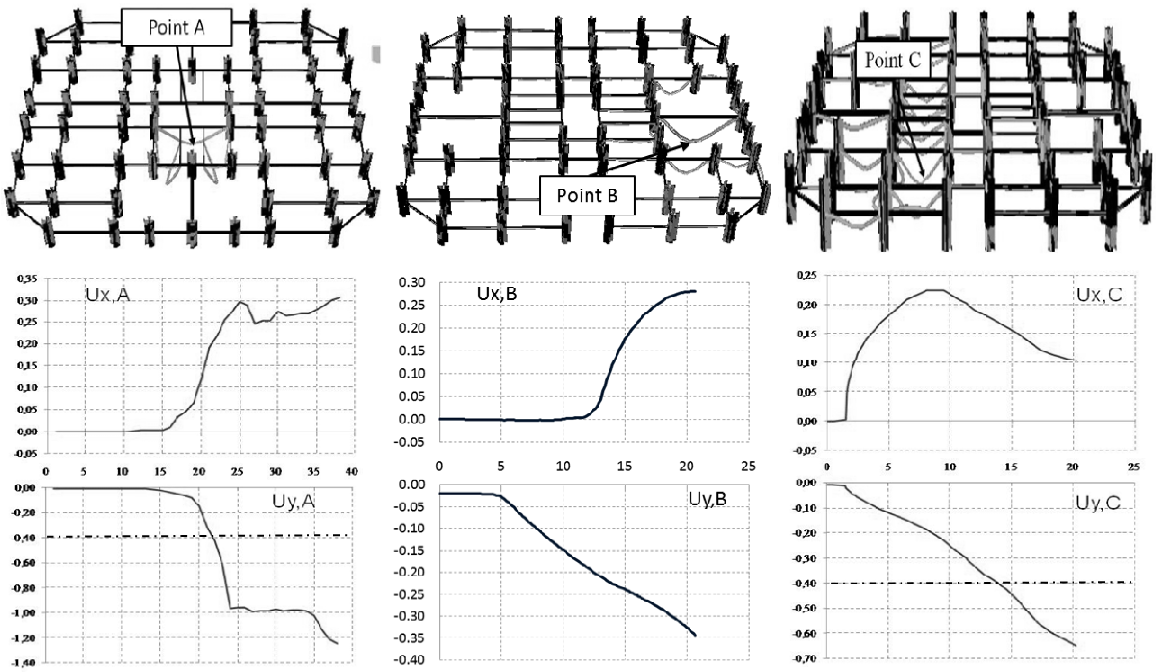


Fig. 4Results for fire scenario 2, 3, 4 (from left to right) in term of deformed configurations (top) and trend of the horizontal and vertical displacements (in m)versus time (in min) of monitored points(bottom).

These nodes identify the mid-span point of the beams that experience the highest displacement in each fire scenario.
It can be seen that all the monitored beams experience high displacements both in the horizontal and vertical direction. In the second scenario however, a discontinuity in the trend of the vertical displacement can be observed, which is followed by an almost horizontal branch that indicates a regain in stiffness of the element. This can be imputed to the triggering of a catenary action, occurring as a consequence of the significant vertical displacement of the beam, which support also two perpendicular beams, and of the high horizontal restrain offered by the columns. The same considerations apply to the monitored beam in the third scenario. In the fourth scenario instead, the monitored beam experience an out of plane buckling, which is followed by a significant but gradual increment in the vertical displacement, so that an actual runaway point cannot be identified.

3 CONCLUSIONS

By comparing the resistance times of the four scenarios summarized in Tab.1, the most critical scenarios seem to be the first and the fourth ones. However, whereas the in the fourth scenario a large portion of the floor is involved in the fire as a result of an assumed loss of the horizontal compartmentalization of the building, in the first scenario only few elements are directly involved in the fire. The second and third scenarios have a longer resistance in term of maximum nominal displacement: however the value of L/20 refers to beam of standard dimension and seem to allow very big displacement for long beams such as the one monitored in the third scenario, which even if experiencing very big displacements doesn’t reach the collapse condition within the analysis time.

Tab. 1 Resistance times in term of nominal maximum displacement and runaway of critical beams.

Scenario Time (in min) at	1 - InB	2 - PointA	3 - PointB	4 - PointC
L/20	9	22	>20	14
Runaway	12	20	---	---

REFERENCES

Bontempi F., Petrini F., Fire-Induced Collapses in Structures: Basis of the Analysis and Design, Proceedings of The Fourth International Conference on Structural Engineering, Mechanics and Computation (SEMC2010), Cape Town (South Africa), 6 - 8 September 2010.

BS 5588-5:2004. Fire precautions in the design, construction and use of buildings - Part 5: Access and facilities for fire-fighting, British Standard, 2004.

Craighead G., High-rise security and fire life safety, Elsevier Science, U.S., 2003

Crooke C., External fire spread via windows - closing report, Building Research Establishment, Fire and Risk Sciences Division, report nr. 203141, United Kingdom, 2002.

EUR 24222 EN, Fire safety of industrial hall - A valorization project, Final report, Research Fund for Coal and Steel, European Commission, DOI: 10.2777/90662, 2010

Galea ER, Berhane D, Hofmann NA. CFD analysis of fire plumes emerging from windows with external protrusions in high rise buildings, Interflam ’96, Cambridge, UK, March 1996.

MSB-6, Multi-storey steel buildings - Part 6: Fire Engineering, Steel Building in Europe design manual, Research Fund for Coal and Steel of the European Community, 2008.

NFPA 101:2009 Life Safety Code®, National Fire Protection Association, MA, U.S., 1988

Song Y.Y., Huang Z., Burgess I.W., Plank R.J., The Behaviour of Single-Storey Industrial Steel Frames in Fire. International Journal of Advanced Steel Construction, 5 (3), pp. 289-302, 2009

Starossek U., Progressive collapse of structures, Thomas Telford Publishing, London, 2009

Usmani A.S., Chung Y.C., Torero J.L., How Did the WTC Collapse: A New Theory, Fire Safety Journal, 38, 6, 501-591, 2003.

NUMERICAL ANALYSIS OF STRUCTURES IN FIRE USING OPENSEES

Jian Jiang [a], Asif Usmani [a], Jian Zhang [b], Panagiotis Kotsovinos [a]

[a] School of Engineering, The University of Edinburgh, Edinburgh EH9 3JF, United Kingdom
[b] School of Built Environment, Heriot-Watt University, Edinburgh EH14 4AS, United Kingdom

INTRODUCTION

Many finite element program codes have been written to simulate the structural behaviour at elevated temperature and the results compare well with the test data from Cardington test. These include specialist programs such as ADAPTIC(Song 1995; Izzuddin 1996), FEAST (Liu 1988; Liu 1996), SAFIR (Franseen 2000), VULCAN (Bailey 1995; Huang 2000) and commercial packages such as ABAQUS and DIANA. The specialist programs can be cost-effective compared to commercial packages but lack generality and versatility because they are always developed to focus on some special feature of structural behaviour in fire. OpenSees is an open-source object-oriented software framework developed at UC Berekeley. OpenSees has so far been focussed on providing an advanced computational tool for analysing the non-linear response of structural frames subjected to seismic excitations. Given that OpenSees is open source and has been available for best part of this decade it has spawned a rapidly growing community of users as well as developers who have added considerably to it's capabilities over this period, to the extent that for the analysis of structural frames it has greater capabilities than that of many commercial codes.
The OpenSees framework is being extended to deal with the frame structures in fire conditions by a team at the University of Edinburgh. New load classes are created to define the temperature distribution across the section of the element, which is built up with "fibres". Existing material classes are modified to account for the temperature-dependent material properties according to the Eurocode3. The existing beam-column element classes are modified to include interfaces to the updated material at elevated temperature and section with temperature field and to take account of the thermally induced forces and deformations. Some benchmark cases are studied to validate the thermo-mechanical analysis in OpenSees followed by the modelling of the restrained beam test at Cardington test. The analysis procedures being developed for structures under fire in OpenSees will make it easier to perform coupled heat-transfer and thermo-mechanical analyses allowing for non-uniform temperature distribution along an element resulting from local fires.

1 THERMO-MECHANICAL ANALYSIS IN OPENSEES

The OpenSees framework was chosen to develop thermo-mechanical analysis of frame structures by adding new thermal load class and modified temperature dependent material classes (shown in Fig. 1). Based on existing beam element *dispBeamColumn2d* (Taucer and Filippou 1991), the flowchart of the thermo-mechanical analysis in OpenSees is shown in Fig. 2. New thermal load class <*Beam2dTemperatureLoad*> is created to store the temperature distribution along the section consisting of temperature and coordinate, the temperature of each fibre will determined by the interpolation of the temperature at the nearest coordinate point according to its location. Also the temperature dependent material properties are modified from the existing bilinear steel material class according to Eurocode 3 (ENV 1993-1-2).

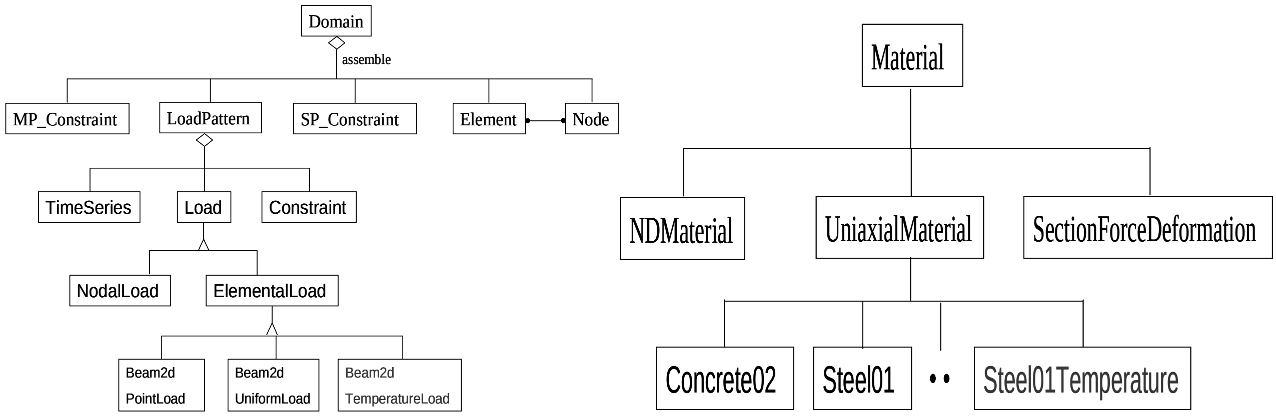


Fig. 1 Class diagram for thermal load and material in OpenSees

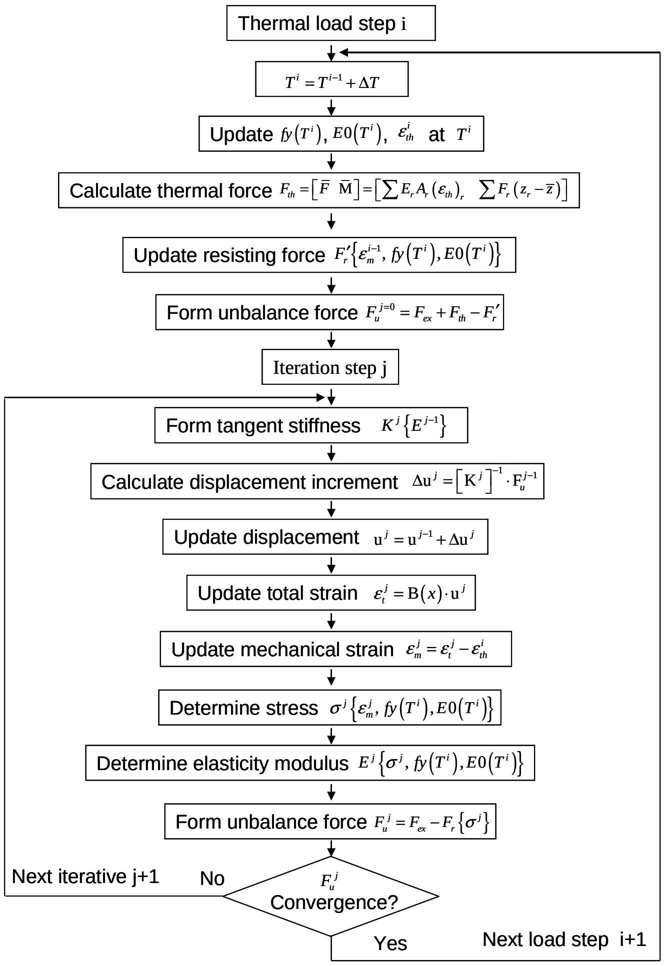


Fig. 2 Flowchart for thermo-mechanical analysis in OpenSees

2 VALIDATION

In order to establish the validity of the thermo-mechanical analysis in OpenSees, a number of benchmark problems are solved involving a beam subjected to temperature gradient and comparing the results against ABAQUS solutions. Further validation is obtained from modelling a uniformly heated steel frame test (Rubert and Schaumann, 1986).

2.1 Simply supported beam with varying restraint to lateral translation

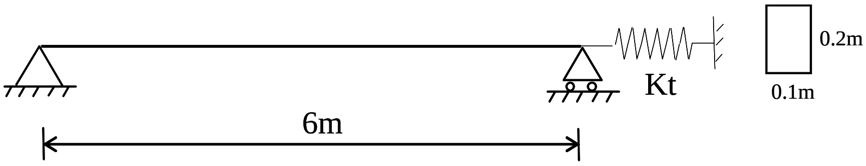


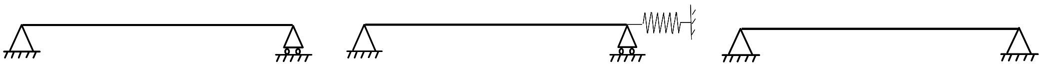
Fig. 3 Simply supported beam with a translational spring at end

The deformation of structures under fire conditions is sensitive to its boundary condition. Fig. 3 shows a 2D single beam with finite end restraint. The beam subjected to a uniformly distributed load (UDL) and a linear thermal gradient along the section height. The finite end restraints are represented by a translational spring of constant stiffness K_t. Different boundary conditions can be obtained by varying K_t. Thermo-mechanical analysis of beam members with these boundary conditions was conducted and the results were compared with ABAQUS.
The temperature at top of the beam is assumed 0^oC and the temperature at bottom varies linearly from 100^oC to 1000 ^oC. Temperature dependent elastic material is used for the model and constant expansion coefficient $\alpha = 12\times10^{-6} / {}^oC$ is assumed. The modulus of elasticity at elevated temperature is varied according to Eurocode 3 (ENV 1993-1-2) with an initial value of 20000MPa at ambient temperature. The details of the model are listed in Table 1.

Table 1. Input parameters of the beam model

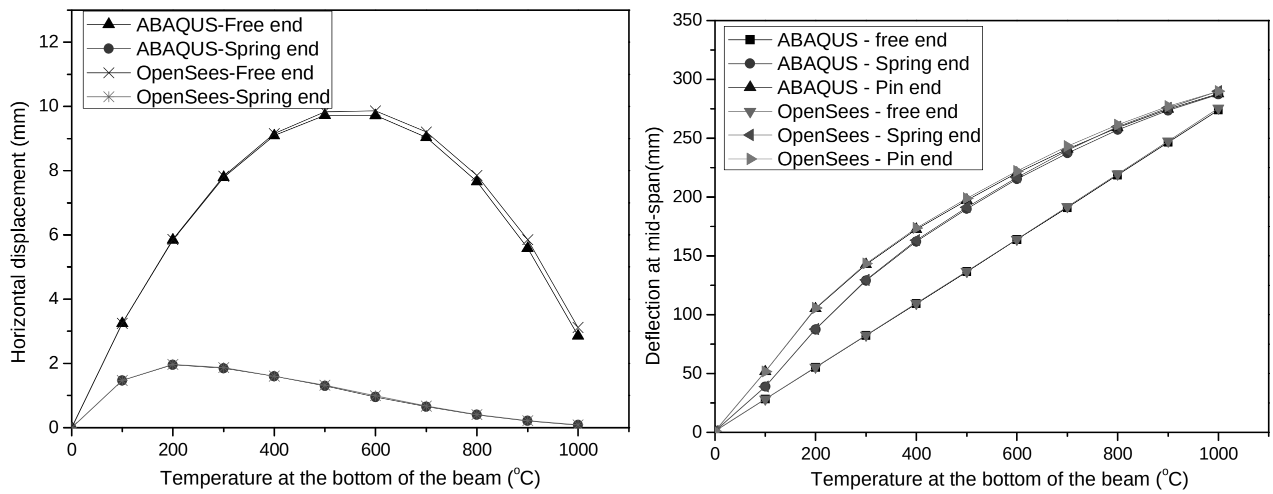
Length (m)	Area (m^2)	E (0oC) (N/m^2)	UDL (N/m)	Thermal gradient (^oC/m)	α (/^oC)	K_t (N/m)
6	0.02	2e11	1000	500-5000	12×10^{-6}	6.7e8

Three kinds of translational restraint were applied to the beams such as free end, finite translational end restraint and pinned end shown in Fig. 4. The predicted results shown in Fig. 5 and Fig. 6 agree well with that of ABAQUS.



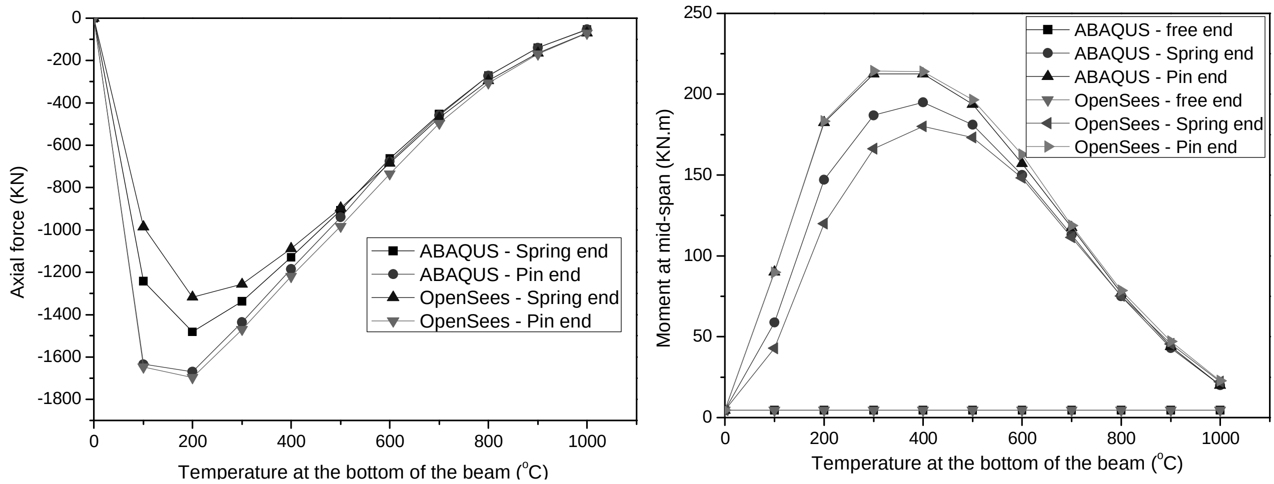
(a) (b) (c)

Fig. 4 Schematic of beams with different translational end restraint: (a) free end; (b) spring end; (c) pin end



(a) (b)

Fig. 5 Displacement of beam with different translational end restraint: (a) Horizontal displacement; (b) mid-span displacement

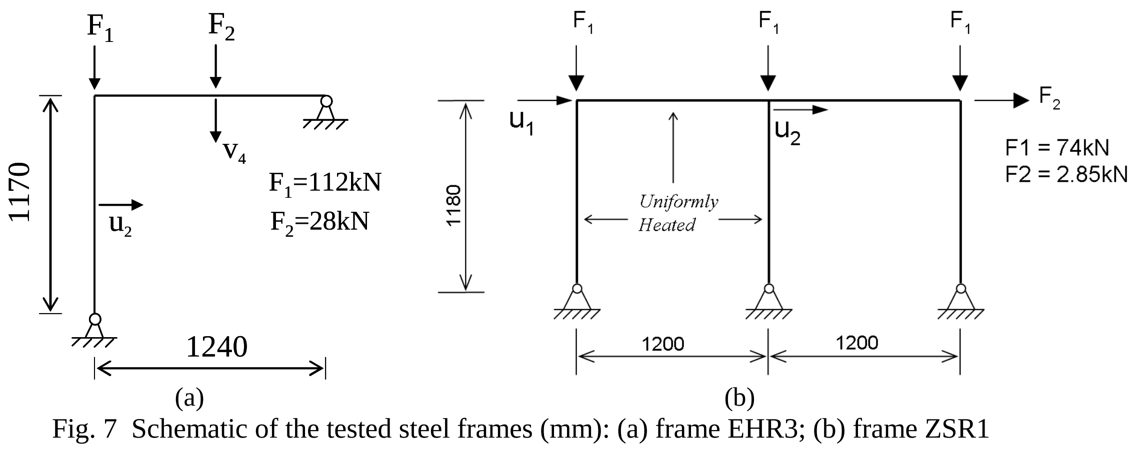


(a) (b)

Fig. 6 Forces in the beam with different translational end restraint: (a) axial force; (b) mid-span moment

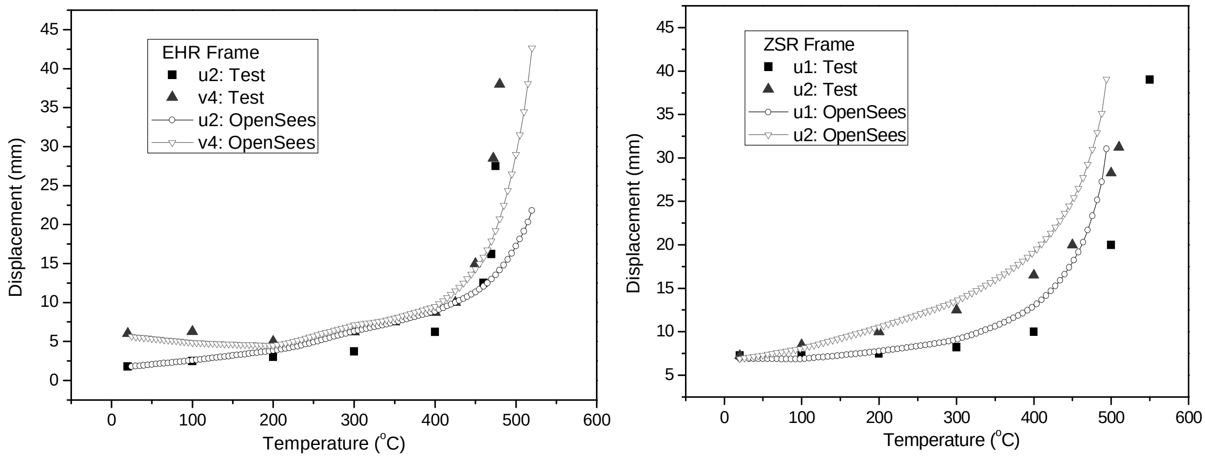
2.2 Steel frame test

Two steel frames as shown in Fig. 7 were tested by heating them directly to a high temperature. The results from an OpenSees analysis shown in Fig. 8 agree relatively well with the experimental data (Rubert and Schaumann, 1986), showing a qualitatively similar behaviour.



(a) (b)

Fig. 7 Schematic of the tested steel frames (mm): (a) frame EHR3; (b) frame ZSR1



(a) (b)

Fig. 8 Comparison between predicted and test deflection results: (a) frame EHR3; (b) frame ZSR1

3 CONCLUSION

The extended OpenSees framework (still under development) seems to be able to adequately deal with the thermo-mechanical analysis of structures under fire conditions. Further work is being done to develop 3D beam element and shell element classes for thermo-mechanical analysis.

REFERENCES

Bailey, C. G. "Simulation of the structural behavior of steel-framed buildings in fire", PhD thesis, Department of Civil and Structural Engineering, University of Sheffield, UK, 1995

ENV 1993-1-2. Design of steel structures, Part 1.2: general rules — structural fire design. Eurocode 3 Part 1.2, 2005

Franssen, J. M., Kodur, V. K. R. and Mason, J. User's manual for SAFIR2001, A computer program for analysis of structures submitted to the fire, University of Liege, Belgium, 2000

Huang, Z. H., Burgess, I. W. and Plank, R.J. "Effective stiffness modeling of composite concrete slabs in fire", Engineering structures 22: 1133-1144, 2000

Izzudin, B. A. "Quartic formulation for elastic beam-columns subject to thermal effects", ASCE Journal of Engineering Mechanics 122(9): 861-871, 1996

Liu, T. C. H. "Thermal modeling of steel portal frame behavior", PhD thesis, Department of Civil Engineering, University of Manchester, UK, 1988

Liu, T. C. H. "Finite element modeling of behavior of steel beams and connection in fire", Journal of Constructional Steel Research 36(2):181-199, 1996

Rubert, A. and Schaumann, P. "Structural steel and plane frame assemblies under fire action", Fire Safety Journal, 10, 173-184, 1986

Song, L., Izzudin, B.A., Elashai, A. S. "Nonlinear analysis of steel frames subjected to explosion and fire loading", Proceedings of the International Conference on Structural Dynamics, Vibration, Noise and Control, Hong Kong, December, 1995

Taucer, F. F. Filippou, F. C. "A Fiber Beam-column element for seismic response analysis of reinforced concrete structures". Report of University of California, Berkeley, 1991

DEVELOPMENT OF HEAT TRANSFER MODELLING CAPABILITY IN OPENSEES FOR STRUCTURES IN FIRE

Yaqiang Jiang [a], Asif Usmani [a] , Stephen Welch [a]

[a] University of Edinburgh, School of Engineering, UK

INTRODUCTION

An advanced analysis for structures subjected to fire typically involves a fire model, a heat transfer model and a structural model. Generally speaking, two categories of software may be used to achieve this purpose (Wang, 2002). The first group represents commercial packages which are generally powerful with well designed GUI based pre-processing and post-processing functionalities as well as rich element libraries. However, due to their intrinsic attributes, these packages may cause inflexibilities in dealing with new and unusual couplings such as an integrated fire-thermal-structural analysis (Welch et al, 2009) and therefore may impose restrictions on the users. The second group consists of in-house programs developed on the basis of research outcomes at certain research and academic institutions. Even if soundly validated, most of them have been developed using procedural programming languages which leads to weak modularity resulting in difficulties in software maintenance and further development. Object-oriented design and programming outweighs conventional procedural paradigm in building and maintaining large and complicated software systems (Rumbaugh et al, 1991). One of the first applications of the object-oriented design to finite element analysis was presented by Forde (Forde et al, 1990). Since then, increasing attention has been given to this topic due to the intrinsic complexity of finite element data structures (Zimmermann et al, 1992; Cross et al, 1999).

OpenSees, the Open System of Earthquake Engineering Simulation, is an object-oriented and open source software framework developed at the University of California, Berkeley (McKenna, 1997). It was initially designed to simulate structural frames subjected to seismic excitations using the finite element method. The framework has been known for its computational efficiency, flexibility, extensibility and portability (McKenna, 2010). One can conveniently introduce into OpenSees a new element, a new material model or even a new analysis procedure without the knowledge of every single piece of codes in the framework.

The authors have been motivated to add a structures-in-fire modelling capability into the OpenSees framework by utilizing its well-designed software architecture (Usmani et al, 2010). To achieve this purpose, a dedicated heat transfer module, addressing transient nonlinear heat conduction in structural members whose surfaces are exposed to spatially and temporally variable heat fluxes, has been developed using the finite element method. By adopting object-oriented programming paradigm, the software structure is consistent with that of OpenSees, which made it possible to reuse some of the existing components such as graph numbering classes and numerical classes. Important class diagrams and the detailed interactions between associated classes are described in this paper. Verification is conducted through running a series of benchmark tests using four-noded and eight-noded quadrilateral elements.

1 SOLUTION ALGORITHM

The discretised nonlinear heat transfer equation using the finite element method and generalised trapezoidal time-integration rule is expressed as

$$\mathbf{C}_{n+1}\dot{\mathbf{T}}_{n+1} + \mathbf{K}_{n+1}\mathbf{T}_{n+1} = \mathbf{Q}_{n+1} \tag{1}$$

where $T_{n+1} = T_n + \Delta t\{(1-\alpha)\dot{T}_n + \alpha\dot{T}_{n+1}\}$

In equation (1), **C** is called heat capacity matrix, **K** is conductivity matrix and **Q** represents the load vector which includes the effects of boundary heat fluxes $\overline{q}(T)$ over all the external surfaces. The users are referred to other publications for more details (Huang & Usmani, 1994). A general expression for $\overline{q}(T)$ describing heat flux boundary conditions is given as

$$\overline{q}(T) = q_{pr} + h(T - T_g) + \varepsilon\sigma T^4 - q_{ir}(t) \tag{2}$$

The first term, second term and the last two terms represent, respectively, prescribed heat flux, convective heat flux and radiation heat flux. It should be noted in the above equation the incident radiation flux does not come from a blackbody with constant temperature, but from all other radiating sources (smoke gas, flame, other solid surfaces, etc.) within a fire compartment. This assumption facilitates the fire-thermal coupling where the incident radiant fluxes can be computed by a separate CFD program.

Besides heat flux boundary conditions(except for prescribed ones), material properties such as specific heat and conductivity can be temperature-dependent as well, which together bring nonlinearities into equation (1). Therefore, an iterative algorithm should be used to obtain the converged solution. A typical predictor-corrector algorithm with Newton iteration method is reproduced here (Winget & Hughes, 1985)

Solution predictor(when $ir = 0$)

$$\begin{aligned} \mathbf{T}_{n+1}^{ir} &= \tilde{\mathbf{T}}_{n+1} = \mathbf{T}_n + \Delta t(1-\alpha)\dot{\mathbf{T}}_n \\ \dot{\mathbf{T}}_{n+1}^{ir} &= \tilde{\dot{\mathbf{T}}}_{n+1} = 0 \end{aligned} \tag{3}$$

Solution Corrector(iterating until converged)

$$\begin{aligned} \mathbf{M}_{n+1}^{*ir}\Delta\dot{\mathbf{T}}_{n+1}^{ir} &= \Delta\mathbf{Q}_{n+1}^{ir} \\ \dot{\mathbf{T}}_{n+1}^{ir+1} &= \dot{\mathbf{T}}_{n+1}^{ir} + \Delta\dot{\mathbf{T}}_{n+1}^{ir} \\ \mathbf{T}_{n+1}^{ir+1} &= \tilde{\mathbf{T}}_{n+1} + \alpha\Delta t\dot{\mathbf{T}}_{n+1}^{ir+1} \end{aligned} \tag{4}$$

The linearized tangent matrix here is defined as

$$\mathbf{M}_{n+1}^{*ir} = \mathbf{C}(\mathbf{T}_{n+1}^{ir}, t_{n+1}) + \alpha\Delta t(\mathbf{K}(\mathbf{T}_{n+1}^{ir}, t_{n+1}) + \mathbf{K}_q(\mathbf{T}_{n+1}^{ir}, t_{n+1})) \tag{5}$$

which approximates the exact tangent matrix by keeping symmetric terms only.

