Science and Technology Developments in Structural Fire Engineering

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Summary

The devastation caused by fire within buildings can result in loss of life and significant economic loss due to direct and indirect costs. In terms of structural performance, the understanding of the behaviour of buildings during a fire has been stifled by the concentration on the performance of structural elements in small-scale fire tests required for approval. Recently, a series of full-scale fire tests has been carried out on various forms of construction which together with the development of fire design approaches has revolutionised the approach to structural fire engineering. The understanding of these full-scale tests, together with the development of codified performance-based approaches, has begun to present designers with the necessary tools to obtain a better prediction of the building's performance during a fire. This in turn allows designers to obtain more economical solutions and identify any weak points within the structural system, ensuring a robust and safe design. This paper presents the latest developments in structural fire engineering, highlighting first the problems of the traditional design approaches and the advantages to be gained by following the more recently developed performance-based approaches.

Keywords: fire; full-scale tests; performance-based design.

Introduction

In the UK the origins of structural fire engineering can be traced back to 1666, where following the Great Fire of London, in which approximately 80% of the city was destroyed, legislation was passed to control the use of structural materials for buildings and the spacing between buildings. Over the years the discipline of structural fire engineering has not progressed significantly. Even today some designers give structural fire safety very little thought compared to other aspects of structural design and rely heavily on simple deemed-to-satisfy (prescriptive) rules to cover buildings ranging from single-storey dwellings to multi-storey iconic structures that define the skyline of our cities.

Today, legislation relating to fire safety of buildings covers life safety, in terms of an acceptable risk, taking into account occupants, firefighters and people in the proximity of the building. Legislation is governed by either functional or prescriptive regulations. For example, the building regulations in the UK[1–3] provide functional objectives for structural aspects of fire safety. These objectives state that the building shall be designed and constructed such that in the event of a fire its stability will be maintained for a reasonable period and that the spread of fire within a building will be inhibited by the division of the building using fire-resisting construction to an extent appropriate to its size and intended use. To meet these life safety requirements, either simple prescriptive rules, as outlined in the approved documents[4–6] or design codes,[7–12] or a more rational performance-based approach[7–12] considering the true behaviour of the fire and structural response, could be adopted.

Recently the discipline of structural fire engineering, which involves the knowledge of fire behaviour, heat transfer and structural response of the proposed building structure, has begun to progress at a significant pace, allowing the confident use of a performance-based approach to structural fire safety. This has, in part, been fuelled by recent research on full-scale structures,[13–18] and the public awareness of various disasters around the world such as the World Trade Center, the Madrid Fire and the First Interstate Bank Fire. Clients are now beginning to ask the question "how robust is my building?" and what is the likely outcome following a fire. As discussed, legislation governs life safety and does not address explicitly the level of protection to the building contents, the building superstructure, heritage, business continuity,

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corporate image of the occupants or owner, and/or the environmental impact. It may be necessary to increase the level of protection above the minimum legislative requirements to address any or all of these issues, which can only be taken into due account using a performance-based approach. In addition, the demand to create more elaborate structures, which cannot be designed using simple prescriptive rules, is becoming more common. The introduction of the Structural Eurocodes,[7–12] which incorporate a separate code for the fire design of primary structural materials, has also raised awareness of the discipline within the design community.

Although structural fire engineering is becoming more prominent in designs, and a more performance-based approach is being adopted, questions are continually raised (and rightly so) by the profession asking "why is a more complicated design approach needed?" To answer this question fully, the underlining principles of the simple prescriptive approach are first explained, followed by the recent developments of structural fire engineering and the resulting benefits.

Prescriptive Approach

Prescriptive approaches are based solely on fire resistance and result in simple deemed-to-satisfy rules such as minimum cover and dimensions for concrete members, magnitude of protection thickness to steel members, minimum geometry of masonry walls and minimum cross-section size for timber members. The definition of fire resistance should be clearly understood by designers, as there is often a misconception that the stated fire resistance (i.e. 30, 60, 90 or 120 min) is directly related to the time that the building will withstand the effects of fire without collapse. Firstly, it is important to emphasise that buildings do not require fire resistance: it is only elements of the structure that require fire resistance. Secondly, the fire resistance of elements is only related to the measure of time that an element, whether it is a structural element, a fire door, or a non-structural compartment wall, will survive in a standard fire furnace test,[19] where the furnace temperature is defined by a prescribed time–temperature relationship (*Fig. 1*). The different fire resistance periods used in the prescriptive approach are crudely defined on the basis of the building occupancy and height.

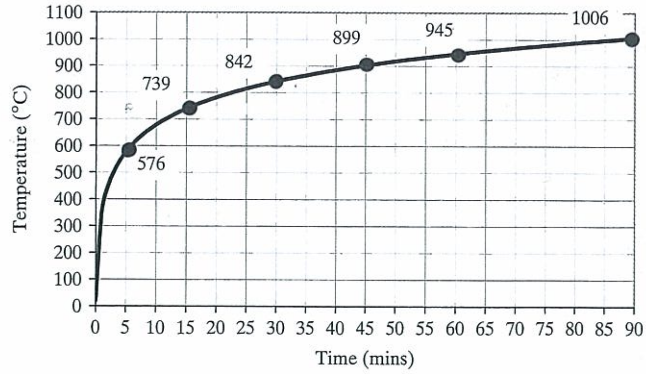
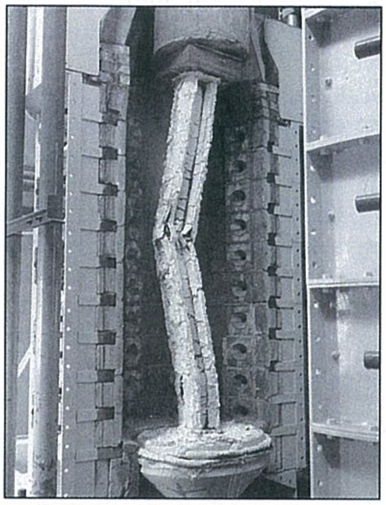
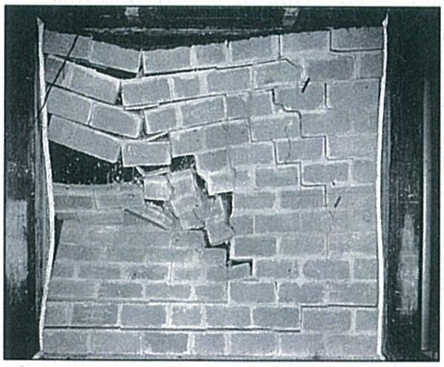


