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DETERMINATION OF FIRE INDUCED COLLAPSE MECHANISMS OF MULTI-STOREY STEEL FRAMED STRUCTURES

A Case Study

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ABSTRACT

Following the events of September 11th 2001, understanding the performance of multi-storey buildings during large-scale fires has assumed greater importance. These events have highlighted the possibility of large uncontrolled fires lasting for several hours (WTC-7). Owners of high-rise buildings are seeking assurance that integrity can be maintained during similar elevated temperature situations. This work is part of a much larger study to evaluate the performance of high-rise steel-framed structures in the event of large uncontrolled fires, using primarily a computational approach. Given a building and its operating conditions, different fire scenarios are established. The choice of scenarios is established on the basis of probability of occurrence and also as a function of damage potential. Computational fluid dynamics models are used to predict critical conditions within pre-determined areas of the building. Emphasis is given to establish a proper thermal boundary condition for the structural elements. A three dimensional numerical model of the structure provides the basis for a structural finite element analysis to be carried out under combined static and thermal loading. Full investigation of the temperatures and stresses generated on structural members due to the design fire chosen are considered. Particular attention to detail is given to those members that are thought likely to contribute to total collapse through localised failure. This is done by combining CFD codes with finite element models. This paper will present a selection of results from the aforementioned investigation, with particular emphasis on the conditions that cause total collapse for the chosen case study.

1 INTRODUCTION

From a fire safety perspective, the design of a building can be approached in two different ways. The first is for the building to comply with current prescriptive guidance in Building Codes/Regulations, and the second approach is a performance-based method based on achieving defined safety goals. Of importance, if a scenario such as the one of September 11th, 2001 needs to be considered as a possible event during the life of the building, performance-based design to achieve defined safety goals is the most robust approach.

Following the increased development of complex computing tools and a greater understanding of the response of structures in fires it is now possible to analyse results from events such as the World Trade Center collapse using a performance-based approach. In relation to achieving safety goals, this method can illustrate the need to understand how structures will behave in the event of a fire. Indeed for this purpose an adequate understanding of the nature of the possible event and the characteristic of the structure and its safety systems is necessary. This requires a detailed understanding of the fire conditions, the interactions between the fire and the structural elements and the sequence of the intervention and evacuation processes. Different methodologies and tools have been developed to study each of these aspects. This paper will concentrate on a methodology that can be used to assess the behaviour of structural elements and the fire. An application example from a fictitious building that resembles an existing multi-storey will be used to illustrate this methodology.

2 BACKGROUND

Traditional design of structures for fire is based on single element or sub-assembly testing in the standard furnace [1]. This approach allows uniform testing of structural elements and other fire resisting components. However, a defining behaviour of structural frames in fire is the response of the frame to restrained thermal expansion effects and resulting geometrically non-linear responses, which cannot be captured by simple unrestrained standard furnace tests on single beams, slabs or columns. Designing structures based on critical temperatures and failure of single elements as a result of material degradation does not address the forces and possible collapse mechanisms experienced by an integrated whole frame structure during a fire. As a direct result of the Cardington Frame fire tests, new understanding of the behaviour of structures in fire has been developed [2, 3]. This understanding has now been broadened so that structures in fire design have a real engineering basis and are not reliant on results from single element testing in the standard furnace. The type of analysis advocated includes detailed modelling of the time evolution of the structure as the temperature increases, this is generally done by means of a dynamic analysis using finite element codes [4, 5].

A landmark series of tests conducted at Cardington (UK) provided the opportunity to establish the validity of this approach. The main conclusions of the tests and the subsequent research projects were that composite framed structures possess reserves of strength by adopting large displacement configurations with catenary action in beams and tensile membrane behaviour in the slab [2, 6, 7]. Furthermore, for most of the duration before runaway failure (not observed at Cardington), thermal expansion and thermal bowing of the structural elements rather than material degradation or gravity loading govern the response to fire [3]. Large deflections were not a sign of instability in those tests and local buckling of beams helped thermal strains to move directly into deflections rather than cause high stress states in the structure. Near failure, gravity loads and strength will again become critical factors.

A thorough understanding of the whole frame response to fire as a result of such analyses allows structural detailing to be incorporated in the design and hence address the structural weaknesses as a result of fire. This leads to more robust fire resistance design based on quantified structural behaviour. This can also be integrated with other design loads, if necessary, such as impact damage.