The residual vector here is defined as

$$\Delta\mathbf{Q}_{n+1}^{ir} = \mathbf{Q}(\mathbf{T}_{n+1}^{ir}, t_{n+1}) - \mathbf{C}(\mathbf{T}_{n+1}^{ir}, t_{n+1})\dot{\mathbf{T}}_{n+1}^{ir} - \mathbf{K}(\mathbf{T}_{n+1}^{ir}, t_{n+1})\mathbf{T}_{n+1}^{ir} \tag{6}$$

2 PROGRAM DESIGN AND IMPLEMENTATION

A comprehensive review on object-oriented finite element programming (specific to structural analysis) was reported and the software architecture for OpenSees was proposed by McKenna (McKenna, 1997). As with its initial purpose, most classes in OpenSees have attributes and behaviours specific to structural analysis only and are not applicable to heat transfer analysis. However, in object-oriented software design, a class should describe objects with similar attributes, common behaviours, common association relationships, and common semantics (Rumbaugh et al, 1991). For this reason, a separate set of classes specific to heat transfer analysis has been created, which will be presented in the rest of this section in detail.

Standard UML (Unified Modelling Language) class diagrams represented by graphical notations are used here for demonstrating the structure of the system (Rumbaugh et al, 1999). Fig. 1 shows the high-level classes for the heat transfer module. A HeatTransferDomain creates the analysis environment for a HeatTransferAnalysis object by aggregating components of a finite element model. Methods to add, remove and access those components are provided. A HeatTransferAnalysis object has to be associated with a specific HeatTransferDomain object to perform the analysis. A HeatTransferNode corresponds to a specific node in FE discretization, which knows its coordinates and holds its temperatures and temperature derivatives (with respect to time).

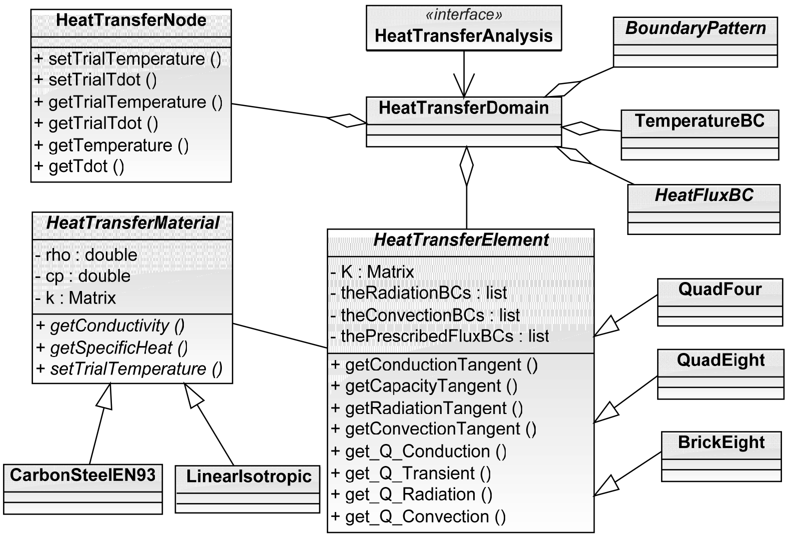


Fig. 1 Diagram for high-level classes for heat transfer module

Methods to modify and query those quantities are provided. Similarly, a HeatTransferElement object represents a specific element in the FE discretization and it is the basic computational unit returning the tangent matrix and residual vector at elemental level. As shown in equation (5) and (6), both the tangent and residual vector can be decomposed into four parts by considering contributions from the transient storage, conduction, convection and radiation. Methods are designed to support this decomposition for a more flexible architecture which facilitates using different solution algorithms and time integration rules. HeatTransferElement is an abstract class and its instantiation relies on the subclasses. Currently four-noded and eight-noded quadrilateral elements are implemented to solve two dimensional heat conduction problems. Each element is associated with several HeatTransferMaterial objects depending on the number of quadrature points used. A HeatTransferMaterial object holds values of conductivity and specific heat at a given temperature and allows access to those quantities. Subclass LinearIsotropic is designed to define any arbitrary temperature-independent material information. Subclass CarbonSteelEN93 is implemented for steel by using material information given in Eurocode 3. Other material models can also be introduced easily by subclassing HeatTransferMaterial. With this design, elements can be assigned different materials and heat transfer in structures with multiple material layers can be modelled.

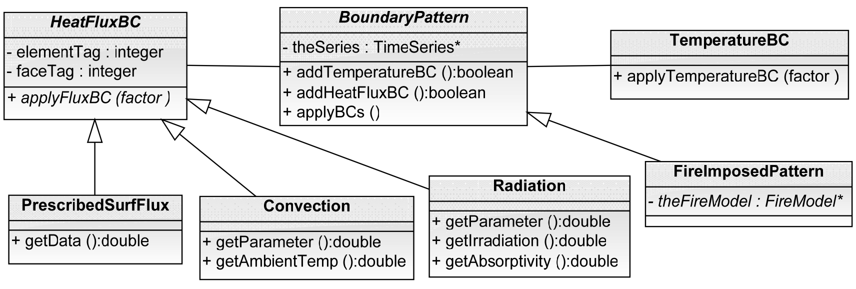


Fig. 2 Diagram representing classes for specifying boundary conditions

A TemperatureBC object specifies the existence of Dirichlet boundary condition at solid boundaries and a HeatFluxBC object specifies Neumman boundary conditions. Some of the attributes and public operations for these two classes are shown in Fig. 2. A HeatTransferElement object keeps a record of HeatFluxBC objects acting on its faces and enables imposition of multiple flux boundary conditions on any of its faces. Objects of TemperatureBC and HeatFluxBC can only deal with constant boundary conditions. A BoundaryPattern class, which has a TimeSeries object as its attribute, is introduced to define arbitrary time-varying boundary conditions. Object of this class can be associated with a number of HeatFluxBC and TemperatureBC objects by invoking addTemperatureBC() and addHeatFluxBC() methods. Subclass FireImposedPattern, which has a FireModel object as its attribute, will be implemented to specify temporal heat flux boundary conditions imposed by a specific type of fire. As indicated by equation (2), there are generally three types of boundary heat fluxes:

prescribed heat flux, convective heat flux and radiative heat flux; thus three subclasses of HeafFluxBC are provided, whose public operations returning relevant quantities are also given in Fig. 2.

Fig. 3 illustrates a general picture of relationships between classes relevant to heat transfer analysis procedures, where HT_TransientAnalysis is a subclass inheriting from HeatTransferAnalysis class and an instance of this class is associated with several instances of other classes to perform a transient analysis. The architecture of this analysis system is fundamentally the same as that of OpenSees. A HT_DOF_Number object employing algorithms based on graph theory numbers all the degrees-of-freedom in the domain with the aim of reducing the bandwidth of the tangent matrix. A HT_DOF_Group object keeps a reference to a HeatTransferNode object in the domain and deals with the mapping between degrees-of-freedom and global equation numbers. Similarly, a HT_FE_Element is linked to a HeatTransferElement in the domain and it provides methods to set and retrieve equation numbers for degrees-of-freedom relevant to that element. It also provides methods to return elemental residual vector and tangent stiffness matrix. A HT_AnalysisModel holds all the HT_DOF_Groups and HT_FE_Elements in an analysis and also provides methods to update the state of the domain. HeatTransferIntegrator class provides methods to assemble the system of equations by adding elemental contributions. Subclasses of HT_TransientIngetrator provide methods to set the solution predictor and updater for a transient problem. BackwardDifference implements the time-integration algorithm shown in equation (1) with α being set one, which removes the need of computing initial temperature derivatives. TemperatureBCHandler is an abstract class, which provides interfaces for imposing the Dirichlet boundary condition on the global system of equations. Subclasses provide the implementation details, such as using penalty method. HT_SolutionAlgorithm implements the algorithms such as Newton's method to solve a system of equations.

By instantiating some of the heat transfer classes and OpenSees' solver classes, one can produce stand-alone executable applications to solve any heat conduction problem. In addition, it is also possible to integrate these heat transfer classes into the main application of OpenSees, which allows users to run heat transfer analyses with scripting languages such as Tcl.

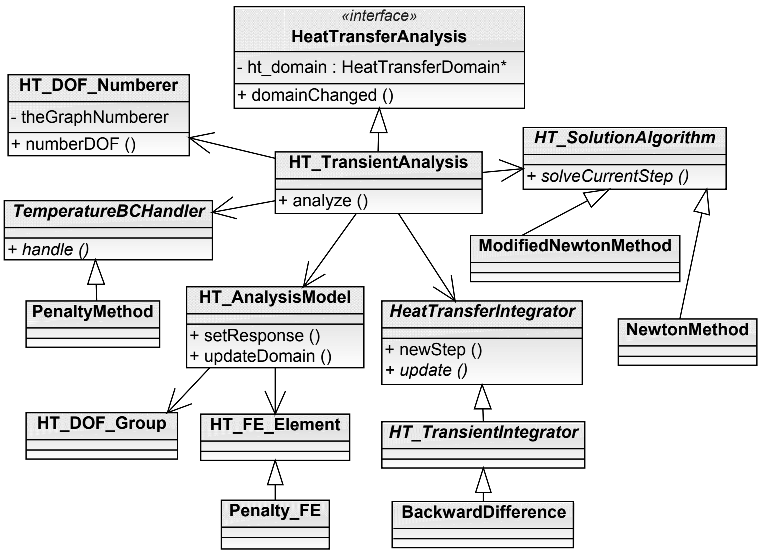


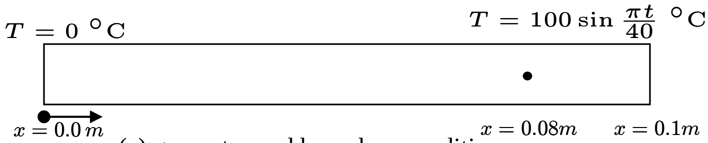
Fig. 3 Class diagram representing heat transfer analysis algorithms

3 VERIFICATION

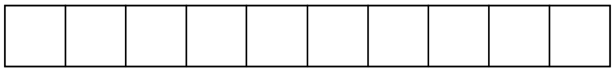
The heat transfer module discussed in the foregoing section is verified through three benchmark tests. The comparison between FE solutions and analytical solutions are given below.

The first example shown in Fig. 4 can be used to examine the general performance of solution procedures of a finite element program and its transient modelling capability in particular (Huang & Usmani, 1994). The physical problem is a two dimensional bar with fixed temperature on its left end and with sinusoidal temperature changes at its right-end. Steel material is assumed here, where mass density $\rho = 7200$ kg/m^{-3}, specific heat $c_p = 440.5$ J/kgK, heat conductivity $k = 35.0$ W/mK.

Both four-noded and eight-noded quadrilateral elements together with backward difference time-integration scheme are used in this test. A uniform mesh with ten elements is shown in Fig. 4(b).



(a) geometry and boundary conditions



(b) FE discretised mesh

Fig. 4 Benchmark test with transient temperature boundary condition

Fig. 5 presents the temporal solution at a target point (x = 0.08m). It can be seen that both the linear and quadratic quadrilateral elements produce results rather close to analytical solutions while the quadratic ones show larger deviation. However, as shown in Fig. 6, eight-noded quadratic elements give better prediction of the spatial temperature distribution especially in the vicinity of the right-end, where the gradient-reversal behaviour is totally missed by using linear elements. One may also notice that 8-noded elements still give a slight error at x = 0.095m, which was demonstrated to be produced by backward-difference time integration scheme and may be improved by using Crank-Nicolson method due to its higher accuracy.

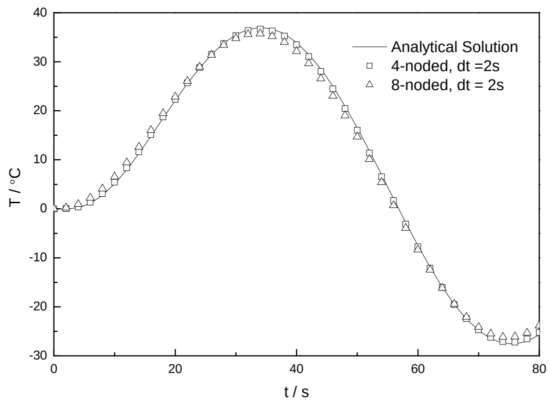


Fig. 5 Temporal solution at target point

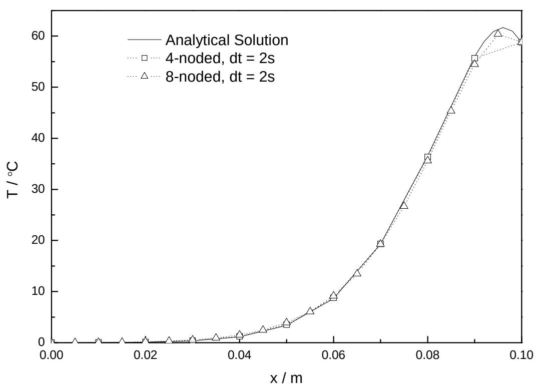
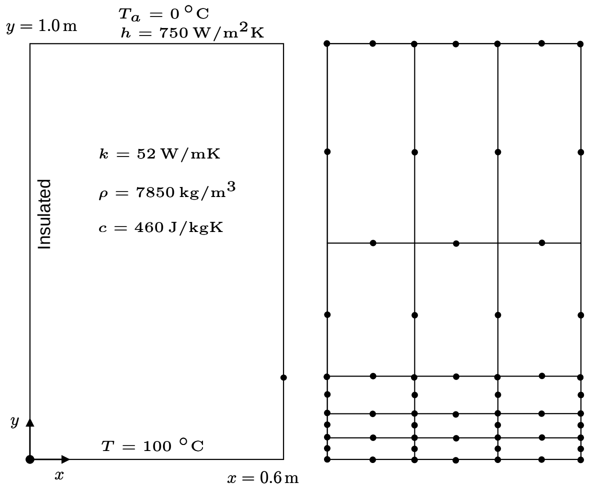


Fig. 6 Spatial distribution along length

The second test problem is shown in Fig. 7, where a two dimensional plate has a fixed temperature boundary condition at its bottom and is insulated along its left-side boundary. The top and right are subjected to the convective boundary condition with ambient temperature $T_a = 0$ °C and convective heat transfer coefficient $h = 750$ W/m^{2}K. Other material properties are listed in the figure.



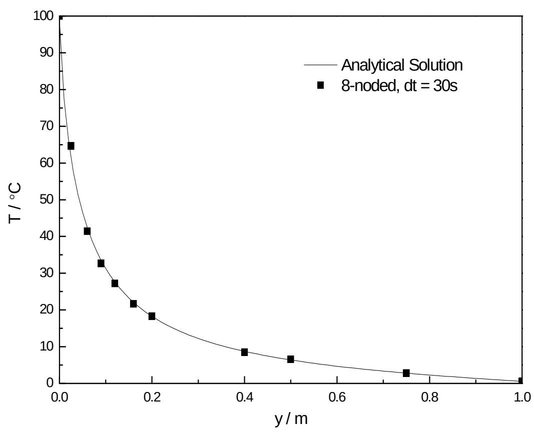


Fig. 7 Benchmark test with convection boundary condition Fig. 8 Temperature distribution along right-side boundary

The steady state analytical solution suggests a strongly nonlinear temperature distribution along the right-bottom boundary. Therefore, a graded mesh shown in Fig. 7 with eight-noded elements is used. The steady state solution given Fig. 8 is obtained by using backward-difference time-integration scheme with a step of 30s. It can be seen that the FE solution agrees with analytical solution very well and the large temperature gradient along the right-side boundary is accurately captured. A local examination at the target point (0.6, 0.2) also gives a good result with 18.28 °C which closely approaches the analytical solution of 18.3 °C.

The purpose of the last test is to examine the program's capability of handling radiation boundary conditions which introduces a nonlinear source into the system of equations. The problem itself shown in Fig. 9 is rather simple, which is again a bar with fixed temperature at its left-end and with its right-end radiating to ambient, where surface emissivity $\varepsilon = 0.98$, Stefan-Boltzmann constant $\sigma = 5.67\times10^{-8}$ $Wm^{-2}K^{-4}$. In the current test, Newton-Raphson algorithm is used and a converged solution of 927.008 K at the right-end is obtained with nil error.

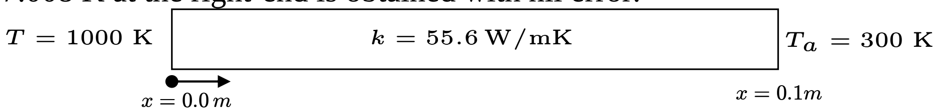


Fig. 9 Benchmark test with radiation boundary condition

3 SUMMARY AND ACKNOWLEDGMENT

A flexible and extensible heat transfer module dedicated to structures-in-fire modelling is developed for OpenSees. A series of benchmark tests are carried out by comparing the modelling results with analytical solutions. It is shown that results from the two agree very well and the module behaves desirably as expected. Future work would be directed to the development of the fire module and the implementation of fire-thermal-structural coupling techniques.

The first author gratefully appreciates the China Scholarship Council and The University of Edinburgh for providing a joint scholarship on his PhD project. The authors also would like to thank Dr Frank McKenna at UC Berkeley for his assistance during the development process.

REFERENCES

Cross J., Masters I., Lewis R., Why You Should Consider Object-oriented Programming Techniques for Finite Element Methods, International Journal of Numerical Methods for Heat and Fluid Flow, 9(3), 333–347, West Yorkshire, England: MCB University Press, 1999.

Forde B., Foschi R., Stiemer S., Object-Oriented Finite Element Analysis, Computers & Structures, 34(3), 355-374, 1990.

Huang H. C., Usmani A. S., Finite Element Analysis for Heat Transfer: Theory and Software, London: Springer-Verlag, 1994.

McKenna F. T., Object-Oriented Finite Element Programming: Frameworks for Analysis, Algorithms and Parallel Computing, PhD Dissertation of University of California, Berkeley, 1997.

McKenna F., Nonlinear Finite-Element Analysis Software Architecture Using Object Composition, Journal of Computing in Civil Engineering, 24(1), 95-107, 2010.

Rumbaugh J., Jacobson I., Booch G., Unified Modeling Language Reference Manual, Reading, MA: Addison-Wesley, 1999.

Rumbaugh J., Blaha M., Premerlani W., Eddy F., Lorensen W., Object Oriented Modeling and Design, West Nyack, NY: Prentice Hall, Book Distribution Center, 1991.

Usmani A., Zhang J., Jiang J., Jiang Y., Kotsovinos P., Zhang J., and May I., Using OpenSees for structures in Fire, 6th International Conference on Structures in Fire, 919-926, Michigan, USA, 2010.

Winget J., Hughes T., Solution Algorithms for Nonlinear Transient Heat Conduction Analysis Employing Element-by-element Iterative Strategies, Computer Methods in Applied Mechanics and Engineering, 52(1-3), 711-815, 1985.

Welch S., Miles S., Kumar S., Lemaire T., Chan A., FIRESTRUC – Integrating Advanced Three-dimensional Modelling Methodologies for Predicting Thermo-mechanical Behaviour of Steel and Composite Structures Subjected to Natural Fires, 9th International IAFSS Symposium, Vol. 22, 1315-1326, Karlsruhe, Germany, 2009.

Wang Y., Steel and Composite Structures: Behaviour and Design for Fire Safety, London and New York: Spon Press, Taylor & Francis, 2002.

Zimmermann T., Dubois-Pèlerin Y., Bomme P., Object-oriented Finite Element Programming: I. Governing principles, Computer Methods in Applied Mechanics and Engineering, 98, 291-303, 1992.

A NUMERICALLY DERIVED MODIFIED CONDUCTIVITY MODEL FOR SOFTWOOD EXPOSED TO PARAMETRIC DESIGN FIRES

Background, Benchmarking and Adaptation for Cooling

Danny Hopkin [a/c], Jamal El-Rimawi [a], Vadim Silberschmidt [b], Tom Lennon [c]

[a] Loughborough University, Department of Civil and Building Engineering, Loughborough, UK
[b] Loughborough University, Wolfson School of Mechanical Engineering, Loughborough, UK
[c] BRE Global, Garston, Watford, Herts, WD25 9XX, UK.

INTRODUCTION

Recommendations for designing timber structures, as well as other structures made of concrete or steel, under fire conditions are given in the relevant parts of the fire Eurocodes. However, unlike other parts, the scope for using the part relevant to the fire design of timber structures (Eurocode 5 part 1.2) is quite limited. This is because, with the exception of a single annex, it is only applicable to structures subjected to standard fire exposure. The only exception to this is Annex A which gives guidance on the charring rates of initially un-protected timber members in parametric fires. However, in the UK the use of this Annex is prohibited as specified in its national annex to the Eurocode.

1 BACKGROUND TO THE MODIFIED CONDUCTIVITY MODEL (MCM)

The complex phenomena present in heated timber elements are difficult to model explicitly and, hence, to date, 'effective properties' are often defined. Such properties implicitly account for the effects of complex behaviour, such as the flow of pyrolosis gases and water vapour, through calibration against known temperatures in limited experimental configurations. Konig and Walleij (2000) have been instrumental in initiating such a process for timber. They calibrated 'effective' thermal properties for standard fire exposure conditions. These properties form the basis of the advanced calculation models contained in Annex B of EN 1995-1-2. However, additional studies by Konig (2006) proved (both experimentally and numerically) that those properties exhibited very conservative predictions of char depth when applied to non-standard (parametric) fire conditions with heating rates in excess of those given by the standard-fire curve. Similarly, the properties from the code were shown to result in non-conservative predictions of timber temperature and depth of char for heating rates lower than that of the standard-fire curve. As a result, EN 1995-1-2 explicitly states that the thermal properties present in Annex B should only be adopted for standard-fire exposure and not for parametric fire exposure.

Konig (2006) previously proposed that consistency between parametric charring measurements in experiments and computational predictions, under standard-fire exposure, could be achieved via subtle modifications to the conductivity-temperature relationships proposed in Annex B of EN 1995-1-2. In addition, he noted that only those properties in excess of 350°C should be modified as they represent the 'effective' properties of the char layer. Phenomena in the char area, such as 'reverse cooling pyrolosis flows', cracking and ablation, appear to be influenced by heating rate. Although Konig (2006) made the observation that the thermal properties present in Annex B of EC5-1-2 were not appropriate for parametric fire applications and that better agreement could be seen through adaptation of the char-layer conductivity, no follow-on research has been conducted to quantify the necessary modification of the char-layer conductivity.

In an earlier publication the authors proposed a modified conductivity model (MCM) for softwood timber (Hopkin *et al*. 2010) based upon the principles outlined in Konig's research and upon EN 1995-1-2 specific heat modifications proposed by Cachim and Franssen (2009). The MCM was derived using numerical calibrations of a fire load- (q_{td}) and heating rate- (Γ) dependent modification factor and the depths of char present in parametric design fires. In the latter case the depth of char in such fires was determined using the Annex A approach of EN 1995-1-2.

The full derivation of the proposed model can be found elsewhere (Hopkin et al. 2010). However, the resulting relations are shown in Tab 1. and Eqn 1.

Tab. 1(a) Summary of MCM

Temperature (°C)	Conductivity (W/m K)
20	0.12
200	0.15
350	0.07
500	$0.09k_{\lambda,\mathrm{mod}}$
800	$0.35k_{\lambda,\mathrm{mod}}$
1200	$1.50k_{\lambda,\mathrm{mod}}$

Tab. 1(b) Specific heat after Cachim & Franssen (2009)

Temperature (°C)	Density ratio G	Cachim and Franssen moisture modified specific heat (J/kg K)
20	1+ω	(1210+4190ω)/G
99	1+ω	(1480+4190ω)/G
99	1+ω	(1480+114600ω)/G
120	1.00	(2120+95500 ω)/G
120	1.00	2120/G
200	1.00	2000/G

$$k_{\lambda,\mathrm{mod}} = k_{\Gamma,\mathrm{mod}} k_{qtd,\mathrm{mod}} \tag{1}$$

with $$k_{\Gamma,\mathrm{mod}} = 1.5\Gamma^{-0.48} \quad , \; k_{qtd,\mathrm{mod}} = \sqrt{\frac{q_{td}}{210}} \;\; and \;\; \Gamma = \frac{(O/b)^2}{(0.04/1160)^2} ,$$

where ω is the moisture content of timber (%), O is an opening factor ($m^{0.5}$) and b is compartment thermal inertia ($J/m^2s^{0.5}K$).

The above, when coupled with specific heat properties and appropriate densities, were found to give rather consistent transient depth-of-char predictions for the heating phase of a parametric fire, when compared to EN 1995-1-2 Annex A. However, from a structural-engineering view point, the definition of the depth of char in Finite Element analysis (FEA) simulations is sufficient to fully characterise the mechanical response of a member exposed to high temperatures. In timber only those temperatures below 300°C are of concern. Above this threshold the timber is charred and friable. As a result, the MCM must not only be able to place the char line correctly within a cross section, but it must also accurately simulate temperature in the intact member. This allows the sectional response to be determined using strength, stiffness and temperature relations. Given the limited number, and limitations, of experiments conducted on timber exposed to parametric fires, the authors were only able to investigate temperature development using the test data developed by Konig and Walleij (1999). The modelling conducted and the comparisons made are discussed in the section that follows.

2 BENCHMARKING AGAINST KONIG TEST DATA

At the turn of the century Konig and Walleij (1999) reported 6 experiments on timber blocks exposed to parametric fires. The experiments, as best as possible, exposed timber panels to one-dimensional heat transfer via a gas-powered furnace following parametric curves. From this it was first observed by Konig (2006) that the thermal properties present in Annex B of EN 1995-1-2 were inappropriate for use with non-standard fire exposure. The timber used in the experiments was a generic softwood with an estimated moisture content of 12% and a mean density of 420-430 kg/m^3. Although Konig and Walleij (1999) attempted to follow parametric curves, this was not entirely possible due to the furnace configuration. As a result, the authors fitted EN 1991-1-2 parametric curves to the measured gas temperature-time relationships via trial and error. The resulting key parameters are noted in Tab. 2. From observation of the test data it is apparent that experiments C1 to C3 follow the standard fire curve and then cool, whilst C4 to C6 follow a different accelerated heating regime. Given this the authors decided to attempt to simulate experiments C4 to C6 as they represent an obvious deviation from the standard fire curve. In addition, as the authors' MCM was developed for the heating phase of parametric fires, only the heating phase of Konig and Walleij's (1999) experiments is considered at present.

Tab. 2 Fitted parametric curve parameters

	C4	C5	C6
q_{td} (MJ/m^2)	93.8	109.4	114.6
Γ (-)	2.7	3.0	4.5

Using these parameters it is possible to determine the appropriate values of $k_{\lambda,mod}$ for each test thus yielding modified conductivity properties.

As the moisture content was estimated as 12% by Konig & Walleij (1999) then the specific heat relationship from EN 1995-1-2 can be adopted without modification. Using the gas temperature measured in each experiment as a boundary condition, coupled with boundary coefficients of ε = 0.7 and α = 35 W/m^2 K, the one-dimensional heat flow was simulated using TNO DIANA. In all instances first-order quad elements with dimensions of 0.5 mm were adopted. Temperatures at depths of 0, 6, 18, 30, 42 and 54 mm, denoted 1, 2, 3, 4, 5 and 6, respectively, were measured by Konig & Walleij (1999). Therefore, temperatures for corresponding nodes are used as output in DIANA (Manie 2010). In addition, simulations were conducted with unmodified conductivity properties as per EN 1995-1-2 for comparison. Plots of the resulting temperature development are shown in Figure 1(a-c).

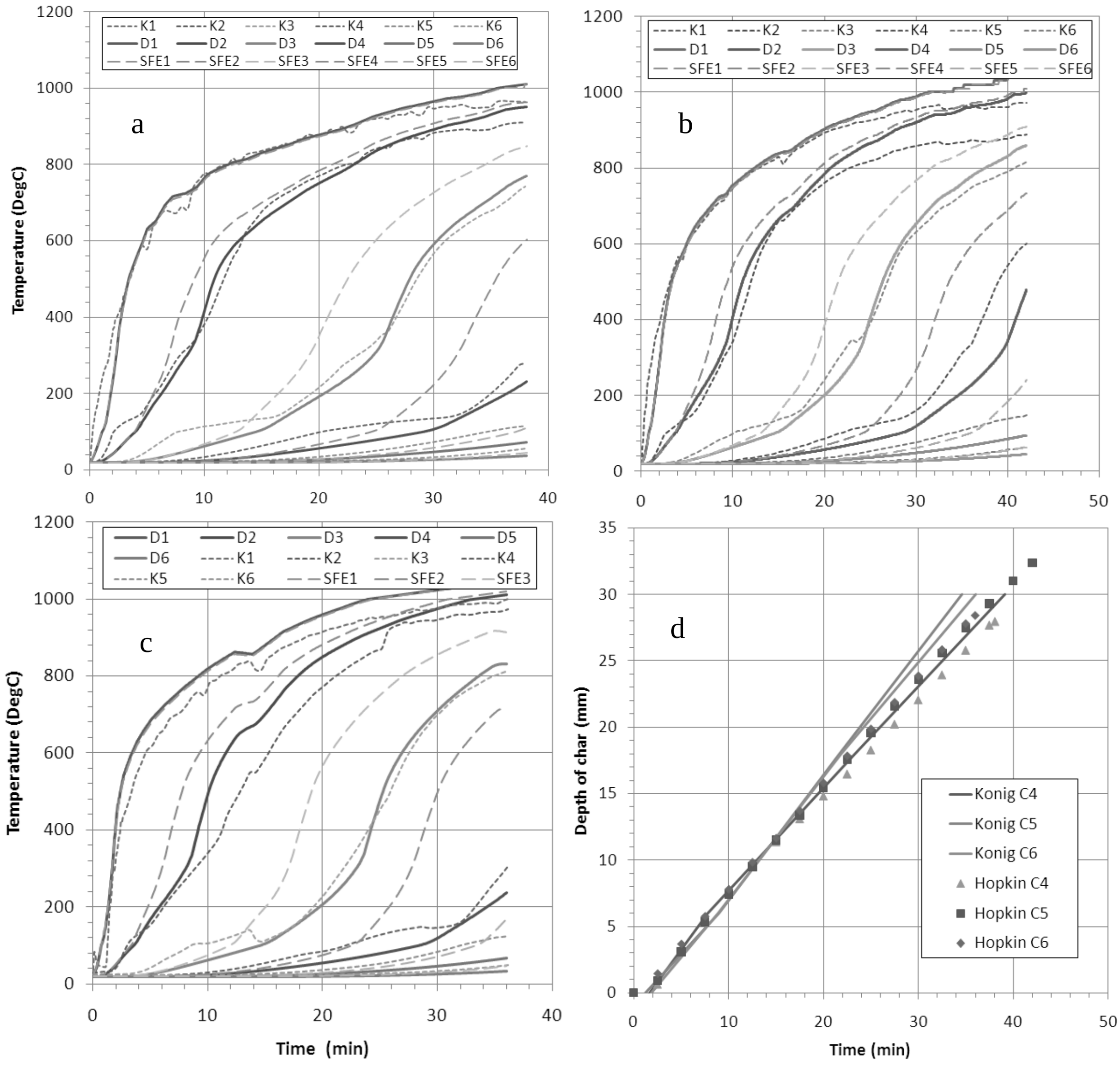


Fig. 1 (a-c) Temperature development, comparison of MCM (D), Konig (K) and EN 1995-1-2 (SFE) simulations/experiments; (d) Transient depth of char- simulation (Hopkin) versus experiment (Konig)

It is apparent from the limited temperature validation conducted that the MCM proposed by the authors for softwood results in a vastly improved prediction of temperature development in timber members exposed to the heating phase of parametric fires (compared to EN 1995-1-2 Annex B properties). However, the experiments of Konig and Walleij (1999) are not well defined, and, as a result, stronger conclusions cannot be drawn without further benchmarking against more robust experiments.