Fig. 1: ISO-834 time–temperature relationship used in a standard fire test

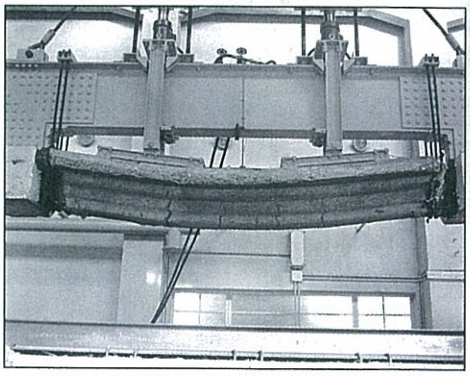


(a) Column (3 m height) after the test

(b) Floor test (4 m span)



(c) Wall test (3 m height)



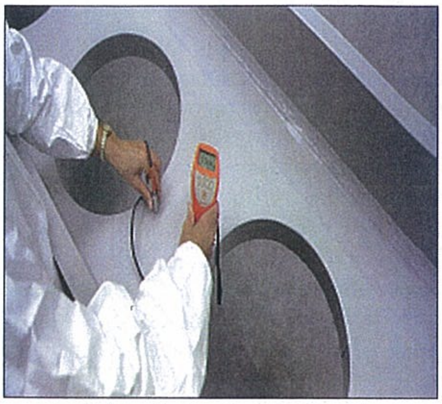
(d) Beam (4,5 m span, heated length 4,0 m) after the test

Fig. 2: Standard fire tests for columns, floors, walls and beams

For structural elements, there are approved furnaces where a standard configuration of a wall, beam, floor or column can be constructed and tested. The dimensions of the elements are 3 m high for columns, 4 m × 4 m (typically) for floors, 3 m × 3 m for walls, and 4,5 m span for beams. *Figure 2* shows fire resistance tests conducted on various structural elements. The failure criteria depend on the type of member tested and are defined in terms of stability, insulation and integrity. Stability is a measure of the ability of the member to support the applied load without exceeding given limits on the maximum displacement and rate of displacement. These deflection limits have been set to reduce the probability of damage to the furnace and have no physical meaning in defining failure of the element. Insulation and integrity are generally associated with compartment/separating walls and floors. For insulation, the maximum temperature on the unexposed side must not exceed a maximum increase of 180°C or an average increase of 140°C. To maintain integrity no significant sized holes must form in the element, which will allow the transmission of hot gases. Failure by integrity is defined by ignition of a cotton pad held close to an opening.

The history of the standard fire test can be traced back to the 1890s[20] when early attempts at establishing the fire behaviour of structural elements were made at the behest of insurance companies and building authorities in the USA, with the first standard published in 1917.[21] The general concept of the standard fire test has not changed significantly since its conception, with only slight modifications being introduced over the years. Although commendable in the 1890s, the standard fire test has stifled the understanding of how buildings actually behave in a fire. The problem is that designers and manufactures tend to only concentrate on the performance of structural elements and systems in the standard fire test, which bears no relation to the actual performance in buildings. In addition, the quality of the system tested will be far higher than that encountered on site. Protection systems will be applied in comfortable conditions by experienced and skilled labour, which is a far cry from the conditions and demands on time encountered on site. For example, *Fig. 3* contrasts the difference in quality of applied protection for an intumescent coating to steel members. *Fig. 3a* shows the coating being checked prior to testing and *Fig. 3b* shows damage on site to the intumescent coating applied to a column. The differences in quality of systems used in the standard test compared to those used on site are typical for all systems (fire doors, all protection systems, compartment walls, etc.) and were highlighted when a full-scale test on a six-storey timber building was carried out where the plasterboard fixed was not of the same quality as that tested in a standard fire test. (The test on the six-storey timber frame building is discussed later in this paper.)

Besides the differences in quality, the standard fire test is also a very poor representation of the actual structural behaviour in a real fire. This is because the building does not represent a collection of individual elements working independently of each other as tested in a standard furnace. The interaction between structural elements in a fire has both a possible beneficial and detrimental effect on the survival of the building as a whole. Beneficial effects are generally due to the formation of alternative load-path mechanisms such as compressive and tensile membrane action, catenary action and possible moment rotational restraint from steel connections, which were designed as "pinned" in the cold condition. Evidence of compressive and tensile membrane action has been provided by various fire tests[22,23] on full-scale buildings. The detrimental effect of several structural elements acting as a unit can be due to restraint of thermal expansion resulting in large compressive forces being induced into elements (particularly vertical elements) which then causes instability. Another detrimental effect can be the behaviour of walls, which in a standard fire test may be shown to perform adequately, but in a real building the movement of the heated structure around the wall may result in premature collapse. This effect was shown[24] for a non-loadbearing compartment wall in one of the fire tests on the steel-framed building at Cardington (*Fig. 4*). In this test, the wall was placed off-grid and the deflection of the surrounding structure caused significant deformation of the wall. Tested in isolation, in a standard furnace, the wall would not be subjected to the additional forces from the surrounding structure.