3 APPLICATION

A typical multi-storey building has been analysed and a series of structural components have been identified as critical to the global stability of the building. In parallel, numerical modelling of the potential fires in the areas where these components are present have allowed identification of those components that have the potential to be exposed to the most severe fire conditions. In the process of identifying these areas, geometrical factors (ventilation and aspect ratio) and fuel loading have been considered. Detailed modelling of the potential fires within the compartment was conducted using FDS [8]. All the appropriate sensitivity analyses were conducted [9] but will not be presented here since the objective is the illustration of the possibility of structural failure and not the description of how to properly conduct such an analysis.

3.1 Fire modelling

For the purpose of this paper only a single critical area is identified. In general the analysis of more than one area might be necessary. The layout of the compartment is presented in Figure 1. Within this compartment there are three columns that represent the critical structural elements. In this particular case the three columns form part of a truss system from which emerges a single column that extends the entire height of the building. The large opening to the right of the compartment provides significant ventilation, while the particular geometry allows for concentration of the heat in the area immediately surrounding the columns. Consideration of the entire structure indicated that failure of this truss or the columns associated with it had the potential to lead to global failure of the building.

Gas-phase temperatures where obtained by assigning thermocouples (in the FDS fire model) in the locations adjacent to the columns of interest. The fuel used for these particular simulations is kerosene and the extent of the fuel coverage was varied seeking the worst-case conditions. The scenario intended to simulate the leakage of a fuel pipe in the event of an intentional fire. Fuel pipes are many times present in buildings where power generation units are available. An infinite supply of fuel was assumed. This is not realistic but is a good way to establish the time period to where structural integrity could be jeopardised. The simulation was conducted only for 600 seconds since the compartment temperature variation became negligible at this point. It is important to note that the evolution of the temperatures of the compartment walls, ceiling and floors was not monitored in detail. Heat feedback from the solid boundaries of the compartment will affect the burning rate and thus continuous evolution of the temperature will occur. Nevertheless, these changes were estimated to be small.

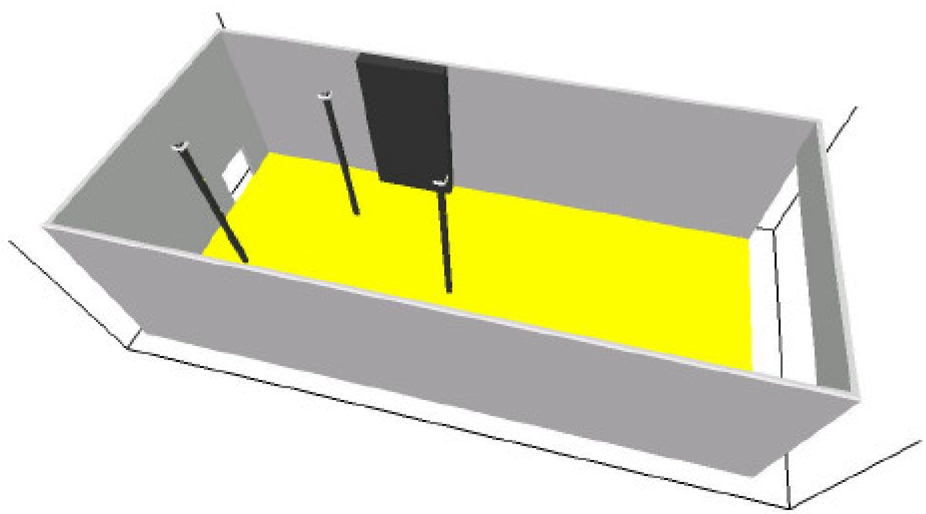

Fig. 1: Schematic of the compartment where critical fire conditions were observed.

The rapid evolution of the fire towards steady state conditions implied that for this particular fire scenario there was no need to establish a “Temperature vs. Time” curve. Instead it was important to establish the spatial evolution of the temperature fields. Temperatures where thus averaged over time.

It is important to note that the spatial distribution of the temperature could have a significant effect on the outcome of the structural model. It could be argued that an homogeneous compartment temperature that corresponds to the peak value observed could be a worst case scenario, nevertheless the dynamic behaviour of the structure is complex and mostly defined by stresses generated by restrained thermal expansion, thus the impact of cold boundaries to a heated structural element could result in a more critical scenario. For this reason the actual temperature distributions will be used for this study. A benchmark case, using peak temperatures homogeneously distributed, will have to be conducted as part of a sensitivity analysis.