3 EXTENSIONS TO COOLING TIMBER

The modified conductivity model (Hopkin *et al*. 2010) was derived by numerically calibrating timber char conductivity to heating rate by positioning the 300°C isotherm (or char line) so that the method yielded the same charring depth as set out in Annex A of EN 1995-1-2. This calibration was conducted so that the depth of char after a period of t0 minutes was consistent when calculated using both FEA simulations and Annex A. The period t0 is defined as the 'constant charring phase'. It describes a linear relationship between depth of char and time. During this period char of a thickness βt0 develops. However, after this period and during cooling, according to EN 1995-1-2, a further char layer with thickness βt0 develops, giving a total depth of char of 2βt0. The term β is the parametric charring rate in mm/min. Given that the MCM was developed for the heating phase of parametric fires (i.e. up to t0), its applicability in the cooling phase of fire development is uncertain. To verify its applicability further benchmarking was conducted against Annex A of EN 1995-1-2 by conducting simulations with the proposed conductivity changes and a fully defined parametric fire (inclusive of cooling). An example finding is shown in Figure 2.

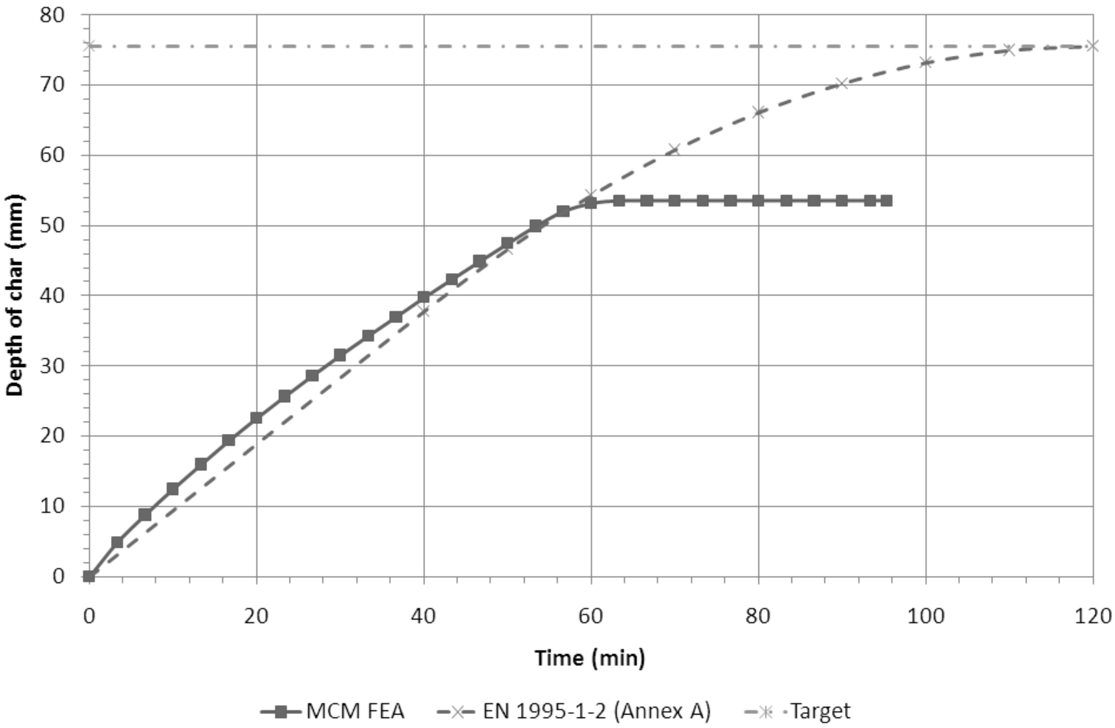


Fig. 2 Comparison of FEA, MCM and EN 1995-1-2 calculation of depth of char

Clearly, even with the proposed modifications to conductivity, char formation in the cooling phase of parametric fires is still not satisfactorily addressed. This could be attributed to a number of reasons, a most critical one is that the pyrolysis and associated energy release of the timber surface is not considered in the design fire characterisation. Konig (2006) proposed that either an artificial fire load, to account for timber member ignition, is incorporated in the cooling phase of a parametric fire, or different timber thermal properties are adopted for the cooling and heating phases of fire development. However, the authors are proposing a more pragmatic solution, which would allow for the performance-based design of timber structures using FEA. This proposal is discussed in the following section.

3.1 An engineered approach to design for cooling

Since charring is a dominant phenomenon, and that transient effects and thermal expansion of timber appear to have little bearing on behaviour in fires, it becomes less important to accurately simulate temperature and char development as a function of time.
By definition, performance-based design is a process whereby a structure is designed to survive the entire duration of a fire, and, in crude terms, the resulting building has infinite fire resistance. It follows that to design a timber member for such an event, it is only necessary to determine the maximum depth of char (at the end of cooling) and the maximum temperature apparent in any undamaged residual timber. This process is semi-independent of time.
Further numerical calibrations performed by the authors showed that, via a slight modification to the fire load-dependent term ($k_{qtd,\mathrm{mod}}$) in the MCM, the total depth of char can be determined accurately using FEA simulation. The calculated char depth is inclusive of the additional char that develops during cooling. The modified term is given by:

$$k_{qtd,\mathrm{mod}} = \sqrt{\frac{4.q_{td}}{210}}\,. \qquad (2)$$

This simple modification yields the following relationships between depth of char and time for different parametric fire exposures, see Figure 3. In all instances q_{td}=210 MJ/m^2:

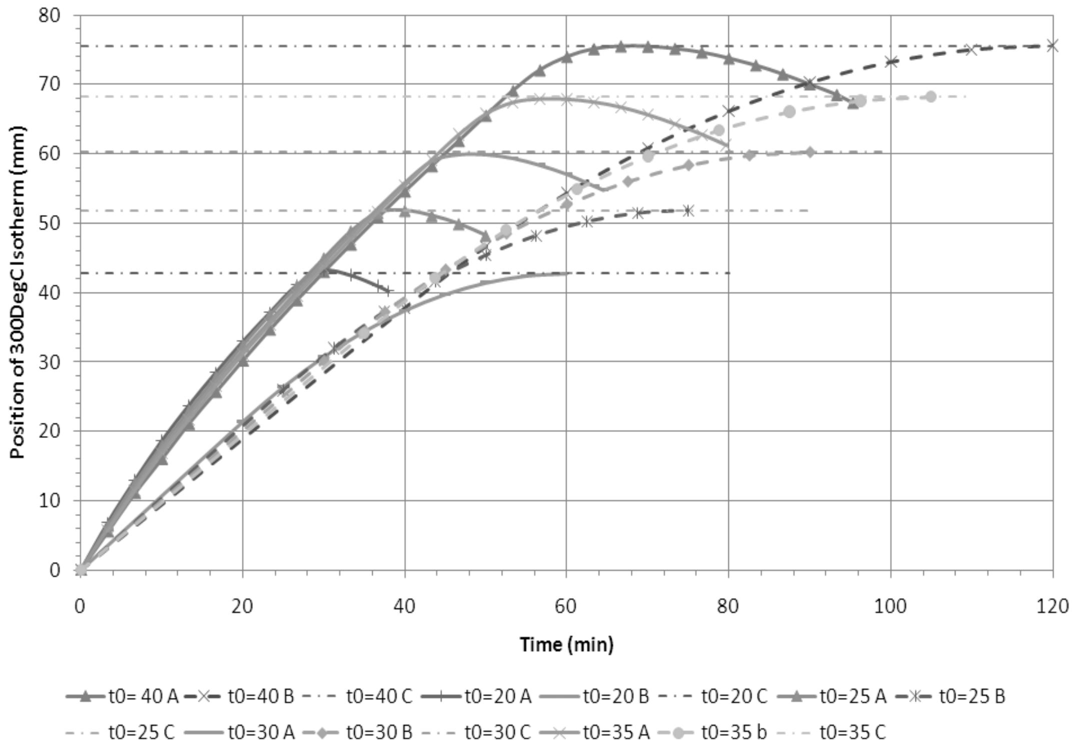


Fig. 3 Position of 300°C isotherm using modified $k_{qtd,mod}$ (A) and EN 1995-1-2 Annex A (B). Target depth of char shown as (C).

Figure 3 shows that in all instances the maximum depth of char determined via simulation is consistent with the Eurocode approach. As a result, although in transient terms the depth of char is inconsistent, the residual cross-section determined in both cases at the end of the fire is identical. From a scientific view point the method proposed does not accurately simulate the physical complexities that occur in timber on cooling. However, this is also the case for the many empirical methods contained in EN 1995-1-2. To gauge the applicability of such an approach, determining the charring depths alone is not sufficient. It must also be shown that ultimate temperature development in uncharred timber is compatible with that apparent in reality. To verify this, further benchmarking will be conducted against the test data of Konig and Walleij (1999).

4 SUMMARY

A modified conductivity model for timber, derived on the basis of numerical calibrations between parametric depth of char and char layer conductivity, has been introduced. The full derivation of the model is outlined elsewhere (Hopkin *et al*. 2010). It was found that with such a modification the depth of char (or position of the 300°C isotherm) can be located with relative accuracy during the heating phase of a parametric fire. In addition, through benchmarking against experimental data provided by SP Tratek, it has been found that the proposed conductivity modifications also result in vastly improved predictions of temperature development in timber members exposed to non-standard fires.

Further benchmarking of depth of char predictions, using the modified properties, and the empirical Annex A charring method indicate that the proposed adaptations still do not fully adequately simulate char formation and temperature development in the cooling phase of a parametric fire. This is likely to be due to char oxidation, which results in additional 'fire load', thus increasing the temperature of a timber member beyond that of the cooling surrounding gas temperature. Such a conclusion was supported by the findings of Konig and Walleij (1999).

The simulation of temperature development in cooling timber members is a complex and difficult task. Konig (2006) suggested that different thermal properties should be adopted in the heating and cooling phases of fire development. From a design perspective this is an awkward approach. As a result, the authors proposed a pragmatic engineered solution, which should, theoretically, but subject to further verification, allow for the design of timber buildings exposed to parametric fires, using computational techniques. This approach will be further investigated, and additional benchmarking will be conducted against any available test data. In general, the developments presented herein are believed to be a meaningful step towards the whole-building analysis of timber structures exposed to non-standard fires. If such an approach can be realised then many of the beneficial aspects of holistic behaviour associated with steel, concrete and composite construction may be capitalised upon to design more efficient large-section timber structures.

5 ACKNOWLEDGMENTS

The authors would like to acknowledge EPSRC, BRE Global and Jürgen König of SP Tratek for their support.

REFERENCES

BRITISH STANDARDS INSTITUTION, 2004. Eurocode 5: Design of timber structures – Part 1-2: General – Structural fire design. BS EN 1995-1-2. London: BSI.

CACHIM, P.B. and FRANSSEN, J.M., 2009. Comparison between the charring rate model and the conductive model of Eurocode 5. Fire and Materials, **33**(3), 129-143.

HOPKIN, D.J., EL-RIMAWI, J, SILBERSCHMIDT, V., and LENNON, T., 2010. An effective thermal property framework for softwood in parametric design fires: Comparison of the Eurocode 5 parametric charring approach and advanced calculation models. Journal of Construction and Building Materials, doi: 10.1016/j.conbuildmat.2010.12.002.

KONIG, J., 2006. Effective thermal actions and thermal properties of timber members in natural fires. Fire and materials, **30**(2), 51-63.

KONIG, J. and WALLEIJ, L., 2000. Timber frame assemblies exposed to standard and parametric fires part 2: A design model for standard fire exposure. I0001001. Stockholm: SP Tratek.

KONIG, J. and WALLEIJ, L., 1999. One-dimensional charring of timber exposed to standard and parametric fires in initially unprotected and postprotection situations. I9908029. Stockholm: SP Tratek.

MANIE, J., 2010. DIANA User's Manual. 9.4 edn. Delft: TNO.

TRAVELLING FIRES IN LARGE COMPARMENTS
Realistic fire dynamics for structural design

Guillemro Rein [a], Jamie Stern-Gottfried [a,b]

[a] University of Edinburgh, School of Engineering, UK
[b] Arup, London, UK

INTRODUCTION

Close inspection of real fires in large, open compartments reveals that they do not burn simultaneously throughout the whole compartment. Instead, these fires tend to move as flames spread, partitions or false ceilings break, and ventilation changes through glazing failure. These fires have been labelled ‘travelling fires.’

Despite these observations, fire scenarios currently used for the structural fire design of modern buildings are based on one of two traditional methods for specifying the fire environment; the standard temperature-time curve, which has its origins in the late 19th century, or parametric temperature-time curves, such as that specified in Eurocode 1 (EN 1991-1-2, 2002). These methods are based on the extrapolation of existing fire test data, which stems from tests performed in small compartments that are almost cubic in nature. This test geometry allows for good mixing of the fire gases and thus for a uniform temperature distribution throughout the compartment.

While both of these methods have great merits and represented breakthroughs in the discipline at their times of adoption, they have inherent limitations with regards to their range of applicability (Rein et al., 2007; Stern-Gottfried et al., 2009). For example, Eurocode 1 states that the design equations are only valid for compartments with floor areas up to 500 m^2 and heights up to 4 m, the enclosure must have no openings through the ceiling, and the compartment linings are also restricted to a thermal inertia between 1000 and 2200 $J/m^2s^{½}K$, which means that highly conductive linings such as glass facades and highly insulating materials cannot be taken into account. As a result, common features in modern construction like large enclosures, high ceilings, atria, large open spaces, multiple floors connected by voids, and glass façades are excluded from the range of applicability of the current methodologies. These limitations, which are largely associated with the physical size and geometric features of the experimental compartments on which the methods are based, ought to be carefully considered when the methods are applied to an engineering design. This is particularly relevant given the large floor plates and complicated architecture of modern buildings.

A recent simple survey of buildings at the University of Edinburgh campus (Jonsdottir and Rein, 2009) underlines the narrow design fire specifications in the Eurocodes. For buildings built over a long period of time starting in the early 20th century, 66% of their total volume falls within the limitations. However, in a newly constructed, modern building that has open spaces and glass facades, only 8% of the total volume is within the limitations. This suggests that modern building trends are moving out of the limits of current design practices.

The limitations of the existing methods arise from the assumption of uniform temperature conditions throughout the whole floor of a compartment. A fire that would cause these uniform conditions burns uniformly within the enclosure and generates high temperatures for a relatively short duration. This is opposed to a travelling fire that burns locally but spreads through the enclosure with time, generating lower temperatures for longer times. Post-flashover fires in open plan offices are unlikely to burn throughout the whole space at once. Real, large fires that have lead to structural failure, such as those in the World Trade Center towers 1, 2 and 7 in September 2001, the Windsor Tower in Madrid, Spain in February 2005 and the Faculty of Architecture building at TU Delft in the Netherlands in May 2008 were all observed to travel across floor plates, and vertically between floors, rather than burn uniformly for their duration.

1 NEW METHODOLOGY FOR TRAVELLING FIRES

While the traditional methods have inherent assumptions of fire behaviour different from that observed in real fires, they have generally been deemed to be conservative, and therefore appropriate, tools for structural fire design in the absence of better and more relevant data. Although these methods might be considered acceptable for most design cases, the need for better optimisation of structural behaviour in fire will eventually require a more performance-based definition of the fire. This is particularly relevant given that computational methods for determining structural behaviour have matured over the last decade and have enabled analysis of more complex structural systems. This has led to an understanding that many modern structures do not behave in the same manner as simpler, more traditional frame based systems. Thus, in order to address these differences, and continue to enable innovation in structural design, a more sophisticated characterisation of fire scenarios is required.

Therefore a methodology is being developed that allows for a wide range of possible fires, including both uniform burning and travelling fires, by considering the fire dynamics within a given building (Rein et al., 2007; Stern-Gottfried et al., 2009, Angus et al. 2011). This methodology also facilitates the collaboration between fire safety engineers and structural fire engineers, which is an identified need within the structural fire community, to jointly determine the most challenging fire scenarios for a structure.

The key aspect of the new methodology is to characterise the thermal environment for structural analysis accounting for the fire dynamics specific to the building, including a wide range of possible fires. In order to achieve this, a simple fire model is selected that enables capturing the spatial and temporal changes of the fire-induced thermal field. This model is then applied to a particular floor of the building accounting for a family of fires that range between one that burns in a small area and travels across the floor plate for a long duration and one that is well distributed across the whole floor plate but burns for a short duration.

2 TEMPERATURE FIELD

The methodology divides the effect of a fire on structural elements into the near field and the far field. The near field is when a structural element is exposed directly to the flames of the fire and the far field is when it is exposed to the hot gases, i.e. the smoke layer away from the flames, as shown in Figure 1. This division of the thermal field allows the methodology to avoid the well-known inaccuracies of flame temperature prediction of most fire models.

In previous work, a CFD fire model (Rein et al., 2007) was selected to study the temperature field as a function of distance from the fire. In this paper, a ceiling jet correlation was used. The ceiling jet correlation developed by Alpert (1972) and given below in Eq. (1) was selected in this case as the simple fire model to study the temperature field as a function of distance from the fire. The use of such a correlation is deemed appropriate if the floor area is large and the smoke layer is thin relative to the floor to ceiling height.

$$T_{max} - T_{\infty} = \frac{5.38\left(\dot{Q}/r\right)^{2/3}}{H} \qquad (1)$$

where T_{max} is the maximum ceiling jet temperature (K)
T_{∞} is the ambient temperature (K)
$\dot{Q}$ is the total heat release rate (kW)
r is the distance from the centre of the fire (m)
H is the floor to ceiling height (m)

Note that while Alpert gives a piecewise equation for maximum ceiling jet temperatures to describe the near field ($r/H \leq 0.18$) and far field ($r/H > 0.18$) temperatures, only the far field equation is utilised in the present case study as the near field temperature is assumed to be the flame temperature.

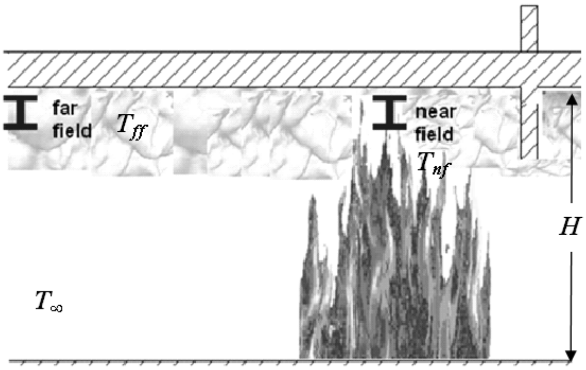


Fig. 1 Illustration of near and far fields

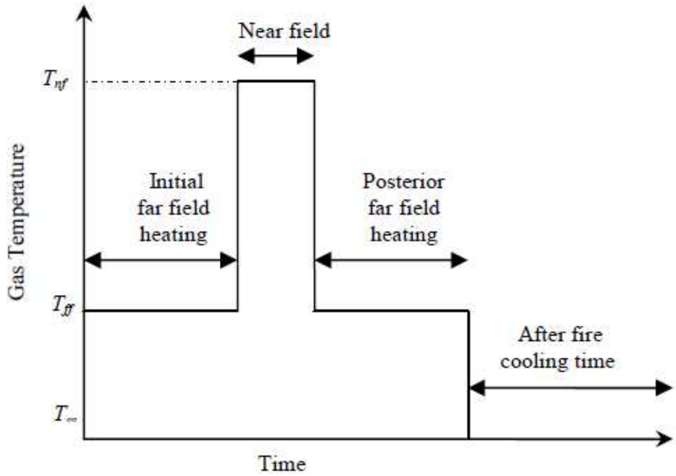


Fig 2 Temperature-time curve at one ceiling location for a travelling fire

The ceiling jet correlation characterises the spatial variation of the temperature field only as a function of the distance from the fire. This simple correlation was chosen to provide a straightforward description of the temperature field that is qualitatively sufficient to progress the development of the methodology. Alternative fire models, including computational fluid dynamics, can be utilised instead.

3 FAMILY OF FIRE SIZES

A family, or set, of fires that covers the range of possible fires, both travelling and uniformly burning, needs to be selected as an input for the temperature field. To do this it was assumed that each fire in the family would burn over a surface, A_b, which is a percentage of the total floor area, A_t, of the building, ranging from 1% to 100%. The burning area of the fire that is equal to 100% of the floor area is a well distributed fire. All other burning areas represent travelling fires of different sizes.

It is assumed that there is a uniform fuel load across the fire path and the fire will burn at a constant heat release per unit area typical of the building load under study. Thus the burning time can be calculated by Eq. (2).

$$t_b = \frac{q_f}{\dot{Q}''} \tag{2}$$

where t_b is the burning time (s)

q_f is the fuel load density (MJ/m^2)

$\dot{Q}''$ is the heat release rate per unit area (MW/m^2)

For the case study presented below, the fuel load density, q_f, is assumed to be 570 MJ/m^2, as per the 80th percentile design value for office buildings. The heat release rate per unit area, $\dot{Q}''$, is taken as 500 kW/m^2 which is deemed to be a typical value for densely furnished spaces, as typical design guidance gives this value for retail spaces. Based on these two values, the characteristic burning time, t_b, is calculated by Eq. (2) to be 19 min.

Note that the burning time is independent of the burning area. Thus the 100% burning area and the 1% burning area will both consume all of the fuel over the specified area in the same time, t_b. However, a travelling fire moves from one burning area to the next so that the total burning duration, t_{total}, across the floor plate is extended. This time is given in Eq. (3).

$$t_{total} = \frac{t_b}{A_b / A_t} \tag{3}$$

This means that there is a longer total burning duration for smaller burning areas. For example, the 100% burning area has a total burning duration of 19 min and the 1% burning area a total burning duration of 1900 min.

In the case of the 100% burning area, all of the structure will experience near field (flame) conditions for the total burning duration (which is equal to the burning time, t_b). However, for the travelling fire cases, any one structural element will feel far field (smoke) conditions for the majority of the total burning duration and near field conditions for the burning time as the fire burns locally to the element. The time one element experiences far field conditions prior to the arrival of the flame is defined as t_{pre} and the time the element experiences far field conditions after the departure of the flame is defined as t_{post}. Figure 1 illustrates the difference between the near field and far field.

The near field temperature, T_{nf}, is taken here as the flame temperature, which for the accuracy levels required in structural fire analysis, is more or less constant and approximately 1200°C to 1300°C for a typical office fire. The far field conditions vary as a function of distance away from the fire. However, it is desirable to express the results in simple terms but without loss of generality in order to be of valuable engineering use. Thus, the far field is reduced to a single characteristic temperature, T_{ff}, which keeps the amount of information passed to the structural analysis manageable. To do this, the far field temperature is taken as the fourth-power average of T_{max} (to favour high temperatures in a bias towards radiation heat transfer and worst case conditions) over the distance between the end of the near field, r_{nf}, and the end of the far field, r_{ff}. This average is calculated by Eq. (4).

$$T_{ff}^{\ 4} = \frac{\int_{r_{nf}}^{r_{ff}} T_{max}^{4}\, dr}{r_{ff} - r_{nf}} \qquad (4)$$

Once the far field temperature is determined for a given fire size, the temperature time history of a point can be described, as shown in Figure 2. Determining both t_{pre} and t_{post} is dependent on the path of the fire and the exact position being examined. However, it is not possible to establish a fire's path of travel a priori, as there are many potential paths in principle; therefore assumptions must be made for worst case conditions.

The growth and decay phases of the gas temperatures for the travelling fires detailed in this paper are assumed to be very fast. This is because the larger an enclosure is, the lower the importance of the thermal inertial of its linings are, thus the faster the growth and decay phases will be. In other words, the transport of the hot gases in the smoke layer is faster than the heat transfer to the surfaces. Note that the cooling is not neglected in the structural analysis, only the decay phase is eliminated from the fire environment.

4 A CONCRETE STRUCTURE

The methodology has been applied to a large compartment with a floor area of 1000 m^2 made of concrete slabs (see details in Law et al., 2011). The resulting family of fires is shown in Figure 3 as the far field temperature vs. total burning duration. When these fire temperatures are fed into a heat transfer model (Abaqus) of the concrete structure, the resulting rebar temperature are those in Figure 4.

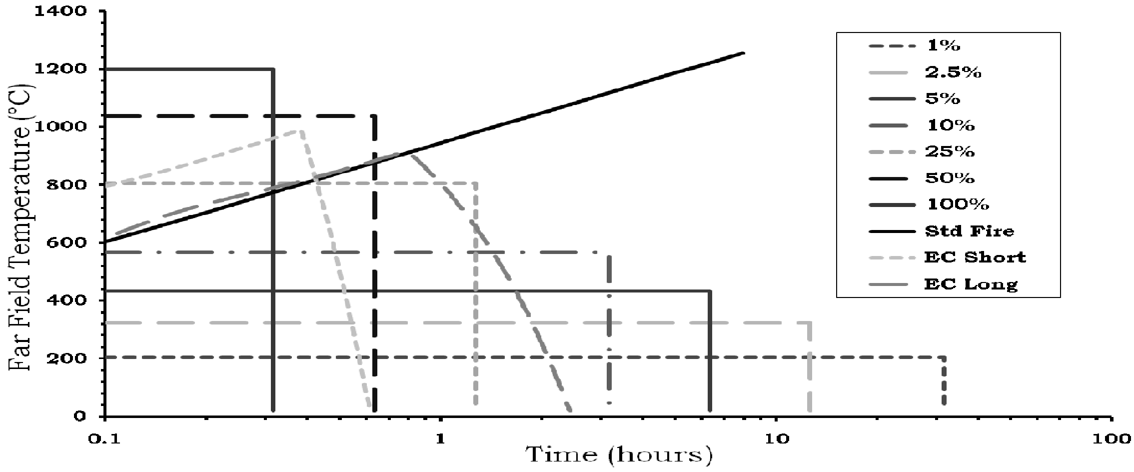


Fig. 3 Fire temperature in the family of fire sizes (Law et al., 2011)

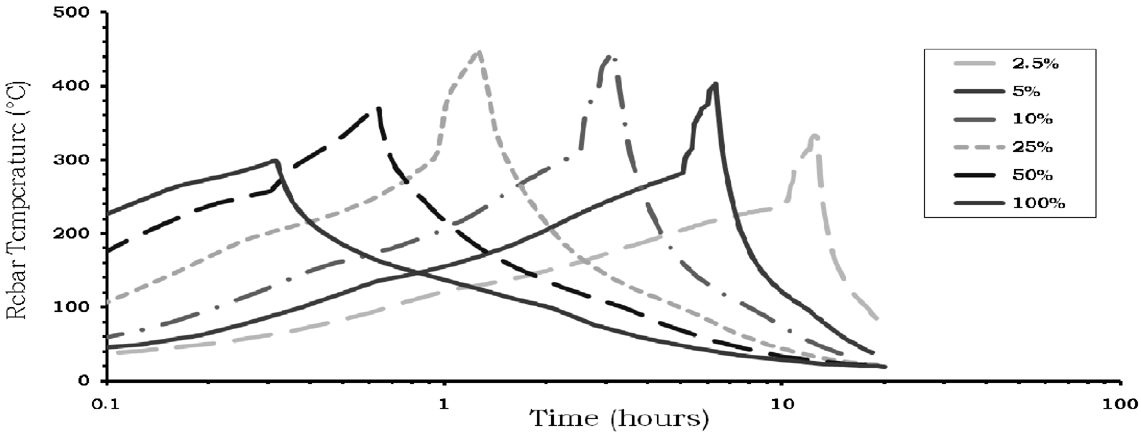


Fig. 4 Resulting temperature of the rebar in the concrete slabs for the family of fire sizes (Law et al., 2011)

5 A STEEL STRUCTURE

The methodology is applied to The Informatics Forum, a modern building in Edinburgh, using simple analytical calculations to obtained the steel temperatures (see details in Jonsdottir et al. 2010). Analysis of three protected steel beams, and comparison to those from traditional design methods (standard and parametric fire curves) are shown in Figure 5.

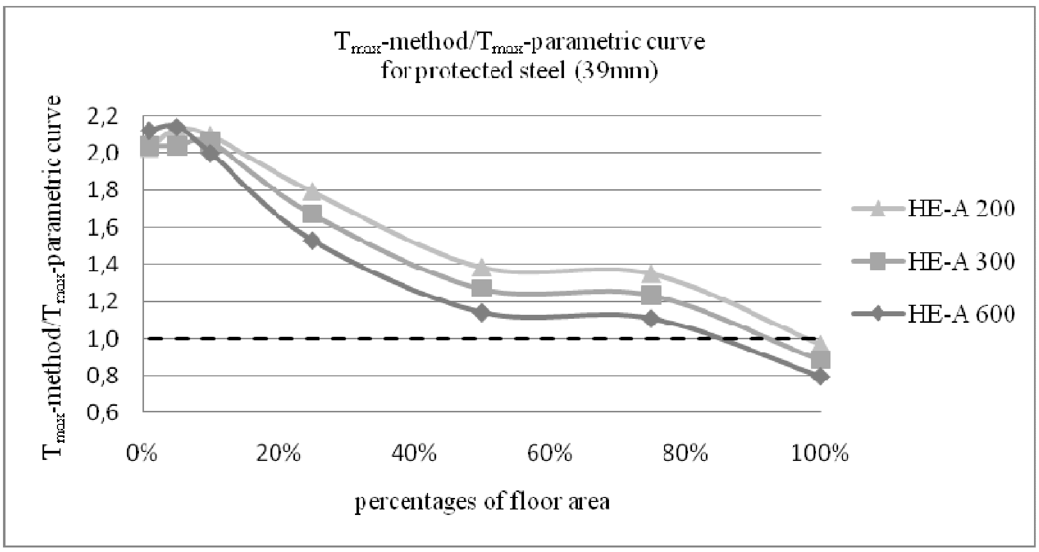


Fig. 5 Maximum steel temperature using the travelling fire method normalised by the maximum steel temperature using the parametric curve - protected steel (Jonsdottir et al. 2010).

The results indicate that more severe conditions are predicted for small travelling fires of size 5 to 10% of the floor plate than for uniform fires assumed in traditional methods. Compared to the parametric fire curve, travelling fires lead up to 95% higher steel temperatures for unprotected steel, and up to 110% for protected with 39mm-gypsum. Traditional methods are more conservative only when compared to travelling fires larger than 85% of the floor plate.

6 CONCLUSIONS

This paper has presented a performance based methodology for specifying design fires for structural analysis using travelling fires. The method allows for inclusion of a family of fires, to cover a wide range of possible fire dynamics in a building, ultimately leading to quantification of a range of possible structural responses. This range includes more challenging cases than are included in the traditional design methods.

7 ACKNOWLEDGMENT

The authors acknowledge collaborators Law, Gillie, Jonsdottir and Torero, and funding from Arup.