(a) Application of coating for testing

(b) Damage to coating on site

Fig. 3: Application of intumescent coatings (a) shows the coating being checked prior to testing (b) shows damage on site to the intumescent coating applied to a column

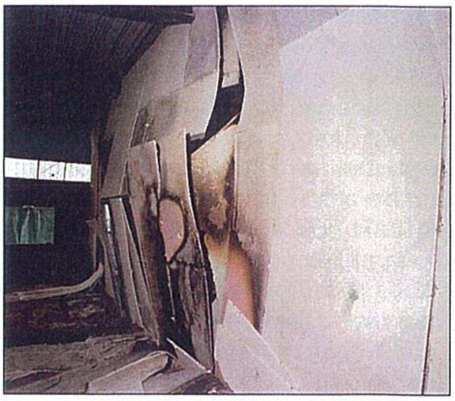


Fig. 4: Deformation of structure, causing failure of compartment wall

Another significant disadvantage of the standard fire test is that the standard curve does not represent a real fire. There are generally three distinct phases to a real fire comprising growth, steady burning and cooling phase. The severity of the fire is governed by the geometry of the compartment, the amount of combustible material, the ventilation conditions and the thermal characteristics of the compartment boundary. Different types of fire can result in different structural behaviour. For example, a short-duration high-temperature fire can cause concrete spalling, and exposure to fire of the steel reinforcement because of the thermal shock; a long-duration low-temperature fire can result in a higher average temperature in the concrete members resulting in greater thermal expansion and a greater overall reduction in concrete strength. In the fire test, once the target is reached (30, 60, 90, 120 min) the furnace is switched off and the load removed. In a real building, the load remains and the fire continues through the three stages of heating.

A further problem of the standard fire test is that, for economical reasons and competitiveness, manufacturers only want to "just" pass the test. For example, when designing and specifying a fire protection system for 60 min, the most economical solution is for the protection system to last 1 s past 60 min. One second before 60 min and the system has failed the test. On the contrary, if the system lasts significantly above 60 min then the protection material will become uncompetitive. The test can be carried out a number of times and only one pass certificate, from an approved testing house, is required. At present, there is generally no requirement from approving authorities to demand a series of fire tests on a system to assess the repeatability of tests. The performance of steel protection systems was highlighted recently in full-scale fire tests[18] on a building system comprising a steel frame support and precast units. The steel support was protected for 60 min specified and fitted by a well-known contractor. The natural fire was designed, by specifying the ventilation openings, to follow the standard fire curve up to 60 min and then to start to cool down. The protection system performed well up to 60 min but then started to fall off the supporting steelwork. *Figure 5* (taken after the test) indicates the areas where the protection has fallen off at the peak of the fire. Unlike the standard furnaces, the full-scale tests represented reality and the load could not be removed or the fire turned off. During the cooling phase of a real fire, the protected steel beams and the precast slabs continue to increase in temperature due to the time-lag caused by the low thermal conductivity of the material. For example,[18] the maximum temperature in the reinforcement in the precast slabs was 554°C even after the fire atmosphere temperature had cooled down from a peak of 1069°C to 541°C. The protection to the beams falling off at the height of the fire once again shows the weakness of concentrating on standard fire tests and not considering the performance of the protection system over the full duration of a fire.

Advanced Structural Fire Engineering

The discipline of structural fire engineering does not have to constitute a "full" performance-based approach to fire safety and there are different levels of complexity of structural fire engineering, which can easily be applied by structural engineers, as part of their design portfolio. These range from the simple prescriptive approach, discussed above, to the detailed performance approach of considering the fire, thermal response and structural behaviour. The process of a performance-based approach to structural fire engineering is analogous to the process of designing structures to withstand wind (*Fig. 6*). As most readers will know, to check buildings for wind loads there are three basic steps, comprising an estimate of the site wind speed, an estimate of the wind pressures over the building and an estimate of the structural response. For a structural fire engineering performance-based approach, the three basic steps comprise an estimate of the severity of a fire, an estimate of the temperature distribution through the structure (thermal response) and an estimate of the structural response.

There are a number of options available to calculate the severity of the fire and the thermal and structural response, as shown in *Fig. 7*, and designers can use different permutations. Of course, increasing the complexity of the structural fire design will result in increased design costs, but with the benefit of a greater reduction in the uncertainty of the building response in a fire and typically a resulting economy in overall building costs. Each of the options will be discussed briefly below.

Fire Behaviour

The first issue is to decide whether the fire remains localised or engulfs the whole compartment. Examples where localised fires may occur are in large high spaces with relatively limited fire load, such as atria, circulation areas in airports, shopping malls, or areas where there are high levels of ventilation such as in open canopies, typically at hotel entrances, under link bridges at airports, or areas where the fire load can be reliably controlled to relatively low levels or spaced such that the fire cannot readily spread from one area of fire load to another. The available methods, in order of complexity, to estimate localised fires are design equations given in BSEN1991-1-2,[7] design equations given in BS7974-1,[25] two-zone models[26] or computational fluid dynamics (CFD) models.[26]

For fully-developed fires, as explained previously, the simplest approach available to the structural engineer is to use the standard fire curve, but this has the disadvantage of not representing real fires and ignoring the behaviour during the cooling stage of a fire. The time-equivalence method[26] is a simple approach that tries to relate the actual temperature of a structural member from an anticipated fire severity, to the time taken for the same member to attain the same temperature when subjected to the standard fire curve. There are a number of time-equivalence methods[27] which take into account the amount of fuel load, compartment size, thermal characteristics of the compartment boundaries and ventilation conditions. Although simple to use, the time-equivalence is a crude approximate method of modelling real fire behaviour and the limitations of the method should be fully understood. The main limitation is that the method is only applicable to the types of members used in the derivation of the adopted formulae. For example, the time-equivalence method present in EN1991-1-2[7] is only valid for reinforced concrete, protected steel and unprotected steel.

