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Model-based analysis of a concrete building subjected to fire

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ABSTRACT

A case study is presented of the Windsor Tower fire in Madrid, a mainly concrete-framed office block, which was involved in a major, multiple floor fire in February 2005. The performance of the structure is documented and examined using all available methods, including analysis of data on the fire and computer modelling of the fire and structure.

Holistic structural performance during a fire is more complex than the effects of fire upon individual members which make up the structure [1]. In concrete structures, fire conditions can have a variety of structural effects, both positive and negative [2], beyond the deterioration of the mechanical properties of the material. In order to properly understand the performance of concrete buildings, comprehensive models, which account for all relevant factors, are required. One of the key challenges is the limited data available from well-instrumented full-scale tests on whole concrete structures, compared to the large amount of data on the behaviour of individual concrete members and the large number of experiments carried out on steel and composite steel/concrete structures.

Here, a systematic approach is adopted for modelling, building up from fundamental, but simplified, analyses. The fire conditions have been computed using simplified analyses and Computational Fluid Dynamic (CFD) modelling. Similarly, the thermal and mechanical response of the structure is assessed using simplified analyses and Finite Element Method (FEM) modelling.

The CFD simulations provide the fire exposure histories of the structural members. The FEM calculations have examined the structural effects of changes to the concrete properties due to increased temperature. Extrapolation of results can examine redundancies within a building and their mobilisation to prevent collapse of the structure in the case of fire.

The paper gives an introduction to the Windsor Tower structure, the fire itself and further details of the methods used in modelling the fire and structural response. Model sizing, structural properties and failure modes are discussed along with initial analysis.

1 INTRODUCTION

The performance of a structure in fire is related to more than the effects of fire upon the individual members which make up the structure [1]. Redundancy within a structure can allow loads to be redistributed even when individual members fail. Members within a structure can expand when heated, which in some cases will have negative effects, for example by pushing columns out of alignment, but may also have positive effects, for example by initiating compressive membrane action in a concrete slab [2].

When examining the effects of fire on a real building using simulation tools, it can be difficult to validate the model due to the relative lack of experimental work carried out on whole structures, compared to the large amounts of data on the behaviour of individual members, derived mainly from fire resistance testing.

The majority of structural experiments have examined the effects of fire on steel and composite steel/concrete structures, such as, for example, the tests carried out on the Large Building Test Facility (LBTF) structure at Cardington. In order to examine the effects of realistic post-flashover fires on concrete structures it is useful to look at case studies of real fire incidents.

In this paper, we will examine the effects of a fire which occurred in the Windsor Tower in Madrid, a largely concrete-framed structure. Computational Fluid Dynamics (CFD) modelling of this fire has been carried out. This modelling aims to characterise the fire development, examining the mechanisms of fire spread within the building (including the compartment of origin, the initial fire floor, and between floors), and has been used as a basis for examining the fire exposures which were imposed on the structural members.

Finite Element Method (FEM) modelling is now being undertaken using the predicted heating regimes to provide the gas-phase boundary conditions for the structural members. The finite element model can then be used to calculate the impact of temperature at a given depth within the structure directly, or simplified one-dimensional models can be used and applied to the FEM models prior to structural modelling.

Structural modelling using FEM allows temperature-dependent material properties to be used, permitting examination of the structural response during a variety of phases of fire development.

2 THE WINDSOR TOWER

The Windsor Tower was built in 1978 and was at one time the tallest building in Madrid. The upper section of the building, above floor three, was a tower block containing offices and consisted of a concrete core, several interior concrete columns, exterior steel columns and a concrete waffle slab floor with permanent clay formwork (see Fig. 1). There were two "transfer structures" in the tower block, which were significantly stronger than the average floor, consisting of a series of concrete walls over the entire floorplan of the tower and a solid floor slab. These were located between the 3rd and 4th floors and between the 16th and 17th floors.