REFERENCES

Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire, European standard EN 1991-1-2, 2002. CEN, Brussels.

Rein, G. et al, "Multi-story Fire Analysis for High-Rise Buildings," 11th International Interflam Conference, London, 2007. http://www.era.lib.ed.ac.uk/handle/1842/1980

Stern-Gottfried, J., Rein, G., and Torero, J. L, "Travel Guide", Fire Risk Management, Nov 2009, pp. 12-16. http://www.era.lib.ed.ac.uk/handle/1842/3184

Jonsdottir, A. and Rein, G. "Out of Range", Fire Risk Management, Dec 2009, pp. 14-17. http://www.era.lib.ed.ac.uk/handle/1842/3204

Alpert, R.L., "Calculation of Response Time of Ceiling-Mounted Fire Detectors," Fire Technology, Vol. 8, 1972, pp. 181–195.

Law, A., Gillie, M., Stern-Gottfried, J., Rein, G., The Influence of Travelling Fires on a Concrete Frame, Engineering Structures, (in press) 2011. doi:10.1016/j.engstruct.2011.01.034

Jonsdottir, A., Rein, G., Stern-Gottfried, J, Comparison of Steel Temperatures using Travelling Fires and Traditional Methods, 12th Interflam, Nottingham, July 2010.

APPLICATION OF A VIRTUAL RESISTANCE FURNACE:
Fire resistance test simulation on a plasterboard membrane

François Cayla [a], H. Leborgne [b], D. Joyeux[c]

[a] Efectis France, Bordeaux, France
[b] Efectis France, Maizières-Lès-Metz, France
[c] Efectis France, Saint-Aubin, France

INTRODUCTION

Furnaces are necessary to perform fire resistance tests on building elements. The European standard **EN 1363 -1** imposes mainly two constrains which must be achieved simultaneously:

- The static overpressure must be maintained to 20 Pa at the top of a vertical tested element placed as the closing wall of the furnace.
- The thermal program delivered by the furnace burners is defined by a time dependant logarithmic curve ranging from 20°C at the start of the test to approximately 1050 °C after 2 hours of test as given by *Eq.* (1):

$$T = 345 \log_{10}(8t+1) + 20 \tag{1}$$

With T in °C and t in minutes
In order to have a better understanding of such fire resistance furnace behaviour, experiments have been carried out and concerned measurement methods (Sultan et al 1986, Sultan 2006). However, few studies are addressed to the numerical simulations of those furnaces (Bressloff et al 1995, Welch et al 1997).
In this work, a fire resistance test furnace simulator is presented. It has been designed in such a way to be representative of the vertical furnace of Efectis France Laboratory. This furnace is warmed up by 12 natural gas burners. Its geometry is showed on Fig. 1.

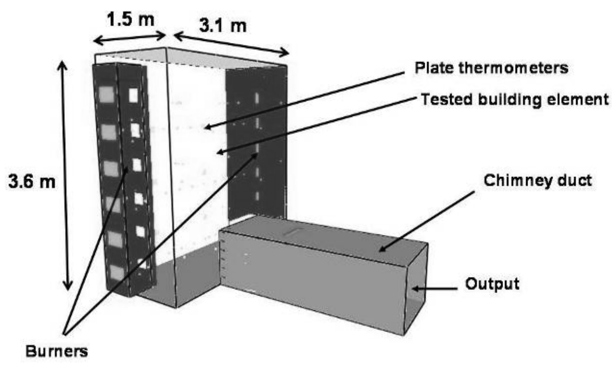


Fig. 1 Outline of the virtual furnace structure

This paper addresses a simulation using the virtual furnace coupled with a plasterboard membrane as tested element. The main characteristics of the simulator and of its coupling with the building element will be summarised in part 2. Results obtained with this simulator on a plasterboard membrane will be exposed in part 3.

1 MAIN CHARACTERISTICS OF THE FURNACE MODEL

The modelling of realistic simulator requires simulating both the furnace behaviour and its interaction with the tested building element.

1.1 The furnace model

The computational fluid dynamics code FDS (Fire Dynamic Simulator) version 5, developed by NIST institute, is used for the furnace design.
This program has been modified to satisfy to the constraints imposed by the European standard EN 1363-1. Thus, automatic controls have been introduced to regulate the 12 gas burners combustion and to assure an imposed output volume flow at the chimney exit. Both combustion mixture and hot gases extraction conditions are iteratively corrected at each time step of the simulation driven on this modified FDS 5 version.
Each gas burner is modelled as a block in which natural gas and combustion air are injected to perform the combustion. Proportions of injected gas and air are based on experimental measurements carried out during calibration tests on the EFECTIS France vertical furnace.
The evolution of temperatures in the virtual furnace is controlled by 6 modelled plate thermometers designed in full compliance with **EN 1363-1** requirements. They are constituted by an Inconel steel sheet insulated on their backside by a refractory board. An Inconel thermocouple is welded on the Inconel steel sheet. This type of thermometer has been developed by the European fire laboratories to harmonize the thermal impact delivered by European furnaces and must be used to perform fire resistance tests (see Fig. 2). This design makes plate thermometers quite sensitive to the radiative heat flux coming from flames of burners and lining of furnaces.
Those plate thermometers are placed as indicated in Fig. 3 in x and z directions and at 10 cm from the tested element in y direction. The steel sheet of each plate thermometer is oriented towards the furnace cavity.

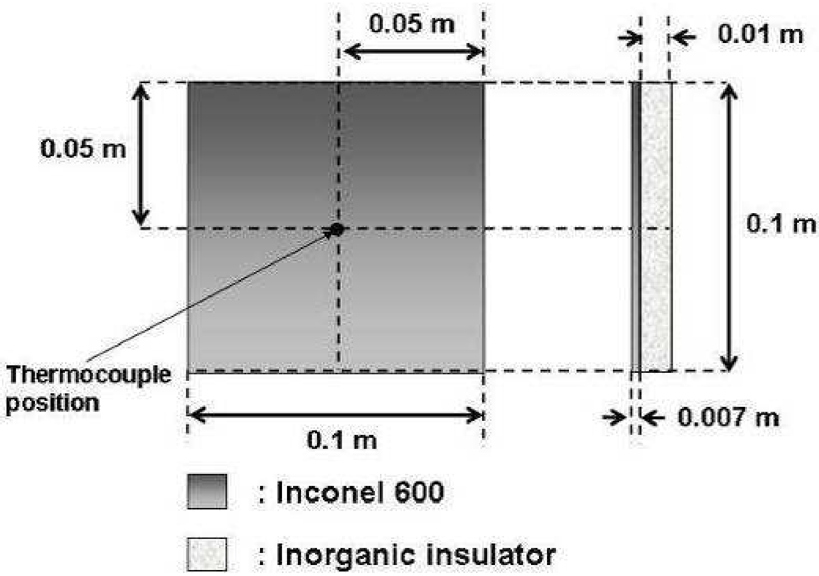


Fig. 2 Outline of the plate thermometer structure

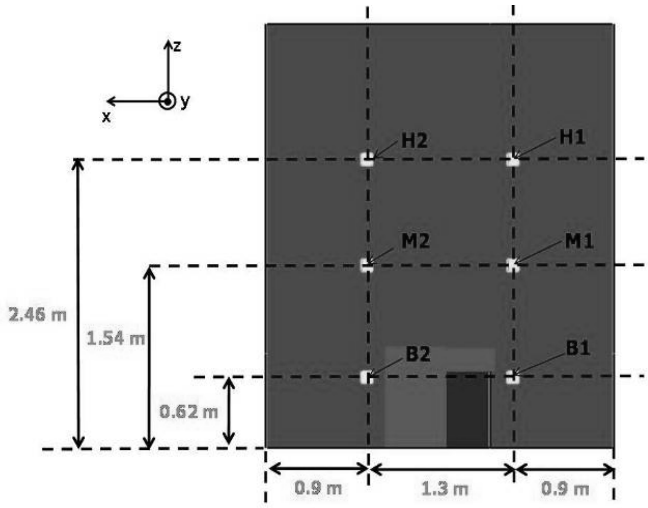


Fig. 3 Position of the plate thermometers

1.2 The building element coupling

The tested building element is modelled by using the code CAST3M. This finite element code is developed by the C.E.A. (French Alternative Energies and Atomic Energy Commission). It is mainly dedicated to thermomechanical behaviour of solids.

An interface has been created between this code and the modified version of FDS 5 to ensure the thermal coupling between the virtual furnace and the element. Thermal constrains delivered by FDS 5 are refreshed regularly on the fire exposed element surface. They are constituted by a radiative flux coming from the furnace lining, a convective flux due to hot gasses in the vicinity of the element and a radiative flux emitted by the exposed side of the tested element (see **Fig. 4**).

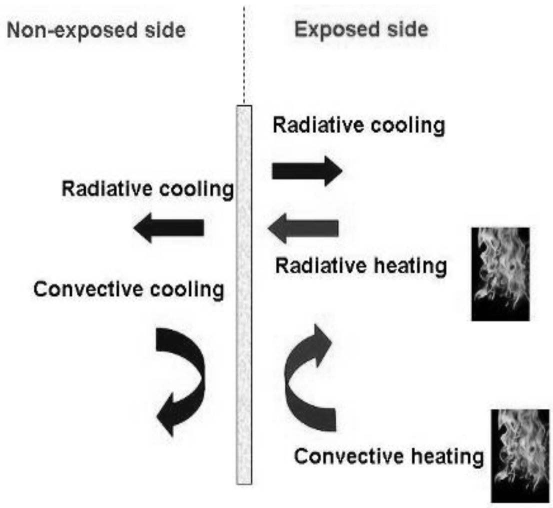


Fig. 4 Heat flux at the exposed and non-exposed sides of the tested element.

Those constraints allowed the determination of the temperature field on the exposed side of the element. This field constitutes the new boundary conditions for the calculation of the thermo-dynamic equilibrium of the furnace inner volume at the next time increment step.

2 RESULTS

The furnace simulator has been assessed in a combination with a full size plasterboard membrane. The dimensions of this membrane are 3.1 x 3.6 x 0.0125 m (w x h th).

The rear side of the membrane is submitted to an ambient temperature equal to 20 °C coupled with

a free convection factor of 4 $W.m^{-2}.K^{-1}$. The emissivity of the plasterboard is taken as 0.7 in this simulation.
On fig. 5 and 6, the temporal evolutions of the temperatures calculated by the virtual plate thermometers are presented. The EN 1363-1 temperature curve and its accepted tolerance (±100°C) are shown too. We observe that the furnace simulator permits to follow strictly the EN 1363-1 thermal program after 10 minutes simulated. Before 10 minutes, temperatures of points M1 and M2 are not lining up with the EN 1363-1 curve because furnace wall are still colder than ambient gas inside furnace.

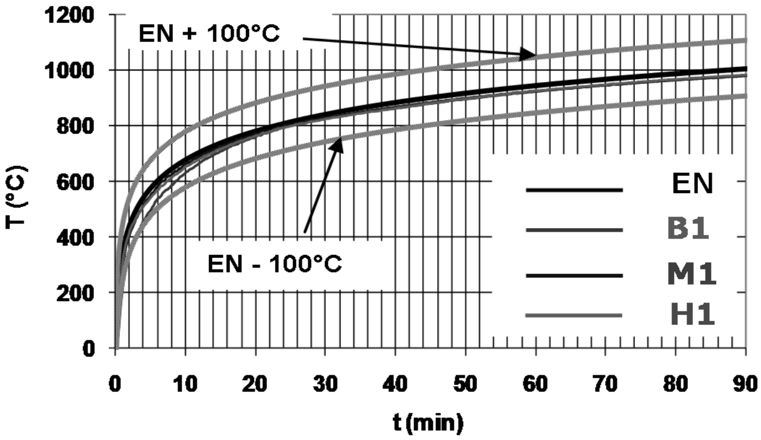


Fig. 5 Temperatures calculated by Plate Thermometers B1, M1 and H1 as function of time.

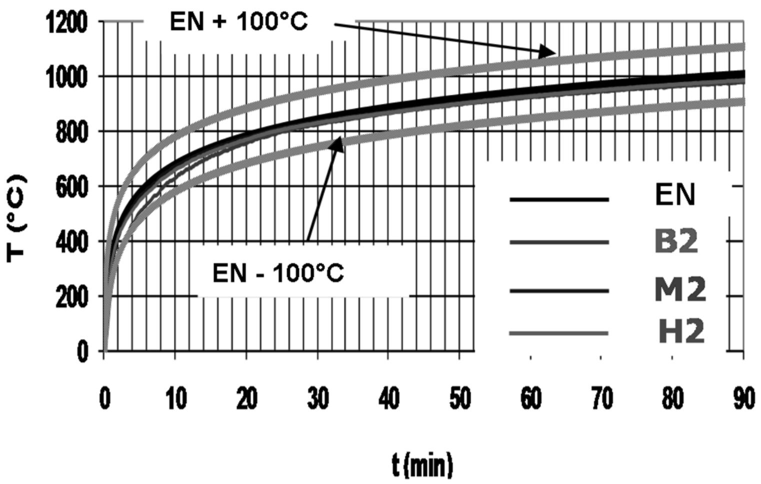


Fig. 6 Temperatures calculated by Plate Thermometers B2, M2 and H2 as function of time.

The Fig. 7 shows the static over pressure time evolution at the top of the tested membrane. We observe that this pressure is pretty well maintained around the 20 Pa level as recommended by the European standard after 10 minutes of simulation. Before this time, the pressure is enhanced because of the fast turning on of the burners, and reaches a maximum of 78 Pa.

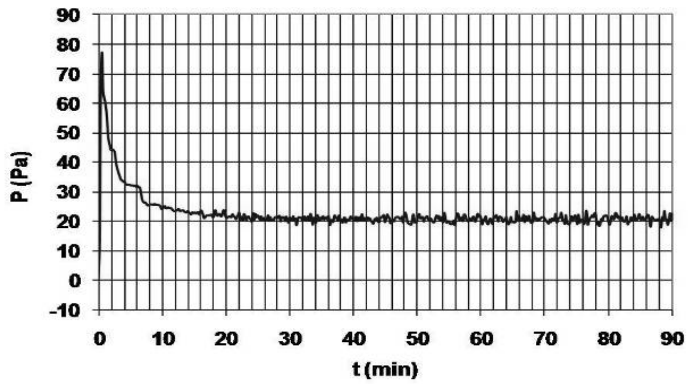


Fig. 7 Over pressure at the top of the furnace as function of time

Fig. 8 presents the temperature gradient evolution in the centre of the membrane. Temperature of the exposed side (a) is below the European standard evolution curve. This is explained by the water evaporation in the core and on non-exposed side of the plasterboard membrane.
On the non-exposed side (c) the hygroscopic effect leads to the slowing down of the temperature around 100°C. This phenomenon is representative of experimental measurements during actual fire tests.

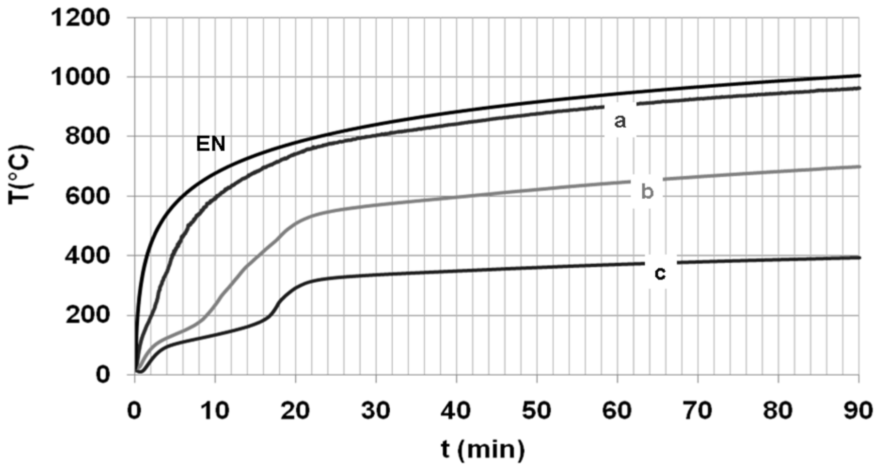


Fig. 8 Temperatures evolutions at the centre of the tested element as a function of time (EN: standard curve, a: exposed side, b: mid-thickness position, c: non-exposed side)

3 SUMMARY AND ACKNOWLEDGMENT

This paper presents a summary of the fire resistance furnace simulator modelled by coupling the code CAST3M and a modified version of the CFD code FDS 5.
The modified version of FDS 5 permits to control temperature and pressure conditions in the virtual furnace to achieve the EN 1363-1 requirements.
The strong coupling of the virtual furnace with the code CAST3M gives results in good agreement with experiments concerning the temperature of the tested element.
To confirm the relevance of the furnace simulator, more sophisticated elements as fire resistant doors will be soon tested. A more complete and elaborate physical model needs also to be developed to take into account more properly the vaporisation plateau supplied by hygroscopic material as plasterboard or calcium silicate boards when submitted to EN 1363-1 thermal program.
We acknowledge P. Verpeaux, A. Millard and Th. Charras of the CEA/DM2S/LM2S (Saclay, France) for their help concerning the use of CAST3M software.

REFERENCES

Sultan M.A., Harmathy T.Z., Mehaffey J.R., Heat transmission in fire test furnaces, 10(2): 47-55, 1986

Sultan M.A. Incident heat flux measurements in floor and wall furnaces of different sizes. Fire and Materials, 30: 383-396, 2006

Sultan M.A. Fire Resistance Furnace Temperature Measurements: Plate Thermometers vs Shielded Thermocouples. Fire Technology, 42(3): 253-267, 2006

Bressloff N., Rubini P. Cox G. Computational Fluid Dynamics Modelling of a Fire Resistance Furnace. 1st European symposium on Fire Safety Science, Zurich, Switzerland, 1995.

Welch S., Rubini P.A. Three-dimensional Simulation of a Fire-Resistance Furnace. Proceedings of 5th International Symposium on Fire Safety Science, Melbourne, Australia, 1997.

THE IMPACT OF ASSUMED FRACTURE ENERGY ON THE FIRE PERFORMANCE OF TIMBER BEAMS
A Numerical Study

Danny Hopkin [a/c], Jamal El-Rimawi [a], Vadim Silberschmidt [b], Tom Lennon [c]

[a] Loughborough University, Department of Civil and Building Engineering, Loughborough, UK
[b] Loughborough University, Wolfson School of Mechanical Engineering, Loughborough, UK
[c] BRE Global, Garston, Watford, Herts, WD25 9XX, UK.

INTRODUCTION

The behaviour of timber at elevated temperature is complex. Timber is not only combustible but also has complex mechanical properties; it is orthotropic and its tensile and compressive strengths and stiffnesses vary at differing rates with increasing temperature. In addition, timber is brittle when exposed to tensile stresses and ductile when subject to compression. All of the above, and the fact that its thermal properties are dependent upon heating rate, make the simulation of timber structures in fire a difficult task.

EN 1995-1-2 gives guidance on the properties, both thermal and mechanical, that should be adopted in the simulation of timber structures exposed to fire. It is apparent from the literature that at ambient temperature strain energy is not instantaneously dissipated at the moment a crack forms in timber under tensile loading (i.e. its behaviour is not perfectly brittle). After cracking, tension softening is apparent whereby the fracture energy is gradually dissipated with increasing crack strain. Modelling this behaviour is not only desirable from a physical point of view but is often necessary to ensure numerical stability in simulations. However, no guidance is given in EN 1995-1-2 regarding a magnitude of fracture energy at ambient or elevated temperature.

In this study the finite element software DIANA has been used to investigate the impact of fracture energy and tension softening regime on the structural behaviour of a simply supported timber beam, subject to a standard fire from below. Timber beams, subject to one-dimensional heating from below in a 2D plane stress formulation, are studied at different load ratios. In enabling this study to be conducted, FORTRAN subroutines were incorporated to determine the elastic modulus of timber depending upon the governing strain state, i.e. tensile or compressive. This user routine is implemented with a Total Strain-based cracking and plasticity model to evaluate the consequences of adopting fracture energies ranging from 600 to 5000 Nm/m^2. In addition, the impact of the shape of the tension softening branch is also evaluated.

1 FRACTURE ENERGY AND TENSION SOFTENING

The fracture energy of timber, more specifically softwoods, is an area well researched at ambient temperature. Many textbooks give fracture energies for different cracking modes, which are shown to be highly dependent upon density (Thelandersson & Larsen 2003). Larson & Gustafsson (1990/91) give one such correlation for notched timber members subject to bending, where:

$$G_f = 1.07\rho - 162 \tag{1}$$

where G_f *is* fracture energy (Nm/m^2) and ρ *is* density in kg/m^3.

For typical softwood this gives fracture energies ranging from 160-480 Nm/m^2 for mixed mode cracking. However, little, if anything, is known about the fracture energy of timber or other brittle materials at elevated temperature.

The numerical mechanical modelling of timber at elevated temperature is rare. To date, simplified models are often adopted using spreadsheets and sectional analysis tools (Konig & Walleij 2000, Schmid, et al. 2010). As a result of such an approach, it is not necessary to define fracture energy as timber in tension can be treated as a perfectly brittle material.

However, such an approach cannot be adopted in more general FEA computations as this may lead to numerical instability. To this end, tension softening regimes are often defined to describe the descending branch of a materials constitutive relation (See figure 1). The definition of such behaviour requires either knowledge of fracture energy (i.e. the integral of the deformation stress curve) or ultimate crack strain, i.e. the strain at which all crack stress has vanished. Neither codes and standards nor academic research give an insight into the effect of increasing temperature on fracture energy or ultimate crack strain. As a result, it is often necessary to assume values, which can have a very large influence upon deformation behaviour and upon the ultimate load carrying capacity.

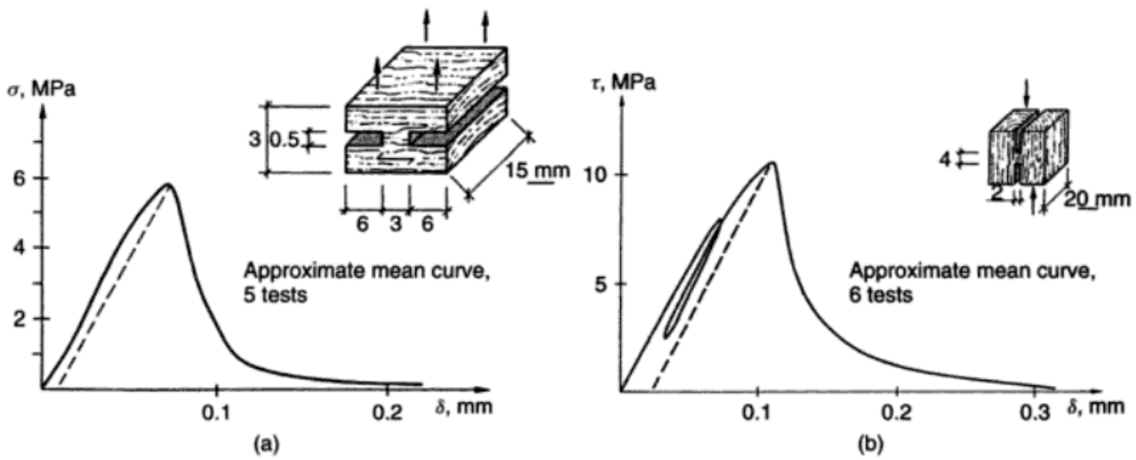


Fig. 1 Indicative fracture behaviour of timber in (a) tension and (b) shear (displacement vs. stress) after Thelanderson and Larsen (2003)

In the DIANA FEA package it is possible to define a number of tension-softening relationships based upon fracture energy, crack bandwidth and/or ultimate crack strain. To investigate the impact of these parameters, a parametric study was designed to study the behaviour of simply supported beams, loaded to different utilisation levels, under standard fire exposure. To undertake the study, it has been necessary to make a number of modifications to DIANA in order to extend characterisation of the behaviour of timber. These developments are discussed in a supporting paper (Hopkin *et al*. 2011). The concept of total strain-based cracking as implemented in DIANA is introduced briefly in the following section. The design of the parametric study is discussed in further detail later on in the paper.

2 TOTAL STRAIN-BASED CRACKING

The DIANA constitutive model based on total strain is developed along the lines of the Modified Compression Field Theory, originally proposed by Vecchio & Collins (1986). The three-dimensional extension to this theory is proposed by Selby & Vecchio (1993). A constitutive model based on total strain describes the stress as a function of the strain. This concept is known as hypo-elasticity, when the loading and unloading behaviour is along the same stress-strain path. In the current implementation in DIANA, the behaviour in loading and unloading is modelled differently with secant unloading.

One commonly used approach is the coaxial stress-strain concept, in which the stress-strain relationships are evaluated in the principal directions of the strain vector. This approach, also known as the *Rotating crack model*, has been applied to the constitutive modelling of reinforced concrete for a long period and has shown that the modelling approach is well suited for such structures.

More appealing to the physical nature of cracking is the fixed stress-strain concept, in which the stress-strain relationships are evaluated in a fixed coordinate system that is fixed upon cracking. Both approaches are easily described in the same framework, where the crack directions *nst* are either fixed or continuously rotating with the principal directions of the strain vector.

The basic concept of total strain crack models is that the stress is evaluated in the directions given by those of the crack. The strain vector ε_{xyz} in the element coordinate system *xyz* is updated with the strain increment $\Delta\varepsilon_{xyz}$ according to:

$$(t+\Delta t_{i+1})\varepsilon_{XYZ} = t\varepsilon_{XYZ} + (t+\Delta t_{i+1})\Delta\varepsilon_{XYZ}\,. \tag{2}$$

This is transformed to the strain vector in the crack directions with the strain transformation matrix **T** giving:

$$t+\Delta t_{i+1}\varepsilon_{nst} = T\left(t+\Delta t_{i+1}\Delta\varepsilon_{XYZ}\right). \tag{3}$$

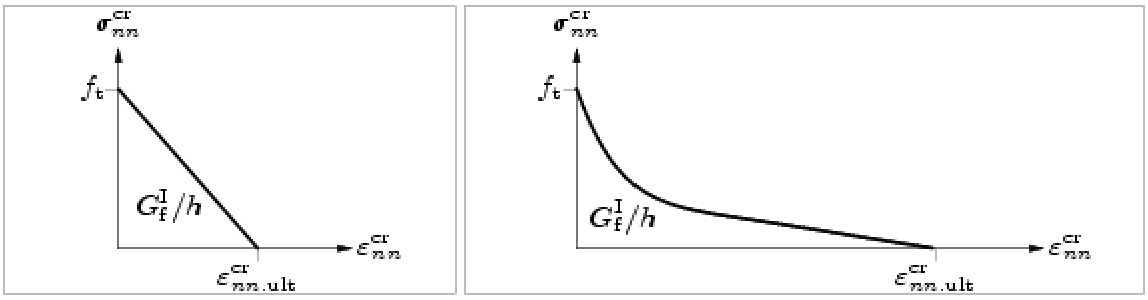
The strain transformation matrix **T** is either fixed upon first cracking or depends on the current strain vector (Rotating crack model). The Total Strain crack models, be it the fixed or rotating crack model, are appealing as they are numerically very stable when compared to smeared strain decomposed alternatives. In such cases the total strain is decomposed into elastic and crack components, i.e.:

$$\varepsilon = \varepsilon_E + \varepsilon_{cr}\,. \tag{4}$$

This decomposition of the strain allows also for combining the decomposed crack model with, for instance, a plastic behaviour of the concrete in a transparent manner. The sub-decomposition of the crack strain ε_{cr} gives the possibility of modelling a number of cracks that simultaneously occur. However simplistically, in a total strain based formulation, the compressive (ductile) and tensile (brittle) characteristics can be idealised within a single material model describing both aspects of physical behaviour.

3 PARAMETRIC STUDY DESIGN AND MODELLING APPROACH

In DIANA the tension softening relations of a material can be described either via fracture energy or ultimate crack strain. Both of these parameters can be specified as a function of temperature. DIANA offers linear, exponential or Hordyk tension-softening regimes, which describe the stress-strain relations of an open crack (see figure 2). More information on the tension softening regimes can be found in Manie (2010).



(a) (b)

Fig. 2 Tension-softening relationships available in DIANA (Manie 2010):- linear (a) and Hordyk (b)

In the parametric study conducted, both of the above were adopted to investigate the apparent failure time of a simple timber beam exposed to fire (ISO834) from below and subject to varying degrees of load level (via a mid-span point load). A simple bi-linear model describes the plasticity behaviour of timber in compression as part of a total strain-based crack model incorporating the above. The beam is modelled as continuum using second-order quad plane-stress elements.

The analysis is conducted as a staggered thermo-mechanical model whereby second-order structural elements are converted to first-order flow elements. Thermal and boundary properties are as per EN 1995-1-2 and EN 1991-1-2, respectively (BSI 2002/2004). Grade C30 timber is assumed throughout with a characteristic density of 300 kg/m^3. Tensile strength is derived according to Thunnel (1941) assuming 80% fractile strength. The Modulus of Elasticity (MOE) as a function of temperature is determined using a subroutine proposed by Hopkin *et al*. (2011).

Timber beams 150 mm deep and 2 m long are subject to different utilisation ratios of 25, 50, 75 and 90%. The required loads to achieve such utilisation levels are derived using the reduced cross section method set out in EN 1995-1-2 for standard fire exposure. Target ‘failure times’ are also derived using this method. Where a “mixed” fracture energy is referenced, this implies an increasing fracture energy with temperatures as per figure 3.

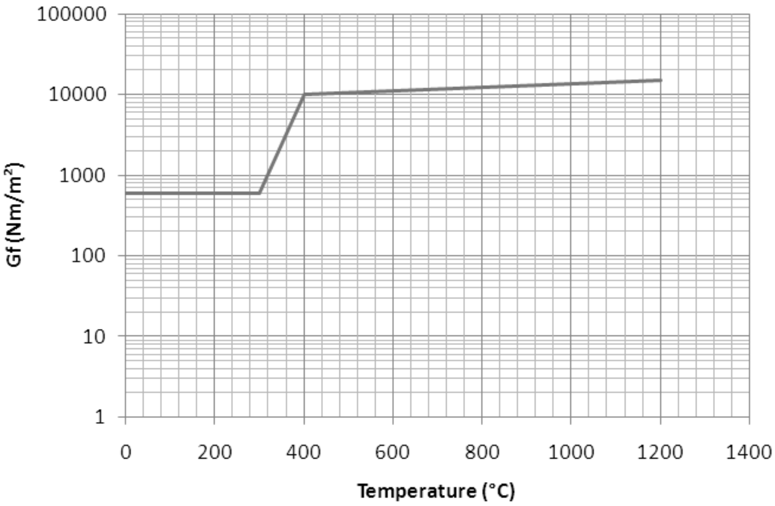


Fig. 3 Mixed fracture energy adopted in simulations

The mixed fracture energy concept is introduced as a potential solution to numerical instability. Large strains can develop in the char zone of a beam, which contributes little to the mechanical resistance yet may govern the termination time of a simulation, should the total strain at the extreme char fibres exceed that of the ultimate crack strain. The application of a single large fracture energy for all temperatures (i.e. 5000Nm/m^2 for all temperatures) may overpredict the load-carrying capacity of a timber beam and, as such, it is important to maintain realistic fracture energy values for uncharred timber.

