PERFORMANCE OF CONCRETE IN FIRE: A REVIEW OF THE STATE OF THE ART, WITH A CASE STUDY OF THE WINDSOR TOWER FIRE

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ABSTRACT

This paper provides a "State of the Art" review on current research into the effects of fire exposures upon concrete. The principal influences of high temperature in concrete are loss of compressive strength and spalling, the forcible ejection of material from a member. Though a lot of information has been gathered on both phenomena, there remains a need for a broader understanding of the response of concrete structures to different heating regimes and the performance of complete concrete structures subjected to realistic fire exposures.

There is a lack of information derived from large-scale tests on concrete buildings in natural fires. Besides undertaking further fire tests, lessons can also be learnt from real fires and the University of Edinburgh has embarked upon detailed studies of the serious fire in the Windsor Tower, Madrid. In order to properly characterise the fire and the performance of the structure a data-gathering exercise has been undertaken and computer modelling tools are being applied in order to obtain better insights into the structural response. There remains some uncertainty about the precise mechanism of fire spread, but an external route is likely, facilitated to some degree by the glazed curtain walling construction; lack of fire protection on the steelwork was the major reason for the subsequent partial collapse of the upper floors and the localised failure of a concrete portal frame can be attributed to the same reason.

1. INTRODUCTION

In order to advance the understanding of the performance of concrete-framed structures during a fire it is important to establish the scope and conclusions of earlier studies and to highlight research gaps. This paper examines the current "state of the art" of our understanding of concrete in fire and overviews notable areas of recent research.

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Gaining an improved understanding of concrete in fire is also greatly aided by the examination of experimental measurements and data obtained from real-world fires. Concerning the latter, a detailed assessment is underway on the fire in the Windsor Tower in Madrid, which occurred in February 2005, with advanced modelling tools being used to assess the fire development and structural response of this mainly concrete-framed structure. Having adequately characterised the actual fire and the performance of the structure, establishing a well-defined case study, the modelling tools will then be used to examine sensitivities to a range of parameters of interest, generalising the conclusions to other possible fire scenarios and structural arrangements.

2. AREAS OF RESEARCH EXAMINED

2.1 Chemical responses of concrete to fire

When subjected to heat, concrete responds not just in instantaneous physical changes, such as expansion, but by undergoing various chemical changes. This response is especially complex due to the non-uniformity of the material. Concrete contains both cement and aggregate elements, and these may react to heating in a variety of ways.

First of all, there are a number of physical and chemical changes which occur in the cement subjected to heat[1,2]. Some of these are reversible upon cooling, but others are non-reversible and may significantly weaken the concrete structure after a fire. Most porous concretes contain a certain amount of liquid water in them. This will obviously vaporise if the temperature significantly exceeds the moisture plateau range of 100-140°C or so, normally causing a build-up of pressure within the concrete. If the temperature reaches about 400°C, the calcium hydroxide in the cement will begin to dehydrate, generating further water vapour and also bringing about a significant reduction in the physical strength of the material. Other changes may occur in the aggregate at higher temperatures, for example quartz-based aggregates increase in volume, due to a mineral transformation, at about 575°C and limestone aggregates will decompose at about 800°C. In isolation, the thermal response of the aggregate itself is more straightforward but the overall response of the concrete due to changes in the aggregate may be much greater. For example, differential expansion between the aggregate and the cement matrix may cause cracking and spalling.

These physical and chemical changes in concrete will have the effect of reducing the compressive strength of the material. Generally, concrete will maintain its compressive strength until a critical temperature is reached, above which point it will rapidly drop off. This generally occurs at around 600°C. This is only a little higher than critical temperatures for steel, but because of the much lower conductivity of concrete the heat tends not to penetrate very far into the depth of the material, meaning that the structure as a whole normally retains much of its strength (timber is similar in being able to retain strength in its depth once surface layers have been attacked by fire).