Below the third floor there was an entertainment and shopping complex, with a different structural layout, and underground parking on several levels; however this part of building was largely unaffected by the fire and did not sustain any structural damage.

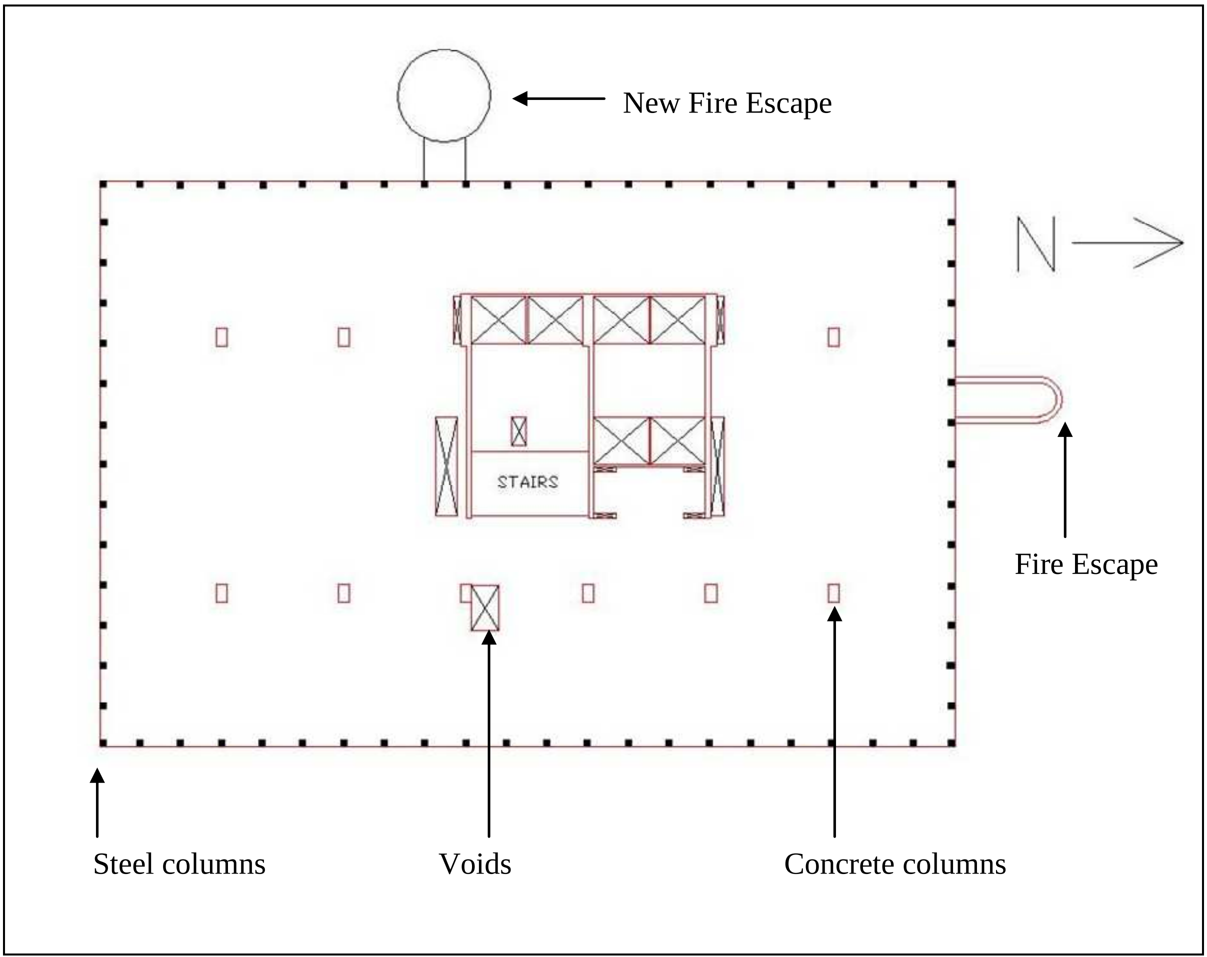

Figure 1 – Windsor Tower Floorplan above second transfer floor

The exterior steel columns were not, when the building was first constructed, fire protected. At the time of the fire a programme of fire protection upgrading was being undertaken, and the steel columns up to the second transfer floor had been protected, except on the 9th floor where two adjacent sides of the building remained unprotected due to the sequential nature of the upgrades to the building. An additional fire escape was also added to the west side of the building.