Parametric fire curves allow the time–temperature relationship to be

Fig. 5: Picture taken after a full-scale fire test showing the loss of protection to the steel beams which occurred at the height of the fire

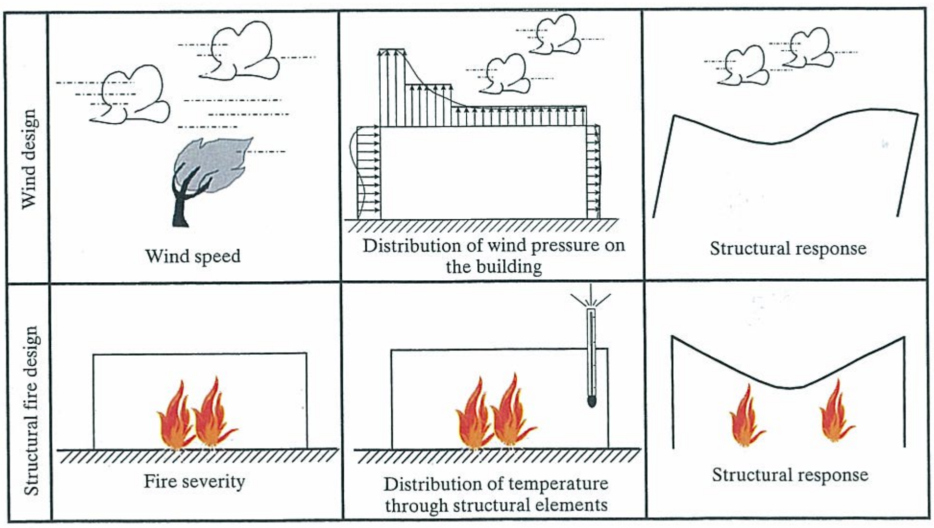


Fig. 6: Process of wind design and structural fire design

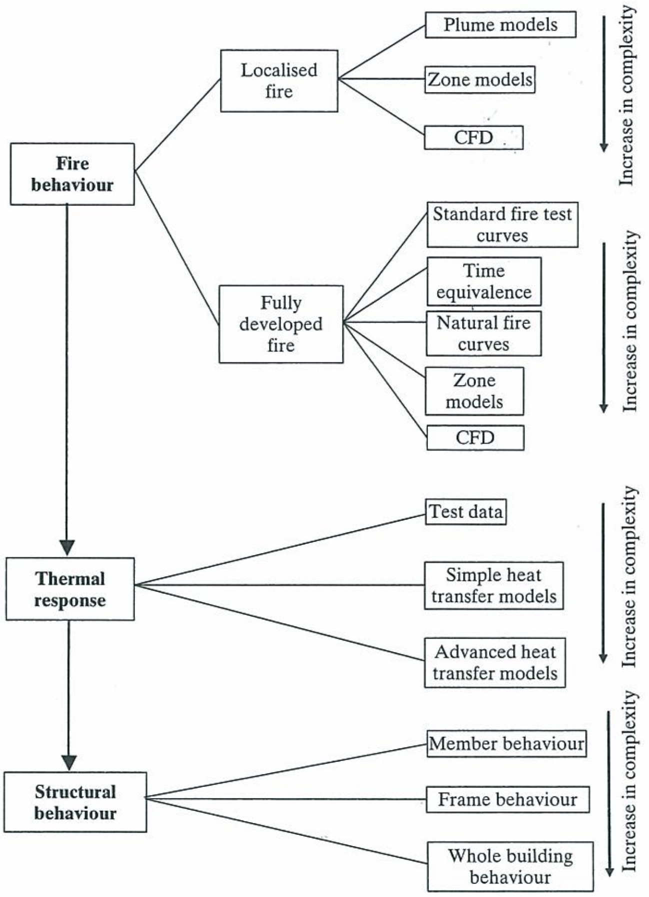


Fig. 7: Available approaches for the three components of structural fire engineering design

estimated over the duration of the anticipated fire. Compartment size, boundary characteristics, fuel load and ventilation are considered. The most validated approach is given in EN1991-1-2[7]: it is simple to use and, with the aid of simple spreadsheets, fire predictions can be easily derived. Once again the limitations of the method should be understood. In the case of the approach given in the Eurocode, limitations on the maximum compartment floor area of 500 m^2 and compartment height of 4 m are stated. In addition, the method is limited to fire compartments with mainly cellulosic type fire loads and compartments with linings that have a value of thermal inertia between 100 and 2200 $J/m^2s^{1/2}K$. The limitations are generally governed by the available test data used to derive the design method. These limitations are discussed in more detail in the National Annexes. For example, the UK National Annex[28] has removed the limitations on compartment floor area and height based on the argument that as the size of the compartment becomes larger the severity of the fire decreases. On the whole, parametric curves present a reasonable prediction over the full duration of the fire and are simple to use, allowing a wide range of fires to be considered. The popularity of parametric fire curves is growing amongst designers and has been used on a wide variety of projects.

Zone models[26] are simple computer models that divide the considered fire compartment into separate zones, where the condition in each zone is assumed to be uniform. The simplest model is a one-zone model in which the conditions within the entire compartment are assumed to be uniform and represented by a single temperature. A more sophisticated modelling technique is the use of CFD to predict fire growth and compartment temperatures. CFD has been shown to be successful in the modelling of smoke movement and has recently been applied to the modelling of fires. Similar to the use of any computer model, both the zone and CFD models require expertise in defining the correct input data and assessing the feasibility of the calculated results.

Thermal Response

Once the compartment atmosphere temperatures are predicted, the temperature of the structure needs to be estimated to allow the calculation of the structural response. The evaluation of temperatures in structural members can be extremely complex, especially for materials that retain moisture and have a low thermal conductivity. The simplest method of defining the temperature profile through the cross-section is to use test data presented in tables or charts, as published in the codes or design guides. The test data is generally based on the standard fire curve. Simple design equations are presented in codes discussed in Refs [9, 10 and 29] (and in some design guides[30,31]) to predict the temperature development of bare steel. The approach considers both radiative and convective heat transfer and although a spreadsheet is required to solve the equation over the fire duration, it is simple to use. Similar equations exist[9,10] for protected steel sections; however, the thermal properties of the proposed protection material must be known, which may be difficult to obtain from manufacturers.