Heat transfer from the gas-phase to the columns was conducted via a total heat transfer coefficient that included a linear component for radiation. The total heat transfer coefficient was defined as $45W/m^2K$ [10]. The evolution of the temperatures of the structures and trusses of interest was established via a simplified analysis. The columns were treated as a fin model and no thermal insulation was included. In the event of insulated structural elements the appropriate heat transfer model for the solid phase will have to be incorporated, this work is expected to be conducted by the authors at a future date.

The problem was divided into two different parts, a transient analysis and a steady-state analysis. It is important to note that this treatment is not necessary since a numerical code can resolve the transient problem completely. Nevertheless, for practical applications it is important to establish analytical methodologies that could enable the designer to concentrate on a parametric study of the different scenarios instead of investing all resources in a complete numerical analysis of the problem. For this particular application, the gain of a transient numerical analysis was deemed marginal, given the thermal inertia of the structural elements, thus a simple methodology to couple the numerical simulations of the gas-phase to those of the solid-phase was developed.

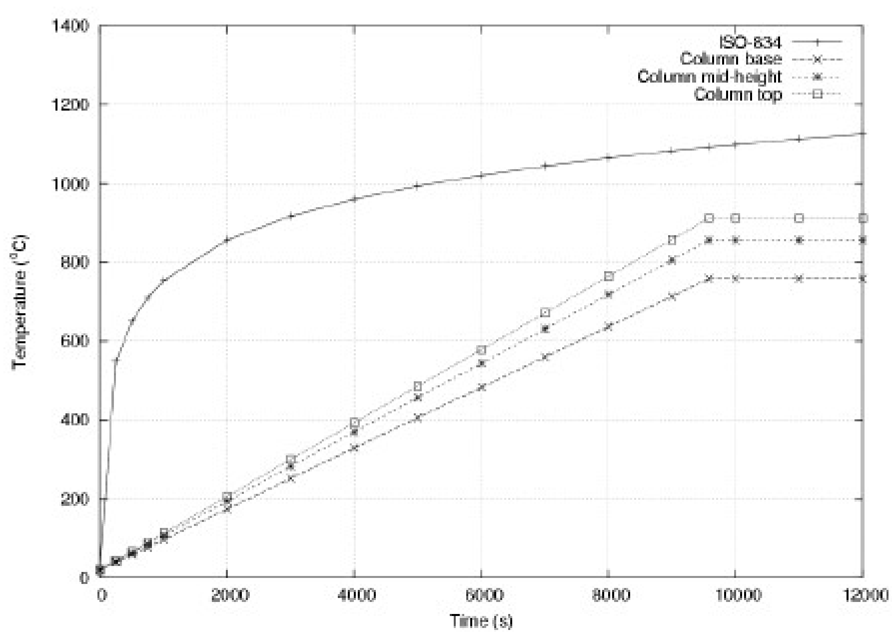

Fig. 2: Time dependent evolution of the temperature of the structural elements. Three cases are presented together with Standard ISO-834 "temperature-time" curve

The time to reach steady state conditions was established by conducting a lumped analysis of the cross section and defining a characteristic time to steady state conditions as the time to reach 90% of the steady state temperature. The results were compared with a numerical transient solution showing that the methodology adequately represented the time evolution of the columns and also that a linear temperature rise was an adequate representation of the temperature histories. Thus, the time dependent temperature evolution of the different steel structural elements was established in this fashion and a set of typical results is presented in Figure 2.

3.2 Structural modelling

Following the calculation of temperatures, structural behaviour is modelled using ABAQUS [11]. The particular methodology followed has been described elsewhere, thus will not be presented here [2, 3]. Figure 3 shows a diagrammatic representation of the modelled structural element. A two dimensional truss model will be presented here but it is important to note that a three dimensional analysis that included large part of the building also accompanied this study. These models incorporate the building geometry, appropriate I-shape member sizes and appropriate loading from floors above. The model incorporates steel and composite concrete. Boundary conditions are such that where out of plane beams would be expected to intersect, lateral movement is prohibited but vertical movement is permitted. Column bases, where appropriate are fully pinned. It is assumed that connections remain intact throughout the duration of the simulation.

Material properties for both steel and concrete were determined from the relevant Eurocodes [12, 13]. These include stress-strain behaviour at elevated temperatures, elastic behaviour and thermal expansion. Plastic tensile strength of concrete was also taken into account.