2.2 Spalling

One of the most poorly understood processes in the reaction of concrete to high temperatures or fire is that of 'explosive spalling'[3]. This is the process whereby chunks of concrete break off and are ejected from the surface of the concrete slab, often at fairly high velocities. The phenomenon is generally assumed to occur at high temperatures, yet it has also been observed in the early stages of a fire[4] and at temperatures as low as 200°C[5]. If severe, spalling can have a deleterious effect on the strength of reinforced concrete structures, due to enhanced heating of the steel reinforcement. Spalling may significantly reduce or even eliminate the layer of concrete cover on the reinforcement bars, thereby exposing the reinforcement to high temperatures, leading to a reduction of strength of the steel and hence a deterioration of the mechanical properties of the structure as a whole.

The mechanism leading to spalling is generally thought to involve large build-ups of pressure within the porous material which the structure of the concrete is not able to sufficiently dissipate, so fractures occur and chunks of the material are forced suddenly outward. While still in its early stages, modelling of spalling is beginning to show promise[3].

The main prerequisites for spalling are relatively well established, these being moisture content of at least 2% and most importantly steep temperature gradients within the material. A value of 5K/mm is a rough minimum and at 7-8K/mm spalling is very likely[6]. Temperature gradients are dependent not only on gas-phase temperatures but also heating rates, so that it is not possible to define a threshold temperature per se. However, these values may be affected by the type of concrete, including the strength of the material and the presence of fibres, as described below.

There has been a large amount of recent research on the potential for inclusion of various types of fibres into concrete to mitigate the effects of spalling. Some studies[7,8,9,10] have included polypropylene fibres into the concrete matrix. The theory is that when the concrete is subjected to heat, the polypropylene will melt, creating pathways within the concrete for the exhaust of water vapour and any other gaseous products, which will thereby reduce the build-up of pressure within the concrete. There has been some debate as to whether mono-filament or multi-filament fibres are better able to mitigate spalling[11]. It has also been suggested that the melted polypropylene fibres can form a barrier to the transport of moisture further into the concrete, preventing pressure build-up at greater depth and forcing the moisture to escape instead[8]. The same report suggests that the polypropylene fibres may provide a mechanism for cracking deeper within the concrete, which may mitigate spalling at the surface, but may have adverse structural consequences. Clearly, more work needs to be done in this area. Other studies have added steel fibres to concrete systems[11]; the theory behind this is that the steel will increase the ductility of the concrete and make it more able to withstand the high internal pressures. Results are, so far, inconclusive[11].

Recently there has been increasing use of ‘high strength concrete’. This material typically has considerably higher compressive strength than normal strength concrete, however it is also considerably less porous and moisture absorbent. While this generally reduces the water content of the cement, it is also harder for water vapour to escape during heating. Spalling has been suggested to be relatively more common in high strength concrete, due to the lower porosity of high strength concrete and hence the increased likelihood of high pressure developing within the concrete structure[7,1]. However, some recent research has shown that this is not necessarily the case, with testing showing higher spalling resistance in high strength concrete than in normal strength[8,2].

An aspect of concrete behaviour in fire that has not been revealed by testing based on standard fire curves is the post-fire cooling stage. The importance of a cooling-off phase in the assessment of a sample’s resistance to heat was demonstrated during a test of some concrete structural elements at Hagerbach test gallery, Switzerland. During the test a concrete sample resisted temperatures of up to 1600°C for two hours without collapsing, but half an hour into the cooling-off phase the sample collapsed explosively[12].

2.3 Cracking

The processes leading to cracking are believed to be essentially the same as those leading to spalling. Thermal expansion and dehydration of the concrete due to heating may lead to the formation of fissures in the concrete rather than, or in addition to, explosive spalling. These fissures may provide pathways for direct heating of the reinforcement bars, possibly bringing about more thermal stress and further cracking. Under certain circumstances the cracks may provide pathways for hot combustion products to spread through the barrier to the adjoining compartment, but this has not been the subject of significant research[12].

Geogali & Tsakiridis[13] have made a case study of cracking in a concrete building subjected to fire, with particular emphasis on the depths to which cracking penetrates the concrete. It was found that this relates to the temperature of the fire, and that generally the cracks extended quite deep within the concrete member. Major damage was confined to the surface near to the fire origin, but the nature of cracking and discolouration of the concrete suggested that the material around the reinforcement had reached about 700°C. Cracks which extended more than 3cm into the depth of the structure were attributed to a short heating/cooling cycle due to the fire being extinguished.