Tab. 1 Parametric study summary

Group No.	Utilisation (%)	Fracture energy (Nm/m^2)	Tension softening	Target failure time (min)
1 (A-E)	25	600 (A), 1000 (B), 2000 (C), 5000 (D), Mixed (E)	Linear	66 (3960 s)
2 (A-E)			Hordyk	
3 (A-E)	50		Linear	34 (2040 s)
4 (A-E)			Hordyk	
5 (A-E)	75		Linear	13.5 (810 s)
6 (A-E)			Hordyk	
7 (A-E)	90		Linear	5 (300 s)
8 (A-E)			Hordyk	

Simulation failure is crudely taken as the last converged step. It is recognised that such a termination can be brought about due to numerical instability and not a physical failure. However, where fractures develop without alternative means of load redistribution, it is highly likely that failure is due to a violation of the stress-strain relationship for the material and thus can be considered as a ‘true failure’. This is particularly the case for instances where large fracture energy values, and thus large ultimate crack strains, are specified for the char layer, i.e. the mixed case.

4 FINDINGS

Without supporting experimental data the authors have chosen to measure the relative impact of fracture energy on 'failure' time by comparing simulation termination times with predicted failure times using the reduced cross-section method of EN 1995-1-2. Results are divided by tension-softening regime and as such plots of apparent simulation failure time and EN 1995-1-2-derived failure time are shown for linear and Hordyk tension-softening regimes in figures 4 and 5, respectively.

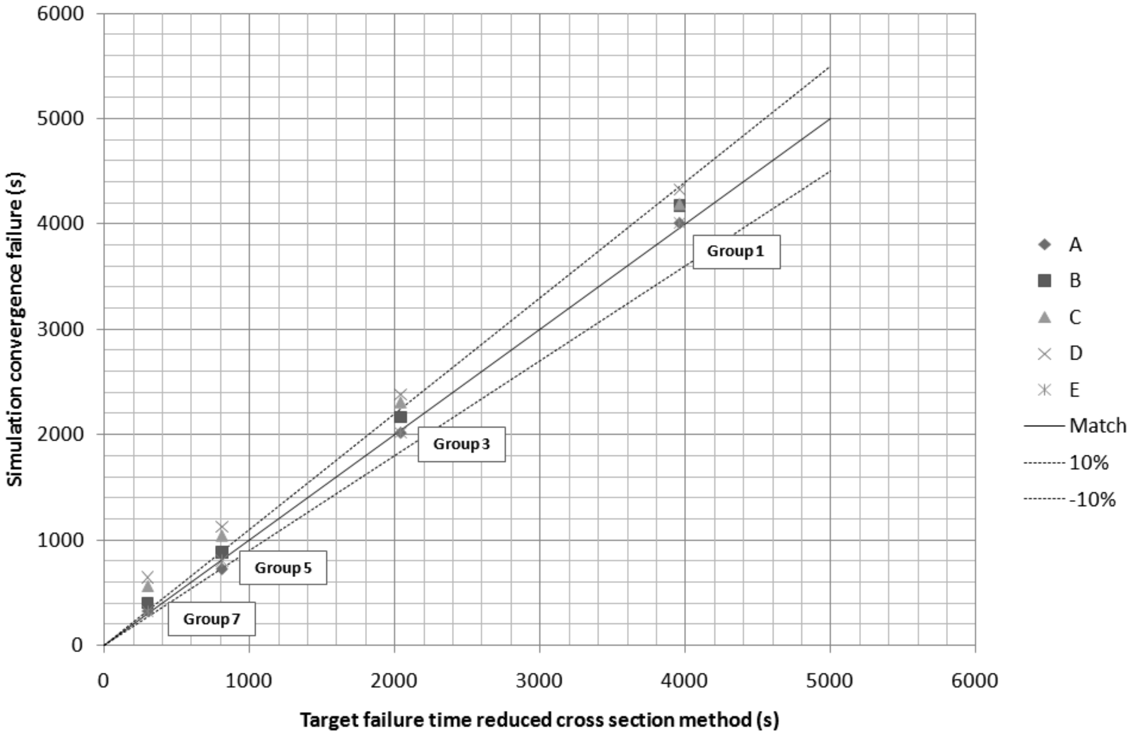


Fig. 4 Simulation termination time *vs*. predicted failure time from EN 1995-1-2 (linear tension softening)

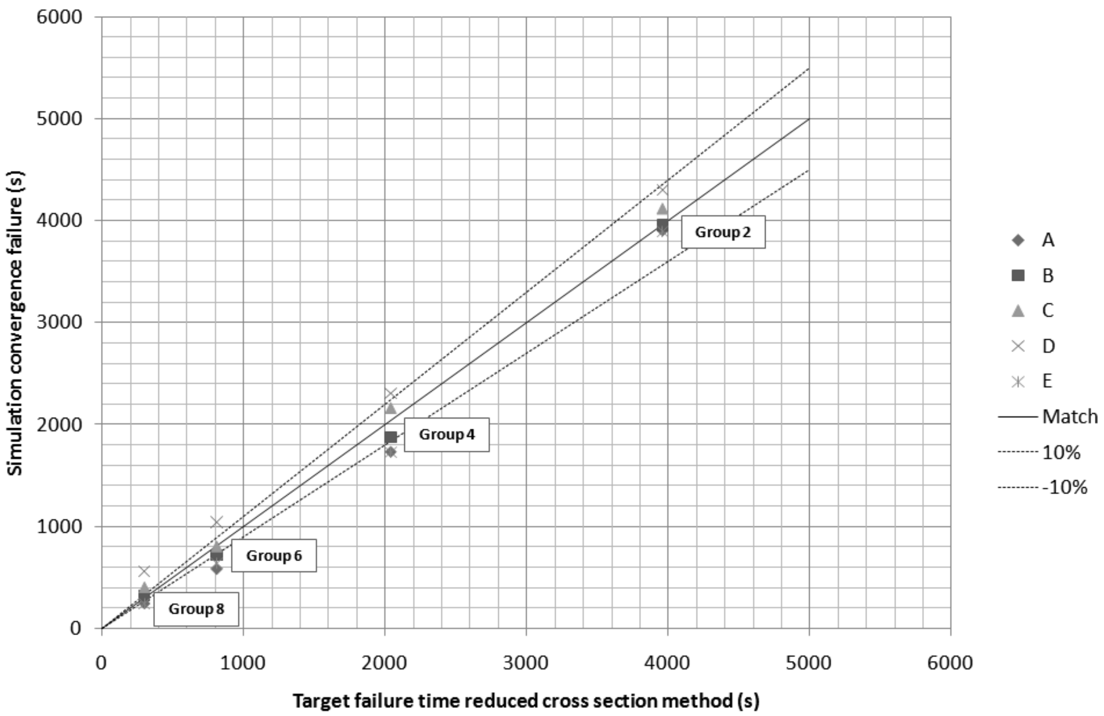


Fig. 5 Simulation termination time *vs*. predicted failure time from EN 1995-1-2 (Hordyk tension softening)

5 SUMMARY

Figures 4 and 5 demonstrate that the assumed fracture energy has an important influence on the simulation termination time when a timber beam is exposed to fire from below and is subject to different levels of load. The larger the fracture energy, the more ductile a structural member behaves as crack stress is dissipated over a much larger crack strain.

In numerical simulations the incorrect input of fracture energy can result in overall reductions in tensile strength as the values specified should be sufficient for the full tension-softening regime to be defined. In DIANA the limiting tensile strength is dependent upon the tension softening regime, fracture energy, MOE and crack bandwidth. Where small crack bandwidths and fracture energies are introduced, reductions in tensile strength can occur, which impact heavily upon apparent ‘failure time’. This behaviour was found to be more critical when Hordyk tension softening is adopted over Linear.

For the purposes of modelling timber beams exposed to fire it has been found that linear tension softening is adequate. A mixed fracture-energy approach (i.e. increasing G_f with temperature) can ensure that numerical instability does not develop in the char zone, where strains are high, whilst also giving realistic strength characteristics and brittleness behaviour in the undamaged residual cross section.

6 ACKNOWLEDEMENTS

The authors would like to thank EPSRC and BRE Global for their support.

REFERENCES

BRITISH STANDARDS INSTITUTION, 2004. Eurocode 5: Design of timber structures – Part 1-2: General – Structural fire design. BS EN 1995-1-2. London: BSI.

BRITISH STANDARDS INSTITUTION, 2002. Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire. BS EN 1991-1-2. London: BSI.

HOPKIN, D.J., EL-RIMAWI, J., SILBERSCHMIDT, V., 2011, The adaptation of TNO DIANA for the modelling of timber exposed to fire. Proc. of ASFE, Prague, Czech Republic.

KONIG, J. and WALLEIJ, L., 2000. Timber frame assemblies exposed to standard and parametric fires part 2: A design model for standard fire exposure. I0001001. Stockholm: SP Tratek.

LARSEN, H.J., and GUSTAFSSON, P.J.H, 1991. The fracture energy of wood in tension perpendicular to the grain. Paper CIB-W18A/24-19-2, Proc. of CIB-W18A Meeting 23, Oxford, UK.

LARSEN, H.J., and GUSTAFSSON, P.J.H, 1990. The fracture energy of wood in tension perpendicular to the grain- results from a joint testing project. Paper CIB-W18A/23-19-2, Proc. of CIB-W18A Meeting 23, Lisbon, Portugal.

MANIE, J., 2010. DIANA User's Manual. 9.4 edn. Delft: TNO.

SCHMID, J., KONIG, J. and KOHLER, J., 2010. Design model for fire exposed cross-laminated timber, V.R. KODUR and J.M. FRANSSEN, eds. In: Proceedings of the sixth international conference on structures in fire, 2-4th June 2010, DEStech pp511-519.

SELBY, R. G., AND VECCHIO, F. J., 1993. Three-dimensional Constitutive Relations for Reinforced Concrete. Tech. Rep. 93-02, Univ. Toronto, dept. Civil Eng., Toronta, Canada.

THELANDERSON, S. and LARSEN, H.J., 2003. Timber engineering. 1st edn. Sussex: Wiley.

THUNNEL, B., 1941. Hallfastetsegenskaper hos svenskt furuvirke utan kvistar och defekter. Handlingar Nr. 161. Stockholm: Ingenjorsvetenskapsakademien.

VECCHIO, F. J., AND COLLINS, M. P., 1986. The modified compression field theory for reinforced concrete elements subjected to shear. *ACI Journal* ***83****(22)*, 219-231.

STOCHASTIC ANALYSIS OF STRUCTURES IN FIRE BY MONTE CARLO SIMULATION

Kaihang Shi [a], Qianru Guo [a], Ann E. Jeffers [a]

[a] Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, USA

INTRODUCTION

Current methods for investigating structural response in fire are based on the assumption that the parameters affecting the fire behavior and the corresponding structural response are deterministically known. However, research in the area of structure-fire interaction demonstrates that a large number of uncertainties exist in the nature of the problem and in our ability to accurately represent the fundamental behaviors with numerical models. Furthermore, as the profession develops performance-based approaches to structural fire design, there is a significant need for design methods that allow structural performance to be assessed in a rational way. Deterministic methods are not well-suited for performance-based design because they do not account for uncertainty and fail to quantify the reliability of a given design, thus preventing a logical assessment of the response given uncertainties in the fire load and subsequent structural response.

To overcome current limitations, a reliability-based framework was utilized to assess the performance of structural systems given uncertain fire effects. The framework involves (i) characterizing the sources of uncertainty, (ii) quantifying the probabilistic characteristics of each uncertain parameter, (iii) defining performance criteria for the structure based on strength, stability, and serviceability requirements, (iv) evaluating the structural response stochastically (e.g., by Monte Carlo simulation), and (v) calculating the probability of failure. Once the probability of failure is determined, the adequacy of the design can be evaluated in terms of an acceptable level of risk. The methodology is illustrated here by an example in which the fire resistance of a steel beam was evaluated based on uncertainties in fire load and structural resistance. Stochastic analyses were carried out by performing finite element analyses within a Monte Carlo simulation. The paper focuses specifically on the implementation of a sequentially coupled stochastic analysis in the finite element analysis software Abaqus (2010) using Python scripting commands. Although the example considered here is relatively simplistic and does not include all possible uncertain parameters, it effectively demonstrates the application of the proposed reliability method and provides insight into the practicalities of extending the approach to more complex structural systems.

1 ANALYTICAL METHODOLOGY

The work described herein seeks to explore the effects of uncertainty in the fire behaviour and structural response using Monte Carlo simulation. Monte Carlo simulation is an iterative method in which random values for each uncertain parameter are generated based on their probabilistic characteristics, and the response is evaluated deterministically for each combination of random values. Once all simulations have been carried out, probabilistic information about the system can be synthesized from the results. For the purposes of evaluating structural reliability, the probability of failure p_f can be determined by observing the number of simulations for which the response exceeds a given failure criterion, i.e.,

$$p_f = \frac{N_f}{N}, \tag{1}$$

where N_f is the number of simulations for which the system failed, and N is the total number of simulations. While the Monte Carlo method has been used extensively to evaluate uncertainty in a wide range of applications, it tends to be very computationally expensive because a large number of iterations are required to achieve a desired level of accuracy. Efficiency can be improved with special sampling techniques such as Latin hypercube sampling (Bergmeister et al., 2009).

As shown in Fig. 1, structure-fire interaction involves a propagation of uncertainty that affects each stage of the sequentially coupled analysis. For example, uncertainties in the compartment geometry, type and distribution of fuel, and ventilation conditions result in a fire load that cannot be predicted with great precision. Additional uncertainties associated with the material properties of the structure, the thermal and structural boundary conditions, and magnitude of applied loads lead to further challenges in determining how the structural system will respond in an actual fire scenario.

The Monte Carlo method was adopted here to simulate the stochastic response of a of a structure given uncertainties in the fire and structural parameters. In the following analysis, random values were generated for each uncertain parameter based on assumed probabilistic characteristics. Three stochastic analyses were then carried out to evaluate the fire behaviour, the thermal response of the structure, and the mechanical response of the structure. Data from the stochastic simulations were then used to determine the reliability of the system using Eq. 1. Details about the analysis are provided in the following section.

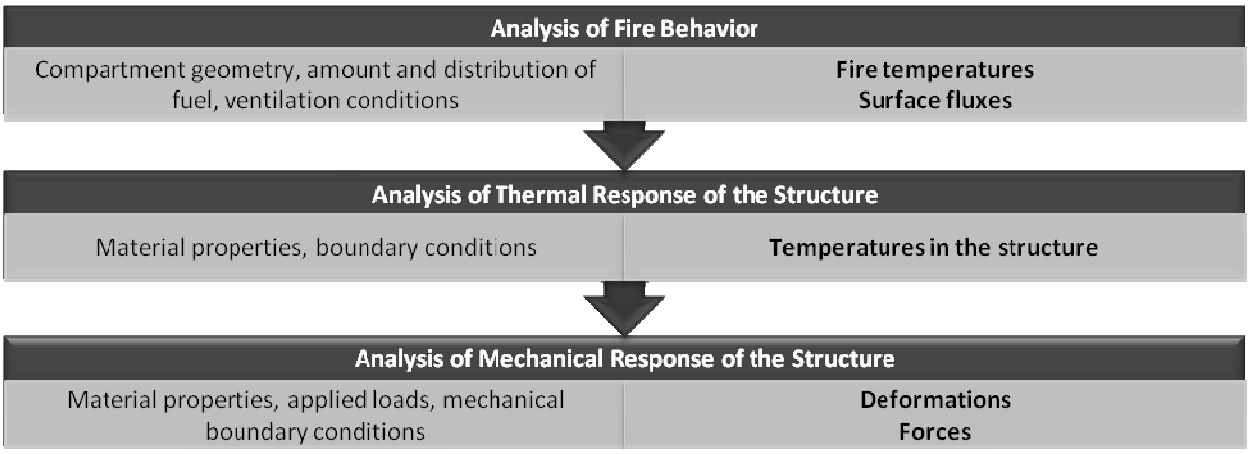


Fig. 1 Propagation of uncertainty in the structural fire simulation

2 PROBLEM STATEMENT

To illustrate the stochastic simulation of structural performance in fire, Monte Carlo simulations were conducted for a protected steel beam exposed to natural fire. As illustrated in Fig. 2a, the beam was simply supported and carried a uniformly distributed load *w*. In addition to the applied loads, the beam also supported a concrete slab, which was assumed to act non-compositely with the beam. The steel had a nominal yield strength of 345 MPa. A cross-section of W28x8 was required to resist the assumed design load based on the U.S. steel design specification (AISC, 2005) and to meet the ANSI/UL 263 requirements for prescriptive fire resistant design in the U.S. The beam's cross-section is shown in Fig. 2b. The beam was protected by a spray-applied fire resistant material such that the beam provided a 1h fire resistance.

The purpose of the analysis was to evaluate the response of the protected steel beam exposed to natural fire given uncertainties in the fire, material, and loading parameters. Natural fire exposure was modelled using the Eurocode parametric fire curve as modified by Buchanan (2002). To evaluate the thermo-mechanical response, two sequentially coupled analyses were conducted in Abaqus (2010). Heat transfer over the cross-section was modelled using two-dimensional continuum elements. The mechanical response was subsequently modelled using two-dimensional beam elements. Temperatures in the flanges and web were obtained from the heat transfer analysis and transferred directly into the structural model by specifying the flange and web temperatures as *predefined fields* in Abaqus.

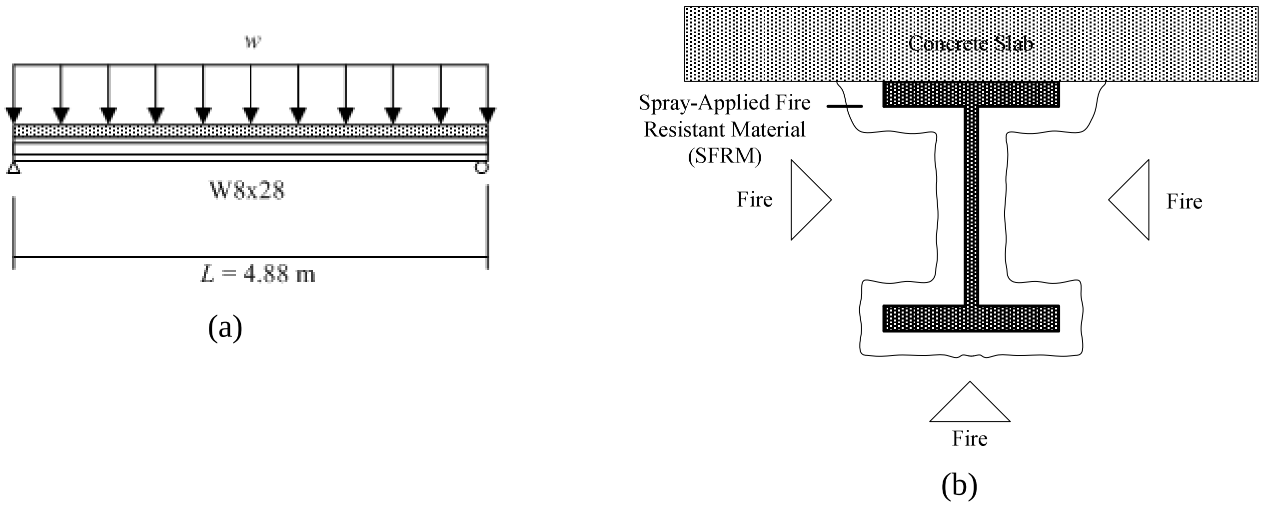


Fig. 2 Problem statement: (a) Loading and support conditions; (b) Cross-section

One thousand Monte Carlo simulations were carried out using Latin Hypercube sampling to reduce the total number of simulations required for the analyses. As illustrated in Fig. 3, two parametric studies (i.e., one for the heat transfer analysis, one for the structural analysis) were run in Abaqus, each of which utilized a Python script file that generated an Abaqus model for each combination of random parameters. To perform the parametric study, the input file (.inp) for the finite element analysis was written in terms of the uncertain parameters associated with the particular heat transfer or structural analysis. Random values for each parameter were generated in Matlab (2010) using the appropriate, mean, covariance, and probability distribution. Values for the random parameters were then entered in the Python script file along with commands to define the combination of parameters for each case and options for executing the analysis. The analysis was executed by running the Python script from the Abaqus command prompt. Once the analysis was completed, a separate Python script was executed to compile the results from each of the simulations.

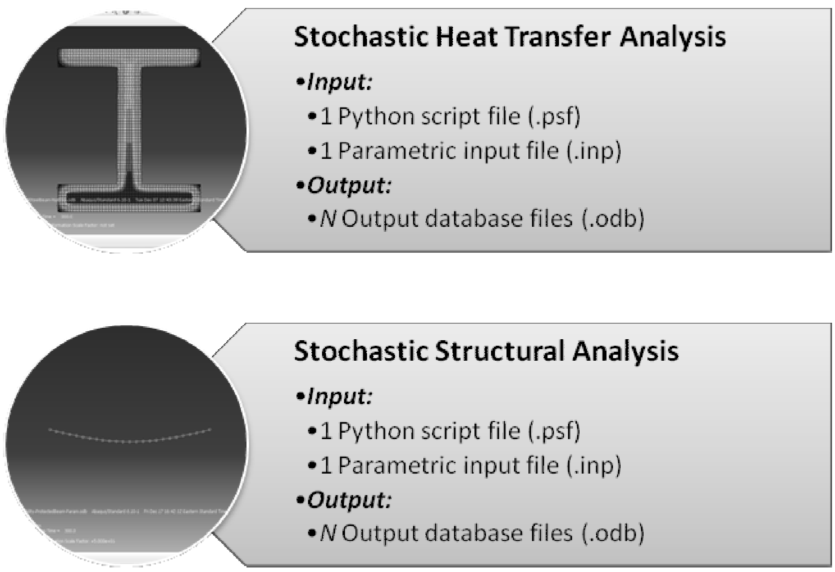


Fig. 3 Sequentially coupled stochastic simulation in Abaqus

2.1 Fire Analysis

Natural fire exposure is modelled using the temperature-time curve recommended by Buchanan (2002). For stochastic simulation of the fire, the fire parameters were assumed to have realistic mean values and probability distributions based on the works of Culver (1976) and Iqbal and Harichandran (2010). Specifically, the ventilation factor F_v was assumed to have a mean value of 0.04, a coefficient of variation (COV) of 0.05, and normal distribution. It was also assumed that the walls and ceiling were made of gypsum board, which has a mean value of b = 423.5 $Ws^{1/2}/m^2K$, a

COV of 0.09, and normal distribution. Based on an assumed compartment geometry, the fire load per total area e_t was determined to have a mean value of 132.54 MJ/m^2 with a COV of 0.62 and Gumbel (Extreme Type 1) distribution.

Random values for each of the fire parameters were generated in Matlab and subsequently inserted in the fire model to obtain a series of natural fire curves that represent the range of potential compartment fires expected in the current case study. As illustrated in Fig. 4, fires varied in duration and intensity, with the expected (mean) fire reaching a maximum temperature of approximately 1100 C and burning steadily for 15 min before decay.

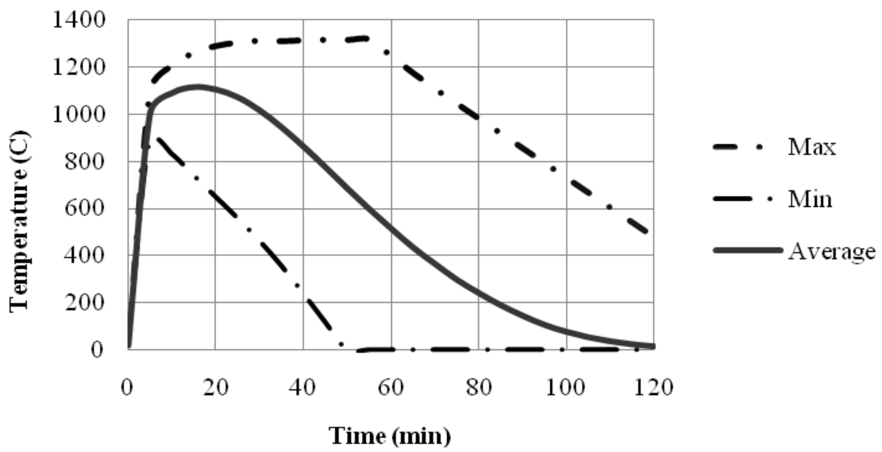


Fig. 4 Compartment temperature

2.2 Heat Transfer Analysis

The fire temperatures computed from the previous step were subsequently used in a two-dimensional heat transfer analysis of the beam's cross-section, which is shown in Fig. 2b. Heat is transferred from the fire to the steel by convection and radiation. Mean values for the convection heat transfer coefficient and effective emissivity ε were taken from the Eurocode, while both constants were assumed to be normally distributed with coefficients of variation of 0.10 due to lack of existing data.

To achieve the 1h fire resistance rating, the SFRM was required to have a thickness of 11.1 mm. The mean value for the SFRM thickness was taken as the nominal thickness plus 1.6 mm, resulting in a mean thickness of 12.7mm. The COV was assumed to be 0.20 and variability that follows a lognormal distribution. The density, thermal conductivity, and specific heat for the SFRM were assumed to be independent of temperatures using mean values reported in the Eurocode and probability distributions given by Iqbal and Harichandran (2010).

Conduction at the steel-concrete interface was modelled by treating the concrete as a semi-infinite medium with constant temperature of 20 C (Incropera and DeWitt, 2002). Thermal properties for the concrete were assumed to be constant and independent of temperature, while thermal properties for the steel were assumed to follow the temperature-dependent Eurocode models. Variability in the steel and concrete properties was ignored in the present analysis for simplicity.

There were a range of steel temperatures obtained due to the variability in the fire temperatures and thermal properties of the beam. Average temperatures from the heat transfer analysis are shown in Fig. 5. Note that the average steel temperatures around 500 C for the expected fire load. However, more severe fire loads combined with low fire protection thicknesses resulted in the possibility of steel temperatures in excess of 1000 C.

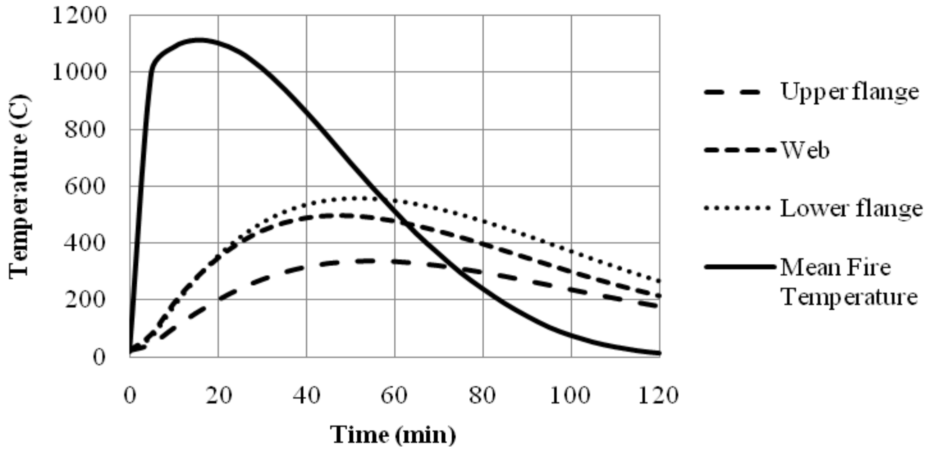


Fig. 5 Average temperatures in the steel

2.3 Structural Analysis

The steel temperatures from the heat transfer analysis were subsequently specified in the mechanical model of the beam that is shown in Fig. 2a. The stochastic simulation considered uncertainties in the yield strength and magnitude of the applied loads. While the beam was designed for a nominal yield strength of 345 MPa, a statistical analysis of data presented by Wainman and Kirby (1988) showed that this grade of steel has a mean strength of 380 MPa and a COV of 0.08 (normal distribution). The uniformly distributed dead and live loads had design values 5.15 kN/m and 3.65 kN/m, respectively, based on typical office loading in U.S. construction. For stochastic simulation, arbitrary-point-in-time dead and live loads were used based on the calculations of Ellingwood (2005). Thus, the dead load had a mean value of 5.41 kN/m, COV of 0.10, and normal distribution, and the live load had a mean value of 0.88 kN/m, COV of 0.60, and followed a gamma distribution.

The mid-span displacement for each simulation was measured at each time step in the analysis. Results are illustrated in Fig. 6. Note that the expected (mean) mid-span displacement reaches a value of 58 mm before cooling. Due to the range of potential fire loads, material parameters, and magnitudes of applied loads, there were many cases in which the deflections became excessively large.

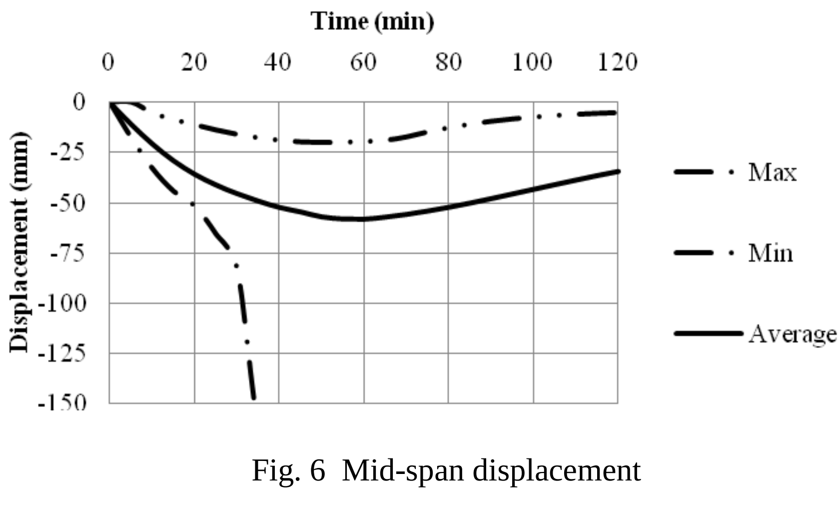


Fig. 6 Mid-span displacement

For the purposes of assessing the reliability of the structure, failure was defined as the time at which the mid-span displacement exceeded a limiting value of $L/30$. The probability of failure was computed according to Eq. 1, yielding a failure probability of 1.3%. Thus, despite the potential for catastrophic failure based on a potential “worst-case” scenario, the beam (as designed according to the current prescriptive codes) appears to be sufficiently designed to resist the natural fire, as expected. Note that the analysis only considered 1000 Monte Carlo simulations, and so the margin of error in calculating the probability of failure is high. Therefore, the findings of this study are inconclusive at this point in time. Further work is being done to explore the problem in greater depth.

3 SUMMARY AND CONCLUSIONS

This paper presents a preliminary study into the stochastic simulation of structures in fire. Specifically, the Monte Carlo method was used with finite element simulation in Abaqus to evaluate the response of a protected steel beam given uncertainties in fire load and structural resistance. While the application shows much promise for future investigations into the probabilistic mechanics of structures at extreme temperatures, the computational demands required to perform three sequentially coupled Monte Carlo simulations with embedded finite element simulations calls for a more computationally efficient approach. On-going work is being conducted to explore the parallelization of the simulation to improve the efficiency of the Monte Carlo method for structure-fire applications.

ACKNOWLEDGMENT

This research was supported by the U.S. National Science Foundation under Grant No. CMMI-1032493

REFERENCES

Abaqus, v.6-10 (2010). Dassault Systemes, Providence, RI.