2.1 The Fire

On 12-13 February 2005, the Windsor Tower was involved in a major fire, of duration 18-20 hours. This broke out in an office on the 21st floor of the building, causing extensive structural damage to the upper floors of the building. Damage to the lower floors, below the second transfer floor, was considerably less. [3]

In the areas above the second transfer floor, large areas of the floor slab exterior to the core collapsed and several of the concrete columns were also damaged. It has been suggested that this was due to a failure of the concrete in the fire; however it must be born in mind that this is also the area of the building that had unprotected steelwork.

Significantly less damage occurred in the lower levels of the building. On the 9th floor, which had large areas of unprotected steelwork, the exterior steel columns were later found to have buckled severely, see Fig. 2 below. However, this part of the building did not collapse. Therefore it must be possible that the concrete structures and the fire protected steel on the floors above and below were capable of redistributing the loads applied to the weakened steel columns.

Figure 2 – Buckled steel columns on 9th floor, photograph from The Concrete Centre

3 METHODOLOGIES FOR APPLYING FIRE EXPOSURE TO STRUCTURE

When applying fire exposures to a structure, a number of methods are available [4], as described below. The simplest approach is to specify a uniform temperature for the surface of the structural elements. This temperature can either be estimated from observational or experimental data, taking into account for example the colour of the flames or the post-fire condition of the exposed materials. In the case of the Windsor Tower, video evidence is consistent with gas-phase temperatures reaching around 800-1000°C after flashover and the surface temperature of the enclosure boundaries may have risen towards this value. A variant of this approach is specifying a gas-phase temperature or incident heat flux and determining the surface temperature in the FEM code.

In the absence of measurements, model-based approaches can also be used in order to represent gas-phase temperatures [5]. This is a highly challenging process, since sufficient details of the fire development are required, but some simplifying assumptions can be made, e.g. with regards to the nature of the burning when the fire becomes ventilation-controlled. The advantage of these types of approaches is that the model can in principle provide spatially- and temporally-varying exposures which are consistent with the assumed fire development.

The CFD method provides prediction of heat fluxes, which can be used to define the boundary conditions for the structural members, rather than relying on gas-phase temperatures. Use of fluxes can provide a more accurate representation of the impact of the fire on the structure, as they describe remote heating - for example, where local gas temperatures are ambient, in inflow regions and lower layers, but where heat fluxes may nevertheless be high. Furthermore, the influence of different optical properties can be considered, since these can affect the amount of radiation reaching a structural element [6].

The simplest approach to RANS CFD modelling, and the most computationally efficient method, is to run a “steady state” simulation of the building during the flashover portion of the fire. During this time, the real conditions within the fire compartments are assumed to remain quasi-steady as the fire is ventilation-controlled. Another method is to use a transient simulation, in order to represent the fire development throughout the building. This will allow the examination of the fire exposure of individual members and also the duration for which they are subjected to different heating regimes from ignition to burnout, i.e. the full spatially- and temporally-resolved thermal exposures. Given that the period between ignition and flashover is generally relatively small compared to the period between flashover and burnout, a useful technique may be to adopt a transient model in order to assess the timescales of the fire and a steady-state model to calculate the fire exposure of the structural members. This allows the steady-state model to be created with a higher resolution than the transient state model, while at the same time allowing the exposure times of the structural members to be calculated.