2.4 Spalling containment

Research has also been undertaken on the effects of wrapping a concrete member in a variety of fabrics in order to assess any improvement of spalling resistance that this may provide[14]. It was found that a metal fabric had a beneficial effect on spalling resistance, with less effect being produced by carbon fibre and glass fibre fabrics. All tests were noted to have a reduction or absence of spalling when polypropylene fibres were added to the concrete mixture[9]. Steel fabric reduces spalling by providing lateral confinement pressure to the concrete member which is greater than the internal vapour pressure causing the spalling. The reduced effect of carbon and glass fibre fabrics is due to the bond strength of these materials reducing at high temperatures and therefore the ability of the fabric to provide confinement being suppressed. It does not appear that the wrapping these concrete members in fabric induces cracking deeper within the structure.

2.5 Effects of reinforcement bars

The performance of steel during a fire is understood to a higher degree than the performance of concrete, and the strength of steel at a given temperature can be predicted with reasonable reliability. It is generally held that steel reinforcement bars need to be protected from exposure to temperatures in excess of 250-300°C. This is due to the fact that steels with low Carbon contents are known to exhibit 'blue brittleness' between 200 and 300°C. Concrete and steel exhibit similar thermal expansion at temperatures up to 400°C; however, higher temperatures will result in significant expansion of the steel compared to the concrete and, if temperatures of the order of 700°C are attained, the load-bearing capacity of the steel reinforcement will be reduced to about 20% of its design value.

Reinforcement can have a significant effect on the transport of water within a heated concrete member, creating impermeable regions where water can become trapped. This forces the water to flow around the bars, increasing the pore pressure in some areas of the structure and therefore potentially enhancing the risk of spalling. On the other hand, these areas of trapped water also mitigate the heat flow near the reinforcement, thereby reducing the temperatures of the internal concrete[15]. A large area of current study is targeted at the effects of using reinforcement constituted by glass or carbon fibres, rather than steel, in concrete[16,17,18,19,20]. Much of this research is motivated by the relative lack of information on Fibre Reinforced Plastic (FRP) reinforcement at high temperatures, and concern over thermally-induced failures. However, most of the testing indicates that with sufficient cover to the reinforcement, FRP reinforcement will have perfectly adequate fire endurance.

2.6 Structural stability and modelling

After a fire, changes in the structural properties of concrete do not fully reverse themselves, as opposed to a steel structure, where cooling will generally restore the material to its original state. This is due to changes in the physical and chemical properties of the cement itself. The non-reversibility of these processes has led to an interesting line of research which aims to assess the severity of a fire (i.e. the maximum temperature to which the structure was exposed) by examination of the state of the concrete structure after the fire[21]. It should be noted that, in some circumstances, a concrete structure may be considerably weakened after a fire, even if there is no visible damage.

Several models are available for the mechanical behaviour of concrete at elevated temperatures. A number of these are reviewed by Li & Purkiss[22], including the model suggested by Schneider[23], in order to produce a model which may be used in finite element analysis of a structure. It is noted that these models break the strain imposed on the concrete into four different types: the "free thermal strain", caused by the change in temperature, "creep strain", caused by the dislocation of microstructures within the material, the "transient strain", caused by changes in the chemical composition of the concrete and the "stress-related strain", caused by externally applied forces.

The models examined by Li & Purkiss each handle these strains differently[22]. "Free thermal strain" is solely a function of the temperature of the concrete member; however creep, transient and stress-related strains are all functions of the temperature, time and stress, making it difficult to separate which particular strains are being influenced during a given experiment. In order to reduce this level of complication, some of the models gather two or even all three of these strains together into one term. Typically, this is the "transient creep strain", incorporating the creep strain and transient strain together.

Based on the results of these models, Li & Purkiss created a new model and used it to demonstrate the significance of transient strain[22]. It was shown that models that do not include transient strain are unconservative for high temperatures, though at low temperatures transient strain appears to have less effect. It was also noted that "it is evident that the full stress-strain curves provided in EN 1992-1-2[24] for higher temperatures are unconservative".