It is possible to use heat transfer models based on one-dimensional heat flow. Alternatively, advance finite-element heat transfer models can be used, but this requires relevant expertise to ensure the models are applied correctly and used within their limitations. An example of the calculated heat transfer through a concrete column is shown in *Fig. 8*.

Structural Response

The simplest calculation methods to predict the structural response are based on the behaviour of individual members, using load and material safety factors at the fire limit state (FLS) which provide realistic estimates of the likely applied load at the time of the fire and the likely material resistance of the member. The approach of designing individual members has evolved from results and observations of standard fire tests. The methods are covered in the codes[8–12] and are predominantly based on strength. The use of member fire design at the FLS will be familiar to practising structural engineers, since the principles closely follow the approach used to check members at the ultimate limit state (ULS). The main differences between ULS and FLS are that for fire design different partial safety factors for load and material resistance are used (to represent an accidental limit state) and the strength and stiffness of the member are reduced on the basis of the temperature distribution through the cross-section.

The prescriptive methods discussed earlier have been derived from standard fire test data, assuming the members are fully stressed at ambient temperature. Member design at FLS has the advantage of allowing designers to predict the response of the structure under the actual likely load on the member at the time of the fire. Possible savings can be obtained, since it is unlikely that members are fully stressed at ambient temperature. This is because serviceability and buildability issues typically result in the specification of member sizes and strength greater than that required to fulfil ULS. However, the simple codified methods give no detail on displacement history, or the maximum displacement, which is important when considering the interaction of floor systems with compartment walls and overall integrity. Utilising member design and incorporating the relevant load and material safety factors, the steel industry has developed[32] various forms of construction that do not require applied fire protection. For example, for typical downstand "I" beams the floor slab can be supported by shelf-angles with the legs of the angles pointing upwards into the slab, a form of construction commonly referred to as "*shelf angle beams*" (*Fig. 9*). The part of the steel section and the supporting angles embedded in the supporting slab can allow 30 min fire resistance to be readily obtained for this type of member. It is possible to increase the fire resistance to 60 min, although the required thickness of the concrete slab may make this form of construction uneconomical.

As with steel beams, it is possible to enhance the fire resistance of steel columns by adopting systems where concrete and masonry materials provide sufficient, although partial, protection. A simple method is to place aerated concrete blocks between the inner faces of the flanges (*Fig.10*). For columns of size 203 ×203 ×46 UC and greater 30 min fire resistance can be achieved. Another method[32] involves filling the area between the flanges with unreinforced concrete. Welded plates and shear fixings allow the load to be transferred from the steel section to the concrete during a fire. This system can achieve 60 min fire resistance and can resist impact damage. A variation of the unreinforced infill column is to provide reinforcement to the infill concrete (*Fig. 10*). Similar to the infill beams, this system is popular in continental Europe and can achieve 2 h fire resistance. Another form of steel column that generally does not use applied proprietary fire protection materials is concrete-filled hollow steel sections (*Fig. 10*). The infill concrete may be unreinforced or reinforced depending on the required load-capacity and fire resistance. For reinforced concrete infill columns, 2-h fire resistance can readily be obtained. To achieve the most efficient concrete-filled column, in terms of fire resistance, the section should be designed in the cold state such that the load-carrying capacity of the steel shell is low compared to the load-carrying capacity of the concrete core.

Some systems, known as slim-floor beams,[32] are constructed such that the beam is encased in the supporting concrete slab with only the bottom flange or plate exposed to any fire (*Fig. 11*). Examples of slim-floor systems are "*Slimflor*" and "*Slimdek*", where beams can readily achieve 60 min fire resistance without the need to protect the exposed bottom flange or plate. The *Slimdek* system, incorporating an asymmetrical beam, has been tested at full-scale[16] and has shown to perform extremely well when subjected to a severe fire.