An important consideration when using CFD models for full-scale structures is the resolution of the numerical grid, since this can have a large effect on the flame spread and fire development, together with the numerical timestep. In LES codes, such as FDS, these two factors are linked, since timesteps are selected automatically to maintain a numerical error estimate (the Courant-Friedrichs-Lewy (CFL) condition) within a specified limit, whilst in RANS codes, such as SOFIE, the timestep can be chosen independently. In either case it is suggested that the finest grid possible be used which will allow reasonable run-times, but this may require parallel computations; then issues of scalability and exchange of information over computational block boundaries may arise.

4 MODEL SIZING

In many cases, it will not be necessary or desirable to base the model geometry on the entirety of the building. A more simplified domain, consisting of either one or several floors, will provide a great deal of information that may be extrapolated to the rest of the building [7].

When examining fire spread models, including those for upward flame spread, the main area of interest is the length of time that individual members are subjected to a given fire exposure. While a model of the whole floor of the Windsor Tower will provide this information, if it is known that directional wind effects were insignificant, as in the current case, symmetry planes can be used to reduce the domain size. It is reasonable to assume that if the floor is divided into quarters along the axes of symmetry then each quarter of the floor will be affected in a similar way – time to flashover and burnout, for example. While on a full floorplan of the floor there may be differences between the fire loading of each quarter of the floorplan, and the floorplan may not be exactly bi-axially symmetrical, it must be considered that the fire loading will vary from floor to floor in any case and that trying to reproduce the exact fire loading on any individual floor and apply it to the entirety of the building will give a misleading impression that no assumptions have been required.

Using these methods it will be possible to reduce the area of building under consideration and therefore decrease the computational time required to run a given model.

When creating an FEM model of the structure, it may again not be advantageous to create 3-D models of the entire floor. Where possible, rather than using “block” type finite elements “beam”, “shell” or “membrane” components should be used, in order to reduce the number of elements necessary. The model can be further simplified by taking a cross-section of the floor and examining it as a beam, as is common practice in structural engineering.

While this approach will preclude accounting for much of the load redistribution that commonly takes place within a 3-D structure, it can be used to provide some simplified models to examine factors such as the failure of individual members within the structure. For example the effect of the failure of the steel edge columns without fire protection upon the floor slab can be examined by simplifying the floor slab as a beam spanning between the concrete core of the building and the steel column.

In an FEM model it is necessary to model multiple floors to gain an understanding of the performance of the whole structure in fire. In particular, it would normally be necessary to model one or more storeys above the fire so that any failure of the compression members which prevent the collapse of the floor immediately above the fire can be taken up by the columns above the fire compartment potentially acting in tension to prevent the collapse.

5 MODELLING OF CONCRETE IN FIRE

In FEM modelling of reinforced concrete as a structural member some of the properties of the material require careful consideration. Firstly, concrete is a good insulator, and therefore the thermal penetration into the member will be low in all but the longest duration fires. Secondly, reinforced concrete is a composite material consisting of both the concrete and the steel reinforcing bar (rebar) within it. The properties of both the concrete and the rebar are temperature dependent, and when loaded a reinforced concrete beam will behave first elastically, then plastically after yield.

5.1 Spalling of Concrete

When heated, concrete often undergoes spalling, a phenomena whereby concrete is ejected from the member [1]. In addition to the removal of some of the concrete section, this also has the negative effect of allowing the steel reinforcement within the concrete to be heated. This will reduce the tensile capacity of the steel in a beam or slab.

The modelling of detailed spalling behaviour is very complicated and challenging, however it must also be considered in the context of the macroscopic effects of spalling on an overall structure. While it will certainly have a large impact upon an individual structural member, this is not necessarily the case that when spalling takes place within a whole structure as it is unlikely to happen to every member at the same time. This allows the redundancies in the structure to support the loading from the spalled structural members.

Spalling on the base of a slab may have a significant effect, however. When a slab is heated and the concrete loses some of its strength the slab can often go into “tensile membrane action” where the load is supported on the tension steel. However, if spalling has taken place to a large degree, the tension steel may not have retained sufficient strength to allow this.
A possible method which prevents the collapse of a slab under these circumstances is compressive membrane action, as described by Bailey [2]. However, this mechanism only remains valid if the peak deflections are under c. 50% of the slab depth.