While it is important to understand the performance of individual concrete members during a fire, the behaviour of the same members within the context of a complete structure can depart widely from their independent responses. This is due to a variety of factors – for example, thermal expansion of members which have been subjected to heating may lead to forces being exerted upon the cooler members due to differential expansion, and upon the hotter members due to restraining forces provided by the rest of the structure. The effects of thermal expansion have long been recognized with steel and composite members[25], but little research is available for concrete structures.

Modelling has largely been undertaken of the effects of increased temperature on individual concrete elements, for example concrete columns[26]. These have been used in particular to compare predictions with the structural Eurocodes and validate against the behaviour of real concrete columns during full-scale fire testing. Further fire testing has been carried out on concrete columns, for example by Benmarce *et al*[27]. This study examines the boundary conditions of the column as well as the effects of heating on the concrete itself, and hence has a closer approximation to the effects of fire on a whole structure. This is important, as the behaviour of a structural member must be related to the structure it is in, rather than being examined in isolation, in order for it to be useful; it is also necessary in considering the effects of the member on rest of the structure. The study concluded that this is an area that has not been examined sufficiently[26], but the tests determined that the additional forces generated were low, around 15% of the design load of the columns. However, the columns tested were 125mm x 125mm cross section x 1.8m high with 108 N/mm^2 high strength concrete, which is very small for a concrete section. Hence it is uncertain whether this data would be scalable to larger members or applicable to members with normal strength concrete.

The main reports on the effects of fire on whole structures have been produced as a result of tests carried out at Cardington by the Building Research Establishment (BRE)[4,28,29]. One of the structures built at Cardington is a concrete building, consisting of high strength concrete columns and normal strength concrete flat slabs. The fire test on this structure, using wooden cribs to provide the fire loading rather than a furnace, was not entirely successful due to problems with the data collection; however it did indicate that a concrete column exposed to a real fire situation was unlikely to fail. Also, large amounts of spalling were induced in the concrete floor slabs. However, these remained intact, which was attributed largely to the "compressive membrane action" as the expansion of the concrete slab was restrained due to the presence of cold surrounding areas of structure, and therefore load was supported by the compressive strength of the concrete. This mechanism differs from "tensile membrane action", where the reinforcement in a concrete slab restrains the slab as it suffers displacement. Compressive membrane action can only take place at relatively small displacements; however it does not rely on the reinforcing bars retaining their strength at high temperatures, as tensile membrane action does. In the case of the Cardington test the reinforcing bars are unlikely to have retained large amounts of strength due to being directly exposed to high temperatures as a result of the significant spalling in the concrete cover.

It is also worth considering that while an individual concrete member may fail, the overall structure may well remain intact, due to the fact that redundancy of the structure allows members to redistribute the loads previously carried by failed members. This is a common phenomenon in composite structures.

The Cardington data has been used to provide input values for a finite element model which made a large number of assumptions with respect to the performance of concrete in fire (for example, the effects of spalling were neglected)[10]. Further study of the effects of fire upon a whole-frame structure would be extremely useful. It is hoped that the examination of the effects of fire upon the Windsor Tower (c.f. section 3 below) will go some way towards developing a better understanding of the effects of a fire upon complete structures, without the necessity of carrying out further expensive large-scale fire tests.

2.7 Composite structures

A common form of construction for floor slabs is known as "Composite Construction". In this method, a concrete slab is cast upon steel beams. The formwork for this slab is a profiled metal sheet, known as decking, which spans between the beams. "Shear studs" are welded to the top of the steel beams, through this profiled decking. These studs allow a mechanical bond to be formed between the concrete and the steel member, and therefore allow the beam and the slab to act as a single member with an increased strength. The steel decking is left permanently in place after the concrete has been cast. Steel reinforcement is typically added above the profiled decking.

There has been a significant amount of work carried out on composite steel and concrete structures, for example the Cardington project carried out full-scale fires on a steel-framed building with composite concrete decking floor slabs[27]. These structures have been found to have considerable resistance to fire[30]. This is in part due to the concrete floor having capacity to act as a tensile membrane, allowing the load to redistribute through the structure when the mechanical properties of steel are reduced. This can lead to a reduction in the requirement for fire protection on the steel areas of the structure, a fact that has been recognised in recent design guidance and is increasingly reflected in engineering design.