Figure 4 shows the collapsed truss following heating. Large vertical displacements can be seen together with a significant hinge in the upper left section of the diagonal truss member. In order to be able to understand the actual collapse mechanism it is important to consider individual parts of the structure with respect to time and temperature. Figure 3 highlights specific nodes that will be used in the following analysis while the evolution of vertical deflections for the selected nodes is presented in Figure 5. Initially there is a small negative deformation due to the static loading of the truss. As the temperature increases, thermal expansion effects appear and an upwards vertical deflection is observed. By comparing the temperatures shown on the x-axis of Figure 5 with the temperature vs. time plot of Figure 2, it can be seen that at approximately 4500 seconds the deflections start to revert their direction due to the degradation of steel strength due to heat, until approximately 6000 seconds when runaway failure is observed. For the purpose of this study this will be the definition of collapse. It is important to note that for this particular case, Figure 2 shows that for the time at which failure occurs, the top part of the truss has reached approximately 580^{o}C while the bottom part of the truss is at approximately 500^{o}C.

Collapse is further illustrated by considering section forces within the truss diagonal experiencing the hinge. Three nodes along its length displaying vertical force with temperature are shown in Figure 6. As the member is inclined at an angle, no vertical force would be expected at the hinge position (node 2109). This is seen to be the case until collapse is seen at about 580^{o}C when large tensile forces are generated. On the top of the diagonal member (node 7107) a column that extends the entire height of the building sits on top of the truss. Initially compressive forces are seen due to static loading, but the effect of thermal expansion generates small tensile forces. As the deflections revert, the forces appear to change back to compression until collapse is observed and extremely large tensile forces are produced as a result of the failure observed in Figure 4. Again a large temperature gradient is seen over the height the member at the time at which runaway forces are observed.

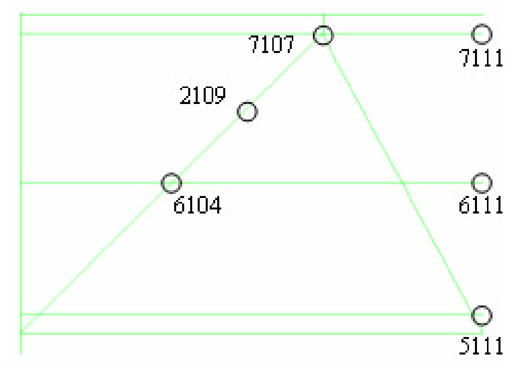

Fig. 3: Truss model with highlighted nodes

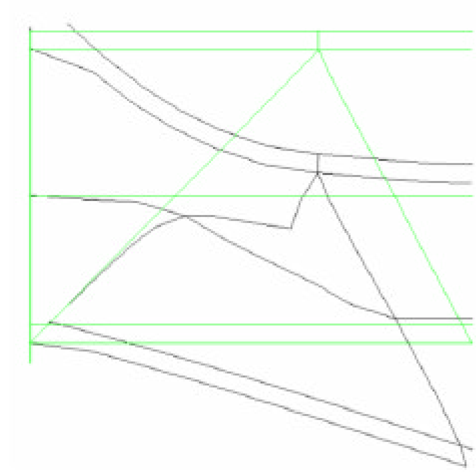

Fig. 4: Deflected truss shape

It should be stressed that in order to predict global failure, a three-dimensional representation of the structure is more accurate and can include the effects of redistribution at failure. This work is currently being undertaken by the authors. The analysis of specific two-dimensional key structural components provides only an indication of the type of global collapse mechanism that may be observed but still remains of valuable importance.