6 FAILURE MODES OF THE WINDSOR TOWER

A number of possible failure modes for the Windsor Tower exist, and the likelihood is that a variety of these occurred in practice. These are:

- Collapse of the floor slab due to the preceding failure of the steel edge columns. This will have forced the floor slab to act as a cantilever out from the core.
- Collapse of the floor slab between the core and the columns exterior to the core. This may have come about due to the large service voids just outside the core, which would have prevented the concrete floor carrying the large tensile load caused by a collapse of the floor exterior to the columns – as the area of floor able to carry this tensile force would have been greatly reduced.

The multi-floor nature of the fire is undoubtedly important, and it is likely that if the fire had only broken out on one floor then the majority of the building would have survived intact. Supporting evidence for this assertion is found in the robust behaviour around floor 9, where local failure of the steel columns was accommodated by load redistribution.

7 STRUCTURAL MODELLING OF THE WINDSOR TOWER

It is not a simple matter to model the behaviour of an entire structure, and it is often desirable to start out with individual elements of that structure and then increase the level of complexity of the model. In the case of the Windsor Tower, the first structural element to be modelled is the concrete floor slab. Due to the nature of the construction of the floor, it cannot simply be modelled as a membrane supported by beams. As a waffle slab, it behaves as a grillage of beams with an additional layer of concrete overlaid on top to provide a floor surface.

The Windsor Tower waffle slab uses a method of construction involving permanent clay formwork. The same waffle slab depth is used over the whole height of the tower block section of the building, though the reinforcement within the beam sections of the waffle varies throughout the building.

When placing the reinforcement for the slab, it is noted that the bottom reinforcement is set onto a channel within the clay formwork. This clay formwork is 10mm thick, and a further 10mm of concrete is used to bring the cover of the bottom steel up to 20mm, as discussed in the INTEMAC report [3], cf Fig. 3. For the purposes of initial modelling of the floor slab, rather than using a different material for the concrete and the clay formwork, the entire cover to the bottom steel has been idealised as concrete. This reduces the complexity of the model, and since this area of the concrete will most commonly be in tension the structural impact of this assumption will be minimal.