3. WINDSOR TOWER FIRE CASE STUDY

The Windsor Tower in Madrid was involved in a major fire, of duration 18-20 hours, on 12-13 February 2005, which caused extensive structural damage to the upper floors of the building. Due to the nature of the building's construction, a largely concrete frame with steel perimeter columns, this fire has provoked intense interest amongst researchers hoping to better understand the performance of concrete structures in fire. In general, the concrete structure appears to have performed very well, and the most severely affected areas of the building appear to be those where the structural steelwork had not yet received fire protection, which was being installed in the building at the time when the fire broke out.

Analysis of records of the fire, together with data gathered on the construction details, has enabled establishment of computer models of the fire development and structural response. These provide a means of characterising the fire and assessing the actual performance of the structure under these heating conditions. However, it is not intended that this be a purely "forensic" exercise, but rather the modelling tools will be used to examine sensitivities to parameters of interest, such as glazing failure, compartmentation failure, external fire spread, fuel source distribution and progressive burnout.

The areas of the building where most damage occurred were the upper storeys above the strong transfer floor (T2), i.e. floors 17-28. The major structural failures can be very simply attributed to the fact that the perimeter steel columns over this height were not fire protected; once they had lost significant strength, then much of the concrete perimetric flooring was unable to support itself as a cantilever and suffered progressive collapse, with failure of a large section on the north-east corner of the building at 01:15hrs. There were some variations in this behaviour though, with no collapse in regions adjacent to the new fire escape, on the west face, presumably due to the additional support for the floor, but some failure of a further section of floor slab together with the supporting concrete portal frame towards the north façade. It is also of interest that there was no significant collapse of the floor slab on floor 9, which also sustained a fully flashed-over fire for a period after 06:00hrs in the morning, and for which there was no fire protection applied to the steel-work on two of the sides; the steel columns here showed severe buckling, but the overall stability of the structure was maintained due to load sharing – with support coming from the protected steel columns both above and below.

Initial studies indicated that it will be quite challenging to model the thermal and structural response of the floor slabs themselves due to their complex method of construction. This utilises a type of clay permanent formwork to create a "waffle slab" profile. As this clay remains in place during the life of the building, there are areas of the concrete which have an additional layer of insulation against fire. This may complicate the analysis as the addition of two layers of insulation to the reinforcing steel, both which will have different thermal properties and one of which is believed to possess no permanent structural properties, will add a large number of variables to any model created. However, it is believed that little work has previously been carried out to examine the effects of fire on this type of concrete slab, and as it is believed to be a widespread form of construction in Spain, at least historically, it may provide a useful precedent for analysis of this form of construction.

Defining the development of the fire is also a very challenging exercise, but necessary in order to facilitate modelling of the structural response. It is fairly well-established that the fire broke out on the 21st floor, in office 2109 at approximately 23:05hrs and was detected at 23:08hrs; a 50cm flame was reported to have been seen there at 23:18hrs, consistent with a hypothesised waste-paper basket fire source, and the fire brigade were called at 21:21hrs; by 23:35 the fire on this floor was fully flashed-over[31]. Simulations of fire development have been performed using the FDS[32] and SOFIE[33] CFD codes but the biggest uncertainty is the time of glazing failure, which has a dominant influence on the fire development. The strategy adopted to overcome this is to run a number of different simulations, each with different glazing failure temperatures, in order to bound the possible behaviours. An initial finding is that if there is no glazing failure at all within the growth period of the fire then the model predicts a decay and eventual extinction; it order to generate a realistic representation of the real fire, a major glazing failure is required in the first 10 minutes or so.