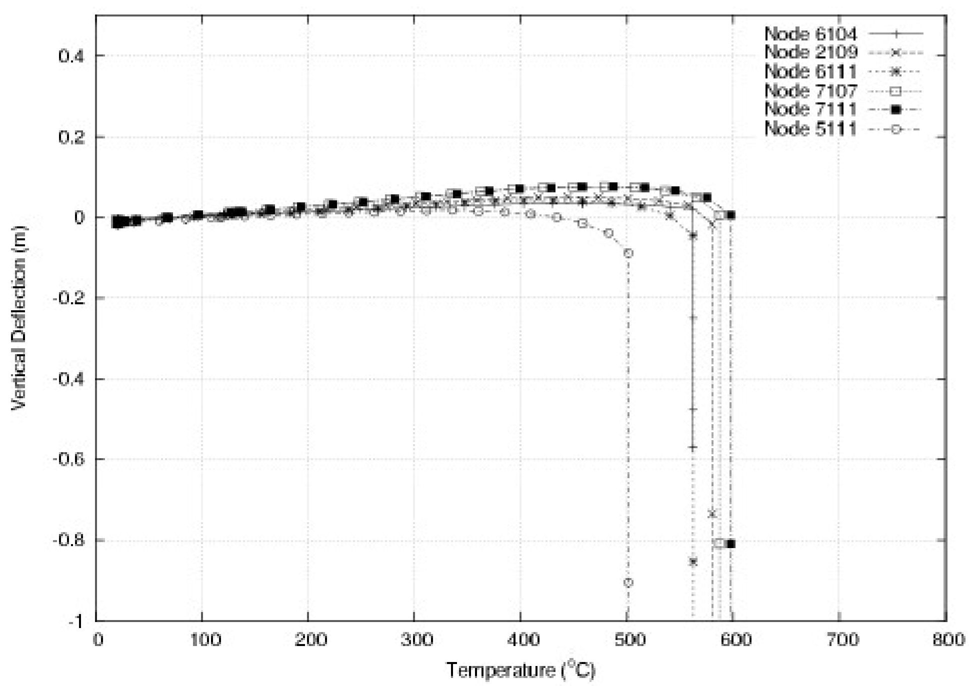

Fig. 5: Evolution of the vertical deformations for selected nodes on structural truss element. See Figure 3 for node positions.

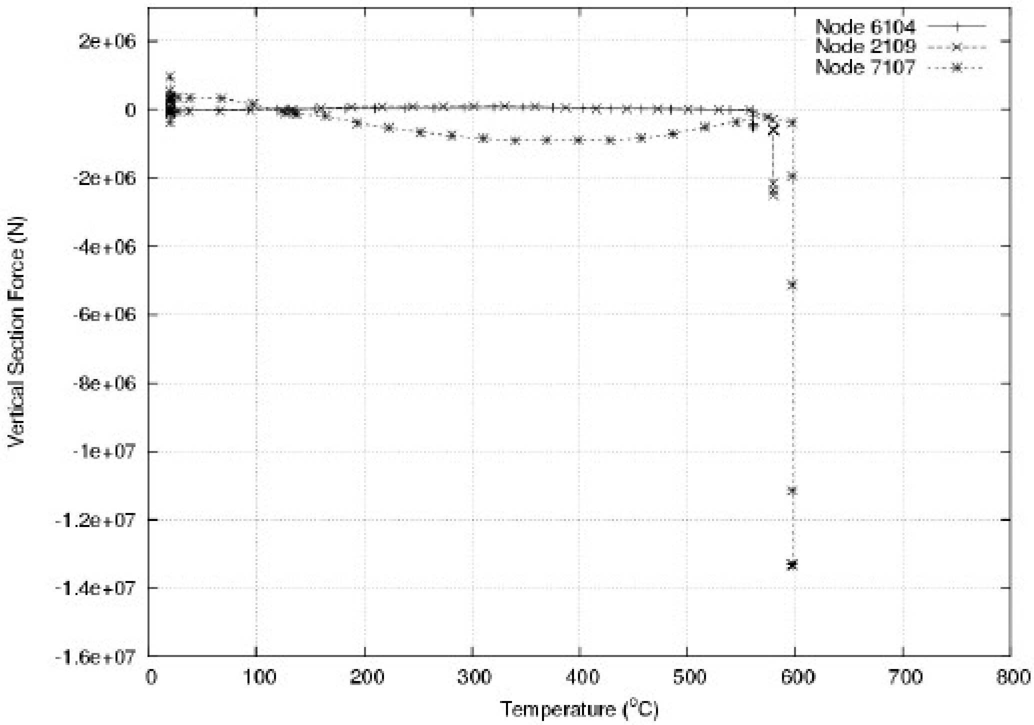

Fig. 6: Total vertical force for selected nodes on the structural element. See Figure 3 for node positions.

4 SUMMARY

A generic building model has been analysed to illustrate a methodology for the prediction of the behaviour in a fire before global structural failure. This paper has emphasised the importance of the use of detailed modelling of the fire that can only be achieved via computational fluid dynamics (CFD) and of the structure using finite element models. The dynamic behaviour of the structure coupled with the non-homogeneous distribution of the gas-phase temperatures requires an analysis that goes beyond the establishment of single average compartment temperatures of the use of test furnaces with characteristic Temperature vs. Time curves. The behaviour of structural members subject to a thermal gradient loading over their length with time has been demonstrated and the potential magnitude in temperature difference at failure along the member has been illustrated. To ultimately determine the behaviour of a structure in fire though, a three-dimensional model will provide the most accurate representation, allowing for full force redistribution at failure.

ACKNOWLEDGMENTS

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KEYWORDS

fire, multi-storey, CFD, finite element, collapse mechanisms, thermal boundary condition