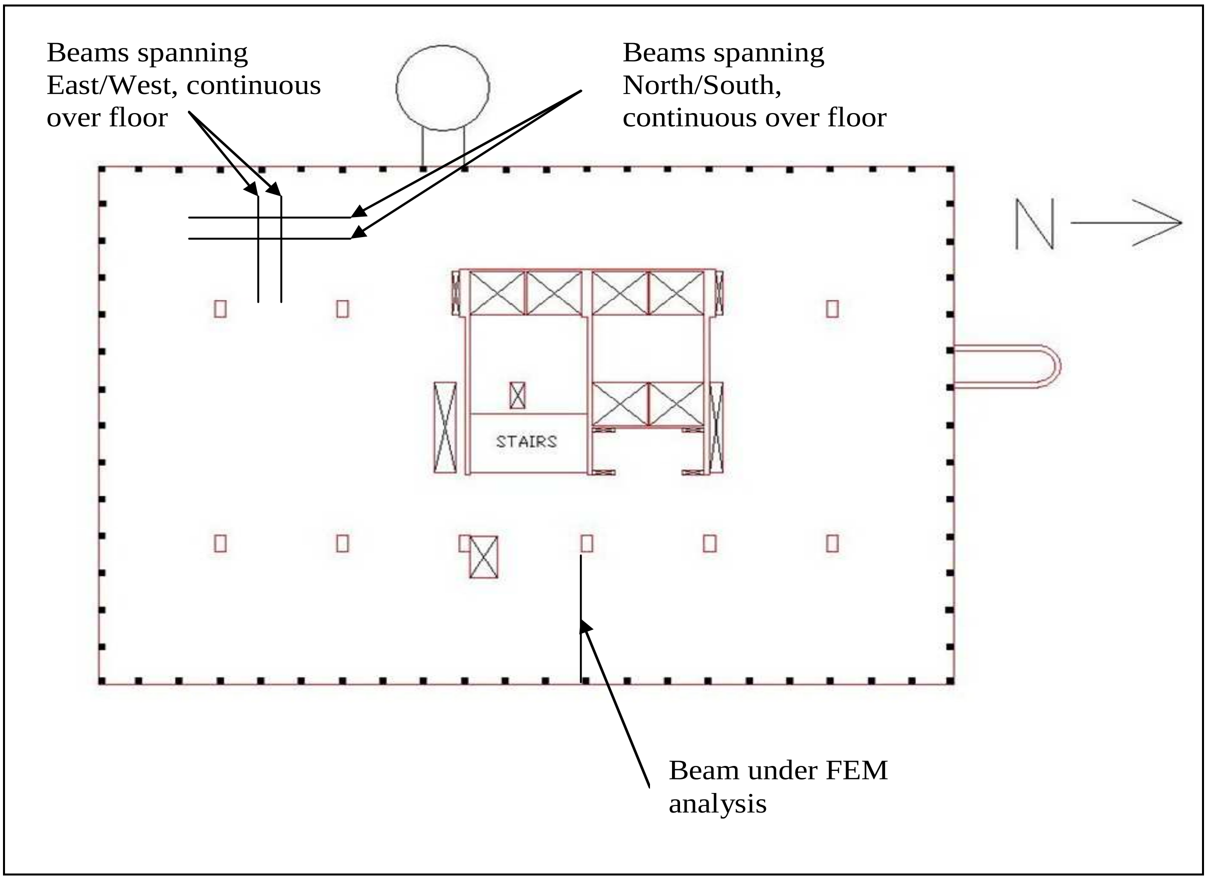

Figure 3 – Layout of waffle slab beams

Examining the plans of the Windsor Tower, it becomes clear that the floor slab acts as a network of beams. These can be modelled as primary beams, which in turn support secondary beams. However, it is apparent when looking at the plan in Fig. 3 that the primary and secondary beams do not necessarily constantly span in the same direction, i.e. while in one section of the building the primary beams may run North/South, in another section they may run East/West. The beams are 0.23m deep and 0.1m wide at 0.6m centres in both directions.

Since the structure can be idealised as primary and secondary beams, a simple model of one of these beams, running from the core of the building to the external columns, has been created. As an initial approach to modelling the slab, the beam highlighted in Fig. 3 is examined; here, the loading acting on the beam per metre can be calculated as 0.6 times the pressure acting on the floor slab.

It is also necessary to make assumptions about the boundary conditions on the beam model. Where the "beam" is attached to the concrete core of the building, it can be assumed that a fixed end connection can be used. At the other end, where the beam attaches to the steel columns, it is assumed that the connection is pinned. While the column is built into the floor slab, the relatively small size of the columns (140mm by 120mm) means that it is unlikely to resist any moments transmitted from the slab.

Once the structure has been defined, it is necessary to examine the effects of heating upon it, specifically the evolution of the heating within the depth of the material, i.e. how far the thermal wave has penetrated the concrete structural elements and the temperatures to which the reinforcing bar has therefore been exposed. This makes it necessary to examine the temperature profile within the depth of the beam.

The primary tool to be used for FEM structural modelling is ABAQUS. While it might be thought that a beam element could be used to model the primary beams, limitations on representing the thermal effects mean that it is more useful to adopt shell elements. With these, the temperature can be defined at a greater number of depths within the member. The concrete beam has therefore been modelled as a narrow, deep shell element.

7.1 Further Modelling

After a model has been developed to examine the effects of loading on an individual concrete element, a 2D “slice” of the building can be represented in order to examine the multiple floor nature of the fire. This will allow us to examine the effects of the steel columns, usually compression members, switching to acting in tension if one floor’s steel columns reach a point at which they are no longer capable of supporting the structure. Further to this, the structure can be extrapolated into three dimensions, allowing load distributions within the floors to be examined.