Having established itself on one floor, there is a great interest in the mechanism of fire spread to other parts of the building. Significant effort has gone into defining the rate of floor-to-floor fire spread, in order to determine the thermal exposure boundary conditions on the rest of the structure, at least in approximate terms. Initial reports indicated that the rate of upward fire spread had been "very rapid", with some suggestion, mainly via news reports and anecdotal evidence, that the fire had reached the top of the building by 00:00hrs. However, more careful investigation indicates that this is probably a significant overestimate. An number of subsequent studies have now reported more precise estimates floor-to-floor fire spread rates, varying from an average of 6.5 minutes per floor (INTEMAC report[34]) to 15 minutes per floor (Japanese study[35]). The latter report provides a detailed time breakdown of the estimated burning histories on each floor; it suggests that the initial upward spread to the 22nd floor took 40 minutes, progress to the 23rd & 24th floors took a further 70 minutes and that the fire reached the top of the building (28th floor) after a further 30 minutes. A time-stamped photograph of the east face, where the room of fire origin - office 2109 - was located, taken at 00:50hrs, is consistent with this, showing obvious external flaming on only 3 or 4 floors, though at this stage there has apparently not been any significant break-out of the fire through the façade on the south face. This suggests that the progress rate upwards from one floor to the next was not indeed that rapid up until this point. Downward spread commenced with involvement of floor 20 at 01:00hrs and then a very steady progression of about 20 minutes per floor, down to floor 12 at 05:40hrs, interrupted only by a long delay of 80 minutes in passing the transfer floor T2. It is interesting to note that on average this downward spread rate actually exceeds that for progress up the building, indicating that a different mechanism was involved (most likely involving inflamed molten solids or liquids physically transferring the fire as they flowed through openings between the floors).

Further to these studies, the report of the National Scientific Police has also provided another estimate of the rate of upward spread which averages approximately 10 minutes per floor[31]. In trying to reconcile this figure with the others quoted it should be noted that there are a number of uncertainties in defining the fire location. Most of the photographic evidence only illustrates what conditions had evolved on the perimeter of the building; had there been internal openings in the floor slabs deeper into the building, e.g. for service ducts, it is possible that the fire may have been spreading more rapidly internally than was apparent from the external footage. It is certainly very clear from the video and photographic evidence that the fire was unevenly distributed around the perimeter of the building, i.e. it does not break-out of the façade on all sides of each floor simultaneously. Initial upward spread seems to have been fastest on the east face, above the room where the fire initial broke out, suggesting that floor-to-floor spread was predominantly occurring in the region of the façade, rather than internally. Together with the possibility of significant fire spread away from the façades this non-uniformity might be part of the reason for the remaining discrepancy in the estimates of the rate of upward spread; another aspect could be terminological differences in defining when a floor is "on fire" (which might vary from having sustained a small localised ignition, to full flashover engulfing the whole floor). Further work is underway to try to establish more clearly the precise course of the fire development throughout the structure.

In conjunction with this, modelling studies have been undertaken in order to examine various hypotheses about the mechanism of floor-to-floor fire spread. Field modelling using

FDS[32] and SOFIE[33], together with the Law external flaming model[36] as adopted in Eurocode 1 (Annex B)[37], suggested that flame temperatures near the glass on the floor above the fire might have been of the order of 850°C. Analyses using the computer package BREAK1[38] reveal that on this basis the time to cracking of a single pane of glass would have been of the order of 250-300 seconds. However, as part of a facelift operation a glazed curtain wall had been added to the building in recent years, so that there was effectively a double skin on the building with a fully glazed external wall and an internal façade consisting of alternating Aluminium (lower) and glass (upper) panels on each floor. Hence, it is insufficient to predict time to failure of the glass, and it is also necessary to estimate time for the glass to fall out, thereby exposing the internal glass to more intense heating and causing it to fail too. Weather reports for the dates of the fire were analysed and it was found that the wind was very light at the time, with hourly averages in the range 0 to 7 km/h during the first 7 hours (averaging 2.7 km/h, and direction varying from N to W, averaging NW); photographic evidence is also in accordance with the fact that the wind was light and had a direction with a westerly component at the height of the building. The low velocities mean that once the external glazing had cracked, with the internal partition still intact, then relatively small pressure differences might be expected between its surfaces. This would be consistent with an average time to failure, i.e. glazing dropping out, being at least a factor of two longer than the time to cracking, since there would be relatively small unbalanced forces on the cracked glass.