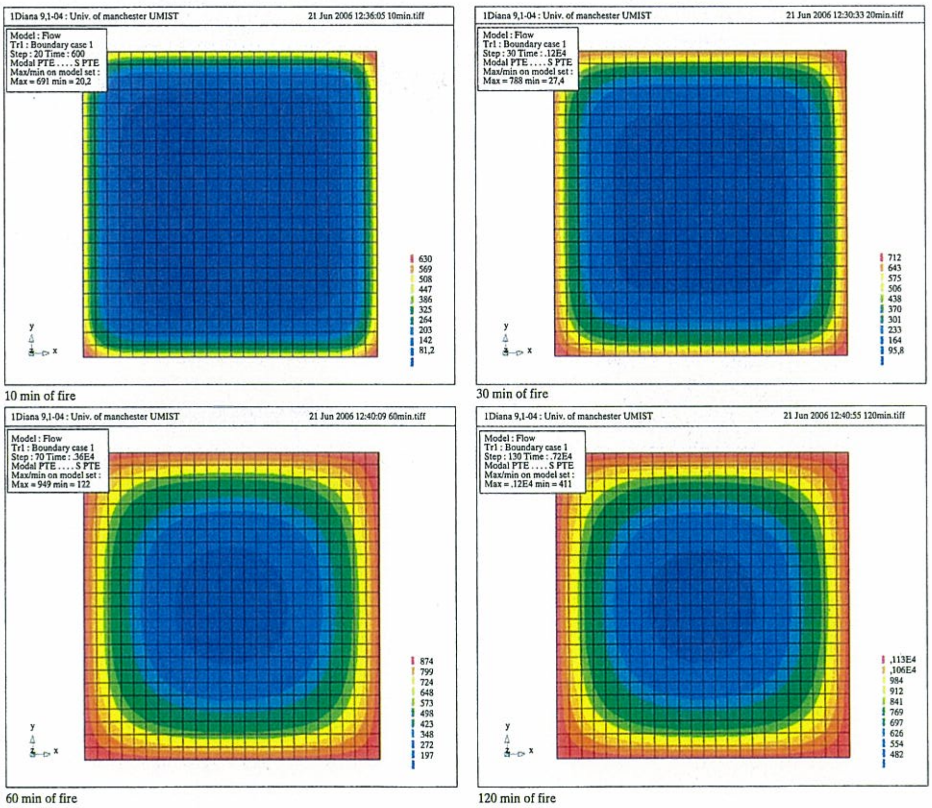


Fig. 8: Temperature distribution through a concrete column

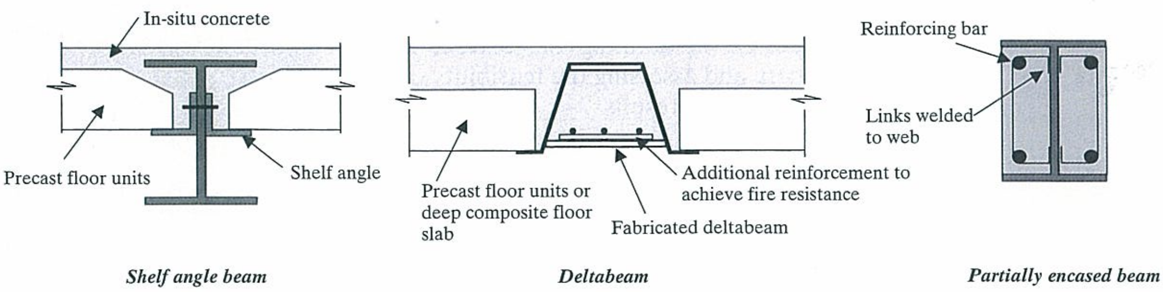
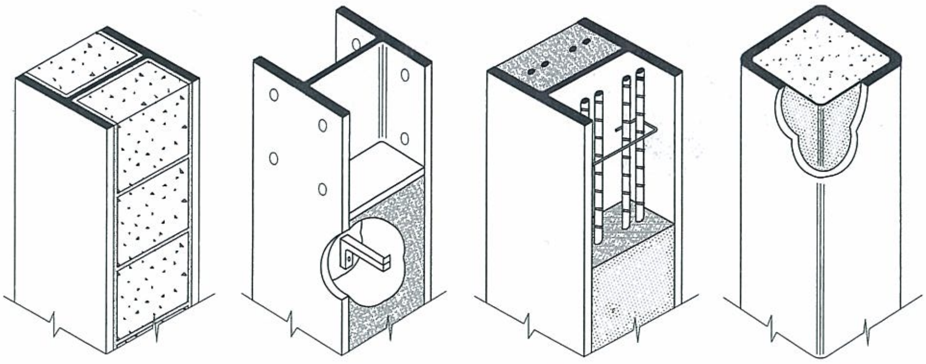
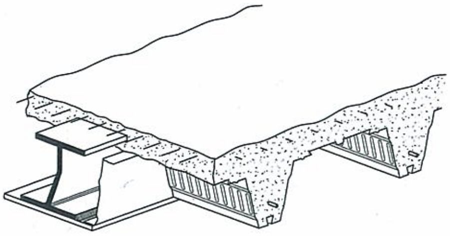


Fig. 9: Partially protected beams

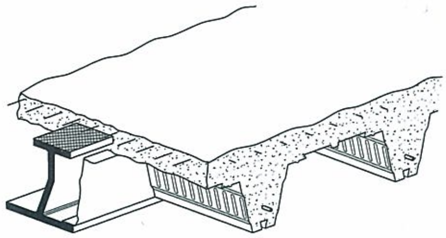


Blocked in steel column | Partially encased steel column (unreinforced) | Partially encased steel column (reinforced) | Concrete filled steel section

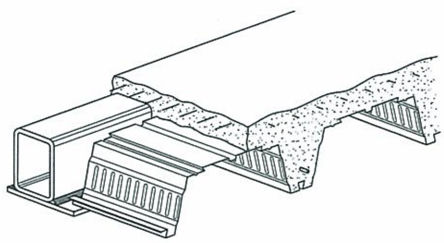
Fig. 10: Partially protected columns



Type of slim-floor ('SLIMFLOR')



Type of slim-floor ('SLIMDECK')



Type of slim-floor ('SLIMFLOR')

Fig. 11: Forms of slim-floor systems

Design methods exist to consider frame behaviour or parts of a structure in fire. In the Eurocodes[8–10] frame behaviour is utilised as a parameter to allow the effective lengths of continuous steel, composite and concrete columns to be reduced from their ambient temperature values. To conduct a frame analysis at elevated temperatures simple computer models are required, which include the effects of thermal expansion of the heated structure and correct boundary conditions.

Although member and frame design at FLS is a significant improvement on the prescriptive approaches, allowing designers to obtain some indication (although limited) of the actual behaviour of buildings in a fire, recent fire tests on full-scale buildings[13–18] have shown that member design is not, in some cases, realistic. To most designers this will come as no surprise since member design methods at ULS and SLS are only an approximation of the real behaviour of buildings. However, provided this approximation is conservative, and results in safe, usable and economic buildings then the design approach is acceptable.

The significant landmark in understanding structural behaviour during a fire was the full-scale fire tests carried out at Cardington in the UK. A total of seven tests[13,14] were carried out on the eight-storey steel-framed building, predominantly funded by the steel industry. *Figure 12* shows one test being carried out and the deflection of the floor slab once sandbags, which were used to represent the imposed load, were removed. The purpose of the tests was to show the inherent robustness of a steel-framed building without the need to protect all exposed steel members, which was the common approach at the time. The approach of applying protection to all exposed steel is expensive and time consuming. Indeed the tests were successful in showing that by utilising membrane action in the composite slab it was possible to leave a large proportion (40–55%) of the steel beams within a floor-plate unprotected. Simple design guidelines, which take into account the membrane action of the floor slab, have been developed by the author and incorporated into industrial design guides.[33,34] The approach has successfully been used on a number of projects both in the UK and across the world. *Figure 13* shows a project in the UK where the method has been used and where the protection is only applied to a limited proportion of the beams. It should be noted that the approach is not simply about removing protection from a proportion of the beams; it is about applying protection where it is needed and having a better understanding of the structural response in a fire. In some cases, the protection applied to the beams identified can be higher compared to designs that follow a simple prescriptive approach.