AISC (2005). Steel Construction Manual, 13th Ed., American Institute of Steel Construction, USA.

Buchanan, A.H. (2002). Structural Design for Fire Safety, John Wiley and Sons Ltd, England.

Bergmeister, K., Novak, D., Pukl, R., and Cervenka, V. (2009). "Structural Assessment and Reliability Analyses for Existing Engineering Structures, Theoretical Background," Structure and Infrastructure Engineering, 5, 267-275.

Culver, C.G. (1976). Survey Results for Fire Loads and Live Loads in Office Buildings, National Bureau of Standards, Washington, D.C.

Ellingwood, B.R. (2005). "Load Combination Requirements for Fire-Resistant Structural Design," Journal of Fire Protection Engineering, 15, 43-61.

Incropera, F. P., and DeWitt, D. P. (2002). Fundamentals of Heat and Mass Transfer, 5th Ed., John Wiley and Sons, Inc., New York.

Iqbal, S., and Harichandran, R.S. (2010). "Capacity Reduction and Fire Load Factors for Design of Steel Members Exposed to Fire," Journal of Structural Engineering, 136, 1554-1562.

MATLAB, v.7.11 (2010). MathWorks, Inc., Natick, MA.

Wainman, D. E., and Kirby, B. R. (1988). Compendium of UK Standard Fire Test Data: Unprotected Structural Steel - 1, Report No. RS/RSC/S10328/1/87/B, British Steel Corporation, England.

INTEGRATED NUMERICAL MODELING IN FIRE SAFETY ASSESSMENT
Current needs, challenges, potentiality and perspectives.

Paweł A. Król

Warsaw University of Technology, Faculty of Civil Engineering, Warsaw, Poland

INTRODUCTION

Building fire safety is one of the six requirements imposed by the local building law act of European Union's member states, which must be strictly complied in the design and execution of buildings and structures. Ongoing adoption of Eurocodes - European standards for structural design imposes new challenges for the engineering community in Poland and elsewhere. It should be expected that, in the near future, as fire safety engineering develops, the global and advanced analyses of civil engineering structures will be absolutely necessary to consider the special design case of a fire. Such analyses will play a vital role in the attempts to assess and ensure high level of structural safety, in particular in the case of buildings and structures, for which the consequences of their destruction are serious, i.e. for those where the risk of human life and health loss is high, and for buildings with a special economic status. EN 1991-1-2: Eurocode 1, Directive 89/106/EEC as well as the state technical and building regulations stipulate that buildings and structures must be designed and built in order that, in the event of a fire: – load-bearing capacity of the construction can be assumed for a specific period of time, – the generation and spread of fire onto adjacent buildings and structures is limited, – residents can leave the building or be rescued in another way, – the safety of rescue teams is taken into consideration. The above requirements are in most cases met by analyzing the load-bearing capacity of the structure in the case of fire impact, using the conventional fire scenarios (nominal fires) or "natural" (performance-based) fire scenarios, whereas in the execution phase, passive and active fire safety measures, respectively, are used. Parts of the constructional Eurocodes identified by numbers 199x-1-2, which relate to the impact on the construction of temperatures generated by fire, discuss specific aspects of passive fire safety, considering the methodology for the design of structures and parts thereof, in order to ensure their appropriate load-bearing capacity, and to minimize the spread of fire. Two approaches exist in the structural design: conventional and performance-based. The conventional approach is based on certain simplified assumptions and uses the concept of nominal fire which generates thermal action described by a function of temperature distribution in time, where the function is not dependent on other physical and chemical parameters, which are often of vital importance. The approach based on performance models uses the principles of fire safety engineering and describes thermal action as a concept that depends on multiple physical and chemical parameters, which values are determined individually, considering the specific features of a given structure, which renders the approach much closer to the actual behavior of the construction in the case of a real fire, but is also much more difficult to describe from the computational standpoint. The state-of-the-art knowledge contained in the provisions of the new construction design standards stipulates that structural fire safety assessment can be made at three levels of detail: at the level of an individual component, part of the structure, or the entire load-bearing system. The approach based on simplified computational models does not reflect the actual action of the construction, or the actual behavior of load-bearing components in the case of a fire. These simplified models are mostly based on the assumptions which stipulate an even temperature distribution over the length and/or cross-section of the analyzed component, and usually lead to overestimated fire load-bearing capacity. Many researchers consider the adoption of such design solutions as not justified from the economic perspective. Fast urban development and technology advance poses new challenges for the actors in the building processes involving highly-complex structures (in particular in the case of high-rise buildings or buildings with complex geometric shapes) and increasingly often necessitates a design approach based on performance, known in the literature as "performance-based design". This approach to structural analysis, especially in the case of large and complex buildings and structures, is much more valid, but requires advanced computational techniques, both to determine temperature fields, which vary over time and space, as well as to determine the response of the structure to the simultaneous mechanical and thermal loads. To respond to the first challenge, computational software is used that draws on the Finite Volume Method and methodologies more broadly used in fluid dynamics. In the case of structural stress analysis, both at the ambient temperature and at fire-generated temperatures, Finite Element Method-based software is in most cases used. Increasingly more often, a need arises to conduct fully combined numerical studies which, in a single computational cycle, consider a two-way interaction between heat transport and the generated temperature fields on the one hand, and the deforming structure on the other. Such combined structural analysis, convenient from the user perspective, generates additional computational problems which, should be identified and at least partially solved.

1 DETAILED PROBLEM DESCRIPTION

Rapid development of fire safety engineering in recent years has stimulated the ever-more widespread interpretation of a fire as a special design case, where limit states of the structure's load bearing capacity are tested. Primary information on the subject is provided in a collection of standards recently published in Europe and being continuously adopted in the UE’s member states as national standards. These are the following Eurocodes: EN 1991-1-2: Eurocode 1, EN 1992-1-2: Eurocode 2, EN 1993-1-2: Eurocode 3, and EN 1994-1-2: Eurocode 4. All these standards consider computational analyses conducted at one of the three levels of detail: single structural component, a detached, representative section of the structure, or global, advanced analysis of the entire structure. The more advanced the level of analysis, the greater its reliability and accuracy of the results.

The primary purpose of analyzing civil engineering structures in the event of a fire is the need to anticipate the fire's impact on the building, and in particular to assess the resistance of the structure in the event of a fire, and its behavior at the stage of temperature rise or cooling. Results of such analysis may find a direct application in the design of passive and active fire safety systems, for safety assessment of new or existing structures, as well as a supplementation of, or addition to, usually very expensive experimental studies.

Although the procedures for the assessment of bearing capacity of a single component are relatively detailed in the provisions of the said standards, details on the analysis of detached sections of structural systems, and on global analysis of entire structures, exist only as general guidelines, providing conceptual assumptions on the methodology only, and do not lead to any clear-cut conclusions which could be used as final and unambiguous guidelines by the individual designer, and which could translate into their direct application in the design practice.

The methodology for the fire safety of civil structures adopted by the said standards is not perfect and results in economically unjustified, overestimated computations. Formulas used to verify the capacity of any individual member in fire situation are designed so that they are close to their corresponding formulas used to determine limit states of load bearing capacity in the event of the persistent design situations. Such verification procedures are valid in the case of single and simple structural elements, but are not fully adequate to reliably assess entire structural systems, as they do not allow a reliable estimation of the thermal and mechanical response of the structure exposed to a fire, leave out the impact of non-linear effects, which can be considered only if the advanced computational models are used. Further, what provokes certain doubts in the research community is the way thermal loads are treated in fire-related static and structural resistance analysis. The conventional design theory assumes that fire-generated loads are quantified using deterministic methods in the same way like in case of typical mechanical loads which come from the structural components’ self-weight (dead load), or such as finishing materials, snow and wind, are quantified. The resistance analysis is essentially limited to pure mechanical factors, and the effect of temperature is considered only indirectly, with the use of adjustment coefficients, which are to account for e.g. structural material resistance parameter degradation.

Next to analytical methods reflected by the design standards, fire-related analysis of civil structures or their components may also be conducted experimentally, or with the use of advanced computer

software, which is authorized based on delegations under the provisions of the relevant technical regulations.
The experimental method appears the most reliable of all, but it has a number of limitations, both of technical and economic nature. Micro-tests, using research furnaces can be applied only for individual structural components, whose dimensions are typically reduced. On the other way the natural-scale tests (the best-known of which are the Cardington and the Mokrsko tests) are very expensive are rarely conducted, although they represent a very valuable source of information on the actual behavior of structures during a fire, fully reflect the nature of physical and chemical phenomena and, when correctly conducted, provide important information on the core of the problem, (Wald at al, 2010). The complexity of the tests in which furnaces are used is also revealed by the some specific uncertainties, which can be partially reduced by e.g. increasing the accuracy of measurement techniques, or cannot be reduced owing to the structure of the research equipment. Disappointingly, in the case of experiments conducted during fires, numerous technical problems arise, related to correct determination of mechanical and thermal factors having a bearing on the tested element or the entire structure during the test, such as ensuring the actual support conditions that fully reflect the conditions which prevail for the actual structure, etc. Also the quantity of information obtained in the course of the experiment is markedly limited. An ideal case of repeating the identical thermal and mechanical conditions is not feasible, and, from the economic perspective, involves huge expenses. Similarly, the determination of certain values on completion of the test is not possible.
The application of numerical methods currently encounters serious difficulties, too. There are no popular and simple-to-use numerical tools available on the market, which would enable structural designers or experts at fire safety to conduct computational studies themselves, and, even worse, the most popular computer programs used for computer-assisted design are not compliant with the provisions of key European standards on construction design in the persistent design situation. Approximate analytical methods which could be used in the design practice provide results which often markedly depart from the actual conditions, in particular in the case of innovative and complex structural systems, or where new technologies and materials are used.
The concept of fire safety based on the natural fire approach provides a more realistic view of the phenomenon over time and space, as compared against other simplified methods based on the conventional fire concept. The design of new civil engineering structures, or the assessment of the existing load-bearing systems based on the performance parameters is one of the methods for approving structural safety stipulated by the European design standards. The need for the fire safety procedures to consider the individual nature of each building, the presence and type of ventilation systems used, passive and active protection systems, as well as other furnishings which have a direct impact on the fire-spread scenario, was formulated in the final conclusions from the European research project developed a few years ago, (Kumar at al, 2008).
The actual understanding of the behavior and actions of the structure in the event of a fire, as well as the assessment of its bearing capacity, may be fully reliable only when using the methods that apply the performance parameters, in particular: type of building, premises or designated fire zone, occupancy method (magnitude of fire load), shape and geometrical dimensions, possible hypothetical changes to the building arrangement, occupancy or furnishings over the life of the building, possible fire scenarios, considering the likelihood of a fire that follows a given scenario, etc. A thorough analysis and understanding of the nature of the phenomenon and a more precise analysis of the structure may demonstrate a higher level of structural safety than previously assessed based on the design standard procedures. From the economic perspective, this approach can lead to "greater value for money".
It is widely expected that the natural fire safety concept-based research tasks can be executed only with the use of advanced numerical tools which enable a simultaneous thermal and mechanical analysis of the structure, when both types of analysis are combined and coupled. The only tools that allow the conduct of such analyses at an advanced level (still beyond the reach of engineering practitioners) are specialty numerical programs (software) based on the Finite Volume Method or the Finite Element Method, which, however, require relatively deep theoretical knowledge. The first

of the two methods allows the adoption of time- and space-variable temperature field factor, and is used in computational fluid dynamics (CFD) (e.g. FDS, ANSYS-CFX, ANSYS-FLUENT, JASMINE, SOFIE). Phenomenon modeling with the use of Computational Fluid Dynamics has been, until recently, relatively often used in engineering and technical sciences to model heat transport processes. The last 20 years have seen a faster CFD development and application of its methods to model fire scenarios, which considerably spurred the development of a research discipline called "fire safety engineering". Until now, however, coordinated attempts at harnessing the potential of the method to assess the occurrence and magnitude of thermal impact on fire-struck buildings have been few and far between.

The FEM is widely used in the structural stress analysis, both for ambient and increased temperatures, and utilizes such software as ABAQUS, ANSYS, LS-DYNA, SAFIR, to name but a few. Fully relevant structural analysis requires a combination and coupling of thermal analysis with stress analysis, where, in a single computational cycle, a two-way interaction is considered between heat transport and the generated temperature fields on the one hand, and the deforming structure on the other, and where secondary static schemes and changes to the degrees of freedom, etc. are incorporated.

2 JUSTIFICATION OF THE PROPOSED RESEARCH METHODOLOGY

Numerical computational models used both for analysis of individual components and global analysis of complete load-bearing systems can be evaluated as fully reliable if they appropriately render the nature of the phenomena they study through, an appropriate selection of the type of analysis, integration methods, system geometry, variability of material resistance parameters, along with the change in temperature field, mechanical support and load conditions, as well as temperature distribution over time and space. Numerical methods (sometimes called as computer simulations) offer a vast potential and are successfully used in many other technical disciplines, such as fluid and solid mechanics, or structural mechanics.

What raises most serious doubts when it comes to the application of using numerical analyses is the question of how reliable the results are, and to what extent those complex numerical analyses are able to anticipate the actual behaviour of structures. It should be noted that formal evidence of reliability of numerical solutions using the FEM exist for linear problems only, where the response of the structure is in proportion to the cause (of the mechanical load). Owing to the nature of the phenomena, linear analyses are insufficient for structures struck by a fire. High temperatures are accompanied by essentially non-linear effects, the sources of which lie in the variable material properties, which change as the temperature rises, geometrical relations between deformations and movements, or which result from the very methods of load application and support, and from interactions between structural components.

3 CURRENT KNOWLEDGE AND EXISTING TECHNOLOGICAL SOLUTIONS

The descriptions of assumptions accompanying building fire safety assessment based on the natural fire safety concept can be found in final reports of the research projects conducted under "Natural Fire Safety Concept" programme, (Kumar at al, 2005 and 2008), whereas results of the attempts to incorporate the method's assumptions into structural design can be found in numerous conference papers, articles in specialty press and some guidebooks (e.g. Parkinson at al, 2010).

The body of literature on advanced computational models for fire-exposed structures is even smaller. There are no comprehensive research studies on integrated fire-exposed structure modeling, with the exception of isolated cases of articles published in recognized international magazines, and papers delivered at international fire safety conferences. Literature on the subject provides some information on the attempts to use numerical tools to model physical phenomena that structural components are impacted by when exposed to high temperatures. Divergent opinions are presented on the potential establishment of standardized procedures for review and validation which could be valid regardless of the case in question. There are also extremely skeptical arguments, which entirely reject the possibility of reliable validation of numerical models. In the case of varied and complex non-linear issues, the going practice in the engineering and technical circles, is to verify the correctness of given solutions based on "a posteriori" tests only, which essentially involve drawing conclusions based on results of the experiments. Over the last decade, thanks to the participation of researchers working in the US National Laboratories, numerous standards and guidelines emerged on the verification and validation of numerical models. The said standards do not provide detailed ways of working, but rather give an overview and terminology on the subject. The standards differ amongst themselves owing to different research foci and specificity of individual applications, and their non-universal nature precludes their application in any given research case.

4 INNOVATION OF THE DESCRIBED RESEARCH METHODOLOGY

The review of the literature on the subject reveals that the proposed research methodology for building fire safety assessment using advanced numerical tools based on the natural fire safety concept is a unique case as compared to the current knowledge on the subject and is in line with the overall research trends with respect to the area covered. Laying the foundations for improved structural safety using the natural fire safety concept represents a need identified by the construction industry, real estate maintenance market, building control bodies and the state fire services, and has recently been a research priority of numerous research centers, both in the European Union's member states and leading American centers, as well. A vast potential and repository of knowledge which still awaits and requires exploration resides in the proposed methodology and the research scope.

Advanced numerical models are primarily used in such areas of science and technology, where, owing to geometrical complexity of the tested structures, or complexity of the tested phenomena, it is not feasible to create ready-made, finite analytical solutions to apply in a given case, as well as in any field where, for various, and often economic reasons, experimenting is not feasible or insufficient. Complex numerical methods come in particularly useful in such computational cases, where geometrical non-linearity of the structures, or non-linearity of material parameters, need to be considered, or where certain values bear a non-linear relation to other values, which are also variable in a non-linear manner. This is the case with both thermal and mechanical response of steel structures exposed to fire temperatures. A natural reaction is that everything new and not thoroughly explored arouses certain controversies and doubts, and sometimes even denial. For instance, in the case of numerical analysis, a controversy is whether it is suitable for correct anticipation of outcomes of the physical problems in question. Adaptation of numerical models to meet the challenge of reflecting physical phenomena usually takes place upon their verification and validation, which is considered as the most objective method for reliability assessment of non-linear computer simulations. Validation should be preceded by verification, which seeks to check the correctness of the broadly understood discreting process that is transformation of the mathematical model into a numerical (discrete) model. Verification usually proceeds by comparing numerical computations with high-accuracy reference-standard analytical or numerical solutions. The core element, although not the only one, of verification conducted by the user is to check the consistency of the solution based on tests with the refined grid of finite elements. In turn, validation consists in comparing numerical solutions with experimental results in order to check whether the numerical and mathematical model correctly reflect the physical model, expressed by the way the structure behaves, by the type of phenomenon in question, etc.

An important role in the process of adapting a numerical model to the physical nature of the analyzed phenomena is played by correct mapping of properties of the materials used in the model, which are typically variable as the temperature rises.

5 PRACTICAL SIGNIFICANCE OF THE PROPOSED RESEARCH METHODOLOGY

The proposed numerical research method is of material importance from the perspective of fire safety engineering economy, design practice with emerging new challenges and owing to the social aspect of the problem. Regardless of leaders' political agendas, safety of the population and property is or should be a prioritized objective. From the economic standpoint, notably in a period of global crisis, it is essential that the proposed design solutions in this respect meet not only the essential requirements relating to structural safety, and safety of people and property as well as rescue teams, but are also cost-effective.

Fire safety has been a subject of research across the world already a dozen or so years ago, but it had not been not until the attacks on the World Trade Centre in New York that the research work accelerated considerably. Many countries across the world, including the European Union's member states, introduce new standards for building design, which necessitate incorporation of a specific case of fire into the structural safety assessment. The procedures adopted by the European standards are not equally proper for the assessment of various structural components, as they rely on certain simplified assumptions, resulting from the adopted validation methods, and from the attempt to align the adopted procedures with those that apply for the analysis of bearing capacity limit states in the persistent design situations. The conventional approach to fire-related structural safety assessment may be accepted only for uncomplicated cases, where load-bearing systems can be quantified with static computations. In the case of complex structural systems, however, the application of the latter may lead to considerable inaccuracies, resulting from the aggregation of non-linearity owing to changes in a number of vital construction material resistance parameters, generation of secondary effects, generation of large additional internal forces in the components following the limitation of deformation freedom caused by the temperature rise. Conventional, analytical methods for fire-related structural safety assessment are marked by significant inaccuracies resulting from the simplified methodology and, as a rule, lead to significant overestimations, whereas actual-scale experimental studies are extremely expensive and have a number of technical limitations.

The utilization of numerical tools to conduct analyses may lead to development of new, advanced and less expensive fire-related method of structural safety assessment based on "virtual testing" of the structure through numbers of numerical analyses, which represent a convenient alternative to the conventional experiment, and allow multiple computations for parameter-based studies or probabilistic simulations. Next to the assessment of new building safety, there is also a need for safety assessment of the existing structures marked by complex load-bearing systems. Since there are no consistent regulations and methodology for the assessment of the actual safety level, large discrepancies tend to occur between safety assessments of various buildings serving the same purpose, and marked by similar technical and geometrical parameters.

It is expected that the development and implementation of the proposed research methodology, notably among local corporations of engineers and architects, will change the present, conventional approach to design, marked by a task-based execution methods.

REFERENCES

Wald F., Kallerová P., Chlouba P., Sokol Z., Strejček M., Pospíšil J., Štroner M, Křemen T, Smítka V., Fire Test on an Administrative Building in Mokrsko. Summary of the Results from the Fire Experiment, 2010

Kumar S., Welch S., Miles S. D., Cajot L.-G., Haller M., Ojanguren M., Barco J., Hostikka S., Max U., Röhrle A., Natural Fire Safety Concept - The development and validation of a CFD-based engineering methodology for evaluating thermal action on steel and composite structures, European Commission Report EUR 21444 EN, 2005

Kumar S., Miles S., Welch S., Vassart O., Zhao B., Lemaire A D., Noordijk L. M., Fellinger J. H. H., Franssen J. M., Integrating advanced three-dimensional modeling methodologies for predicting thermo-mechanical behaviour of steel and composite structures subjected to natural fires, Research Fund for Coal and Steel, Contract No RFSR-CT-2003-00030, Final Report, 2008

Parkinson D. L., Kodur V., Sullivan P. D., Performance-Based Design of Structural Steel for Fire Condition. A Calculation Methodology, ASCE Manuals and Reports on Engineering Practice No. 114, 2009

NUMERICAL INVESTIGATION OF PLASTERBOARD SEPARTING ELEMENTS SUBJECTED TO FIRE

Hung Tran [a,b,c], Hervé Leborgne [b], Bin Zhao [c], Alain Millard [a,d]

[a] Ecole Normale Supérieur de Cachan, Avenue du président Wilson, 94235 Cachan Cedex, France
[b] Efectis France, Espace technologique – L'orme des merisiers – Immeuble Apollo, 91193 Saint-Aubin, France
[c] Centre Technique Industriel de la Construction Métallique (CTICM), Espace technologique – L'orme des merisiers – Immeuble Apollo, 91193 Saint-Aubin, France
[d] Commissariat à l'Energie Atomique (CEA), 91190 Gif-sur-Yvette, France

INTRODUCTION

Plasterboard separating partitions are widely used in the complexes, commercial and industrial buildings. They consist of regularly spaced steel studs fixed on both sides with the facings by screws. Each facing is composed of a single or a double layer of plasterboard. The plasterboard is reinforced with help of glass fibres to ensure its mechanical resistance. The steel studs which have in general a lipped C-section or I-section play the role of a supporting frame. The advantage of such type of partition is lightweight, easy for installation and its high fire resistance performance. The building fire regulations always require the partition elements to have sufficient fire resistance in terms of load-bearing stability, thermal insulation and integrity criteria. For the time being, the common way to validate the fire resistance of such type of partition is to conduct full-scale fire tests. However, these tests are often very expensive and many tests need to be carried out to finalize a new product. Moreover, the limited maximum height of the furnace (5 m at present) constitutes a problem for the manufacturers when they need to validate higher partition elements.
Therefore, the development of a numerical tool becomes very useful in order to carry out on one hand the preliminary design of new products as well as their optimization and on the other hand, to extend the application of experimental results to configurations different from those of the tests (change of dimensions, of components, of boundary conditions ...). Up to now, many researchers have studied and proposed heat transfer models to predict the temperature distribution in the plasterboard partitions but there was little research on their structural behaviour under fire conditions.
In consequence, this paper presents a study which led to the development of a numerical model capable of predicting the thermo-mechanical behaviour of plasterboard partitions when they are subjected to standard thermal action of EN 1363-1 on one side. The developed model allows treating not only non-linear heat transfer but also the non-linear structural behaviour of the plasterboard partition under thermal stresses taking into account the rigidities versus temperatures for the punctual screwing of plasterboards on the steel studs. The prediction of the numerical model was compared with experimental results in terms of temperature and out-of-plane displacements of the partition. The comparisons permit to show the capability of the numerical model to predict appropriately the fire behaviour of plasterboard partitions.

1 PROCEDURE FOR MODELLING OF PLASTERBOARD PARTITIONS

In order to model the fire behaviour of plasterboard partitions, the first step was the development of a thermal model in order to determine accurately the temperature distributions through the partition. These temperatures were used as input data for the thermo-mechanical model. One assumed a weak coupling of the two models.

1.1 Heat transfer model

The heat transfer numerical analysis was performed using the software Cast3M from which was determined the temperature distribution in the cross-section of the partition when it was exposed to the standard fire. The assumptions used for the thermal analysis were as follows:

- The temperature field was uniform throughout the height of the partition. This assumption leads to a 2D thermal model instead of a 3D one, which reduces largely the computation time;
- Double plasterboards layers constituting the two facings were considered as an equivalent homogeneous material. The contacts between the different elements of the partition were assumed to be perfect and the fixing system (screws) was not modelled.

Thermal analysis was conducted on a 2D model using 4-node linear elements (Q4). The facings were subdivided into several layers depending on the thickness of the facing. The thermo-physical properties of steel are those given in Eurocode 3 (Part 1-2), while those of the plasterboards were identified versus temperature by a characterization program and shown in fig. 1 and 2. It must be noted that the evolution of the water content, the phase change due to evaporation of water and mass transfer due to migration of water vapour in the material were implicitly taken into account via the variations of its properties.

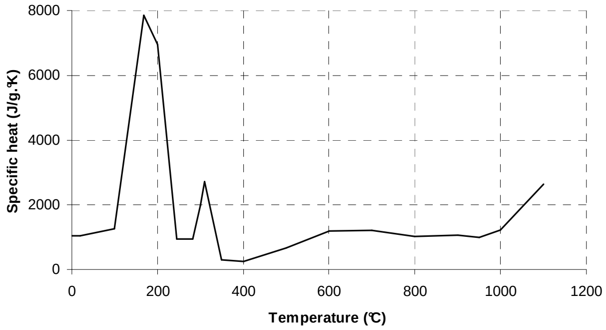


Fig. 1 Specific heat of plasterboard

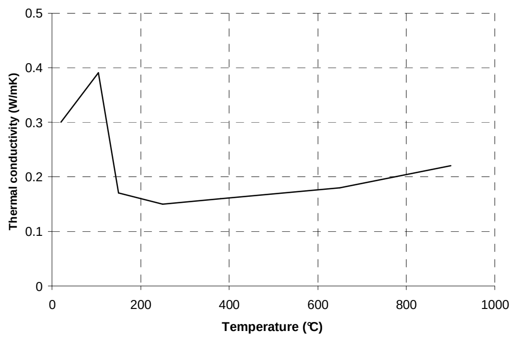


Fig. 2 Conductivity of plasterboard

In the model, one introduced a parameter t_{fall} in order to represent the time at which the exposed facing fell. One supposed that after this time, only the unexposed facing was still in place. This time, which was determined from the test, was used to define the two periods of calculation: period 1 (from 0 to t_{fall}) and period 2 (from t_{fall}). The boundary conditions and thermal loading for the two periods (example t_{fall} = 100 minutes) are presented in fig. 3 and 4.

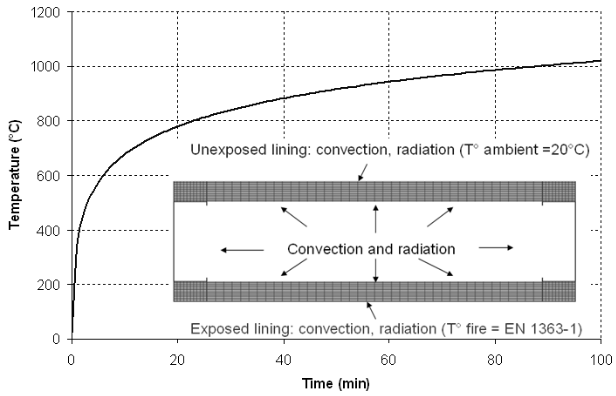


Fig. 3 Boundary conditions and thermal load for period 1

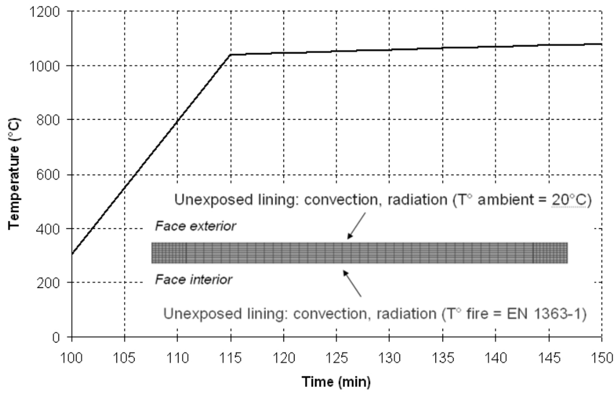


Fig. 4 Boundary conditions and thermal load for period 2

1.2 Thermo-mechanical model

The numerical modelling of structural response to fire action was carried out using the software Cast3M. The assumptions made for the thermo-mechanical analysis were as follows:

- The facings were considered as a continuous membrane without interruption along the full height (no horizontal joints between boards of a same layer). The double layers of plasterboard were considered as a equivalent homogeneous material;
- The exposed facing was supposed to remain in place during the simulation. Its loss was taken into account through the drastic reduction of its thermo-mechanical properties.

The plasterboard partition was modelled in 3D as an assembly of finite multi-layer shell and interface elements. The finite multi-layer shell elements with non-linear evolution of the temperature between the two faces were adequate. To do this, Cast3M uses the Kirchhoff theory (thin plate) for the formulation of its four-node shell element. The thermal expansion, geometric and material non-linearities were taken into account for all elements. Screws for connections were modelled by the interface element (JOINT3D) of Cast3M (Pegon et al, 2001) with an elastic behaviour. The Eurocode 3 (Part 1-2) steel properties were used. For plasterboard, Young's modulus and yield strength were determined by a 4-point bending test at different temperatures up to 400°C. When the temperature of the gypsum exceeds 400°C, i.e. when the cardboard of plasterboard is totally charred, its elastic modulus and yield strength can be assumed to decrease down to zero at 1200°C. It is noted that with a new product, the same characterization program must be performed to identify its thermo-physical and thermo-mechanical properties. Fig. 5 and 6 show the variation of the thermo-mechanical properties of plasterboard versus temperature:

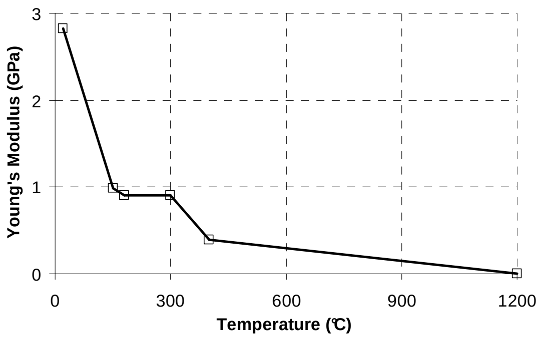


Fig. 5 Elastic modulus of plasterboard

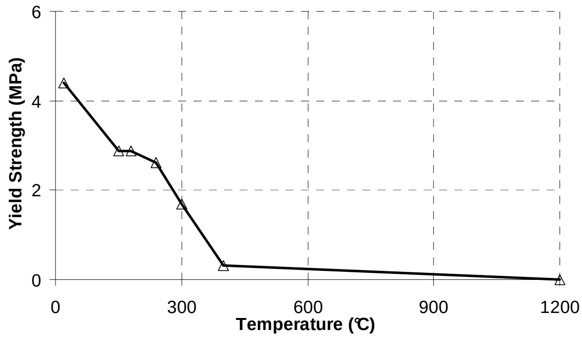


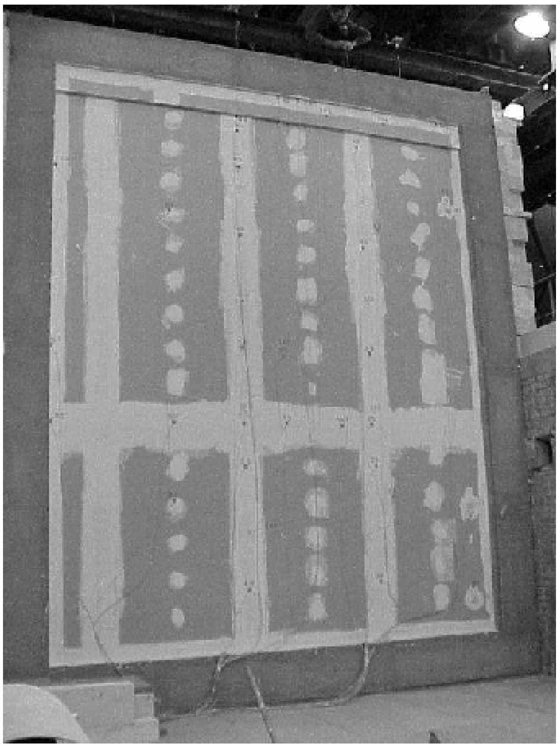
Fig. 6 Yield strength of plasterboard

2 VALIDATION OF DEVELOPED NUMERICAL MODEL

2.1 Experimental reference test

The reference fire test was conducted with a 5x4 m (h x w) partition. The frame of the partition was made of 8 lipped C-section steel studs 100 x 50 x 0.6 mm (h_{web} x w_{flange} x th) spaced at 600 mm. A 50 mm gap was provided at the top of the steel studs to allow free expansion. The facings were made of two 15 mm layers of fire quality plasterboards. The joints between boards of both layers were staggered. Boards of both layers were screwed together along horizontal joints. The facings were fixed on studs by screws at 300 mm centres and at 100 mm centres on upper and lower tracks. Fig. 7 shows the plasterboard partition.