A useful overview of the mechanisms of fire spread in multi-story buildings with glazed curtain wall façades is provided by Morris & Jackman[39]. For example, this mentions that the observed vertical spread mechanism for the fire in the 62-storey First Interstate Bank fire, Los Angeles, California, in 1988[40], was via flames "breaking out of the external skin followed by break-in on the floor above, supplemented by failure of the fire-stopping between the floor slab and the curtain wall system over a gap of approx. 100mm". Two other cases are identified in which multi-floor fires developed: the office block fires in a 38-storey tower in Philadelphia, Pennsylvania[41] and a 12-storey building in Basingstoke, UK[42] (both 1991). There is some evidence that the equivalent fire stopping in the Windsor Tower was not fully installed at the time of the fire (it is believed to have been missing in many places) and this may have provided a path for spread. Another alternative to the above described external spread scenario might be that fire broke into the cavity within the curtain walling and propagated directly to the floor above via this internal space followed by failure of the original inner skin glazing and entry of the fire into the next compartment. On the other hand, if the glazing of the inner skin extends only over the upper half of the floor height then the internal pane might actually have been more robust to cracking than the larger external panes in the curtain wall. To what extent the internal cavity could have generated a "chimney effect" remains uncertain, as it is not clear how hot gases might have escaped from the top. However, considering the lack of any evidence to the contrary, the presumption must be that this space does not provide a well-ventilated high temperature combustion region until such as time as the façade opens up properly via failure of the external glazing; at this point, the situation is similar again to a purely external spread route and there seems to be no reason to assume that spread via this mechanism would be particularly rapid. Hence it is debatable whether a route via an internal cavity in the curtain wall would in practice speed up the rate of spread in the current case, and indeed evidence from the fire reported above suggests that spread rates were not particularly rapid even when this mechanism was of relevance.

Morris & Jackman draw attention to the fact that there some inherent structural weaknesses in glazed curtain walling systems, which have a tendency to allow fire to spread from storey to storey by a variety of mechanisms[39]. This view is supported by observations from measured failure times in special fire tests on multi-storey buildings, which included glazed curtain walling. Failure times of between 5 and 13 minutes were reported for the tests with glazed façades, with a big influence of the fire load. With fire resisting panels instead of glass these times were more than doubled; however, the comparison is not strictly correct for the current case because standard glazing was not tested. Though there is no reason to suspect that fires are any more likely to develop in buildings with glazed façades the tendency for substantially greater consequential losses per fire is a concern to insurers; this fact is clearly exacerbated when such fires lead to structural failures of major parts of a building, as occurred in the Windsor Tower, with the costs of repair greatly increased. It is hoped that by better understanding these types of fires, and the behaviour of concrete structures, these types of losses could be minimised in the future.

4. CONCLUSIONS

In general the behaviour of concrete in fire is not very well characterised at present, and further research is required in almost every aspect of this field. Specifically the mechanism and causes of spalling, currently one of the greatest concerns for those interested in concrete building safety in fire and high temperatures, are not adequately understood. There is a need for more systematic studies which examine the effects of varying heating conditions, both on spalling behaviour and more generally.

The majority of past research work on the response of concrete to fire (or, more frequently, high temperatures) has looked at the effects of fire upon individual structural members, and most commonly when subjected to heating from standard fire tests. There is a great need for development of models which consider the effects of fire on the whole structure under more realistic heating regimes. There is also a fundamental requirement for further large-scale testing of concrete structures, to observe the behaviour of complete concrete structures in real fires and also for validation of advanced computer modelling tools.

The University of Edinburgh are working towards filling some of the knowledge gaps by the detailed examination of the effects of fire upon the Windsor Tower, gathering data and applying modelling tools to bound and characterise the behaviour. Initial results suggest that the observed response was well within the bounds expected for a structure of this type; the lack of fire protection of certain parts of the steelwork was a major reason for the partial collapse of the building, including some concrete elements, and whilst the particular nature of the glazed curtain-walling may have permitted upward fire spread, this might not have been any worse than performance of a building lacking an additional façade skin.

5. ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support and guidance of The Concrete Centre/British Cement Association and EPSRC, via an industrial CASE award, and generous help from colleagues at GIDAI at the University Cantabria, together with ArupFire.

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