The fire tests on the Cardington steel frame also allowed the development

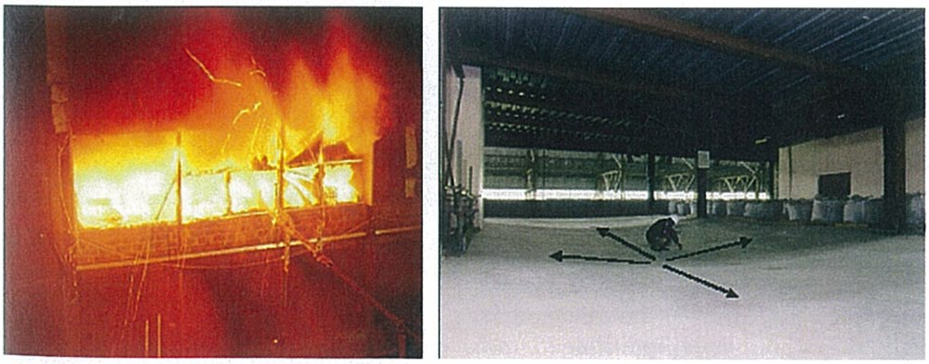
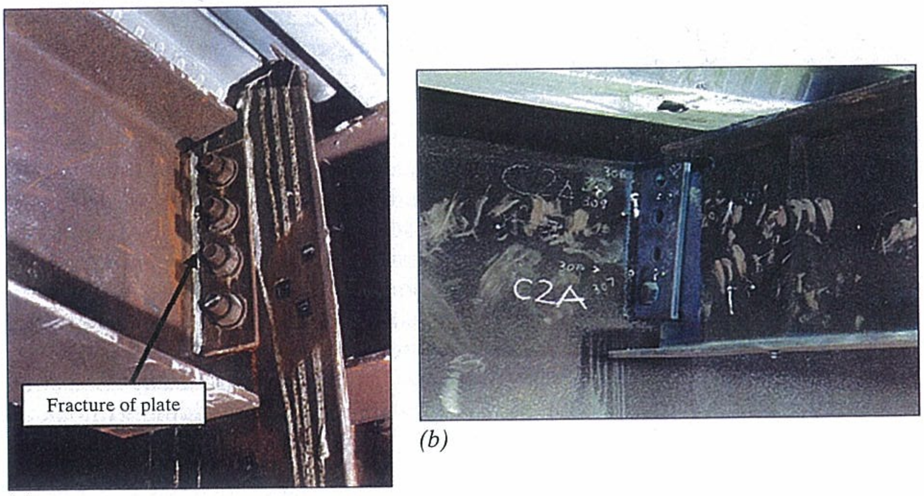


Fig. 12: Fire test on the eight-storey steel-framed building and deflection of the floor slab following a fire test (with sandbag loading removed)

Fig. 13: Project showing unprotected beams and protected beams utilising latest design methods incorporating membrane action



(a)

(b)

Fig. 14: Photographs showing (a) fracture of plate (end-plate) and (b) shear failure of bolts (fin plate)

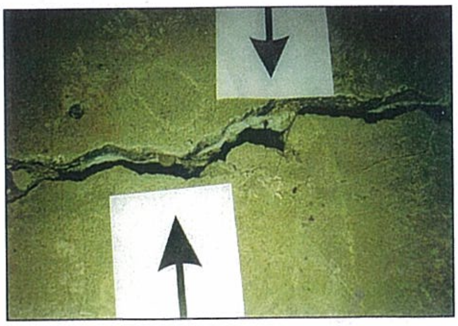


Fig. 15: Localised cracking of the slab over a beam (upward arrow shows shear stud, downward arrow shows fracture of mesh)

of finite-element models[35–38] which are being readily used in the design of buildings. Once again, these models allow an optimum solution of fire protection to be specified. They also identify "weak points" in the structure where additional protection can be applied. In most cases, although the protection is removed from some of the beams, the need to increase the protection on other beams and columns results in a more robust structure compared to one where all the exposed steel is protected based on the simple prescriptive rules.

Besides identifying beneficial behaviour the Cardington steel frame tests also identified detrimental behaviour, which is now being addressed by designers. For example, it was shown[13] that connections were subjected to large tensile forces during the cooling stage as the beams cooled down from an inelastic state (*Fig. 14*). Irrespective of wherever beams are protected or unprotected, connection behaviour should be considered during the full duration of the fire. Either the connection should be designed for the full tensile force encountered during the fire or designed to accommodate the force through ductility. For example, in the Cardington tests the web cleat connections failed by shear of the bolts (*Fig. 14*) whereas the flexible end-plates accommodated the force and maintained vertical support. Another area of concern is localised failure. For the composite slab this was in some cases due to inadequate lapping of the mesh reinforcement, which was designed as an anti-crack mesh in normal design but contributed to the strength and ductility of the composite floor during a fire. In other cases, localised failure of the composite slab was due to the high concentration of strain over supports (*Fig. 15*). If continuity of the mesh reinforcement is assumed in the design approach it is important that fracture does not occur.