This partition was heated on one side according to the standard fire curve EN 1363-1 and expressed by the following formula: $T_f = 345\log(8t+1) + T_{amb}$, where t is the time in minutes and T_{amb} is the ambient temperature in the furnace at 100 mm from the exposed side, in Celsius degrees. Temperatures measuring sections and out-of-plane displacements at different levels were distributed over the partition. On the studs, the thermocouples were placed on the exposed and unexposed flanges as well as at ¼ and ¾ height of the web. On both facings, thermocouples were installed at interface between double layers and on unexposed sides. The out-of-plane displacements were measured in 3 locations by displacement sensors placed on the unexposed side. Locations of the instrumentation are shown in Fig. 8.



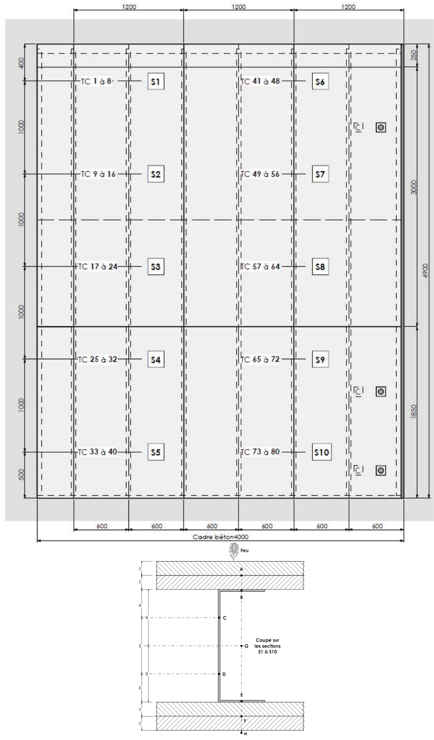


Fig. 7 Plasterboard partition

Fig. 8 Locations of the instrumentation

During the fire test, the partition bent towards the furnace, due to the expansion of the studs. One observed that the horizontal joints were a weak point of the partition. They tended to open earlier than the vertical joints. This opening allowed the passage of hot gases inside the cavity, which raised quickly the temperature of the steel studs and also increased considerably the deformation of the studs. At 100 min around, the first fall-down of parts of exposed facing occurred. This phenomenon was progressive and did not affect the whole exposed area of the partition at the same time. When the exposed facing fell, the steel studs were directly exposed to fire and their temperatures increased rapidly and quickly exceeded 900°C, a sufficient temperature to ensure that there was no more load-bearing capacity still present in any part of the studs. The studs eventually buckled and failed locally. After this moment, one observed the displacement of the partition towards the side opposite to the fire because of the contraction of the unexposed facing as it was still attached to the studs and both tracks. At the end of test, unexposed facing was still in place and one did not observe any rupture on it.

2.2 Calibration results

The developed numerical model is validated against a full-scale fire test of plasterboard partition which was described above. Before modelling the structural behaviour, a thermal analysis was conducted by using the thermal model to predict the temperature distribution within the cross-section of the partition. To simplify the model, it was reduced to a common part between two consecutive studs of 600 mm along width (see fig. 9). This choice led to consider as a closed internal volume between the two studs and adiabatic conditions on both lateral edges of the model. Fig. 10 shows a mesh of the model. Based on the experimental test, one assumed that the exposed facing fell at 100 minutes (t_{fall} = 100 minutes). The predictions from the thermal model were compared with experimental results at several points in the cross-section. The comparisons are presented in fig. 11 and 12. One can note a small difference between average experimental temperatures and those calculated.

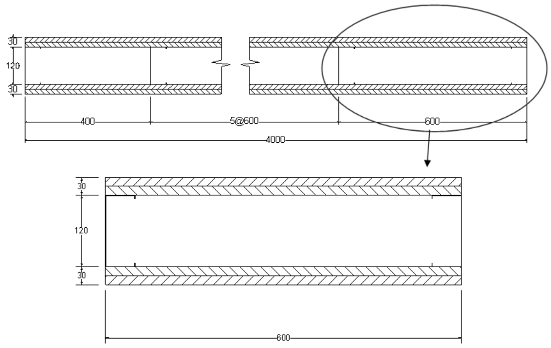


Fig. 9 Geometry for the thermal analysis

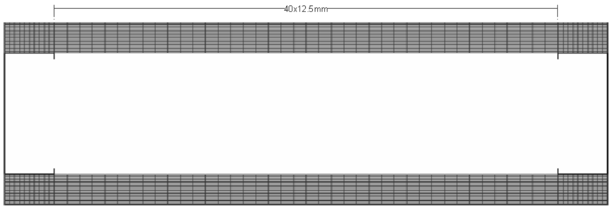


Fig. 10 Mesh of finite element model

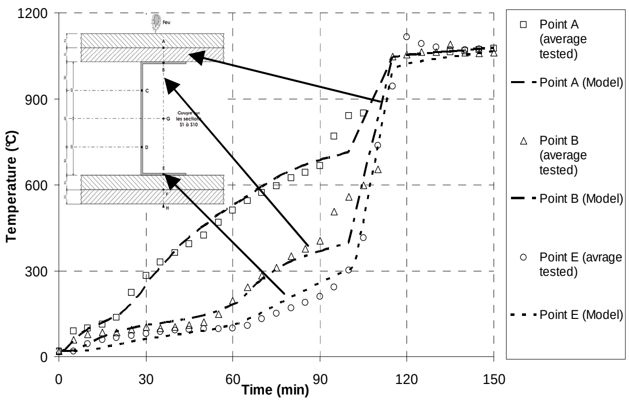


Fig. 11 Comparison between model and measure

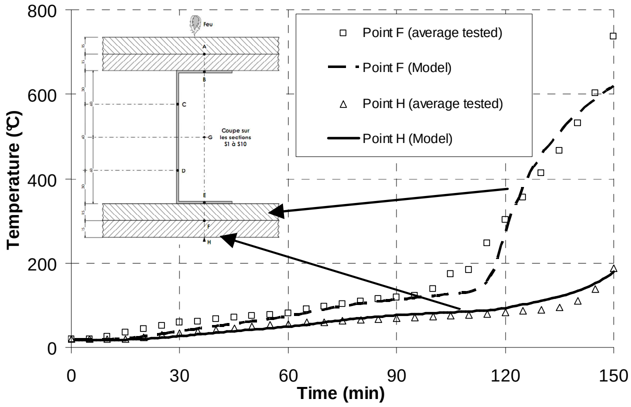


Fig. 12 Comparison between model and measure

Non-linear temperatures distributions calculated from the thermal analysis were incorporated into the temperature input file in order to simulate the structural behaviour of the partition. Fig.13 shows the finite-element mesh and boundary conditions considered for the thermo-mechanical model.

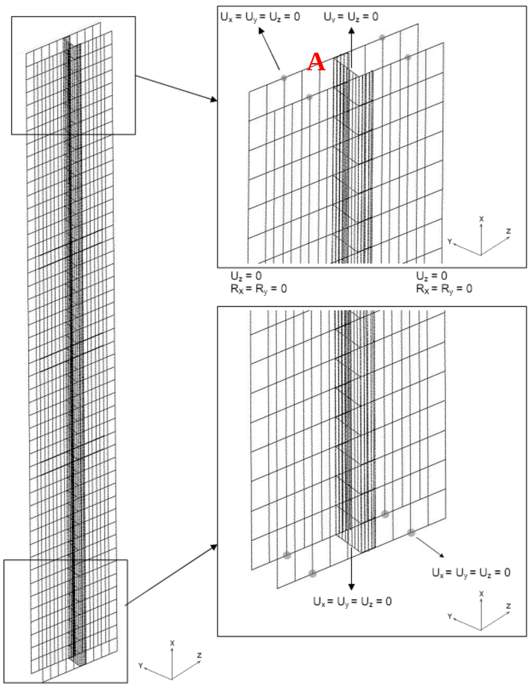


Fig. 13 Mesh and boundary condition

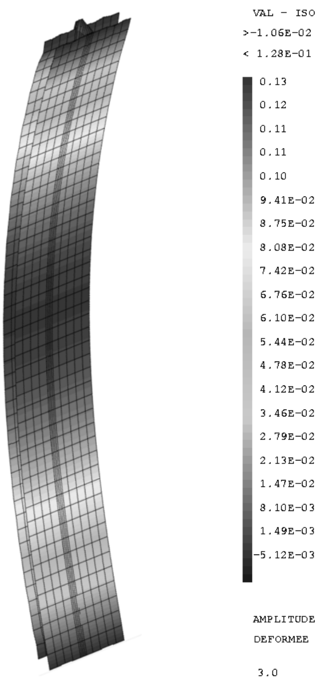


Fig. 14 Deformed shape at 107 minutes

Both longitudinal sides of facing, rotations about the X and Y axes and displacements in the Z axis were restrained. All displacements of eight points of the two facing were blocked in order to simulate the screwing on top and bottom tracks of the partition. For point A (see fig. 13) on the stud, displacements were restrained in the Y and Z directions, and allowed in the X direction.

The coefficient of thermal expansion assigned to the exposed facing was assumed without pre-drying. On the unexposed face, during the first 60 minutes, its temperatures were less than 100°C. It can therefore be assumed that the facing was subjected to a pre-drying for the first 60 minutes at least. The latter led to use different thermal expansion coefficients for the exposed and unexposed facings. Fig. 14 shows the deflection shape of the partition at 107 minutes (maximum deflection). Fig. 15 shows the comparison of the maximum deflection at mid-height of the partition between the prediction of the model and the test result. One can observe that the predictable displacement from the model was under-estimated of about 20 mm when compared to measured displacement. This difference might be caused by the deformation of the concrete frame used as supporting construction for the tested partition and not taken into account in the model.

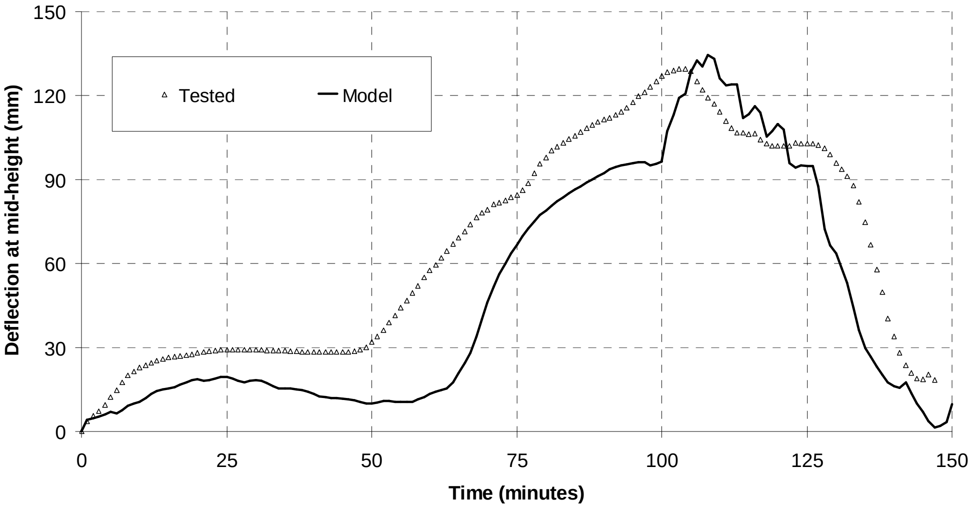


Fig. 15 Comparison of predicted deflection at mid-height of the partition with test result

3 CONCLUSIONS

In this paper, the development of thermal and thermo-mechanical models to simulate the plasterboard partition in fire has been described. The proposed model allows dealing with non-linear heat transfer as well as non-linear structural behaviour of the plasterboard partition from thermal stresses, taking into account specific rigidities versus temperatures for the punctual screwing of plasterboards on the steel studs. The predictions of model show a good correlation with the test results.

REFERENCES

Cast3M, Finite element software (version 2010), http://www-cast3m.cea.fr/

Cramer S. M., Friday O. M., White R. H., Sriprutkiat G., Mechanical Properties of Gypsum Board at elevated temperatures, Fire and materials 2003: 8th International Conference, 2003

Feng M., Wang Y. C., Davies J. M. Thermal performance of cold-formed thin-partitioned steel panel systems in fire. Fire Safety Journal 38, 365–394, 2003.

Chen C. J., Lee T. L., Jeng D. S. Finite element modeling for the mechanical behavior of dowel-type timber joints. Computers and Structures 81, 2731-2738, 2003.

Eurocode 3 – Part 1.2: Calculation of steel structures – Fire behaviour, 1993.

Pegon P, Pinto A.V., Géradin M., Numerical modelling of stone-block monumental structures, Computers and Structures 79, 2165-2181, 2001.

FIRE SIMULATION APPLICATION IN FIRE SAFETY DESIGN FOR TUNNEL STRUCTURES

Aleš Dudáček [a], Isabela Bradáčová [a], Petr Kučera [a]

[a] VŠB – Technical University of Ostrava, Faculty of Safety Engineering, Ostrava, Czech Republic

INTRODUCTION

The developing Trans-European Rail network made up of the high-speed and conventional rail networks must ensure safe, uninterrupted, efficient, high-quality and user-friendly transport for users as well as operators.

The rail transport network also includes tunnel structures. A possible occurrence of a tunnel incident may have considerable negative impacts on people and property.

The consequence of a tunnel incident is then traffic suspension of quite long duration together with direct endangering people in the tunnel and possible great material loss of and damage to cargo, rail vehicles and tunnel structure and its equipment.

When comparing the probability of an incident in a road tunnel with that of an incident in a railway tunnel it is possible to state that the probability of an incident in a railway tunnel is, owing to the nature and method of rail traffic control, substantially lower than in the case of road tunnels. With regard to a large number of people carried by passenger trains, a potential incident can however affect a markedly greater number of people under worse conditions of their evacuation (difficult quick leaving the coach, limited width of the escape walkway along the train, etc.).

On the basis of risk analysis, scenarios for specific incidents in tunnels, such as collision, derailing, fire, explosion, toxic gas release, and spontaneous evacuation have been developed.

To ensure interoperability and required level of safety, European regulations lay down essential requirements that must be fulfilled in the trans-European rail system:

- safety
- reliability and availability
- health
- environmental protection
- technical compatibility.

In the area of fire safety, the satisfaction of the essential requirements is shown by means of fire safety design on all levels of land-planning and design for tunnel construction. Minimum safety limits are set, e.g. in Commission Decision 2008/163/EC (TSI, 2007), and other requirements are stated in thematically relevant national standards (ČSN 73 7508, 2002).

However, the complexity of showing the fulfilment of generally formulated requirements calls, more and more frequently, for choosing approaches different from standard (conservative) approaches which do not provide any integrated view of the situation being solved, and which often are not able to consider it at all.

More and more experts all over the world are thus concerned with more general, mathematically describable models based on selected fire scenarios. Subsequently, efforts appear to describe, as trustworthy as possible, a fire, to model its development, smoke and combustion product spread, to express opacity, to deal with the transfer of heat through a structure, and to describe other important manifestations of the fire.

1 EVACUATION ANALYSIS

To verify the safe evacuation of people in case of fire on a train set in a railway tunnel, the following tasks were solved:

- development of temperatures during a fire in a tunnel
- smoke stratification during a fire in a tunnel
- evacuation time assessment.

2 MODELLING OF TEMPERATURE DEVELOPMENT AND SMOKE STRATIFICATION

In the design of escape walkways, the worst variant is usually considered, namely a fire on a coach in the vicinity of entry into the escape walkway (cross-passage or escape shaft). People thus can escape merely along unprotected escape walkways leading to the entry to a neighbouring cross-passage or escape shaft or to the exit through tunnel portals. In addition, in the identification of the most unfavourable site of the fire it is necessary, in the case of a longitudinally inclined tunnel tube, to consider the stack effect owing to which people escaping in the direction chosen incorrectly may be directly exposed to the products of combustion.

In the determination of temperature and smoke distributions in the tunnel, the program FDS (Fire Dynamics Simulator), current version 5 was used.

2.1 Geometry of a Tunnel Model

In Fig. 1 the part of the tunnel tube that will be used not only for modelling the spread of fire and combustion products but also for the assessment of evacuation of people is given. People can escape along an unprotected escape walkway towards the entry to an escape shaft situated 605 m from a portal. The model tunnel length is selected at *610 m*, width at *12 m* and the maximum height of the tunnel arch at about *8 m*. Fire simulation time is 20 minutes.

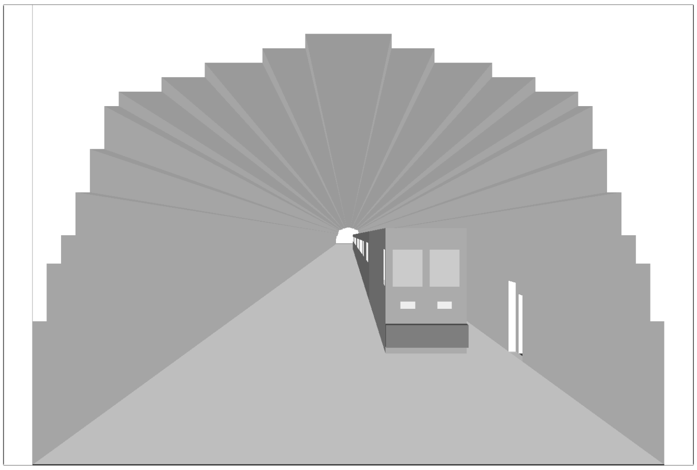


Fig. 1 Simplified cross-section of bi-directional tunnel tube

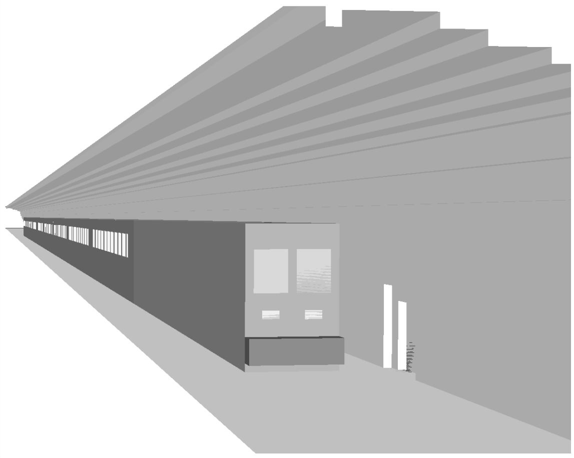


Fig. 2 Geometry of train set

2.2 Geometry of a Train Set

For simulation, a passenger train set consisting of eight coaches and a locomotive of the total length of 225 m is selected. The shape of train of this train set is simplified to one elongated rectangle divided into the locomotive and 8 passenger coaches, divided from each other by partitions. Along the sides of the coaches, window openings are distributed evenly. The front of the train is placed near the exit portal. A fire in the first coach is assumed.

2.3 Definition of a Fire

A fire is defined by means of heat release rate determined on the basis of data from the German railways (Deutsche Bahn AG, 2000). Values of passenger train fire development during the first 20 minutes grow gradually to 21 MW.

2.4 Graphic representation of results concerning the distributions of temperatures and smoke in the tunnel

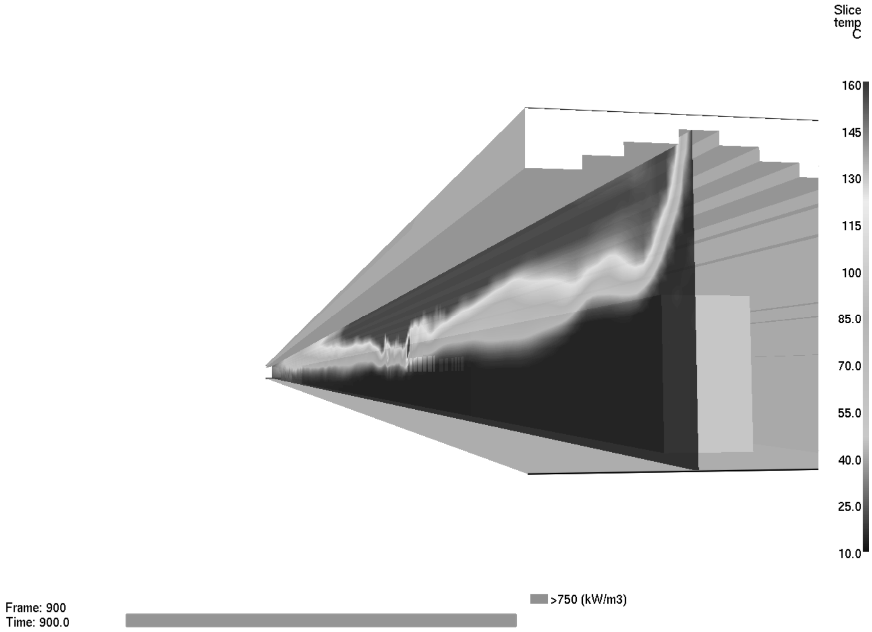


Fig. 3 Isotherm in the longitudinal cross-section of tunnel in the vicinity of fire seat in the 15th minute

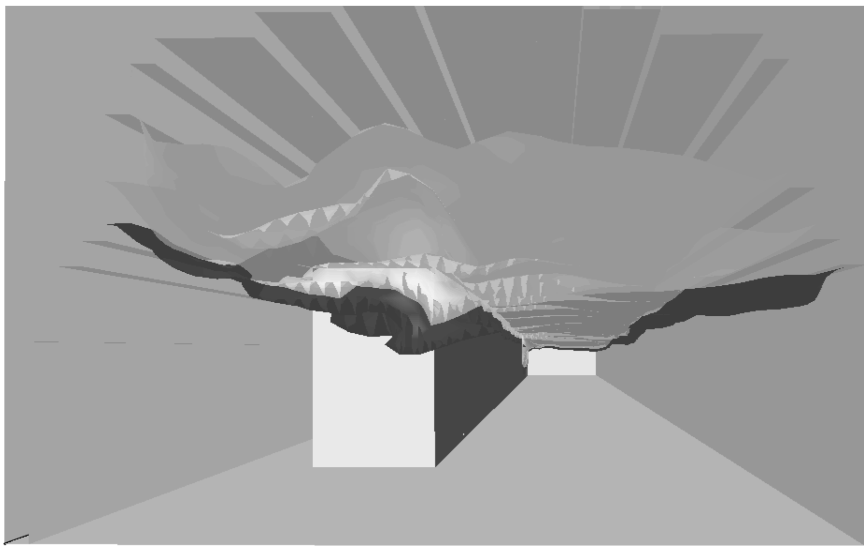


Fig. 4 Isotherms just behind the train set at 40 °C - 60 °C in the 15th minute

Note: These limit temperatures will not occur at heights less than 2.5 m on the walkway; they will not endanger in any way people escaping towards the entry to the escape shaft. Total evacuation time according to program SIMULEX is about 12 minutes.

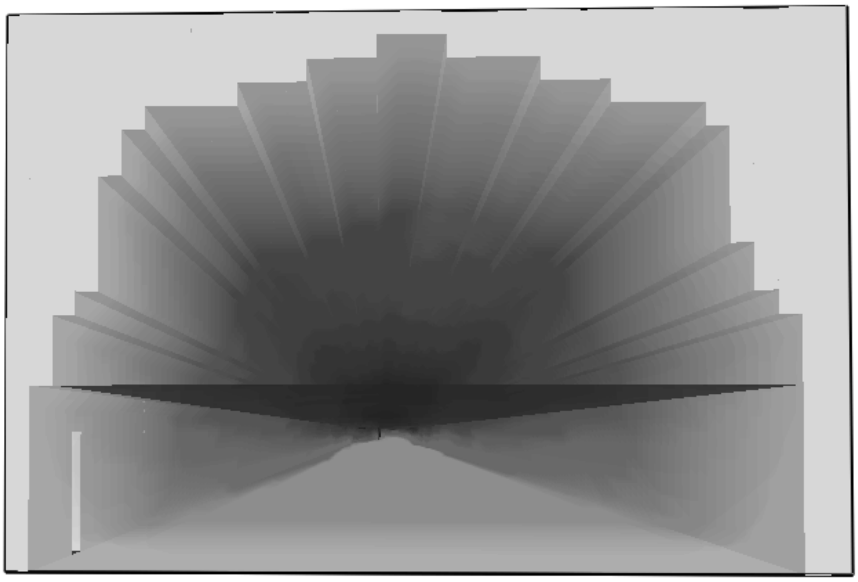


Fig. 5 Smoke layer at the entry to the escape shaft in the 12th minute
(line across the tunnel tube represents the 2.5 m height)

Note: Cooled smoke layer will diminish visibility on the escape walkway already at the end of evacuation; however, escaping people will not be endangered.

3 EVACUATION TIME ASSESSMENT

The aim of evacuation assessment using a SIMULEX model is to identify critical points for evacuation in the railway tunnel concerned, and to verify whether the designed escape walkways will enable people to leave through the tunnel escape shaft the space of affected tunnel tube in a sufficiently short time.

3.1 Geometry of the Tunnel Model and Preconditions for Evacuation

For the purpose of evacuation assessment, the evacuation of people from the train set, which considered a stop of the train set near the exit portal, in the point of smallest height, was selected. Evacuating people thus can escape in two directions along the unprotected escape walkway along the tunnel tube – towards the tunnel portal and towards the entry to the escape shaft. A distance between the portal and the entry to the escape shaft is 605 m, escape walkway width is 1.1 m and width of door to the tunnel shaft is 1.4 m. The estimated number of passengers in the set is 840.

The escape of people from the train set takes place in both the directions; that is why at setting calculations in the program SIMULEX, one half of the passengers of the train set (320 passengers) is designed to escape towards the portal and the other half of the passengers towards the entry to the escape shaft. People making for the portal will walk along the far escape walkway to avoid the effects of heat from the burning coach.

The walking speed of people is 1.0 m/s on a flat surface (corresponds to the dimensions of an average man). However this speed is not constant; it changes depending on the density of people at a certain place and time.

Before starting the evacuation of people, a 30 s time delay, which expresses the time required for informing the passengers by the train crew about the start of evacuation, is considered.

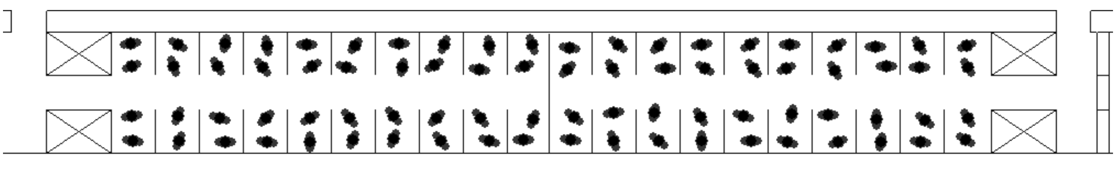
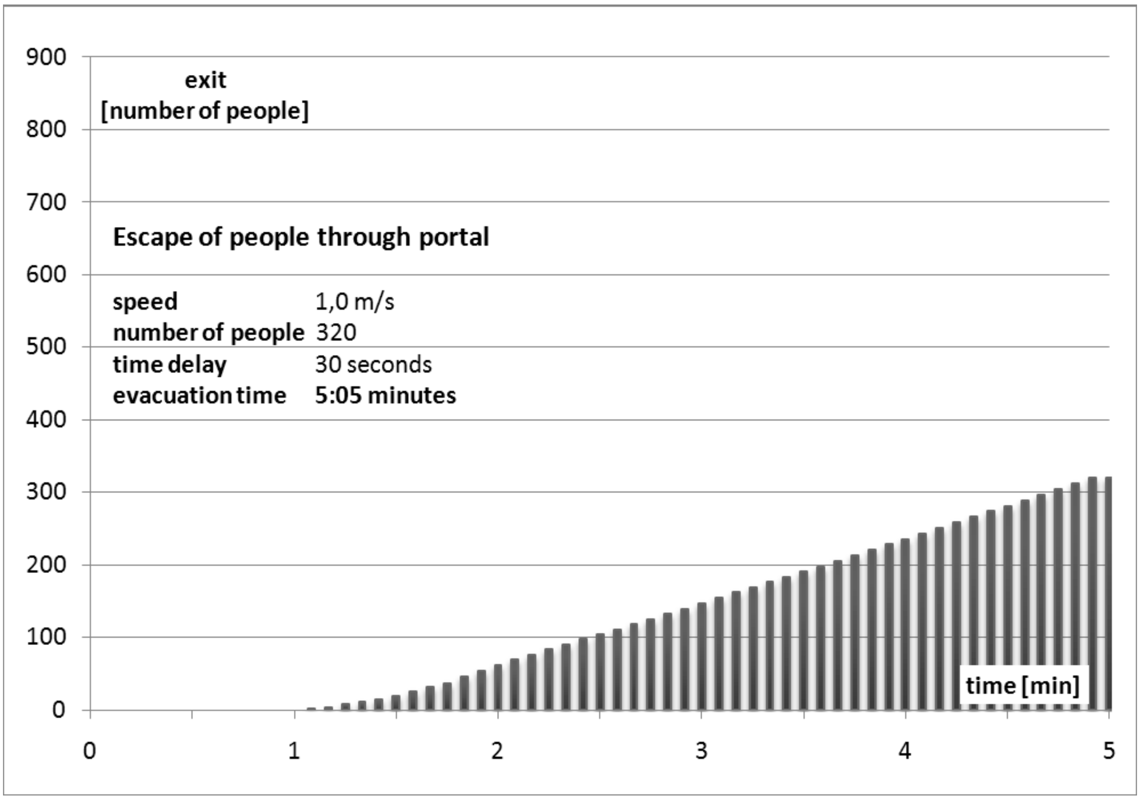


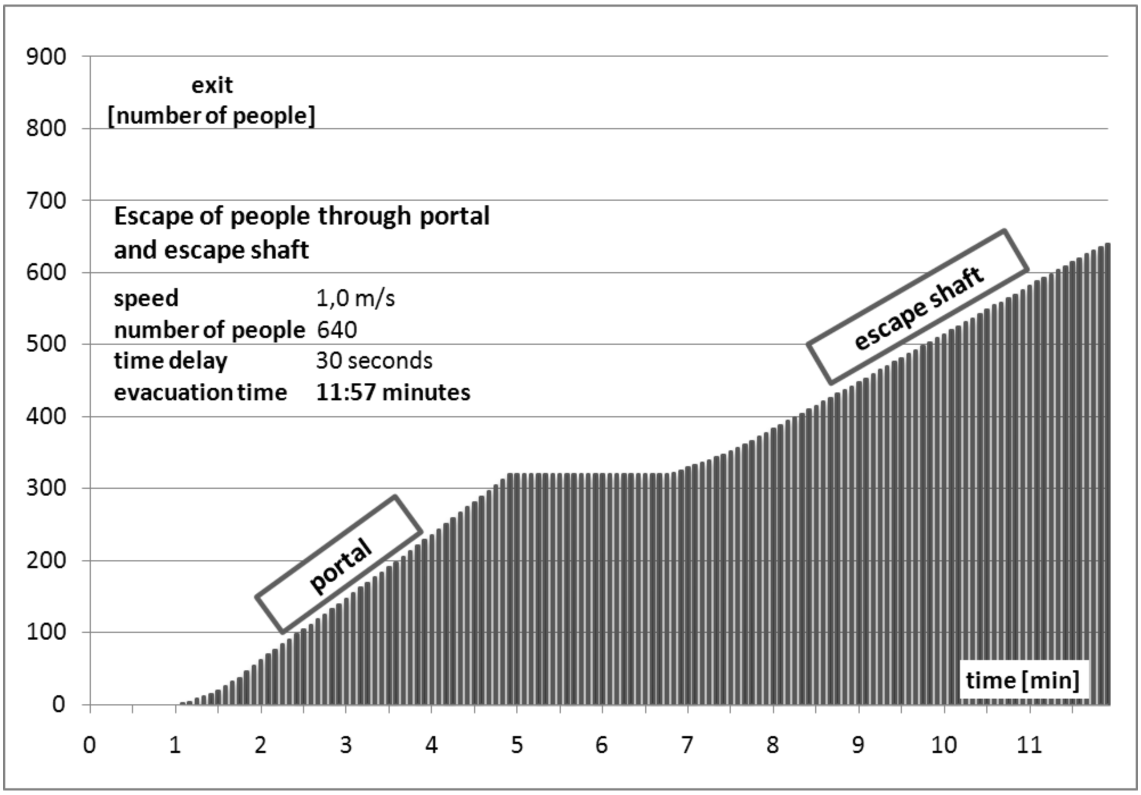
Fig. 6 Visualisation of placement of people in a coach

The placement of people in individual coaches is even. The space is divided longitudinally into two parts by an escape aisle. To direct the evacuation better an auxiliary dividing line is there in the median part of the coach, which guides evacuating people towards the nearer escape door.