8 ANALYSIS

Models of fire development on the floor of fire origin (21) have been described previously [5]. In these simulations some attention has been given to the aspect of ventilation failure, i.e. the timing of glass fall-out, with two cases considered – glass failure based on a heat detector threshold of 150°C, and a more conservative case where the glazing is removed at ignition. Also considering uncertainties in the fire loading and distributions of combustible materials, further simulations were run in which the supply rates of fuel corresponded to roughly twice what might be expected to be the ventilation-controlled heat release rate per floor ($\dot{m} = 5.5 A_w \sqrt{h}$), which is determined as c. 350MW, based on an assumption of a window height of half the floor-to-ceiling distance of 3m. At longer fire durations, some of the aluminium panels located in the base of the openings were also lost, therefore giving access to even more ventilation, but at this stage the fuel load of the fire may have been substantially diminished due to burn-out. Based on these ventilation assumptions, the peak gas-phase temperatures computed exceeded 1100°C and the corresponding temperatures of the surfaces of the concrete columns were about 200°C lower. There was little sensitivity of this prediction to the size of the fire, which was expected since when the ventilation-controlled limit is exceeded extra fuel burns mainly outside the compartment.

These assessments are further supported by the post-fire analysis of the concrete strengths [3], which confirmed that the surface temperature of the columns and slabs had exceeded 800°C. It may also be possible to make an estimate of the concrete surface temperatures based on the methodology of examining the colour change of the concrete and changes to the microstructure [8, 9].

In examining the performance of the structure using FEM codes, a key input is the temperature conditions in the structure. FEM codes such as ABAQUS can of course compute the thermal response on the basis of known exposure boundary conditions, either temperatures or fluxes, but this normally involves use of brick elements, which are inefficient for the structural modelling. Hence, the thermal response has been examined using independent thermal models, the results informing the set-up of the ABAQUS simulations. Using shell elements, different temperatures are specified for each layer, and therefore the reinforcement bars. One important consideration here is the depth of the elements in relation to the depth of the thermal wave which has propagated into the material. If the latter is very shallow, then the temperature distribution within the structure cannot be adequately represented unless there are sufficient nodes to properly resolve it. If there are insufficient nodes, the effect of the heating will tend to be exaggerated and this can have a knock-on effect on the mechanical modelling.