Other full-scale tests have been carried out in the UK, including the tests carried out on a seven-storey concrete building[15] (*Fig. 16*), a six-storey timber-framed building[17] (*Fig. 17*), a large floor-plate for slim-floor beams[16] (*Fig. 18*) and recently two single-storey steel-framed and precast floor buildings[18] (*Fig. 19*). Again, the test on the seven-storey concrete building showed both beneficial and detrimental behaviour, which would be impossible to identify in small-scale tests. For example, compressive membrane action was identified which showed that although spalling exposed the bottom reinforcement (*Fig. 20*), the slab maintained its load-carrying capacity. The detrimental behaviour observed in the test comprised lateral movement of the columns, which could result in shear failure. This was witnessed in a real fire in Brazil in 2004, as shown in *Fig. 21*, although it should also be noted that overall stability was maintained. Careful detailing of the reinforcement in the columns would alleviate this problem.

A large-scale compartment fire test[17] was carried out on the six-storey timber-framed residential building, also constructed at the UK Cardington facility (*Fig. 17*). The compartment comprised a single flat within the building on the second floor. The primary objective of the test was to assess that structural stability was maintained and that the compartmentation enclosing the flat was effective in stopping the fire spreading to the rest of the building. The test lasted 64 min, at which time the fire brigade were ordered to extinguish the fire. The reason for the intervention was due to structural tim-

Fig. 16: Fire test on a seven-storey concrete-framed building

Fig. 17: Fire test on a six-storey timber-framed building

Fig. 18: Fire test on a 12,2 m × 12,2 m slim-floor system

Fig. 19: Fire test on 7,0 × 17,9 m precast units supported by protected steelwork

Fig. 20: Extent of spalling on the Cardington concrete fire test

Fig. 21: Shear failure of a concrete column following a fire (due to thermal expansion)

bers supporting the floor above the fire being exposed to the fire due to the loss of the insulating plasterboard. Although the plasterboard was fixed by a well-known contractor, the workmanship, especially the nailing of the plasterboards, was not to the correct standard. Again, this highlights the difference between small-scale fire tests, where the workmanship is of high quality and checked thoroughly, and a full-scale test, where the workmanship is not comparable. To ensure sufficient fire resistance, especially for timber-framed buildings, the standard of workmanship is critical; in particular, the correct type and spacing of fixings for the plasterboard must be used.

Another large-scale fire test[16] has been conducted on a slim-floor system

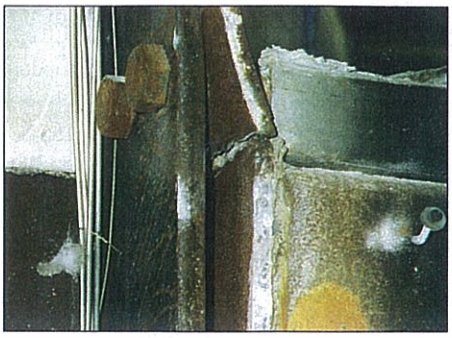


Fig. 22: Fracture of end-plate

(12,2 m × 12,2 m, *Fig. 18*), comprising asymmetric beams, rectangular hollow section beams and a composite floor slab. The aim of the test was to show the difference between the small-scale standard test and a more realistic large-scale test. The available data, together with observations from the test, provided a useful insight into the behaviour of the slim-floor system in its entirety. A significant observation from the test results was that the behaviour of the beam-to-column connections, which were assumed to be pinned in the cold design, had a significant beneficial effect on the overall structural response of the system due to their rotational capacity that was utilised during the fire. However, in one instance a connection fractured, as shown in *Fig. 22*, although vertical shear was still maintained. This fracture resulted in a sudden increase in the vertical displacement of the connecting beam as the connection lost its flexural capacity.

Recently two large-scale fire tests[18] on a hollowcore floor-plate (7,0 m × 17,86 m), supported on protected steelwork, have been carried out. The two tests were identical except for the connection details between the floor units and supporting steel beams. In the first test, the units simply sat on the top of the beams, whereas in the second test the units were tied to the beams using shear studs and U-bars. The main objective of the tests was to show that premature shear failure experienced in small-scale standard tests on one precast unit does not occur when the floor-plate is considered in its entirety. The tests were purposely subjected to a very severe fire created by specifying unrealistically small ventilation openings, compared to modern office construction. The hollowcore floor-plate performed very well (*Fig. 5*) supporting the full applied static load for the duration of the tests. Besides showing that shear failure does not typically occur in reality, and is induced by the constraints of small-scale tests, the tests highlighted a beneficial load-path mechanism created by lateral thermal restraint to the floor units, which has not previously been considered in design. Once again the full-scale tests showed that the small-scale standard fire tests, used to assess fire resistance, can be very unrealistic and ignore the beneficial effects of the structural behaviour. One interesting conclusion from the test was that there was no significant difference in behaviour, in terms of vertical deflection, between the two tests even though the tying details between the precast units and supporting steelwork were different.

Conclusions

This paper presents an overview of the development of structural fire engineering. Until recently, most structural engineers did not venture into fire design due to their lack of knowledge of fire behaviour, and relied on simple prescriptive rules for structural fire design. Likewise, fire engineers also relied on the same simple prescriptive rules, mainly due to their lack of knowledge of structural engineering and understanding of how structures behave under fire load. As explained, the prescriptive approach is based on fire resistance as measured in small-scale standard fire tests. Although the standard test does allow comparison to be made between different systems it does not represent reality both in terms of fire and structural behaviour. Unfortunately the introduction of the standard fire test has led designers and manufacturers to concentrate only on the performance of systems tested to obtain approval, and ignore the actual behaviour in buildings.

The development of performance-based approaches has been presented in this paper allowing designers to incorporate more realistic fire and structural behaviour in their designs. This approach allows the designer more "control" over defining the performance of the building by incorporating beneficial effects of whole building behaviour and identification of any "weak-links" within the structural system. There are various levels of a performance-based approach which have been discussed. These range from obtaining a better understanding of member behaviour to considering whole building response. By using the recent advances of structural fire engineering the designer can assess the robustness of buildings and increase the levels of safety (if required by the client) above the minimum legislative requirements for life safety, to better protect the building's superstructure and contents, business continuity, heritage, corporate image and environmental impact.

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