3.2 Output Data



Graph 1 – Dependence of number of people escaping through the portal on time



Graph 2 Dependence of number of people escaping through the portal and the escape shaft on time

4 SUMMARY

By the program SIMULEX the time necessary for the evacuation of people was determined. The evacuation was carried out in two directions – towards the tunnel portal and towards the entry to the escape shaft. In spite of the fact that the division of evacuating people was made into two equal parts, the times of evacuation in individual directions were markedly different. The evacuation of people towards the tunnel portal took only 5 minutes, whereas the evacuation towards the entry to the escape shaft lasted 11:57 minutes and was thus the factor decisive of the determination of total evacuation time. The cause of the longer evacuation time was especially the distance-to-exit, i.e. the length of unprotected escape walkway.

Using the program FDS, the distribution of temperatures, the level of smoke layer and visibility during the fire on the train set in the tunnel were determined.

On the basis of simulation performed using the program FDS for the selected fire scenario (14 MW in the 15th minute at flow of 2 m/s) it is possible to state that the isotherm of 40 °C below the upper hot layer will not shift in the course of evacuation (12 minutes) below the height of 2.5 m above the escape walkway. Toxic combustion products will be accumulated mainly in the upper hot layer, it means above this isotherm. The isotherm of 40 °C was selected as acceptable temperature for the short-term stay of people during evacuation in the affected tunnel tube.

By means of program FDS visibility was also verified; the limit value of visibility is 10 m. In the course of evacuation, the visibility changed minimally, the value of 30 m decreased merely in the vicinity of the burning coach; otherwise, the visibility above the escape walkway during evacuation of people will not be less than 30 m.

5 CONCLUSION

By modelling using the computer programs it has been verified that people will leave the tunnel within 12 minutes and that in the course of evacuation they will not be endangered by high temperatures and smoke. Moreover, it has been verified that the visibility (opacity) along the walkways is satisfactory.

REFERENCES

TSI – Commission Decision 2008/163/EC concerning the technical specification of interoperability relating to safety in railway tunnels in the trans-European conventional and high-speed rail system 2007.

ČSN 73 7508: Železniční tunely, ČNI, září 2002.

Deutsche Bahn AG – Ergebnisbericht zur Beurteilung von Branden an Schienenfahrzeugen als Bemessungsbrände zur brandschutztechnischen Auslegung von oberirdischen Personenverkehrsanlagen der Deutschen Bahn AG; Deutsche Bahn AG, DB Station & Service, Fachstelle Brandschutz, Frankfurt am Main, September 2000.

ČSN 73 0802. Požární bezpečnost staveb. Nevýrobní objekty. ÚNMZ 2009.

SIMULEX 11. 1. 3 – User´s Guide, Copyright Integrated Environmental Solutions Ltd. 1998.

FIRE ANALYSIS OF RC PRECAST SEGMENTAL TUNNELS

Giovanna Lilliu [a], Alberto Meda [b]

[a] TNO DIANA BV, The Netherlands
[b] Universita' di Roma "Tor Vergata", Italy

INTRODUCTION

Tunnel construction with Tunnel Boring Machine (TBM) is today the most extensively adopted method both in soft soil and rock. The tunnel lining is usually made of precast reinforced concrete segment elements. Study of the tunnel behaviour under fire conditions is a relevant topic (Pichler et al. 2006, Savov et al. 2005, Ulm at al. 1999, Varley and Both 1999), also due to several accidents that have occurred in recent years.

The effect of fire exposure is not related to the material degradation solely, but the actions that can arise due to the thermal expansion can jeopardise structural safety. Regarding the material degradation, the main effect on this kind of structure is related to concrete spalling: in fact, precast segments are usually produced with high strength concrete, in order to speed the production process. However, spalling can be avoided with the use of a proper mix design or by adding polypropylene fibres. The other aspect is related to material degradation under elevated temperature: this effect should be considered coupled with the stresses that can occur due to the thermal expansion of the tunnel lining.

The analysis of a lining with the typical characteristics of a tunnel built in soft soil is considered in this paper. Analyses are performed with the FE program DIANA. In the analysis, excavation of the tunnel is modelled first in order to predict stresses in the lining due to the soil pressure and after this the fire exposure is considered. The nonlinear behaviour of the reinforced concrete lining is taken into account in order to simulate the actual behaviour.

The results show the importance of a complete analysis of the structure, the ability to consider the interaction with the soil and the degradation of the concrete lining.

1 TUNNEL MODELLING

The tunnel is modelled in 2D, under the assumption of plane strain. A construction phase analysis is performed in order to simulate the different construction phases and the exceptional event due to the fire.

In the first phase, only the soil is considered and stresses are initialized adopting the K0 procedure. Three layers of soil are present: the most superficial layer, 1 m deep, is modelled as linear elastic. The second layer, with a depth of 10 m, is made of non-cohesive material with Young's modulus E= 340 MPa and friction angle $\varphi = 35°$. Finally, the third layer is a cohesive soil with a depth of 50 m, E=400 MPa, cohesion c=3000 Pa and $\varphi = 27°$. The second and third layers of soil are modelled with a Mohr-Coulomb model (Fig. 1). In order to prevent volume expansion in case of unloading of the soil, the Young's moduli adopted in the analysis are those obtained in a triaxial test, during unloading.

In the second phase, excavation and installation of the tunnel are considered, with the tunnel segment being connected to the surrounding soil with interface elements following a friction Coulomb material model with tension cut-off. The thickness of the tunnel segment is 400 mm. Reinforcement is placed at the extrados and the intrados of the tunnel, with a cover of 50 mm.

Finally, effect of fire is considered on the inner part of the tunnel segment. The adopted fire curve (temperature versus time) is the RWS law (Efectis Nederland 2008), typical for hydrocarbure fire (Fig. 2). Fire is considered only on the tunnel lining above road level.

For modelling the effect of the fire, a partially-coupled thermo-mechanical analysis is performed. In the thermal analysis the effect of increasing temperature in the structure is modelled by solving the Fourier equation for heat transfer considering conduction, convection and radiation effects.

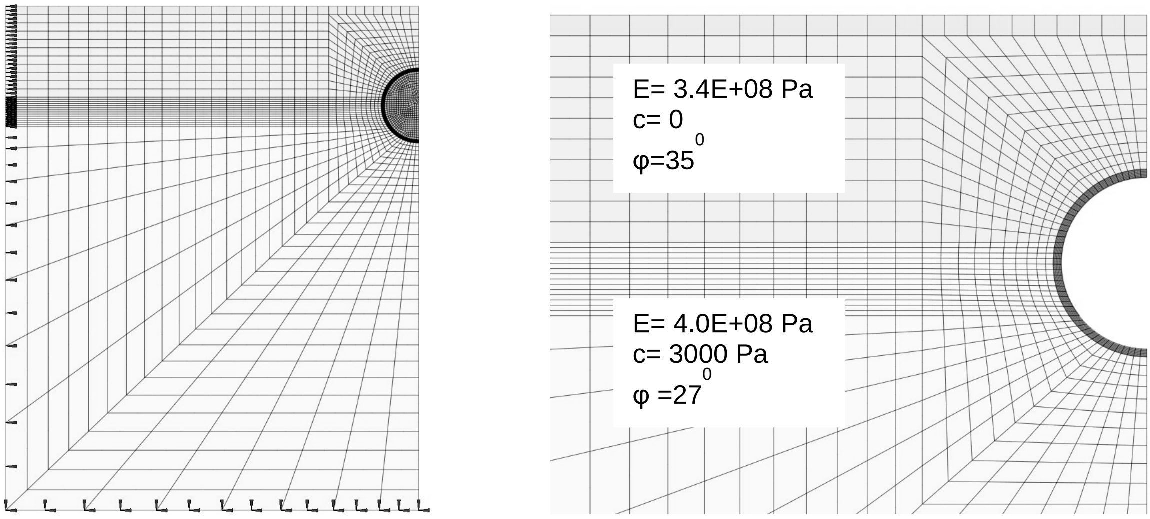


Fig. 1 Soil model.

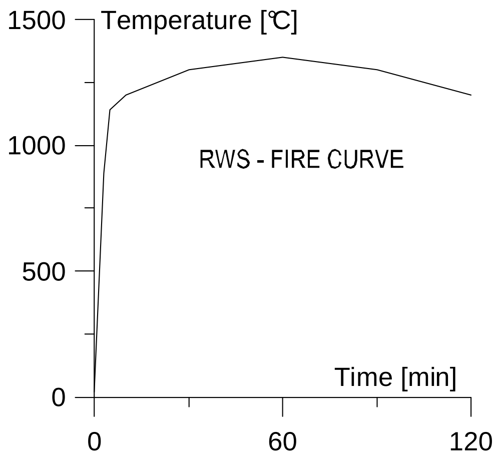


Fig. 2 RWS fire curve.

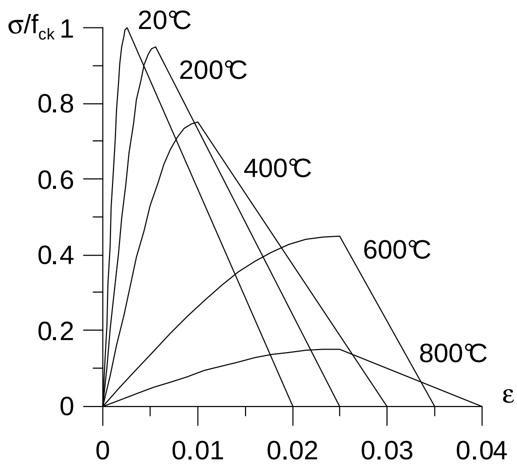


Fig. 3 Concrete in compression.

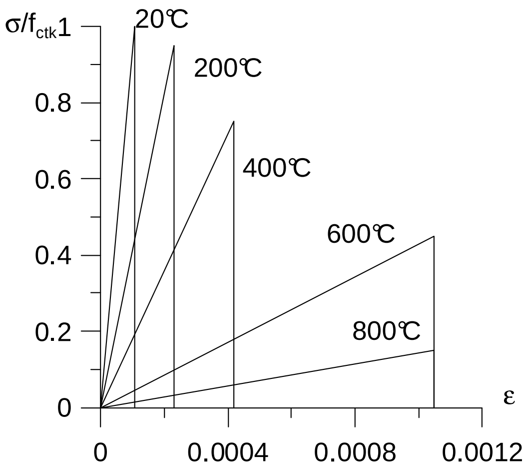


Fig. 4 Concrete in tension.

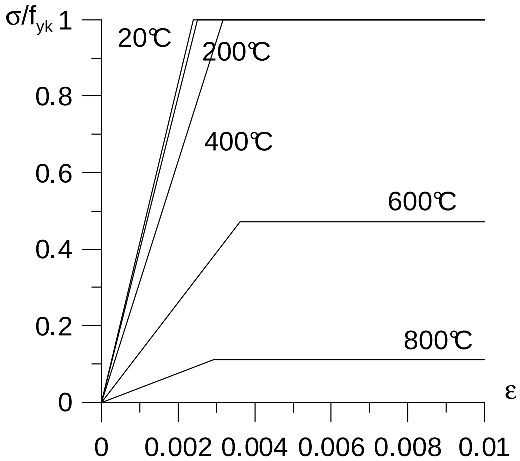


Fig. 5 Steel reinforcement.

In the stress analysis, the adopted constitutive material laws are those suggested in the Eurocode 2 (EN1992-1-2). The stress-strain curve for concrete in compression is shown in Figure 3. Eurocode 2 does not give any indication for concrete in tension. In this analysis it has been chosen to use an

elastic-brittle relationship for concrete in tension, with tensile strength and Young’s Modulus decaying with temperature as they do in compression (Fig. 4).
The steel reinforcement, considered embedded in the concrete elements, follows an elasto-plastic behaviour with yielding strength decaying with temperature as described in the Eurocode 2 (Fig. 5).

2 RESULTS

Figure 6 shows results at the analysis step corresponding to installation of the tunnel. On the left, the vertical stresses in the soil surrounding the tunnel is depicted, while the principal stresses in the lining are shown on the right. It can be observed that the lining is subjected to prevalent bending moment with part of the structure under tension. This is a typical situation of a tunnel excavated in soft soil near the surface.

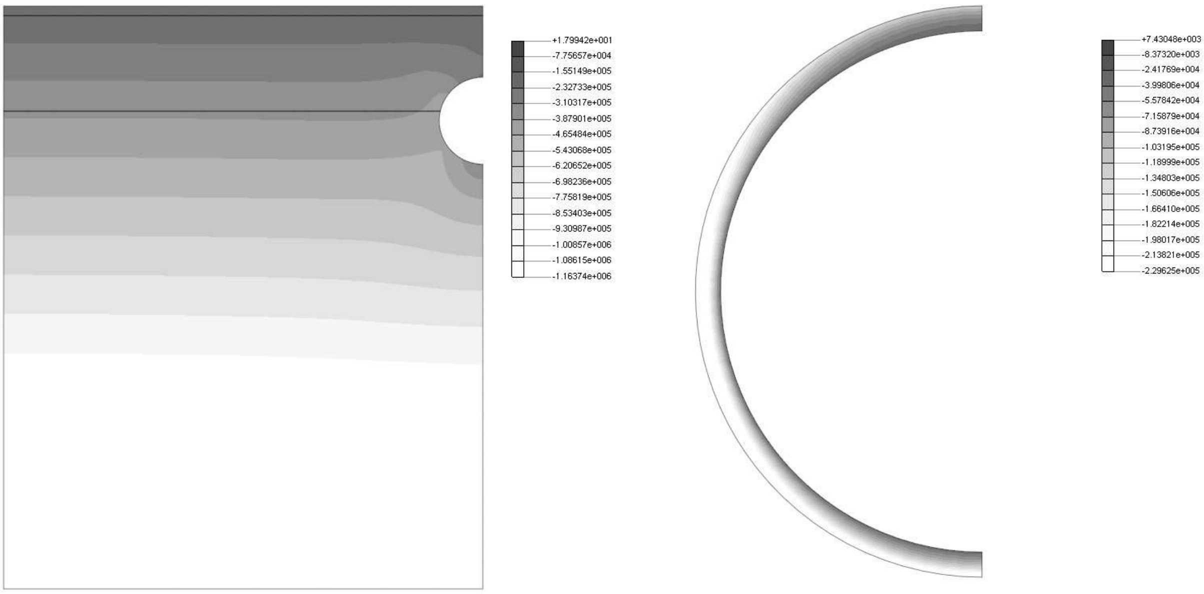


Fig. 6 Stresses in the soil and in the lining after installation of the tunnel (values in N/m^2).

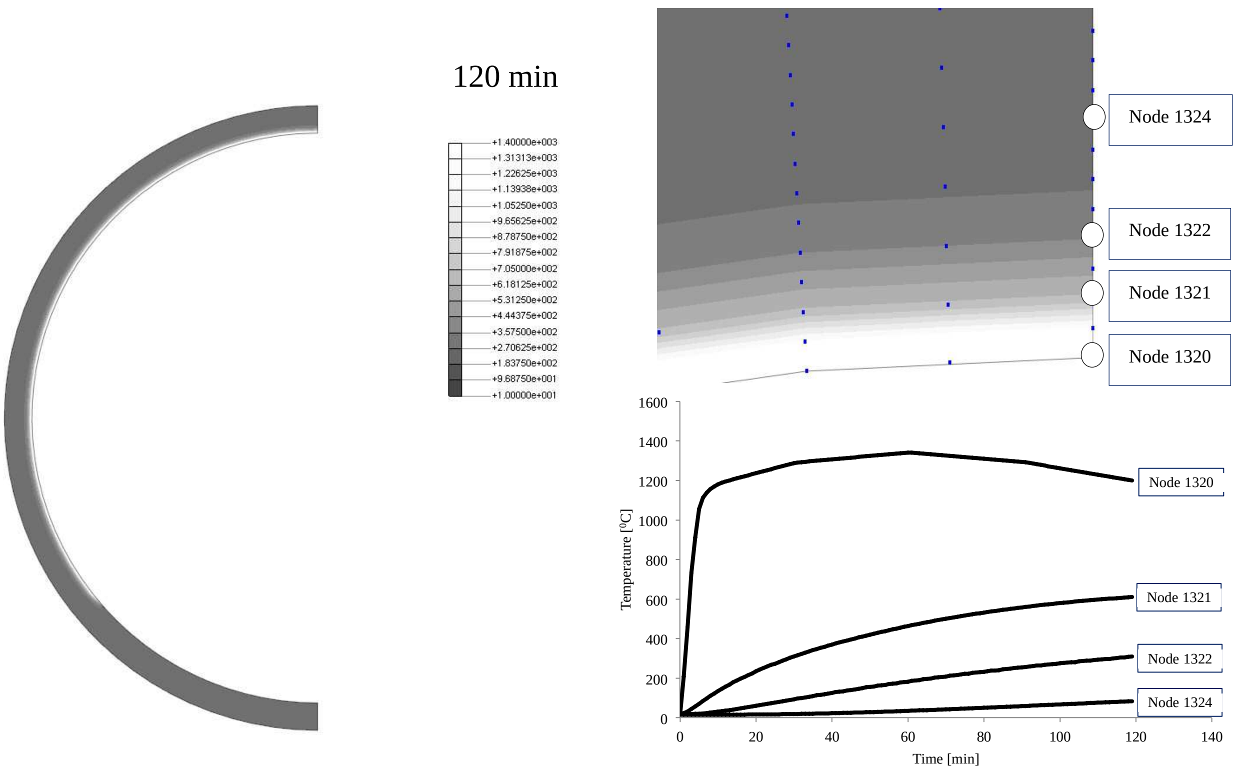


Fig. 7 Temperature development.

After this step the fire is considered. The thermal analysis is performed in steps of 1 minute up to 120 minutes of fire exposure. Figure 7 shows the temperature distribution in the lining after 60 minutes of fire exposure. It can be observed that after 120 minutes of fire exposure, only the first 60 mm of the lining reaches temperatures higher than 400 °C while temperatures of about 100 °C develop at a depth of 150 mm.
Figure 8 shows the crack propagation in the lining. The first cracks appear after 25 minutes of fire exposure. Several cracks are present after 30 minutes, with a concentration in the lower part of the lining at the road level. After 60 minutes all the lining is cracked.
Figure 9 shows the number of newly formed cracks versus time: it is evident that most of the cracks develop in the first 45 minutes of fire exposure.
It can be noticed that cracks are present at the outer part of the lining (Fig. 8) and they are caused by stresses induced by the differential thermal expansion. This aspect is important on the structural point of view: crack propagation causes a stiffness reduction in the lining, thus limiting the effect of the differential thermal expansion.

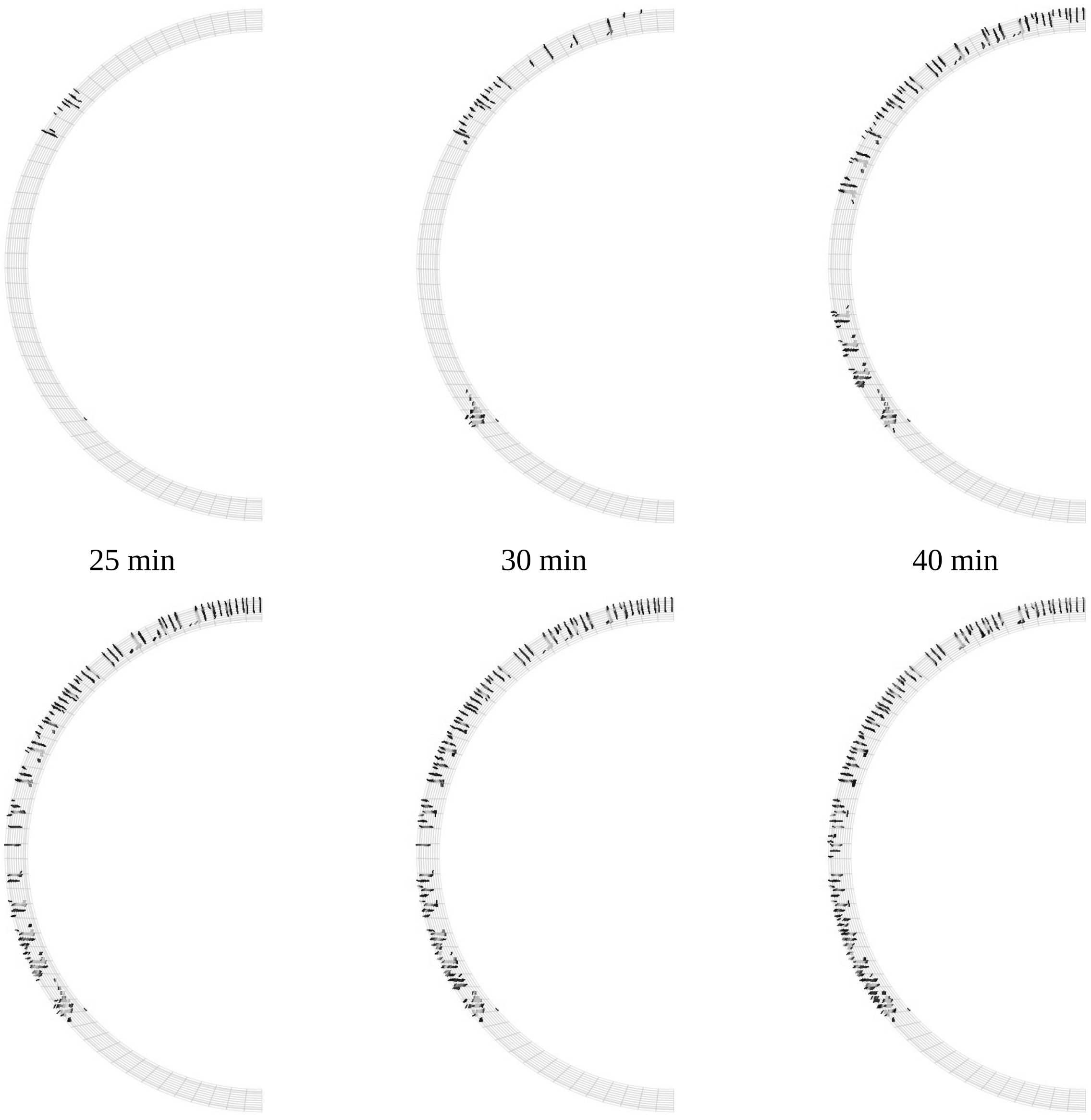


Fig. 8 Crack patterns during fire.

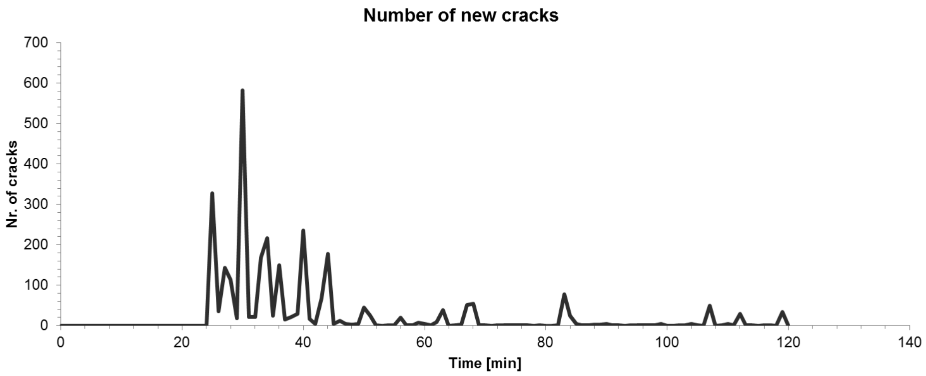
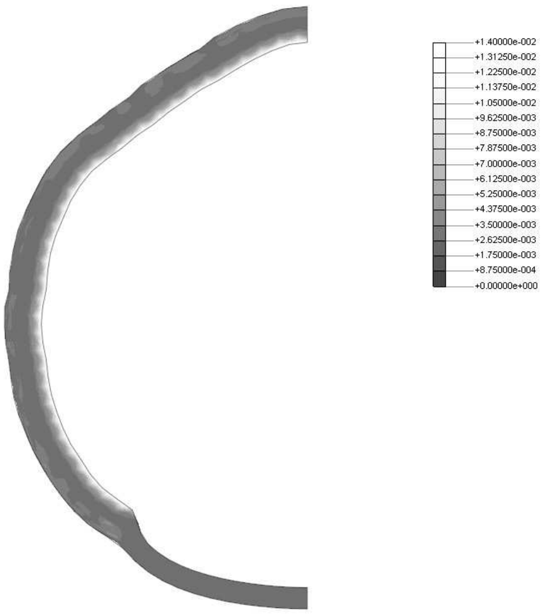


Fig 9 Crack development versus time.

At the inner part of the tunnel, only a small portion of the tunnel thickness appears to be damaged: this effect is not related to a high stress level but to the dramatic material degradation, due to the high temperatures that are reached. Figure 10 shows the equivalent plastic strain in the lining after 120 minutes fire duration.

The reinforcement remains elastic at the extrados whereas it yields at the intrados, due to the thermal degradation. Figure 11 shows the plastic strains in the reinforcements at the intrados, after 120 minutes fire exposure.



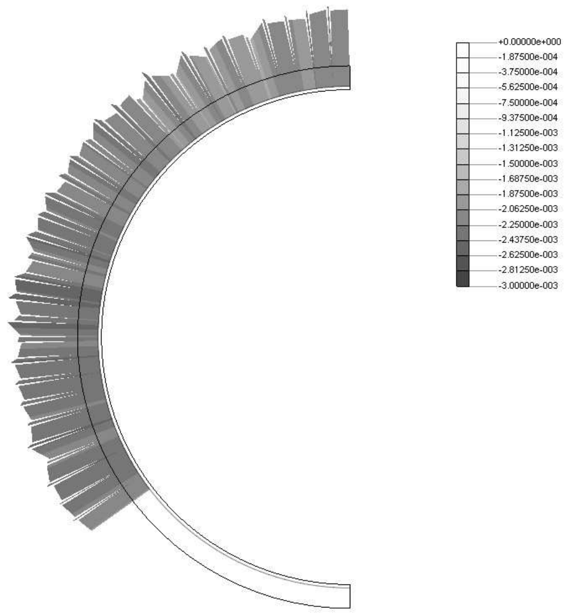


Fig. 10 Equivalent plastic strains.

Fig. 11. Plastic strains in the reinforcement.

RESULTS

This paper presents a procedure for analysing the behaviour of segmental lining tunnels in soft soil under fire. The analysis considers construction of the tunnel and application of the fire afterwards. A partially coupled thermo-stress analysis is performed for modelling effects of fire, that cause both degradation of the material and stresses due to differential thermal expansion.

The results show that the lining is subjected to extensive cracking that allows a reduction of the stiffness and, as a consequence, a reduction of the stresses induced by the thermal expansion. Furthermore, damage in the concrete at the inner part of the tunnel is very limited and caused by the material degradation induced by the fire. Despite cracking and yielding of reinforcements, structural stability of the tunnel is ensured up to 120 minutes of fire duration.

This type of analysis requires the use of an adequate non-linear finite element code, able to consider soil-structure interaction, non-linear material behaviour and cracking in concrete, and the coupling between the thermal and the mechanical problem.

REFERENCES

DIANA Users' Reference Manual, release 9.4, 2010.

Efectis Nederland, Fire testing procedure for concrete tunnel lining, 2008.

EN1992-1-2. Eurocode 2: Design of Concrete Structures - Part 1-2: General rules – Structural Fire Design. December 2004.

Pichler C., Lackner R., Mang H.A., Safety assessment of concrete tunnel linings under fire load, ASCE Journal of Structural Engineering. Vol. 132, No. 6, 2006.

Savov K., Lackner R., Mang H.A., Stability assessment of shallow tunnels subjected to fire load, Fire Safety Journal, Vol. 40, 2005.

Ulm F.J., Acker P., Levy M., The ''Chunnel'' Fire. II: analysis of concrete damage, ASCE Journal of Engineering Mechanics. Vol. 125, No. 3, 1999.

Varley, N., Both, C., Fire Protection of Concrete Linings in Tunnels, Concrete, May 1999.