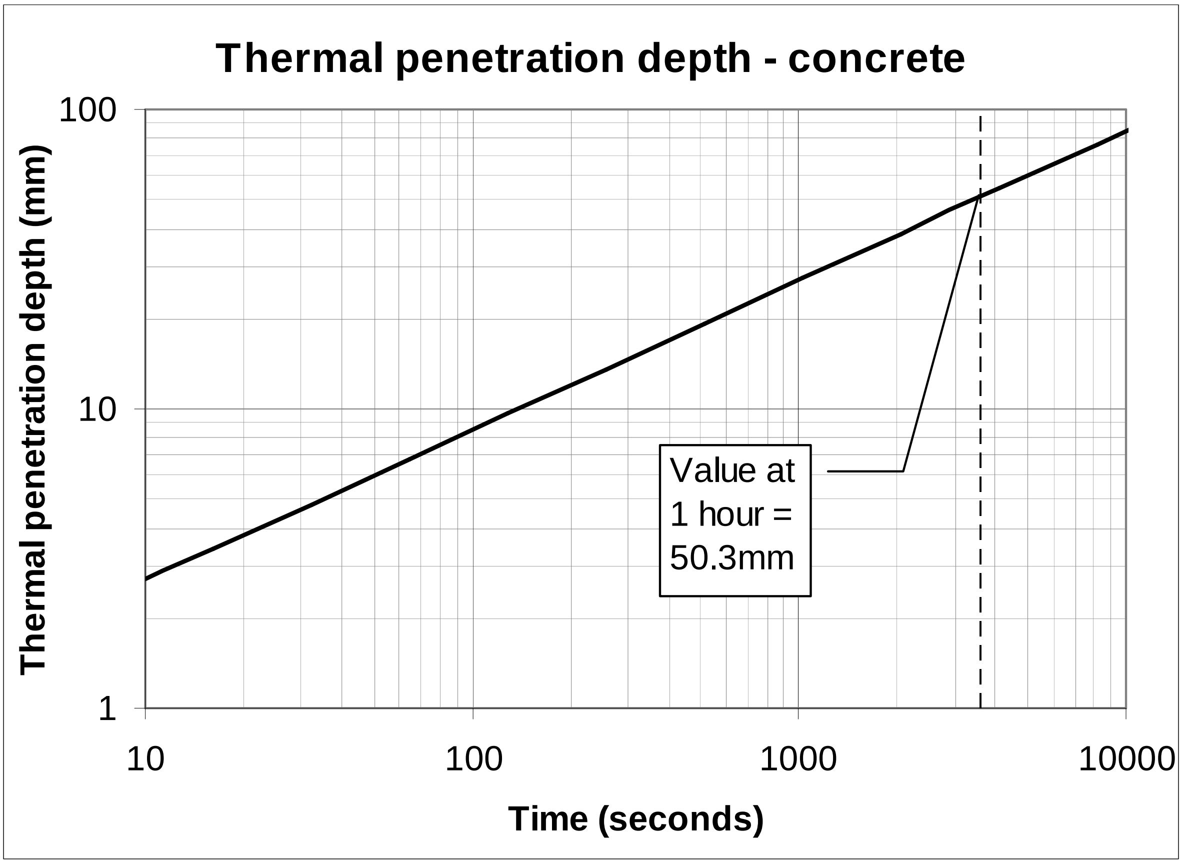

Figure 4 – Thermal penetration depth against time

Therefore, in order to inform this modelling a simple calculation of the evolution of the thermal penetration depth against time has been performed. The specified concrete strengths of the structure were 24.5 MPa in the columns and walls, 29.4 MPa in the deep beams and 17.2 MPa in the floor slabs, though there were some variants on this in practice [3]. Assuming a normal-weight siliceous aggregate concrete, density was taken to be 2400 kg/m^3, thermal conductivity as 1.2 W/m/K and the specific heat capacity as 880 J/kg/K, giving a thermal diffusivity of 0.57x10^{-6} m^2/s. Fig. 4 shows the evolution of the thermal penetration depth with time; fire duration is uncertain, but there is some evidence that the main burning phases lasted for the order of one hour [10]; the relevant penetration depth would therefore be equal to about 50mm. By comparison, studies of the concrete strengths after the fire suggested that in small regions of the ceilings the damage contour, based on a criterion of 500°C isotherms, had exceeded a depth of 200mm [3]. In this case there may of course have been out-of-plane effects including heating from the sides of the beams which make up the waffle slab (dimensions 100mm x 200mm, excluding the slab on top).

Using the default of five nodes per shell element, the spacing in the model of the 230mm deep beam and slab assembly is 58mm, traversed by the thermal wave in about 1400s. Before this time, the thermal response would therefore not be resolved, leading to some degree of error in the model predictions. Fig. 4 suggests that a resolution of at least 10mm is required in order the adequately describe the thermal response of the slab in the first 100 seconds of simulation.

9 CONCLUSIONS

A comprehensive approach to assessing the performance of the mainly concrete-framed structure of the Windsor Tower is being undertaken. An assessment of likely thermal exposure histories has been performed, referencing data gathered from the post-fire assessment of the structure and from various modelling approaches. The thermal response of the structure is thereafter assessed, in order to define the inputs and boundary conditions for structural models, including FEM. Based on this a strategy for modelling the mechanical response of the structure has been developed, with a view to examining possible failure mechanisms in relation to the observed effects of the fire.

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